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CONTENT

Prof. Rejhana Dervišević
Editor in Chief

Dear readers, it is our great pleasure to offer to a scientific and professional public insight at the new issue of "Journal of Faculty of Mining, Geology and Civil Engineering".

In almost six decades of our Faculty's work, besides education was also conducted a research work, achieved through numerous significant domestic, European and international projects that contributed to the development of Bosnia and Herzegovina economy, mining, geological and civil engineering profession and science. Today's organization of our Faculty with five departments emerged from all general departments for scientific and educational work and those are: Mining, Geology, Civil Engineering, Borehole Exploitation of Mineral resources, Safety Studies as well as 15 scientific fields at which scientific work has been carried out (science fields 1.5, 2.1 and 2.7; Frascati).

Work on promoting and raising the quality, as well as affirming this publication, is a great challenge for every editor in chief. To accomplish this goal, current scientific and professional work, as well as systematic work, and successful co-operation of members of the Editorial and Advisory Board, reviewers and authors are necessary.

We would like to thank to the authors that have chosen our Journal for publishing their papers. We expect to continue and extend cooperation in the future, by contributing to the affirmation of the publication, and promotion of scientific thoughts and scientific results as well.

DOI 10.51558/2303-5161.2022.10.10.9 Scientific Paper

GEOMORPHICAL MESO-ENTITY SEMBERIJA LOWLAND PLAIN

Alen Lepirica[1.], Željka Stjepić-Srkalović[2.], Dado Srkalović[3.]

SUMMARY

In this paper, it was necessary to prove that Semberija is in the geomorphological sense, the southern part of the Pannonian Basin, which is in contact with the Dinaric mountain system. Its done on the basis of the geomorphological analysis of the terrain, primarily morphotectonic, morphological and morphogenetic similarities with the plain terrain of Pannonia. This was the basis for a complex geomorphological regionalization which included added similarities and connections with the Pannonian Basin in climate, pedogeography and biogeography sense. Thus, after the complex geomorphological regionalization, it was determined that Semberija is a meso-entity, a subunit of the geomorphological macroregion Sava Basin, ie the southernmost part of the geomorphological megaregion Pannonian Basin.

Key words: Complexed geomorphological regionalization, geomorphological meso-entity Semberija lowland plain, Pannonian Basin, Bosnia and Herzegovina.

1. INTRODUCTION

Semberija is the lowest, northeastern part of Bosnia, whose area according to J.Đ. Markovic (1988) with surface is about 300 km2. The boundaries of Semberija, which resemble an isosceles triangle, are very clearly - neotectonically, lithologically, geomorphologically and hydrologically determined.

The structural boundary of this lowland in the north is the Sava tectonic trench through which the Sava river flows. In the east, the border stretches along the Drina primary fault through which the Drina river meanders on the stretch from Janja to the confluence with the Sava. The southwestern border is structurally expressed by the local fault Crnjelovo Gornje - Janja. On this stretch, the border of Semberija between North Majevica hills and the Brčko plateau is morphologically clearly expressed in the surrounding relief. (OGK 100, Interpreter for Brčko) (See Figures: 2 and 3). De facto, it is also a morphotectonic boundary filled with proluvial-deluvial deposits that lithologically build holocene fans formed at the fault contact of the low flat surface of the Semberija plain with the surrounding low slopes of north Majevica hills.

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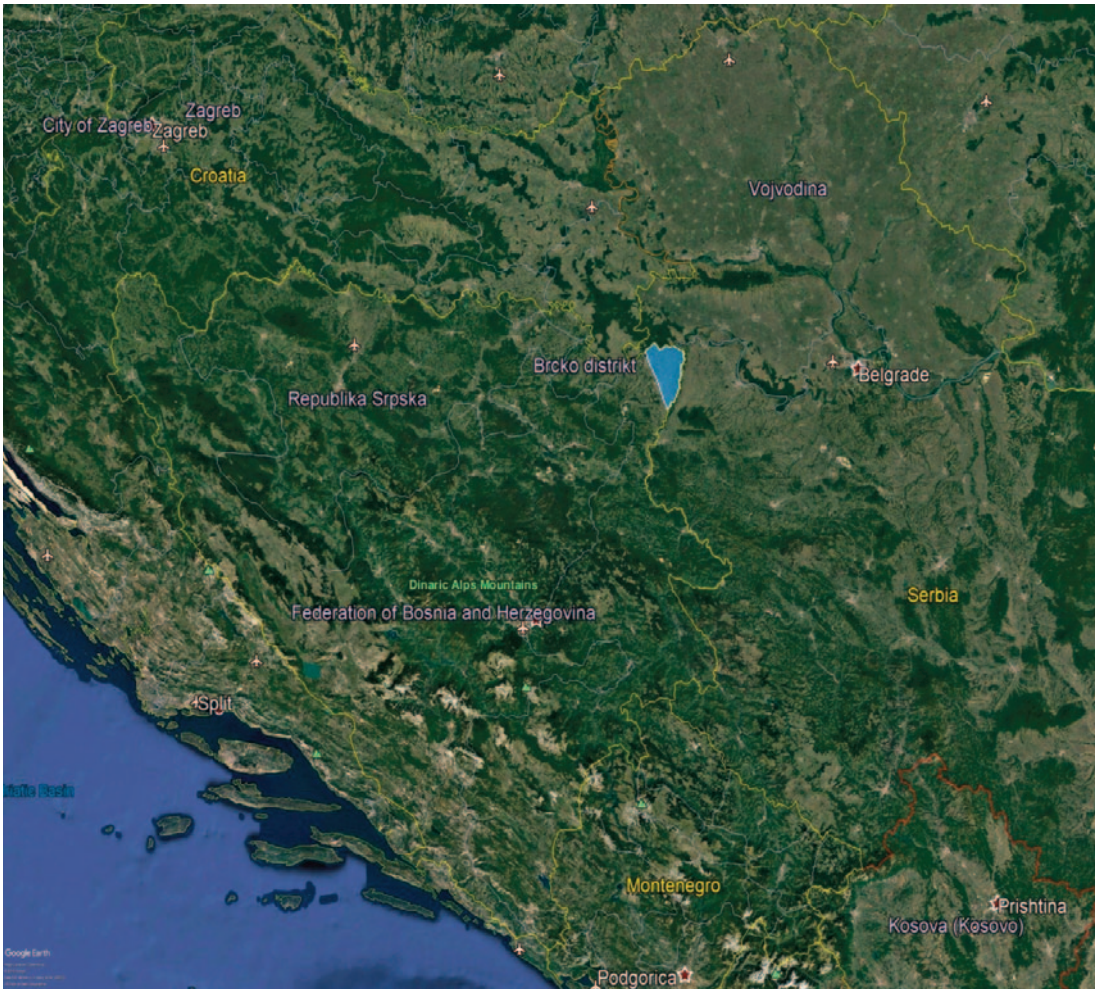

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Figure 1. Geographical position of the Semberija lowland plain (blue color), Source: Google Earth 2013.

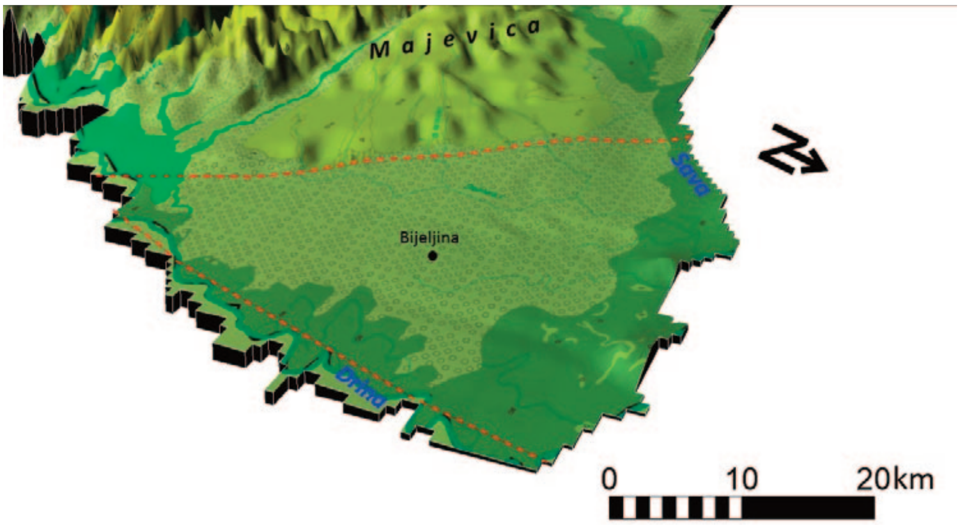


Figure 2. Semberija; The Drina River (east border) and the Sava River (north border). In the wide bank of these rivers you can see a higher floodplain (dark green), a terrace plain covering more space (light green with circles). The Crnjelovo-Janja fault (red dashed line, above of Bijeljina City) can be clearly seen as the morphological and morphotectonic boundary of the Semberijas terraces plain with the slopes of the North Majevica hills. (DEM: Dado Srkalović, 2019)

In the regional sense, Semberija has been treated as a part of the Pannonian or Peripannonian space in most of the physical-geographical and geomorphological works presented so far. This refers to the works of Yugoslav geographers: B.Ž. Milojević (1952), R. Petrović (1957), S. Ilešić (1961), J. Đ. Marković (1970, 1988), V. Rogić (1973, 1987) and others. The relief of the mentioned lowland mesoregion, considered geotectonically, was formed in the Sava zone, the youngest structural unit of the Inner Dinarides (Andjelković 1984). It is a zone of horsts and trenches (K. Petkovic, 1960) (Figure 3). The Pannonian Basin covers parts of the Pannonian structural complex and the Inner Dinarides (Supradinaric) (M. Herak 1990). Wrenching and tectonic inversion: NE-SW striking system of the dextral strike slip faults during Oligocene-Miocene was followed byNW-SE striking wrenching in Early and Middle Miocene, affecting South Pannonian Basin, Western thrust belt and Adriatic foreland. It is reflected as tectonic inversion in Mid-Bosnian Schists Vranica Mts. and as the system of the right-lateral strike and oblique slip faults, creating the large Sava and Drava depressions of the South Pannonian Basin. To the N and NE, Dinarides units were overprinted by tectonic processes controlling the evolution of the petroliferous Neogene Panonian Basin (V.Tari 2002.).

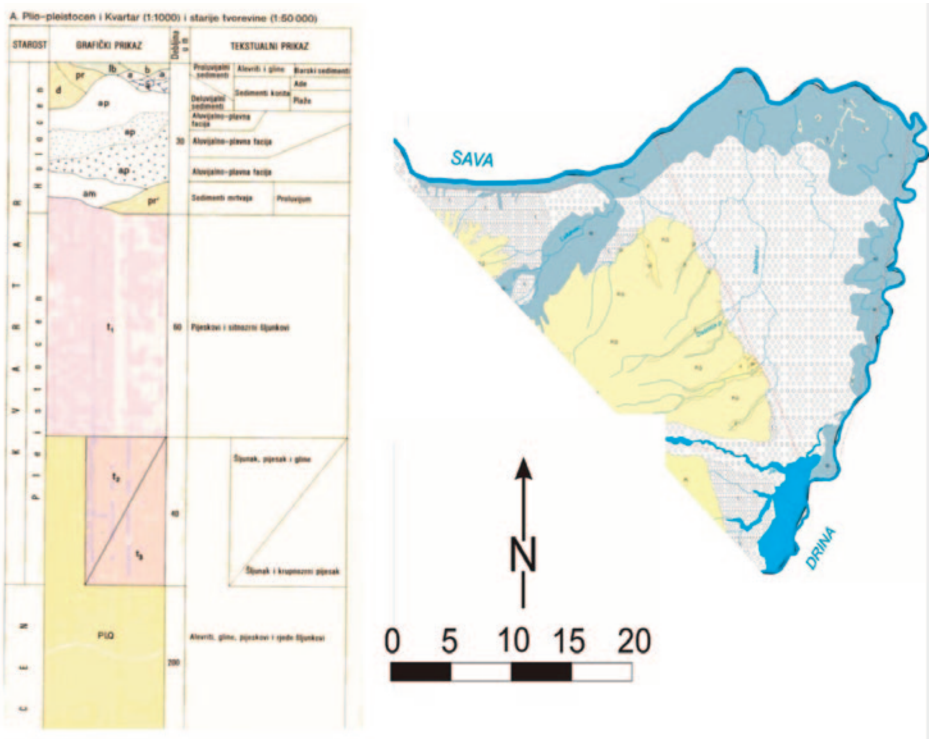
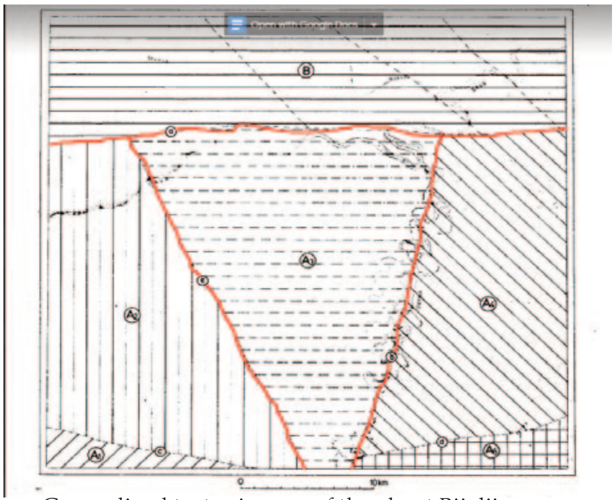


Figure 3. Geological map of Semberija

The active uplifting of the horst anticlinorium of Majevica took place simultaneously with neotectonic subsidence, which affected the surrounding paleodepression of the Semberija (Fig. 3 and 6.). Vrhovčić J. Buzaljko R. and M. Mojičević M. Mojičević (1986), (authors OGK 100, sheet Bijeljina, for SR BiH) state that the investigated mesoregion in the stratigraphic-structural sense enters the unit of Posavina and Semberija where it belongs entirely to the Dvorovi tectonic microblock (Fig.3 and 4). The modern Holocene period is marked by the activity of shear faults - the system of the Sava graben and the Drina fault and the secondary fault Gornje Crnjelovo - Janja (Fig. 3 and 4). The recent runoff directions of the Sava and the lower course of the Drina are neotectonically directed by subhorizontal wing displacements, strike-slip faults of the Sava and Drina trenches (A. Lepirica 2013) (Fig. 3 and 4).



Generalized tectonic map of the sheet Bijeljina

A - Unit of Posavina - Semberija: A_1 - block of Ugljevik, A_2 - block of Dragaljevac, A_3 - block of Dvorovi, A_4 - block of Mačva, A_5 - block of Lešnica;
B - Unit of Srem. Faults: a - Sava fault, b - Drina fault, c - Modrane fault, d - Lešnica fault and e - Crneljevo - Janja fault

Figure 4. F. Horváth, (1985b) in his work: "Thickness of Neogene Quaternary basin fill [Pannonian Basin system]", includes Semberija in the Pannonian Basin which is visible along the border with the Vardar zone and clearly represented by a blue dashed line (Fig. 5).

The fault neotectonic movements significantly determined the morphoevolution of the relief of this macroregion. With the uplifting of mountain elevations (altitude over 500 m above sea level) and the regression of Paratethys (or Pannonian Sea) in the pont, the morphoevolution development of the landforms of northern Bosnia and Semberija begins (Fig. 6).

In the analysed area, the average thickness of the continental crust is about 25 km. Semberija, from the lithological point of view, is characterized by heterogeneous lithofacial complexes (Fig. 3). It is mainly dominated by thick deposits of marine and lake sediments, genetically related to Paratethys and Paleogene and Neogene northern Bosnian basins. (Čičić, S., 2002) (Fig.2 and 3).

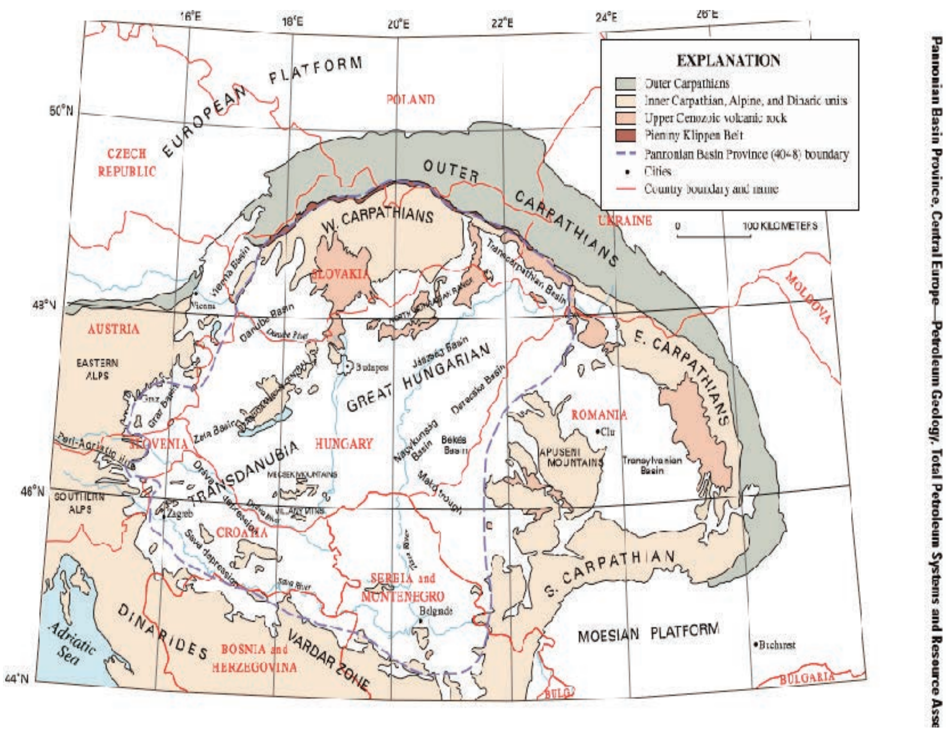


Figure 5. Horváth, F., 1985b, The Pannonian Basin, (Retrieved from: A study in basin evolution: American Association of Petroleum Geologists Memoir 45, map 1.)

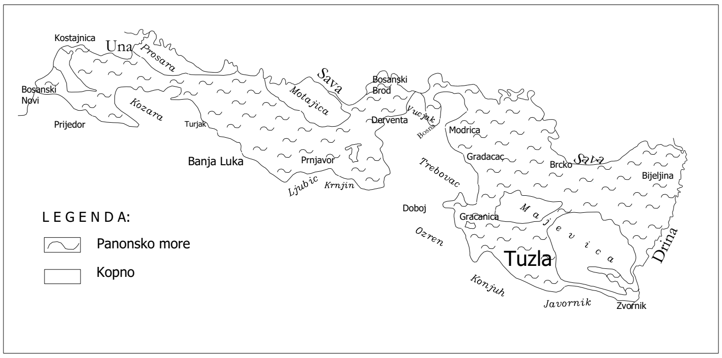


Figure 6. Paleogeographic map of northern Bosnia in Baden (Vrabac, S., Ferhatbegović, Z. & I., Đulović I., 2006)

2. MATERIALS AND METHODES

Thus, we used the method of complex geomorphological regionalization according to A. Bognar (2001). So we were primarily based on morphotectonic, lithological, morphological and morphogenetic similarities with the Pannnonian Basin. In the other hands, using the methodology of modern geomorphological regionalization, we took it into account additional, climatic, hydrological, pedological and biogeographical characteristics that resulted in the regional geomorphological determination of the Semberija.

The morphotectonic analysis have included: of the relations between relief, lithological substrate and neotectonic structures were analyzed.

The types and river network density were made by morphotectonic analyses using primary the topographic maps scale 1:25 000 and Google Earth satellite map.

The morphological analysis were included:of the orographic structure, hypsometric relations, slope inclinations and energy relief were made.

We used topographic maps scale 1:25000, 1:50000 and 1:100000, geological maps 1:25000, 1:100000, digital map Adria topo 2.10, and the Google Earth satellite map. In the terrain researches and analyses, we used a GPS (Garmin, Montana 600) for the observations of the microlandforms, in situ. Afterwards, the base of morphometric data was made, which was processed in the GIS using the Map Info Professional 9.5 application, and for creating the thematic maps and we used digital elevation model (DEM) and Satelit imagery google 2017.

Thus, the goal of this research is to determine the geomorphological position and regional hierarchy of the Semberija plain.

3. RESULTS AND DISCUSSION

In a strictly geomorphological sense, Semberija expresses in relief the northeastern part of the geomorphological macroregion "Mountains, hills, valleys and valleys of northern Bosnia" (Lepirica 2009).

From the aspect of geomorphological regionalization, it represents a mesoregion. It is a smaller, south-eastern part of the geomorphological macroregion of the Sava Basin. Semberija with the neighboring Mačva is the largest macro-flood in this part of Sava drainage system.

Figure 7. Photo: Ervin Lepirica, View of the low plain Semberija from Majevica from Bilalić (Teočak); on the left side of the picture on a flattened Sember terrace, in the distance, you can see a part of the town of Bijeljina.

It is part of the Pannonian Plain south of the Sava according J. Đ. Marković, 1988. Neotectonic subsidence of Semberija, 2-4 mm / year, active during the Quaternary (according to the map of vertical movements of the lithosphere crust, P. Jovanović, 1971) stimulated increased fluvial accumulation and lateral erosion of river flows. Geomorphologically, this was reflected in the development of floodplain and terrace plains (See Fig. 3).

Thus, fault neotectonic movements, but also climatic fluctuations during the Pleistocene and Holocene, significantly determined and directed the Quaternary fluvial and fluvio-marsh morphoevolution of the Semberija relief. The relief structure of Semberija is represented predominantly by flattened fluvial terrace landforms and elongated floodplains of the rivers Drina, Sava and their smaller tributaries. (Fig. 7).

From the hypsometric aspect, the lowest, lowland belt (0-200 m above sea level) is the most common in this mentioned area. The highest terrains extend of about 115 m A.S.L., at the contact of the terrace plain with the slopes of the North Majevica hills, southwest of Bijeljina.

It is tectonic trace of the Gornje Crnjelovo- Janja fault.

The terrain of Semberija is very pronounced, slightly, gradually sloping towards the erosion base - the riverbed of the Sava, which stretches to about 77.5 - 78 m A.S.L. Thus, the altitude in Semberija generally gradually decreases from southwest to northeast. The lowest values of vertical disintegration (0-5 m / km^2) and slope (0 ° - 2 °), (according to E. Hammond, 1964), predominantly characterize the terrains of the Semberija floodplains and terrace lowlands (See Fig.3 and 7).).

Therefore, in this lowland area, morphogenetically dominated by accumulation and fluvial lateral erosion (planning process) because this eastern part of the Sava Basin is neotectonically lowered between the Sava and Drina faults (Fig.2,3,5 and 7).

Semberija is mostly represented in relief by terraces and floodplains built by Quaternary alluvium and proluvium sediments deposits of the Drina (mostly) and the Sava, in the far north (according to the Interpreter of the Basic Geological map -100) (Fig.2).

Also in this lowland relief, the appearance of kilometer-long elongated swamp line depressions such as: Svora, Lukavača, Prugnjača, Popovača, Biogradska bara, Makova bara and others can be noticed. These are fragments of the former cut-off meanders of the Drina and Sava, which represent recent marsh. (See Fig 8).

Figure 8. Meandering of River Drina downstream from the Janjasettlement; IN the both sides in the picture you can see agricultural land of the terraced plains;floodplain s along the river flow (in the central part), the fragments of former riverbed (cut off meander) covered with hygrophilous vegetation, sandy-gravel bars with underwater hidrodynas, vegetation traces of former oxbow lakes (left) (Source: Google Earth 2011.)

The widest fluvial landform is a deep Sava riverbed, with an average width of 250-350 m, regulated for river navigation. A multi-kilometer long defensive embankment stretches above the Sava.

In these plain areas, the shallowly cut, unstable riverbed of the Drina, 170-300 m wide, is fulfilled with sandy-gravel deposits. Geomorphologically, Drinas meandering river bed it is marked by the braided channel patterns with dynamic development of fluvial microforms: river passages and channel bars with hydrodynes on bottom, accumulated point bars and opposite natural levees of the cut banks, river islands and oxbow lakes in initial phase on cut off meanders. On this part, of the lower Drina flow, the river islands are more than 1 km long as Buljuklića ada, Kolaričeva, Limanska ada, Nalića ada, Sitnjak ada and other numerous river islands, are covered by dense hygrophilous vegetation of willows and poplars.

The transition to the older phase of free meandering is marked by the moment when river currents, erosively cut the narrow meandering neck at higher flows. At the places where the meandering peaks are cut off, oxbow lakes are formed, filled with fine-grained muddy-clay sediments. With the subsequent intensification of organogenic-bar processes, the oxbow lake become swamp with further development, completely dry up with dense vegetation (Fig. 8).

Recent swamps, hundreds and thousands of meters away from the recent riverbeds of the Sava and Drina, indicate their former, wide, lateral migrations. During the colder winter months, they are often frozen. The concentration of oxbow lakes and swamps is mainly noticeable in the northeast part of area, around Velino Selo, where the fluvio-swampy lowland relief dominates. During the winter temperature minimums, the mentioned northern Bosnian river flows are partially frozen. It is a short time situation, expressed in places of slight falls of the longitudinal profiles, where the river flow are very slowly. The free meandering of the lower course of the Drina, through neotectonic subsided areas of Semberija and Mačva, formed a whole network of new and fossil river beds with channel bars, backswamps, abandoned river islands, fluvial beams and numerous oxbow lakes (See figure 8).

The average annual flows of this largest tributary of the Sava are 395 m^3 / sec. (Source: D. Dukić: Enciklopedija Jugoslavije, 1990.) Slightly curved meanders of smaller river flows Janja (a tributary of the Drina) and Dašnica (a tributary of the Sava) up to twenty meters wide intersect the Semberija terrace plain. "Active fluvial landforms include alluvial fans, terraces, deltas, and floodplains. The latter are often further subdivided into natural levees, channel bars, backswamps, infilled channels, and crevasse deposits" (P. F. Hudson 2017). The torrent Drina riverflow, which is in the upper course of the nival-pluvial river regime into the low Sava valley, flows from the high mountainous hinterland whose peaks often exceed altitudes of 2,200 m above sea level. The upper course of this mountain stream drains Dinarides mountain system of the southeastern Bosnia, northern Montenegro and Albania. The Drina at the confluence with the Sava, especially in the period of its very high water levels of the Drina, affects the characteristic occurrence of slowing water runoff, which is one of the causes of flooding of the surrounding plains and the formation of dynamic forms of low floodplains. Spatially, the largest alluvial plains, in Bosnia and Herzegovina, reshaped by the lateral planning of the Drina and Sava, are located in Semberija (Fig 2.). In the geomorphological sense, they represent higher alluvial planes formed 2-3 m above the rivers that are higher part of the riverbed. They express very high water levels of annual discharge in the lowland relief. Their flattened areas, kilometers wide, extend above the low river banks of the Drina and Sava. More alluvial lithologically are represented by alluvial-proluvial and marsh sediments on which alluvial and gley soils have developed. They are dominated by rounded, loose alluvial deposits (gravel, sand, fine-grained clayey sand, clays and sandy clays) (Fig 3.). Periodically flooded terrains of the northern Bosnian floodplains are marked by numerous smaller permanent wetlands, overgrown with reeds and sedges (Lepirica 2013). This lowland are hydrologically fed by flood, groundwater and precipitation waters. Micro-relief depressions of higher fields are often represented by oxbowlakes, traces of past, old riverbeds, channels and periodic swamps, recognizable by dense hygrophilous vegetation which, with the additional application of modern agro-technical measures, turned higher floodplains into fertile agricultural areas (Fig. 8).

J.Đ. Markovic (1970) states that Semberija is for the most part a fluvial macrofan of the Drina River, whose sediments from the Pleistocene to the present day have shifted the Sava river low to the north. Northeast of the confluence of the Drina and the Sava, there is a classic example of the movement of the Sava riverbed by about 5 km to the north. South of the current flow of the Sava, the entire old flow of the Sava has been preserved, in the abandoned meander of the flow of Resavica and Banovo polje (Tumač 1986.). Thus, in Semberija, in the morphogenetic sense, the fluvial relief of the terrace lowlands dominates (Fig 3.). As part of the terrace plains, sand, gravel, silt and clay, over 80 m thick, participate (Fig 2.). Those plains are dissected by meandering, unstable riverbeds. Partially denuded forms of the terrace lowlands of Semberija appear in three elevation levels. They extend above the alluvial-flood plains. They represent older, draining planar fluvial forms, sandy-gravelly-clayey composition. The morphoevolution of terrace forms is marked by succession of lateral and deep fluvial erosion caused by neotectonic changes and climatic fluctuations.

The lowest first river terraces, cut by fluvial erosion in Quaternary sediments, were leveled by subsequent lateral erosion. The Semberijas part of the first Drina terrace is located 4 to 5 m above the normal level of the Sava and Drina rivers, and at altitudes of 80 to 100 m. It is situated between the Drina and the alluvial floodplains of the Sava (Interpreter of the Basic Geological Map 100, 1986). On the surfaces of the first river terrace, the Bijeljina City developed, as well as many other settlements. (See Fig.3) Spatially, the largest are morphoevolutionarily older, other Pleistocene river terraces (t2), are slightly inclined towards the bottom of the basins. They were formed at about 12-20 m, above the remains of the older, higher, third Pleistocene terrace (t3), they are fragmented. They are located above low fields, at altitudes of about 105-115 m A.S.L. Due to the mechanically more erodible Quaternary lithological composition, their terrace sections were denudation-destroyed. The edges of the higher terrace areas, in contact with the slopes of the low hills in the south, are covered with proluvial-deluvial deposits. Proluvial-deluvial fans landforms formed primarily by torrents and leaching from the surrounding slopes during rainfall and during periods of melting of the snow cover. In relief, they represent mainly the southwestern border with the North Majevica hills. They are formed in contact with the more flat terrains of the Semberija lowland, where the kinetic energy of torrents decreases sharply. Anthropogenic - technogenic relief is mainly concentrated around Bijeljina City where it is represented by the urban center of Bijeljina and numerous newly built satellite settlements not far from the city with supporting infrastructure (residential, industrial and energy facilities, asphalt roads, railways, canals, transmission lines). The settlement of Janja and other smaller rural settlements with agricultural areas belong to this genetic type of relief. It is important to point out that a large part of the fertile terrace of the Semberija lowland plain has been brought to agricultural use. Then, the ethno village "Stanišić", the infrastructure swimming-pool in Dvorovi and especially the technogenic relief are expressed by numerous canals (Dašnica, Drinica) and the defensive embankment along the Sava River.

Climate significantly affects the characteristics and intensity of exogenous geomorphological processes. Most of northern Bosnia is dominated by a temperate-continental climate with more pronounced continental characteristics of thermal and pluviometric regimes. "The main maximum precipitation falls in spring or early summer and the secondary in autumn." (R.Milosavljević, 1984-85.)

Table1. Mean air temperatures and average values of annual precipitation in the period 1951-1970.

Weather station	**Altitude** (m)	**Medium per year** air temperature (^{0}C)	**Average annual precipitation** (mm)
Brčko	96	11,2	781
Bijeljina	90	10,9	751

(Source: Ecological-vegetation regionalization of Bosnia and Herzegovina.- Special editions no. 17, Faculty of Forestry, 51, Sarajevo, 1983).

The lowest average values of annual precipitation of 600 -750 mm characterize the low area of Semberija, in the northeast (See Table1). Vegetation in this basin area exists on fluvial, fluvio-swampy sediments soils such as predominant eugley, fluvisol, semigley.

Semigley soils are very high quality agricultural soils, and are found in Lijevče polje and Semberija (Ahmetbegović, S., Stjepić-Srkalović, Ž., Gutić, S., 2017). In the Sember lowland, from a pedological sense but also from a bioclimatic point of view, oak forests and other moist and semi-moist forests of alder, field ash, willow, poplar, shrub and shrub (Almion glutinosae, Almion incanae, Populion alba) are partially developed. (Source: Intrazonal forest vegetation in SFR Yugoslavia, Encyclopedia of Yugoslavia 1990)

4. CONCLUSION

Semberija is a part of the Pannonian Basin in the far south, as evidenced by works related to the scientific geological research of the F. Horvat (1985), M. Herak (1990). and V. Tari (2002). Therefore, the lowland plain of Semberija, in geomorphological sense belong the Pannonian Basin. There is no doubt that morphostructural and morpholitogenic elements are decisive for the regional differentiation of the state space (Bognar 1995). Apart from them, the predominant interference of other natural elements - climatic, hydrological, pedological and biogeographical, which represent the principle of special spatial connections, was of great importance in the separation of regions in individual cases. The mentioned principle is often used in natural geographical - homogeneous regionalization, but also in modern geomorphological regionalization (Bognar, 1997, Lepirica 2009, 2013). A. Bognar (1995), regionalizing the territory of Croatia, singled out the Pannonian Megaregion of Croatia, the southwestern part of which tectonically belongs to the Inner Dinarides. Of course, a similar approach of homogeneous, physical-geographical and geomorphological regionalisation in this paper was applied to the Semberija. Therefore, on the basis of the performed geomorphological regionalization, it was determined that the geomorphological mesoregion of the Semberija Lowland Plain. It is the southeastern part of the Sava Basin macroentity. In the regional separation of the analyzed area, the criterion of specific spatial connections used in modern geomorphological differentiation of the area was strictly taken into account. Thus, complex geomorphological and physical-geographical analyses of the plain terrain obtained a very high degree of correlation or natural homogeneity with the geomorphological megaentityof the Pannonian Basin in relation to neotectonic subsidence, landforms(river terraces, alluvial plains, meandering riverbeds), morphology (lowland plain, relief energy, altitudes), climate (continental precipitation regime, mean annual precipitation height 600-700 mm.). Also, the complex analysis included general Pannonian hydrographic features (mechanisms of runoff of lower river watercourses).The Quaternary hydrodynamics of the Drina river flow reflected in the morphoevolution of two lowland plains - Semberija in the west and neighboring Mačva in the east.

These fluvial processes also directly affected on the pedogenesis of fluvial-marsh soils (eugley, fluvisol, semigley) and the vegetation features (oak, hornbeam, malt, poplar and willow) which confirms that Semberija is the southern part of the Pannonian Basin geomorphic megaunit.

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GEOLOGICAL CHARACTERISTICS OF THE KOTEZI MINING DISTRICT IN THE BUGOJNO COAL BASIN

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SUMMARY

This paper presents geological characteristic of the deposit Kotezi of the Bugojno coal basin. Regional geological researches were conducted from 1983 to 1987, and detailed from 2014 to the end of 2018.
After field research and laboratory tests, the analysis and interpretation of the obtained results was carried out. Based on the determined borders of surface distribution and the research results of the coal deposit Kotezi, proved are four coal layers: II bottom, I bottom, main and roof.
Coal of the deposit Kotezi belong to the younger coals, soft to medium hard, no luster (matt), brown to black colored, and have brown streak. Their fracture is plate-like and particleboard. He do not have a distinct lignite structure, except in the lower layers. With their general habitus, they resemble younger brown coals, and belong to humic coals which are relatively low in carbonation.
Coal deposit Kotezi is the most important deposit in the Bugojno basin, which is very important due to continuity of coal exploitation.
Considering the raw material potential and the possibility of expanding the existing raw material base, the Bugojno coal basin has particular importance for the long-term development of lignite exploitation and its use for thermal energy purposes.

Key words: the Bugojno coal basin, Kotezi, coal layers, raw material potential.

1. INTRODUCTION

The deposit of coal Kotezi is located on the territory of municipalities Bugojno and Donji Vakuf. From a structural-geological point of view, it belongs to the northwestern part of the Bugojno coal-bearing basin (Figure 1).

Western boundary of the deposit is represented by peripheral parts of the settlements Poriče, hamlet Miličevići and settlement Ždralovići. Settlements Potkraj and Prusac are in the northwest. Settlements Fakići and Guvna, as well as hamlets Grabovci and Krčevine are all located on the north. On the south the peripheral parts are settlements Karadže and Gaj Berića, while the eastern border is represented by the settlements Milanovići and Udurlije and the hamlet Orčevići. In the central part of the productive area are settlements Gornji and Donji Kotezi. The deposit is about 5 km long along the longer axis, and about 3 km along the shorter axis [1,2].

Regional geological researches were conducted from 1983 to 1987, and detailed from 2014 to the end of 2018. [3-13]. Mining district Kotezi contains very significant but insufficiently explored lignite reserves. It should be considered that only shallower parts of the deposit have been explored, so it is justified to predict significantly higher reserves in the deposit.

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Figure 1. Panoramic view of the coal deposit Kotezi (Forčaković Dž. 2021)

2. GEOLOGICAL CHARACTERISTICS

Neogene sediments are represented with freshwater lake formations, situated discordantly over the older basement. This basement consists of Middle Triassic and partly Upper Cretaceous sediments.

In the development of the Neogene, the Middle and Upper Miocene were separated (Figure 2), which is divided into seven lithostratigraphic units: basal ($^1M_{2,3}$), the second bottom coal layer ($^2M_{2,3}$), clay, clayey sandstones and marls ($^3M_{2,3}$), first bottom coal layer ($^4M_{2,3}$), marly limestones and marls ($^5M_{2,3}$), main coal layer ($^6M_{2,3}$), clays and sandy clays ($^7M_{2,3}$) [2,7,11,12,14,15]. Miocene is followed by Pliocene-Quaternary (Pl,Q) and Quaternary (Q) sediments.

The deposit is elliptical in shape, running direction northwest-southeast. Coal layers dip slightly to the northeast at an angle of 0-27°. This represents an unfavorable circumstance due to the increase in the depth of the main coal layer, because the thickness of the overburden cover, i.e. the overburden coefficient increases.

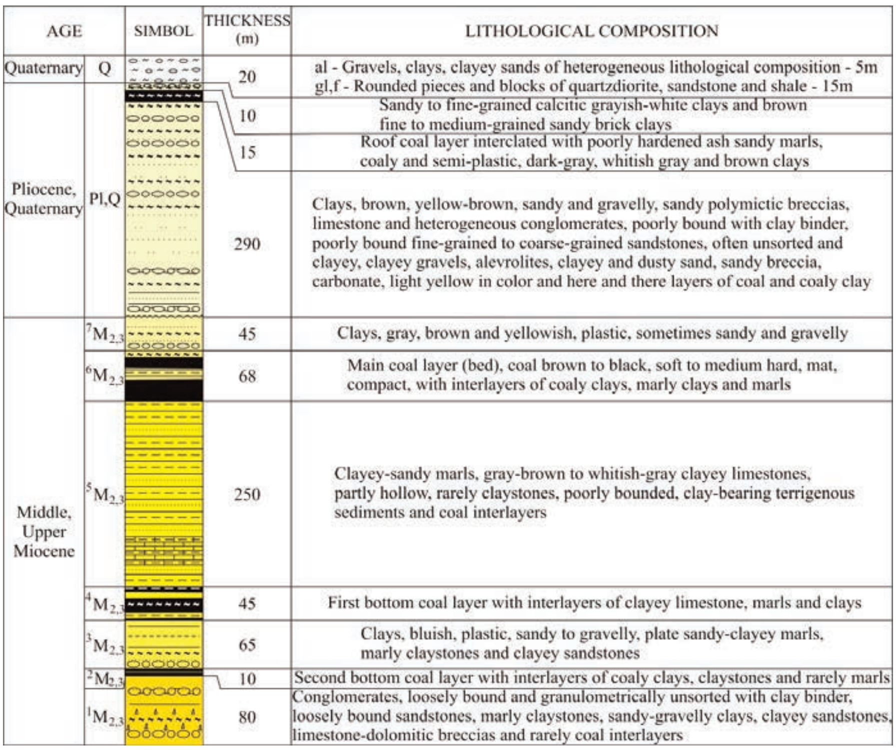


Figure 2. Sinthesized geological column of the coal deposit Kotezi (Forčaković Dž. 2021)

There are four coal layers of different economic importance in the deposit: II floor, I floor, main and roof.

According to the composition, depth and exploitability of coal layers in the Kotezi deposit, the following are distinguished:

- Roof coal layer - quite complex structure (conditionally exploitable) - It is spread over a relatively small area Karadže-Gaj Berića-Gradina.
- Main coal layer - relatively complex structure (exploitable) - Has the greatest distribution in deposit.
- The first bottom coal layer - very complex structure (non-exploitable). Only in the area from Silin to the west of Guvna, where it stretches along the Rigavac stream bed to Zgonovo near Prusac, could it be exploitable, because the outcrops were mostly discovered in the Rigavac bed, and it is 7.4 m thick (borehole B-43).
- Second bottom coal layer - extremely complex structure (non-exploitable). It has a smaller distribution than the first bottom coal layer.

Basal unit ($^1M_{2,3}$)

Going from the northwestern peripheral belt of the basic highlands, from the outcrops towards the morphologically lower parts of the terrain (towards Vrbas river), the basal unit ($^1M_{2,3}$) is covered by thick deposits of younger coal-bearing Middle and Upper Miocene and Pliocene-Quaternary sediments.

In the lithological composition of the basal unit, coarse-grained to fine-grained and clayey terrigenous sediments alternate: weakly bound and granulometrically unsorted conglomerates with a clayey binder, weakly bound sandstones, marls, calcitic clays, sandy-gravelly clays, argillaceous sandstones, limestone-dolomite breccias and rarely interlayers of coal with a thickness of 0.1-1.0 m, as well as plate-like interlayers of sandy marls.

Sediments of the basal unit are predominantly gray-green in color. Their dip is about 12^o. No macrofauna was found in them, and the microfauna is modest. Determining pollen particles from samples of the basal unit, from the borehole B-18: *Verucatosporites favus, R. Pot. (Polypodiaceae), Pityesporites mikroanalatus R. Pot. (Pinus), Subtriporopollenites simlex, R.Pot. (Carya), Polyporopollenites undulosus, Wolf (Ulmus)* and by the method of correlation with the results of analyzes from coal layers and sediments that build the floor of the upper coal layers, it can be concluded that the basal unit and the coal-bearing lithostratigraphic members were deposited during the Middle - Upper Miocene [16]. Lithological composition of the basal unit has significant practical value for borehole suspension. Thickness of this lithostratigraphic unit is up to 80 m.

The second floor coal layer ($^2M_{2,3}$)

The second floor coal layer lies concordantly on the basal unit and belongs to the oldest coal layer in the basin.

Its natural outcrops with elements of spatial orientation 110/14 were registered at the locality near the village of Ždralovići, along the western edge of the basin [27]. The outcrop zone is located in the southwestern part of the basin immediately above the settlements of Alibegovići, Poriče, and towards Ždralovići and Prusac, while the northeastern and eastern borders are formed by the Lubovo and Spahinac streams.

The greatest depth of the second floor layer was determined northeast of Fakić in the area of borehole B-31, where it lies to a depth of 424.5 meters, and the shallowest part is south of Prusac (locality Potkraj) in the area of borehole GO11, where it lies to a depth of 13 m. Floor sediments were drilled at 14 meters, and they consist of sandy dark brown and calcitic green clays.

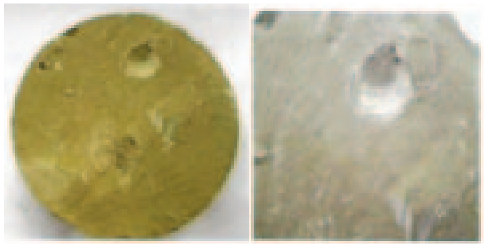
In the phase of regional geological research until the end of 1987, it was drilled with 16 boreholes. During 2017 and 2018 (Phase of detailed geological research) it was drilled with 6 more boreholes (K40, K45, K80, K118, GO11 and GO12) [10,18].

Geological thickness of the coal layer is variable. The average thickness of the layer is 5.6 m, and the average thickness of clean coal is 4.34 m. Interlayered tailings consist of clays, carbonaceous clays and, more rarely, marls. East of the Spahinac stream, the second floor coal layer suddenly loses thickness and disappear. No fossil content was determined in material taken from this lithostratigraphic unit. The thickness of this lithostratigraphic unit is up to 10 m..

Unit of clays, clayey sandstones and marls ($^3M_{2,3}$)

This lithostratigraphic unit represents the immediate and higher cover of the second floor coal layer. The sedimentation cycle begins with plastic clays, followed by the sedimentation of calcitic clays, argillaceous sandstones, and sandy to gravelly clays of predominantly gray-green and dark red color.

Packages of plate-like sandy marls with poorly preserved and fragmented fossils of freshwater fauna are subordinately represented in the northwestern part of the Bugojan coal-bearing basin. In the gray sandy marl of borehole K45 at a depth of 348.4 m, a fossil fragments of bivalve lids Pisidium sp. (Figure 3) [14] were found.



a) b)

Figure 3. Lid of bivalve *Pisidium sp.* (a-core, b-enlarged part of the core, the length of the lid is 11 mm) in the core of the borehole K45 from a depth of 348.4 m (Vrabac S., and Đulović I. 2017)

The first floor coal layer ($^4M_{2,3}$)

This zone is built of coal and interlayers of sandy-clayey limestone with shells, marls, bituminous sandy marls and clay (Figures 4 and 5).

It was drilled with a total of 28 boreholes (5 boreholes were drilled in 2017 and 2018: K40, K45, K80, K118 and G5).

In the phase of regional geological research until the end of 1987, it was drilled with 23 boreholes. The thickness of the first floor coal layer varies between 1.1-42.0 m. The average thickness of the layer is 12.1 m, and the average thickness of clean coal is 6.1 m. Distance between the second and first floor coal layers varies between 1.56 m (exploratory borehole BŽ-1) and 184.9 m (borehole B-32). The thickness of this lithostratigraphic unit reaches 42 m.

Unit of clayey limestones and marls ($^5M_{2,3}$)

Outcrops of sandy-clay and tufa limestones and marls are rarely found on the surface. They were discovered on the left bank of the Poričnica riverbed in the cuts of the ascent road towards the hamlet of Miličevići and in the vicinity of Prusac, Guvna, Ždralovići and Gorica (the cut of the road to the former Tito's villa). This superpositional lithostratigraphic unit is represented between the first floor layer and main coal layer.

Sedimentation of this lithostratigraphic unit begins with sandy marls with ostracods and rarely represented freshwater macrofauna.At the location of borehole B-9, the immediate cover of the first floor coal layer is represented by clayey limestones with shells, Pisidium sp. et al. In the immediate roof of the first floor coal layer, plate-like to thin layered microcrystalline limestones with calcite veins (lake "chalk") [7] can be found.

The most common lithological unit in this zone is clayey limestone, gray-brown to whitish-gray in color. In places it is a poriferous fossil-bearing clayey limestone. In addition to clayey and microcrystalline limestone, marls, sandy marls and less common clays are present in the zone, followed by weakly bound, clayey, terrigenous sediments and interlayers or layers of coal 1-2.2 m thick. The content of $CaCO_3$ in limestones is very high (93-98%), which indicates the possibility of their application in the paper and rubber industry.

The zone of clayey limestones and marls is the most fossil-bearing. However, the freshwater microfauna from clayey limestone layers is characterized by thin soft shells. That is why it is poorly preserved. It is often broken, crushed, and identification of the species is very difficult even in the selected samples.

As part of detailed geological investigations of the Kotezi coal deposit in the northwestern part of the Bugojno coal-bearing basin, micropaleontological investigations were also carried out. Damaged shells of the gastropod Valvata cf. Abdita Brusina (Figure 4) were found in the material from well K45, with a maximum size of 0.8 mm [14].

Figure 4. Shells of the gastropod Valvata cf. abdite Brusina from the core of the borehole K45 from a depth of 139.0-139.2 m (Vrabac S., and Đulović I. 2017)

In the gray marl of borehole K45 at a depth of 179.1 m, a fossil with damaged lids of the bivalve Sphaerium cf. Ožegovići Brusina was found (Figure 5).

Figure 5. Lids of the bivalve Sphaerium cf. Brusina burns (lids length up to 24 mm) in the core of the borehole K45, from a depth of 179.1 m (Vrabac S., and Đulović I. 2017)

In the light gray sandy marl from borehole K80, at the interval 121.8-122.0 m, a gastropod shell fragment (?Lymnaea) and fragments of carbonized plants were found (Figures 6 and 7) [14].

Figure 6. Fragment of gastropod shell (?Lymnaea), borehole K80 (Vrabac S., and Đulović I. 2018)

In the gray sandy marl of borehole K80, at the interval 296.8-297.0 m, a fragment of a Quercus sp. leaf impression was found. (Figure 7), as well as fragments of bivalve lids and a fragment of a gastropod mold [15].

Figure 7. Fragment of a Quercus sp. leaf impression, K80 (Vrabac S., and Đulović I. 2018)

As part of detailed geological investigations of the Kotezi coal deposit in the Bugojno coal-bearing basin, micropaleontological investigations were also carried out (Vrabac and Đulović, 2018). In the material from the borehole K80, at the interval 296.8-297.0 m, damaged shells of the gastropod Valvata cf. abdita Brusina (Figure 8) with a size of 0.7 mm [15] are found.

Figure 8. Shell of the gastropod Valvata cf. abdita Brusina (Vrabac S., and Đulović I. 2018)

Based on the paleontological remains of molluscs and ostracods from the coal-bearing deposits of the Kotezi region, which were determined during the research in 2017 and 2018, it is not possible to precisely define the stratigraphic affiliation of the coal-bearing sediments. Considering that the leading fossils, which belong to proboscideans and rodents, indicate the Middle Miocene and Upper Miocene age of the coal-bearing sediments of the Bugojno basin, it is justified to include the investigated coal-bearing sediments of this area in the Middle and Upper Miocene ($M_{2,3}$) [14,15].

The thickness of this lithostratigraphic unit is significant and tends to increase from the western part of the basin towards the center of the basin. In the western part of the basin, it reaches from 137 m (borehole BP-2, locality Guvna) to 247.95 meters (borehole B-18, locality Kotezi). Its lithological composition has significant practical value for suspending boreholes on the main coal seam. The thickness of the lithostratigraphic unit ranges up to 250 m.

Main coal layer ($^6M_{2,3}$)

The main coal layer in the northwestern part of the Bugojno basin is located on layers of sandy marls, marls or clayey limestones (Figure 9). The immediate roof consists of sandy clays, which in places have the characteristics of plastic, greasy clays. They are most often contaminated with heterogeneous, slightly rounded or angular unsorted roof deposits and, less often, carbonized fragments of flora.

During the geological research until 1984, the main coal layer was determined in some boreholes as roof, floor, etc. The dual interpretation of the same coal layer created difficulties in programming the execution and suspension of further exploration works, until its extent and spatial position in the deposit were determined.

According to the results of geological research conducted in 1985, this layer was determined as the main coal layer in the northwestern part of the Bugojno basin [16]. Based on the correlation with the rest of the Bugojno basin, this layer was determined as the main coal layer ($^{6}M_{2,3}$).

The greatest thickness of the main coal layer with barren interlayers was found at borehole B-26 (locality Kotezi)and it is 68.6 m. The thickness of clean coal is 44.55 m [19]. The average thickness of the main coal layer is 24.8 m, while the average thickness of pure coal in the layer is 19.82 m.

The main layer contains interlayers which are represented by clays, marls and rarely clayey limestone. In the lower part of the layer, the barren parts consist mainly of marls, clayey limestones and carbonaceous marls. The upper part of the layer is most often intercalated with clayey matter and carbonaceous marls.

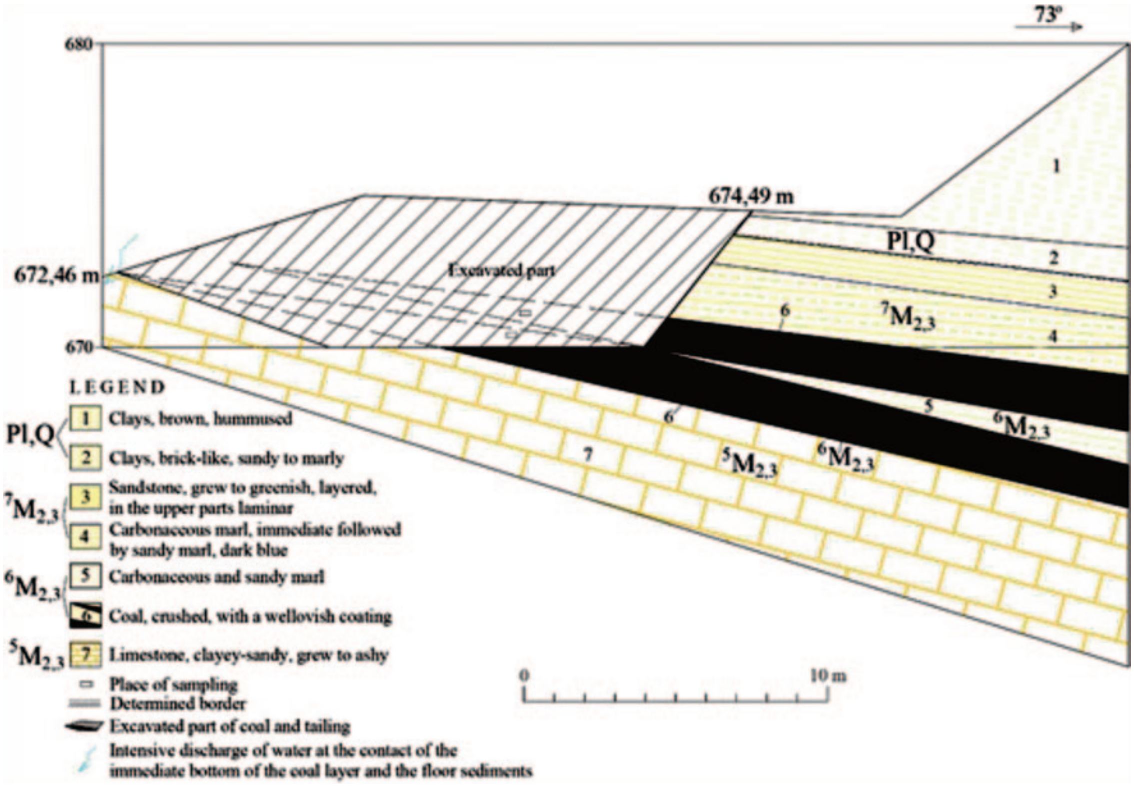


Figure 9. Geological open pit R11, above the Poriče (Forčaković Dž. 2018)

The distance between the main and first floor coal layers is between 8.6 m (exploratory borehole B-2) and 220.5 m (exploratory borehole K80). Petrographic composition includes clayey limestones, gray-brown to whitish-gray in color, poriferous fossil-bearing clayey limestone, marls and sandy marls and rarer clays, as well as weakly bound, clayey, terrigenous sediments and coal interlayers. According to the data, the thickness of the pure coal, i.e. the stratification of the layer, is greater north of the Spahinački fault, which cuts the deposit transversely in the west-southwest-east-northeast direction (WSW-ENE).

In the central and western part of the deposit on the southern fault block of the Spahinački fault, the thickness of coal is greater. Going towards the Vrbas-Voljevac fault, it decreases again, and along the Poriča fault on the uplifted northern block, the main coal layer is distinguished by intense stratification, and the outcrops give the impression that it has no economic value. However, it should be borne in mind that the coal layers are largely burnt when they come into contact with air and moisture, due to their flammability. This is confirmed by the remains of the burned part - which stands out for its brick-red color. Going to the deeper parts of the deposit, the thickness, purity and heat value of the coal give it the main economic importance. Favorable geological factors which show that in certain parts of the deposit the coal of the main coal layer will be able to be profitably exploited by the surface method adequately increase its economic importance.

The following fauna was found in the main coal seam: *Prososthenia* sp., *Sphaerium* sp., *Sphaerium ožegovići* Brusina, *Pisidium* sp., *Orygoceras* sp., *Planorbis pulići* Brusina, fish teeth and bones, *Congeria zoisi* Brusina, *Melanopsis* sp., *Valvata* cf. abdita Brusina, *Planorbis* sp., *Limnea* sp., *Chara* cf. *escheri* Unger, *Oogonije Chara,* Candona sp. [7,10,20,21].

According to the total determined subfossil remains and the finding of Middle and Upper Miocene forms: Betula, Tilia, Ulmus, Salix, etc., it can be concluded that the main coal layer and floor layers were created in the Middle and Upper Miocene. The thickness of this lithostratigraphic unit (6M2,3) varies up to 68 m thick.

Unit of clays and sandy clays ($^7M_{2,3}$)

This lithostratigraphic unit is composed of grey, brown and yellow-brown clays which are mostly plastic, greasy, and in places are sandy and fine gravelly or coaly. Sometimes these clays alternate laterally with sandy marl or it is a sandy clay.

The immediate roof of the main coal layer is almost always composed of thinner deposits (0.1–7 m) of greyish, quite crumbling clayey limestone with a lot of faunal remains (gastropodes). These deposits are often sandy, with quartz and chert as admixtures. Over these deposits comes a much thicker series of marls with a higher or lower content of $CaCO_3$ and terrigenous matter.

The following fauna was found in the roof of the main coal layer: *Congeria friči Brusina, Melanopsis cognata* Brusina, *Melania verbasensis* Neum, *Lytosthoma grammica* Brusina, *Sphaerium sp., Sphaerium ožegovići* Brusina, *Pisidium* sp., *Dreissena* sp., *Congeria zoisi* Brusina, *Congeria dalmatica* Brusina, Melanopsis pygmaea Hornes, *Stenothyra stenostoma* Brusina, *Lithoglyphus* sp., *Orygoceras* sp., *Prososthenia eburnea* Brusina, *Limnea* sp., *Planorbis manteli, Valvata* sp., *Prososthenia* sp., *Planorbis* sp., *Lithoglyphus*? panicum Neum, *Lithoglyphus* cf. *triapoli* Brusina, *Candona (Pseudocandona)* sp., *Cypris* sp. [9,20-24].

Based on the available age data, this unit is assigned a Middle and Upper Miocene age ($^7M_{2,3}$). It is considered that Pliocene sediments were deposited upwards from this lithostratigraphic unit. Thickness of this lithostratigraphic unit ($^7M_{2,3}$) reaches 45 m.

Pliocene-Quaternary sediments (Pl,Q)

The deepening of the lake followed the deposition of plant material from which the main coal layer will be formed. It is followed by the clay unit ($^7M_{2,3}$), as a coal immediate roof. Then begins the cycle of deposition of fine to coarse-grained sediments, which represent the final part of lake sedimentation. The complex of loosely bound and unbound lithological members of the Pliocene period consists of: clays, brown, yellow-brown, reddish, greenish and gray-blue colors siltstone, sandy and gravelly in all granulations, often unsorted, sometimes calcitic and passing into sandy marls, then sandy to gravelly calcitic clays, sandy polymictic breccias, limestone and heterogeneous conglomerates that are weakly bound by a clay binder. Beside limestones and quartz diorites in the composition of heterogeneous conglomerates, there are also dolomites, Verfen sandstones and Permotriassic sericitic slates and siltstones. Composition of the Pliocene sediments also includes loosely bound sandstones that are fine-grained to coarse-grained, often unsorted and silty, then silty gravel, siltstones, clayey-dusty sand. Interlayers of coal and carbonaceous clay are present in places. The highest (youngest) parts of the complex of terrigenous sediments, according to current knowledge, belong to the Quaternary period.

Roof coal layer

According to its superpositional position, it is estimated that the deposition of Pliocene sediments ends with the sedimentation of the roof layer and its immediate roof. Roof coal layer was isolated in a relatively small area, between the settlements of Karadže, Gaj Berić and Bugojno. Its dimensions are 1.3 km along the strike direction and 1.2 km along the dip direction [1]. In the current phase of research, and in the absence of fossil remains, based on the superpositional position of this coal layer, it is considered to be Pliocene age [7].

Roof coal layer was drilled with a total of 5 boreholes. The thickness of roof layer based on data from boreholes BŽ-21, B-11, B-34 and K128 is 12.17 m. The thickness of pure coal is 7.7 m. It is characterized by intense stratification. Interlayer tailings consist of weakly hardened ash-gray sandy marls, followed by carbonaceous and semi-plastic clays of dark gray, brown and whitish-gray colors. Njegovu neposrednu podinu izgrađuju poluplastične do sitnošljunkovite kalcitične gline sivo- Its immediate floor is made up of semi-plastic to fine-grained calcitic clays of gray-whitish and brown or dark gray color, and the immediate roof is made of sandy to fine-grained calcitic clays of gray-white color and brown fine- to medium-grained sandy clays that have the characteristics of good brick clay.

The distance between the roof and the main coal layer is between 221.15 m (exploration borehole B-11) and 258.78 m (borehole B-34). Petrographic composition of these sediments includes gray, brown, and yellow-brown clays that are mostly plastic, greasy, and in places sandy and fine gravelly or carbonaceous. These clays sometimes alternate laterally with sandy marl or it is calcitic-sandy clay, loosely bound sandstones which are fine-grained to coarse-grained, often unsorted and clayey, then clayey gravels, siltstones and clayey-dusty sand.

Thickness of the Plio-Quaternary sediments in the deeper parts of the basin reaches up to 336.5 m (borehole B-9, locality Udurlije).

Quaternary (Q)

Due to the neotectonic uplift of the Bugojno basin, lake waters receded. After the long Neogene lake phase of deposition, a spacious river basin and fields remained.

During the Quaternary period in the area of the Bugojno basin, the process of raising the terrain and retreating of the lake waters took place more slowly, so the lake sedimentation regime continued through the Pleistocene. Quaternary sediments appear along Vrbas, riverbeds, small streams and larger streams, then at the foot of steep slopes, etc. Terrigenous unbound or semi-bound rocks (clays, silty-clay sands, crumbling and silty sandstones, silty conglomerates and breccias, and unsorted gravel deposited by the arrival of occasional torrential flows into lake waters) could not be separated from similar depositional formations in the Pliocene. Considering that the lower border of Quaternary lithological members on the wider surface of the research area and lake deposits is not defined, they are defined as Pliocene-Quaternary sediments (Pl,Q). In the other isolated Quaternary formations, the following facies are represented: river terrace sediments (t_1, t_2 and t_3), deluvial facies (d), proluvial facies (pr), floodplain facies (ap) and alluvium (al). Lithological composition of these facies is variable and depends on the geological structure of the sub-strate, as well as the age and way of creation [17].

Quality

Quality of the coal layers of the coal deposit Kotezi (Prusac-Guvna-Kotezi-Karalinka-Gaj Berića) was determined by laboratory and industrial tests (Table 1) performed within the framework of geological research [10-12,25,26].

Table 1. The average values of immediate analyzes of the coal layers of coal deposit Kotezi

Components	Coal layers			
	Prusac-Guvna-Kotezi-Karalinka-Gaj Berića			
	II bottom	I bottom	Main	Roof
Air-dry moisture [%]	24,99	22,37	28,11	23,95
Hygro moisture [%]	6,67	6,06	10,12	6,6
Total moisture [%]	31,66	28,43	38,23	30,55
Ash [%]	23,6	19,52	18,18	20,49
Volatile substances [%]	26,07	30,94	26,65	30,77
Combustible substances [%]	44,74	51,73	46,03	49,19
C-fix [%]	18,70	20,77	16,34	18,41
Coke [%]	42,27	40,62	33,91	38,67
Combustible sulfur [%]	1,45	2,19	1,0	0,48
Bound sulfur [%]	2,32	1,87	1,26	3,20
Total sulfur [%]	3,77	4,06	2,26	3,68
Upper calorific value [kJ/kg]	13.000	12.700	11.000	12.800
Lower calorific value [kJ/kg]	11.300	11.500	10.000	11.500

Reserves

According to the complexity of the geological structure and the degree of tectonic disorder, the Kotezi coal deposit is classified in the II group, and according to the variability of the coal layers (morphology, thickness and quality) in the II subgroup [10,28]. Based on the results of regional and detailed geological research, the coal reserves of the Kotezi deposit were calculated and confirmed final with the date 07/31/2018 (Table 2).

Table 2. Overview of coal reserves by layers of the Kotezi deposit

[10^6 t]

Deposit	II bottom			I bottom			Main							*Roof			
Kotezi	A	B	C_1	A	B	C_1	A	B	B_{vb}	C_1	C_{1vb}	C_2	D_1	A	B	C_1	C_2
Total	-	-	9,3	-	-	16,8	35	59,5	73,8	1,57	48,6	104	12,5	-	-	-	10,7

*Contoured only in the area Karadže-Gaj Berića-Gradina

Through the synthesis of relevant data, the contoured areas with the balance coal reserves of the main coal layer in the amount of over 90 million tons were determined. Based on the projected producti^{o}n of about 300 x 10^3 tons of coal with predicted exploitation losses of 10%, the established reserves can ensure continuous production for several decades.

4. CONCLUSION

In the area of the Kotezi, the Middle and Upper Miocene were separated, which is divided into seven lithostratigraphic units: basal ($^{1}M_{2,3}$), the second bottom coal layer ($^{2}M_{2,3}$), clay, clayey sandstones and marls ($^{3}M_{2,3}$), first bottom coal layer ($^{4}M_{2,3}$), marly limestones and marls ($^{5}M_{2,3}$), main coal layer ($^{6}M_{2,3}$), clays and sandy clays ($^{7}M_{2,3}$). Miocene is followed by Pliocene-Quaternary (Pl,Q) and Quaternary (Q) sediments.

The deposit contains four coal layers of different economic importance: II floor, I floor, main and roof.

Mining district Kotezi contains very significant but insufficiently explored lignite reserves. It should be borne in mind that only shallower parts of the deposit have been explored, so it is justified to predict significantly higher reserves in the deposit.

Coal deposit Kotezi is the most significant deposit in the Bugojno basin, which is very important due to the continued exploitation of coal from the main coal layer.

Considering the raw material potential and the possibility of expanding the existing raw material base, the Bugojno coal basin has particular importance for the long-term development of lignite exploitation and its use for thermal energy purposes.

Determ$_{i}$ned content of $CaCO_3$ in limestones ($^{5}M_{2,3}$) is very high (93-98%), which indicates the possibility of their application in the paper and rubber industry. The use of limestone for these purposes must be verified by additional laboratory and industrial tests.

Lithological composition of the clayey limestone and marls (unit $^{5}M_{2,3}$) has significant practical value for stopping boreholes on the main coal layer.

Based on the paleontological remains of molluscs and ostracods from the coal-bearing deposits of the Kotezi region, which were determined during the research in 2017 and 2018, it is not possible to precisely define the stratigraphic affiliation of the coal-bearing sediments. Considering that the leading fossils, which belong to proboscideans and rodents, indicate the Middle Miocene and Upper Miocene age of the coal-bearing sediments of the Bugojno basin, it is justified to include the investigated coal-bearing sediments of this area in the Middle, Upper Miocene ($M_{2,3}$).

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SAFETY RISK MANAGEMENT IN BIH COAL MINES – OBJECTIVE NEED OR A BURDEN

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SUMMARY

Safety risk management in Bosnian-Herzegovinian underground coal mines is not based on a systematic approach and standardized risk management methods. Mostly there is a traditional approach present, i.e. reaction to incidents/accidents that happen. This paper aims to point out the need for proactive approach introduction with an emphasis on importance of ventilation systems failure mechanism simulation analysis in planning of people and property defense and rescue in crisis situations. Based on ventilation parameters measuring results for a given research site (brown coal mine), a model was developed on which a simulation analysis was performed for three characteristic ventilation failure cases: spontaneous oxidation at the longwall exit, collapse at the entrance to the longwall and methane outburst - preparation of a new excavation field. VnetPC and CFD "Fluent" software packages were used to simulate mine ventilation.

Keywords: risk management, brown coal mine, safety, ventilation system, longwall, simulation analysis, VnetPC, Fluent.

INTRODUCTION

The principle of "normative safety" is dominant in Bosnia and Herzegovina's (BiH) coal mines, with frequent and common basic principles and obligations neglection, a routine approach to safety and efforts to meet the law, regulations and technical norms requirements. The scope of consequences and probability of certain dangerous conditions as well as their treatment in accordance with risk management standards is not considered.

The Law on Mining Industry of the FBiH obliges mines to develop a Defense and rescue plan for collective risks and accidents, which must be adjusted to the specific circumstances and specifics of the mine, updated in a timely manner, provide all preventive and operational measures and activities related to potential hazards.

All decisions envisaged by the mine Defense plans mainly depend on the "persons responsible experience" and possible information from the endangered part of the mine, provided that communication with workers is not interrupted and that the gases and ventilation parameters remote control system is not damaged or interrupted.

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The current safety concept and technical solutions for protection in underground coal mines in BiH do not provide adequate protection of human life and health in isolated areas in case of mining accidents, and even during some events that do not last long and after which production can be re-established.

The mine defense plans analysis has shown that crisis management conventional methods in coal mines in case of components or ventilation system as a whole failure do not provide an adequate level of workers life and health safety [6].

In modern risk management theory, adverse events domain extends to so-called "missed opportunities", thus the risk management process goes beyond the traditional safety framework, and practically encompasses the entire decision-making and management process [3] [7]. Everything that has not been done in a favorable and feasible way in given conditions becomes risky.

The new risk management concept is a significant shift from the current system of "normative safety", according to which accidents in mines that are not the result of violating the regulations or non-compliance with prescribed standards are considered "force majeure". Risk management implies the identification of the "risk owner" and his obligation to take all objectively possible measures to maintain risks at the accepted level, i.e. to prevent an adverse event.

2. VENTILATION SYSTEM FUNCTIONING HAZARDS

Underground coal mines are complex systems in technical and technological terms, especially in terms of ventilation. Mine ventilation system reliability during its exploitation period implies the ability to preserve designed ventilation parameters that ensure prescribed ventilation regime and air properties (gas, thermodynamic and climatic relations) in mining premises.

Numerous models for mine ventilation systems risk assessment have been developed in the world, while in our country there are no criteria or methods for risk parameters calculation. Current mining regulations in BiH treat the ventilation systems reliability according to a simplified procedure by determining limit values of the equivalent orifice, fan pressure potential and differences in the potential of the ventilation system that must meet prescribed limit requirements for normative stability of ventilation.

Factors with the greatest impact on mine ventilation system reliability and ventilation risks are [2]:

- aerodynamic resistances of the ventilation system elements,
- fan operation reliability and ventilation facilities functioning,
- network topology (organizational schematics, system organization),
- occurrence of sudden threats such as explosions, mine fires, rock burst, gases and materials outbursts, collapse,
- ventilation facilities maintenance planning and implementation,
- conditions and methods of exploitation,
- initial data in calculations (accuracy) and
- methods of applied ventilation systems calculations.

An adverse event of a mine ventilation system means a state of functioning capacity loss, violation of the designed or prescribed regime, or a state in which the ventilation facility (process) or some part of it partially or completely loses its working capacity.

The ventilation system always works with a certain risk, and the occurrence of ventilation system failures in mines cannot be ruled out even when extensive prevention measures are taken. For efficient protection against unexpected events in coal mine, it is important to predict possible ventilation conditions and the development of individual failures to effectively manage risks. In case of severe failures, key information is usually missing or the reliability of awareness about processes and phenomena is limited. Rescuers enter the mine with unclear picture of the situation. Experience with coal mining accidents shows that the existing communication systems in mines are not reliable enough, and that one of the first challenges is to establish a connection with endangered workers.

In recent years, many published scientific and professional papers have been dealing with numerical simulations of ventilation system failures, and they have also been used for major mining accidents analysis.

Research by Greuer R.E. [5] indicates the physical principles and principles of mine ventilation systems operation as well as the skills of ventilation modeling. The paper then describes several computer programs for modeling the interaction of mine ventilation and fire. Yuan and Smith [10] [11] conducted research on spontaneous heating, by CFD numerical modeling, investigating spontaneous heating in old workings.

Papers [4] and [8] present a case of the practical CFD methods application in mining accidents research, and provide a complete experiences analysis. The paper [1] analyzes local increase in volume flows and flow velocities in a large profile room (longwall) by controlled recirculation, and the impact of this process on explosive and toxic gases concentrations in mine workings.

3. EXPERIMENTAL RESEARCH METHODOLOGY

Underground coal mines are a specific and very complex problem for analyzing the reliability of systems required to ensure working and living conditions due to long distances and depths, long ventilation paths, the presence of methane and other explosive and flammable gases and substances, sudden occurrence of toxic gases, harmful coal dust and a number of other detrimental effects on workers.

The selected research site (mine B) has a pronounced stochastic nature of the process characterized by specific technological and natural conditions related to gas, gases dynamics, tectonics, hydrogeological conditions, etc. that cause unpredictable hazards.

With the aim to develop a technical model for predicting and preventing adverse effects of the ventilation system (or its components) failure, a series of "in-situ" and laboratory measurements were conducted within research, as follows:

- effective fan pressure (depression),
- air flow rates,
- air and rock mass temperatures,
- coal tendency to spontaneous combustion testing,
- testing dust flammable and explosive characteristics.
- volumetric air flow,
- aerodynamic resistance,
- air chemical composition analysis,
- monitoring the occurrence of oxidation processes (analysis of fire indicators)

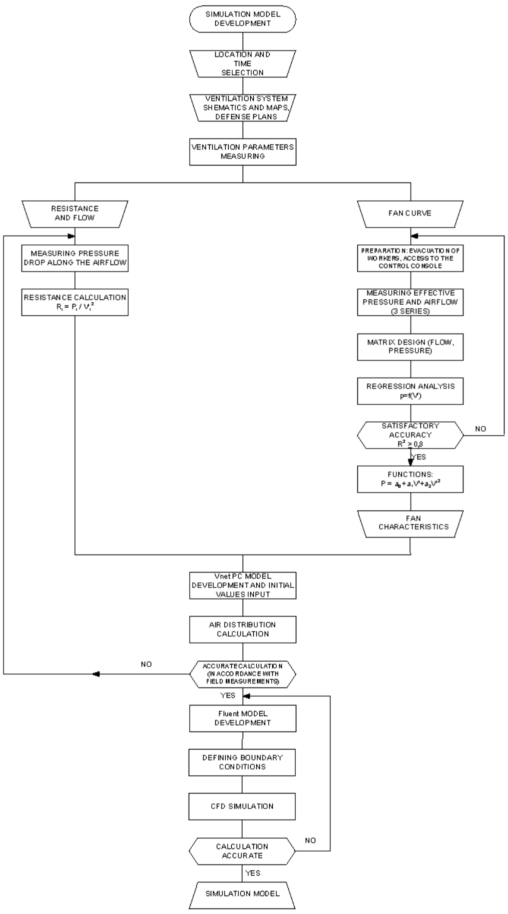

Figure 1. Schematic presentation of "in situ" ventilation parameters measuring plan and simulation models development [6]

Based on the obtained research results, a model was developed for computer simulation of contaminant distribution processes and changes in ventilation parameters with Hardy-Cross method using VnetPC software package and CFD software package „Fluent", which is illustrated by the flow diagram presented in Figure 1.

4. COMPUTER SIMULATION OF VENTILATION SYSTEM FAILURE IN COAL MINE B

Figure 2 shows a linear scheme of underground Mine B ventilation system with two main inlet air flow rooms and one outlet to the main ventilation heading inclined roadway.

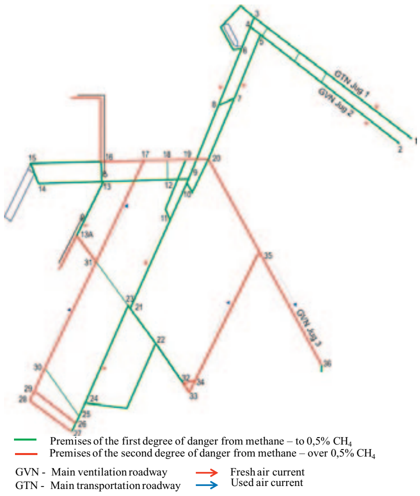

Figure 2. Mine B linear ventilation layout [source: mine documentation]

Working with three active longwalls in a thick coal seam prone to spontaneous oxidation imposes the necessity of analyzing the process within the system itself, which is the "heart" of both the production system and mine ventilation.

For the combined analysis, a ventilation compartment with two longwall excavations was selected, as shown in the normal operating mode in Figure 3.

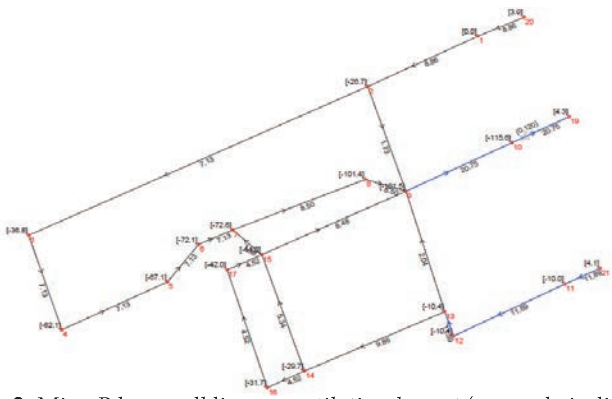

Figure 3. Mine B longwall linear ventilation layout (normal air distribution) [6]

In the normal state of ventilation, and at the time when three longwalls are active, as shown in Figure 3, two ventilation compartments (VO) can be observed: in one of them there is a mechanized longwall excavation, and in the other one there are two classic longwalls. As can be seen from Figure 3, the two VOs have a joint outlet air flow so that the total outlet air flow has a volume flow of 20.75 [m^3/s]. The inlet air current in the northern part of the excavation has a volume flow of 8.86 [m^3/s], and 7.13 [m^3/s] goes towards the longwall itself. Classic longwall excavations are ventilated with a total of 11.89 [m^3/s] of air.

In order to consider potential consequences of characteristic and possible failures on the entire ventilation system functioning, a computer simulations of characteristic cases were performed on a previously calibrated model for ventilation calculation, which is presented in the following subsections:

1. Spontaneous oxidation at the longwall outlet where 300 [ppm] CO is released;
2. Collapse at the inlet to the longwall (inlet air current) where the flow is reduced to 20% of the initial value;
3. Methane outburst - preparation of a new excavation field longwall, simulated methane concentration 25%.

The simulation analysis have determined ventilation branches that failed in case of simulated danger. The assessment of the contaminants distribution and their intensity determined zones where workers and the production process are endangered.

The Mine defense plan analysis for simulated failures have shown that the personal protective equipment provided by the mine to workers is not adequate due to the limited protection level and that it is necessary to plan secondary protection measures for most possible failures, Figure 9.

Simulation analysis have also indicated a high probability of the secondary failures occurrence which are not treated by the Defense plan, and protection measures are not provided for them.

4.1. SPONTANEOUS OXIDATION AT THE LONGWALL EXIT WITH 300 PPM CO RELEASED

The oxidation process at the longwall outlet was simulated, with a concentration of 300 [ppm] of carbon monoxide registered at the oxidation site, as illustrated in Figure 4.

Immediately when the air is connected to the air current from the first classical longwall (point 9), the CO concentration decreases by 60%, and only the rooms of the outlet air current are contaminated.

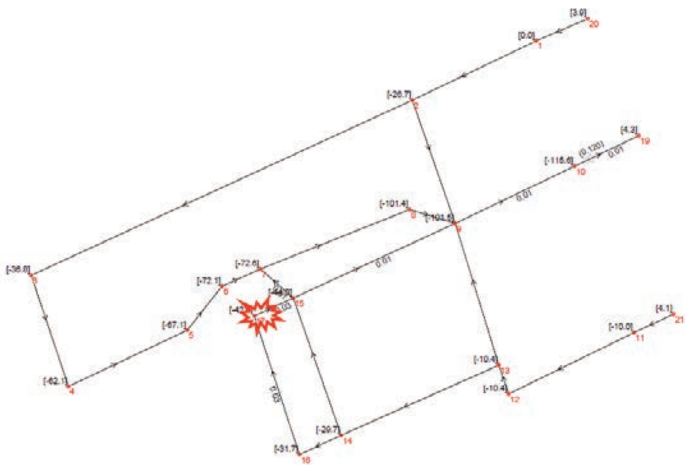

Figure 4. Simulation of spontaneous oxidation [6]

The increase in carbon monoxide concentrations is evident in branches 16-17 and 17-15, i.e. in the longwall rooms where oxidation occurs and the connecting roadway of the exit room from that longwall to the place where it connects with the air flow from the second ventilation compartment longwall.

From the aspect of "failure" and workers endangerment in this phase, we can talk about the endangerment of workers in rooms of the outlet air current. Since the outlet air currents in this mine are used for the materials transport and other types of "auxiliary transport", which is very pronounced in this case, the failure of this type can endanger workers and the process in those rooms. Long ventilation paths, slopes, unfavorable conditions for movement and possible workers disorientation can increase evacuation time out of the scope of protection with personal protective equipment, and it is necessary to consider other protection strategies to reliably save lives from short-term and sudden increases in CO and other oxidation process products.

The defense and rescue plan for collective risks and accidents envisages this danger as part of the group: "plan for protection against fire, explosion and gases" (Figure 5).

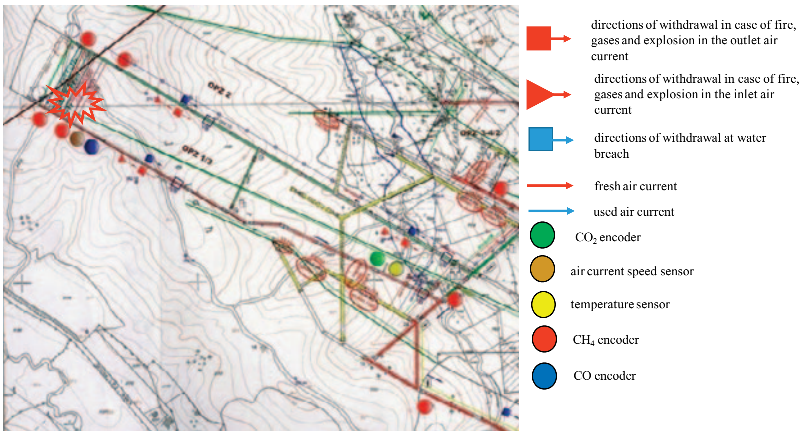

Figure 5. Detail from the Defense plan for simulated hazard [source: mine documentation]

The plan for this group of dangers also predicts the air flow reversion. The decision on reversion is made by the managers based on the situation "experientially". Recommendation for the Defense plan improvement is that such decision-making is preceded by simulating the various possible scenarios and measurements, and detailed, clear and precise prescribing of procedural steps. Air flow reversion can result in contamination of other sites in coal mine and serious endangerment of workers if they are not evacuated in time. This danger cannot be ruled out, given that the Defense plan does not consider such secondary ventilation failures.

4.2. COLLAPSE AT THE LONGWALL ENTRANCE (INLET AIR CURRENT), VOLUMETRIC AIR FLOW REDUCED TO 20% OF INITIAL VALUE

Figure 6 shows simulation of collapse at the longwall entrance.

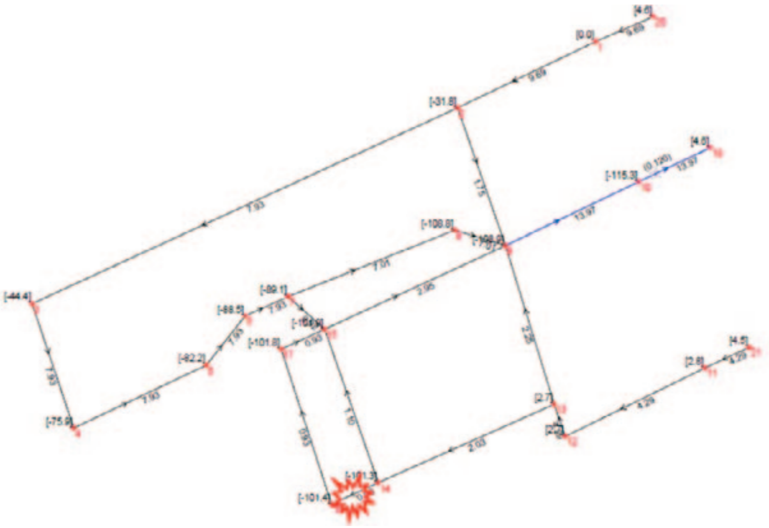

Figure 6. Simulation of collapse at the entrance to the longwall [6]

The collapse or reduction of the effective room profile in any part can have a great impact on ventilation. Even when stacking large quantities of materials or timber, the profile may decrease, resistance may increase and ventilation may be compromised.

Table 1 shows simulation results for the case when the entrance room profile is reduced to 20% of effective compared to the nominal calculated.

The combination of reduced room profile due to pressure or poor maintenance, roof support damage, installation of transport and other equipment and stacking materials in room can, in conditions that seem normal from the aspect of all processes functioning, endanger workers lives. If there is oxidation hotspot in rooms, the situation is further complicated.

Table 1. Changes in airflow by branches during stimulation of collapse at the longwall

From	To	Flow -normal conditions (m^3/s)	Flow during the collapse at longwall (m^3/s)	Flow difference, %
1	2	8,86	9,69	9,37%
2	3	7,13	7,93	11,22%
3	4	7,13	7,93	11,22%
4	5	7,13	7,93	11,22%
5	6	7,13	7,93	11,22%
6	7	7,13	7,93	11,22%
7	8	8,50	7,01	-17,53%
8	9	8,50	7,01	-17,53%
9	10	20,75	13,97	-32,67%
11	12	11,89	4,29	-63,92%
12	13	11,89	4,29	-63,92%
13	14	9,86	2,03	-79,41%
14	15	5,34	1,10	-79,40%
15	9	8,48	2,95	-65,21%
14	16	4,52	0,93	-79,42%
16	17	4,52	0,93	-79,42%
17	15	4,52	0,93	-79,42%
13	9	2,04	2,25	10,29%
9	2	1,73	1,75	1,16%
7	15	1,38	0,92	-33,33%
10	19	20,75	13,97	-32,67%
20	1	8,86	9,69	9,37%
21	11	11,89	4,29	-63,92%

The red color in Table 1 indicates branches where the air flow volume is reduced. In some branches, an increase in volume flow is observed, which is caused by the air reduction in branches marked in red.

As a particularly serious dimension of this problem, it is necessary to investigate the possibility for such a disturbance to occur without being registered by remote control system or not being registered until the moment when a more serious disturbance of gas relations occurs. It is difficult to install and maintain remote control sensors in the rooms immediately around the longwall, and regular measurements and controls at ventilation inspection stations are outside the affected area because standard ventilation inspection stations are installed at the beginning of the excavation fields.

The defense plan envisages measures for this danger, which are reduced to providing the air through perforated pipes, and then removing the rubble.

This type of disturbance needs to be treated more carefully in defense and rescue plans, especially in terms of reliable detection of any changes in resistances and flows in rooms that are most important in the ventilation system: those that make up the production system, always occupied with many workers and which are exposed to the highest heat and gas loads.

4.3. METHANE OUTBURST – PREPARATION OF A NEW EXCAVATION FIELD LONGWALL, SIMULATED METHANE CONCENTRATION 25%

In the case of methane outburst in preparation of a new excavatiᵒn field for a longwall with 1.78 [m^{3}CH$_4$/s] (Figure 7), the working space and the return airways are gas filled.

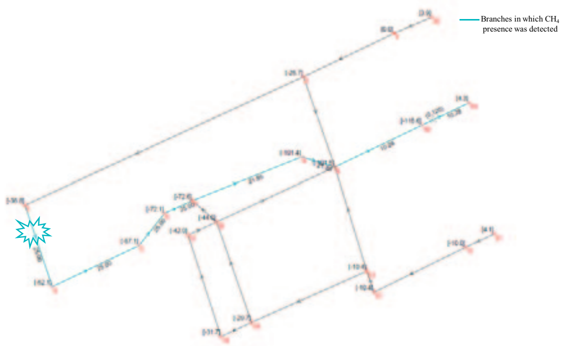

Figure 7. Simulation of methane outburst at the longwall [6]

Flammable or explosive methane concentrations are present throughout the outlet air current, which implies why it is necessary to carefully plan the exclusion of potential ignition sources. The immanent presence of oxidative processes on the open coal seams surface, as well as along the air leakage, results in possibility of methane ignition, in cases where a outburst would occur.

In such accidents, the life danger lasts relatively short, and the potential evacuation directions are not contaminated, so it is necessary to ensure conditions for survival only until the workers evacuation to the surface in usual directions.

Figure 7 shows the part of the mine where the methane outburst at a concentration of 25% was simulated. Table 2 shows methane concentrations for simulated hazard from which it is evident that flammable or explosive concentrations are present in all branches.

Table 2. Methane concentration by branches during the outburst in longwall

Branch	Air flow - normal conditions [m^3/s]	CH_4 concentrations [%]
3-4	7,13	25
4-5	7,13	25
5-6	7,13	25
6-7	7,13	25
7-8	8,50	21,85
8-9	8,50	21,85
9-10	20,75	10,28
10-19	20,75	10,28

Methane outburst is a serious situation that occurs in (methane) gassy mines, and is manifested by mining operations in the area with high methane pressure in the coal seam, roof, floor or by a crack system connected to a remote methane "reservoir" under pressure. It is usually a "point" source of methane, which can create dangerous concentrations in local conditions, even when it comes to small amounts of this gas. In the Defense plan, this danger was treated in the group "plan for fire protection, explosion, and the gases occurrence" with a general approach and measures for protection and rescue.

In the case of methane presence, action strategies must include three dimensions:

1. Early detection of dangerous concentrations and exclusion of possibilities of gas ignition by works termination and workers evacuation. If there is spontaneous oxidation or smoldering, it can be a source of ignition, so the scope of this measure is limited. In the event of a methane explosion, the mechanical effect of the explosion and its consequences can be serious for workers in the immediate vicinity.
2. The methane explosion can create conditions for the propagation of coal dust explosion, which is one of the explosions with the most severe consequences in mining history. It is necessary to pay special attention to this possibility.
3. Explosion products can endanger workers from the place of origin to the mine exit.

4.4. CFD SIMULATION OF RESEARCH POLYGON VENTILATION PARAMETERS

Although previous Hardy-Cross-based simulations did not identify complex cases of ventilation risks, in case of three large-profile longwalls in a relatively small space, a much more extensive and objective view is provided by CFD simulation based on the "finite volume" method.

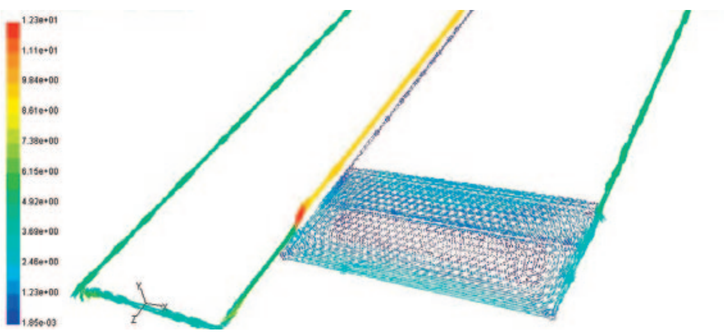

Figure 8. Air velocity vectors [m/s] [6]

Figure 8 shows three analyzed longwalls, where space is discretized on model, as well as the interaction with the old workings and the surrounding rocks. Arrows and different colors show air velocity vectors inside the rooms.

The air flow inside the two classic longwalls on the right side of the image can be seen as particularly complex in terms of ventilation. Velocity vectors indicate air circulation and leak, which is why in the case of increased cracking, the presence of faults or excavations below previously exploited levels, a spontaneous oxidation can be expected.

5. DISCUSSION OF RESEARCH FINDINGS

Figure 9 shows four typical cases of protection and rescue measures, depending on duration of ventilation disorder due to accident, and the safety time provided by each of the 3 levels of workers protection.

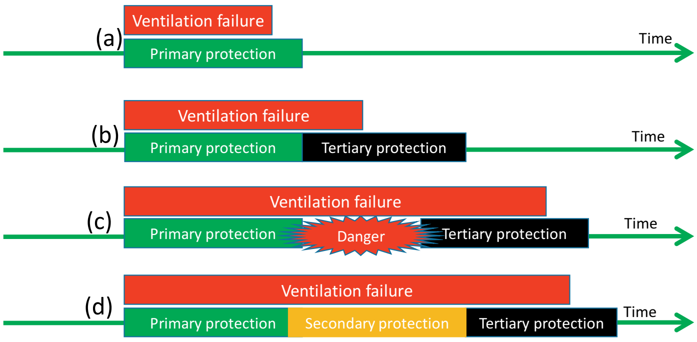

Figure 9. Measures for people protection and rescue from the consequences of ventilation failure

In case (a), primary protection, as it is by regulations in mines in BiH, provides sufficient time to evacuate or otherwise rescue vulnerable workers, when a secondary or tertiary level of protection is not required. When a failure occurs, as in case (b), tertiary protection is necessary, which in combination with primary protection is sufficient and adequate for rescuing endangered miners.

The case under (c) is characterized by a longer period of workers vulnerability, so that primary protection does not provide sufficient time for survival, and tertiary protection cannot provide conditions for evacuation quickly enough. In this case, there is a "time of uncertainty" which is characteristic of cases of trapping and isolation of workers in the mine workings.

Based on experiments and simulations, it has been concluded that in real conditions, probable cases illustrated by the situation "c" from the previous figure are possible.

Primary protection systems are used in coal mines in BiH, and in case of accidents, tertiary protection is applied. The lack of a "second level" - secondary protection - results in frequent "battles with time" and uncertainty about the fate of trapped miners underground. The air needed for survival is not the only thing that needs to be provided in secondary protection measures. It is necessary to solve problem of supplying the drinking water and food, as well as communication and lighting in case of prolonged ventilation interruption.

6. CONCLUDING REMARKS

A detailed review of actual safety concept and technical solutions for protection in underground coal mines in BiH has revealed an objective need to move off the dominant principle of "normative safety" to a proactive approach. In order to create necessary preconditions for successful and efficient safety risks management, it is essential to start with immediate regulation and legislation implementation in accordance with the modern approach and practice of countries with developed mining industry.

Major accidents in mines usually do not occur solely as a result of "force majeure", but result from unwanted and unsafe conditions, and often leads to a numerous induced hazardous conditions. The distinctly stochastic nature of the underground coal mining process, where it is practically impossible to predict all aspects, is a serious problem that must be given more importance and attention. Mining operations take place in environments that are highly heterogeneous and potential sources of danger can vary widely within a relatively small area. This implies that it is necessary to use different risk control methods and strive to manage them adequately.

Practice confirms that a proactive approach to risk management reduces incidence of adverse events and reduces consequences of potentially tragic incidents in underground coal mining production systems. Contrary to the normative approach, when only strictly enforced regulations are expected from the responsible authorities, it is necessary to impose an obligation on mining companies to treat risks as situations that occur, to record them, make a register, but also adopt procedures for their monitoring.

The results of experimental research and conducted simulation analysis of failures clearly indicated that hazard identification, assessment and risk management of underground ventilation systems by analyzing the mechanism of primary and secondary failures of elements and systems are a necessary prerequisites for defining workers exposure to hazardous conditions in terms of time, type and intensity of contaminants during the state of failure in working environment and rescue and evacuation planning.

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INFLUENCE OF VARIOUS LONGWALL FACE VENTILATION REGIMES ON SPONTANEOUS MINE FIRE OCCURENCES AND INCREASED GAS RELEASE IN „RASPOTOČJE" MINE OF ZENICA BROWN COAL MINES

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SUMMARY

Brown coal mining conducted at greater depths, characterised with severe mining-geological conditions often results in deviation from the applied mining methods. Experiences gained in mechanised longwall mining of roof plate in deeper parts of Raspotočje mine, using method of roof caving without safety pillars left behind the working panels indicates to shortcomings of the applying mining method. Frequent occurences of spontaneous mine fires, of hazardous gasses well above the limit values, and constant exposure to rock bursts of various intensity resulted in application of three longwall face ventilation regimes: conventional „U" ventilation system, ventilation with gas channel and middle hallway, and separate ventilation of the upper part of the longwall face. This paper presents experience based information gained in a five years period of mining, analysed using one-factor and two factor regression analyses applied to a number of technical-technological parameters (ventilation, mining system, etc) influencing occurences of spontaneous mine fires and increased gas release in various ventilation regimes at longwall face aimed to give basic guidelines for minimisation and elimination of certain hazards.

Keywords: ventilation regime, spontaneous fires, gas release, mechanised longwall.

1. INTRODUCTION

Complex mining conditions in subject mine, especially in deeper parts (below K± 0 m level) chategorised this mine in a group of very complex mines with increased potential hazards, that results from:

- Coal seams are chategorised in a group of seams extremely prone to self-combustion
- Explosive properties of coal dust: p_{max} = 8,6 bar; $(dp/dt)_{max}$ = 394 bar/s; $K_{st,max}$ = 107 m·bar/s)
- Increased content of methane in coal seams (qa= 6,52 $m^3CH_4/t_{čus}$)
- Occurrences of rock bursts were registered below the K ± 0 level, excavation depth was at level 550 – 750, and that is above the critical depths of H_{kr} = 400 m
- Levels of water intake are high (3,0 – 5,0 m^3/min)
- Occurrences of other hazardous gasses were registered as well (CO_2, CO, H_2S, SO_2 and NO_2).

Methane releases occur from coal and accompanying rocks (porous lime stone) in the form of escalation and blowers of various intensity and time duration, mostly in the zone of tectonic disturbances.

Beside the natural conditions, the technical-technological factors have crucial influence on fire occurrences and gas release intensity [1] [8]. Giving that fact, it is necessary to adapt this factors to deposit conditions, i.e. negative occurences related to natural deposit conditions have to be minimised.

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Basic influential technical-technological factors include [7]: mining method, ventilation regime, manner and sequence of development works, gob sealing method, advance rate and intensity of mining works, scope of degassing conducted in previous seams.

2. ROOF PLATE MINING IN THE MAIN SEAM OF „RASPOTOČJE" MINE

The deep part of "Raspotočje" mine is one of the mining districts being mined out in the most severe natural conditions occurring in Bosnia and Herzegovina mines.

Roof plate mining of the main seam, Raspotočje mine, was carried out using mechanised longwall face method with retreat advance. Changes in length of working face is determined based on geometry and micro tectonic conditions in the block and occurrences of oxidation processes, that caused shortening of planned working face lengths [10].

Basic properties of mining method are as follows [3]:

- working face length 70-160 m
- minimal mining height 1,70 m
- maximum mining height 3,40 m
- working mining height 3,0 m
- dip angle of working face 6-34°
- productivity 4,50 t/m^2.

Total thickness of the main coal seam is aprox. 8,30 m, whereas 5,0 m thick coal plate in the floor part was left, except in the parts where floor coal plates were disturbed when crossing over the secondary diagonal faults (thickness of 3,0 m) Figure 1.

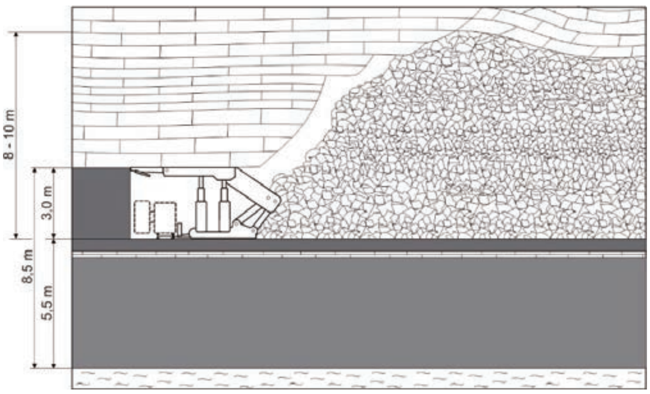

Figure 1. Longwall mining of the roof plate in the main seam

Roof caving was mostly conducted sucessively with working face advance. Rarely, forced blasting was conducted in the case of failled collapsing of the immediate roof plate. Along with advance of working face the entries were closed in the panel (gas channel, middle hallway, connecting gates and other auxilliary rooms), except of transport hallway that was maintained to serve as a gas channel in the next panel.

Arch steel support was dismantled from the gas channel upon passing of working face, for reason of complete caving and to reduce air flow through the gob.

Steel support in the middle hallway and other rooms was replaced with wooden support before arrival of working face, whereas connecting inclines and auxilliary rooms were stowed with filter ash.

3. VENTILATION AND POSITION OF VENTILATION GALLERY AT LONGWALL FACE MINING

Mine ventilation is conducted using mechanical depression method with two axial „Korfman" KGL-220 type fans. Fans are of same characteristics, one used in operation and the second as a reserve. Main mine ventilation is in category of diagonal systems, whereas main district ventilation systems are planned as central and fall in the category for increased fire risk systems due to possible self combustion of safety pillars for district openings (great pressures on openings and significant difference in potentials between entry and exit rooms).

Entry air for longwall face and development workings runs through opening coridors while return air runs down the main ventilation incline. Separate ventilation units (working panels and lower development panels) are separated by an air crossing constructed in the safety pillar for development rooms.

Ventilation of working panel is conducted using conventional „U" system, intake air is driven down the transport hallway to the longwall face whereas air is returned down the ventilation hallway. Lesser quantities of air pass through the connecting inclines in the working panel.

Geological conditions, gass and spontaneous combustion occurences resulted in apllication of 3 ventilation regimes listed below [6]:

«A» conventional „U" ventilation system (safety pillar left toward the gob of the previous working panel)

«B» ventilation with gas channel and central hallway (open contact with the gob of the previous working panel)

«C» separate ventilation of the longwall face upper part (safety pillar left toward the gob of the previous working panel).

3.1. «A» VENTILATION REGIME

This regime was applied for mining of the first working panel in the tectonic blocks and initial parts of the working panels that were longer than previously mined ones. This system is characterised with deep flowing of air through the longwall face gob and by high dependence of gass occurences on air quantity and dynamic of working front (Figure 2) [5] [6]. Figure 3 shows canonical ventilation scheme for the „A" regime.

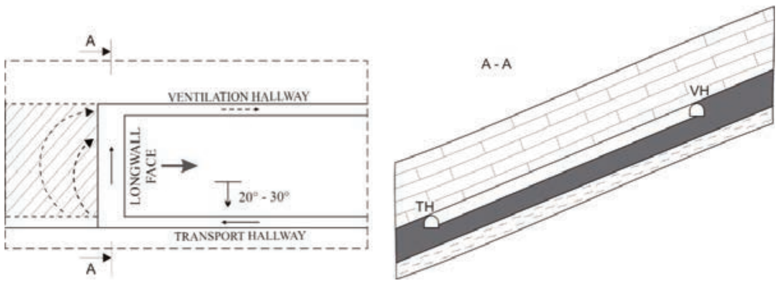

Figure 2. Longwall face mining of the roof plate using conventional ventilation regime

Total length of the longwall working face is 100 m. First problem occurred relating the caving of the immediate roof, after 28 m advance of the face. Upon roof caving, gas release into working face took place along the entire longwall face with methane concentration (CH_4) exceeding 1,0 %, while at the exit from longwall face it exceeded 3,0 %. Gas status went back to normal after couple of days, while 2,0 % methane concentration remained in the top and exit part. During extreme conditions, with drop in barometer pressure, methane release exceed 25.0 m^3/min.

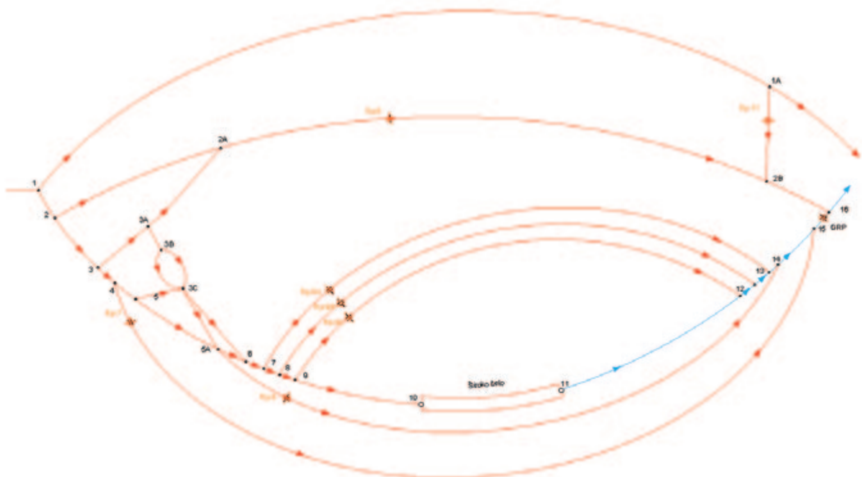

Figure 3. Canonical ventilation scheme – ventialtion regime «A» [5]

Projected longwall face ventilation air quantity is 2000 m^3/min. This required alteration in operation regime of the main fan, i.e. alterations in position of main fan rotor blades. Test operation of the main fan proved stable , up to the blade position on rotor of fan no. 4. This fan operation regime resulted in increase of air quantity from 3200 m^3/min to 4100 m^3/min, with gradual increase of fan depression to 3139 Pa.

Increase in fan depression caused increased activity of methane blowers and increase inflow of CH_4 from the isolation units along the main return air ways. Methane content in front of the main fan reached 50.0 m^3/min, while CH_4 concentration exceeded value of 1.0 %. Finally, parallel operation of main fans set on blade position of fan No 3.5, secured the air quantity of 4500 m^3/min, with simultaneous depression to 2747 Pa.

Rehabilitation of rooms and isolation units on main return air ways was required, in order to reduce methane concentration in front of the main fan, while longwall face operations were conducted with the strict controls of face and air ways (continuous CH_4 control, on site or using methane control station, and removal of electric installations and devices from the entire length of air way.

Installation of hard-rib ventilation pipeline ø 800 mm, from the main ventilation incline to the top of the longwall face, resulted in reduction of methane concentration to the limit values. Pipeline had ø 2,0 m tin funnels mounted at the ends. Funnels fixed in the main ventilation incline was used to create depression in the pipeline, whereas the funnel fixed on the top of the longwall face, at the end of the ventilation hallway toward the gob, was used to capture methane. Quantities and concentrations of gasses in pipeline were controlled regularly, with intention to maintain the methane concentration in pipeline above the 20 %.

Established ventilation regime with gas channel and middle hallway resulted in significant reduction in released methane with simultaneous increase in carbon dioxide quantity (CO_2). Such an exchange of gasses (CH_4 and CO_2) through gas channel secured more favorable mining conditions in the remaining part of the working panel. Problems occurring in the later mining period were related to rock bursts, which became stronger and more intense as the face line approached to the fault zone.

3.2. VENTILATION REGIME «B»

Ventilation regime «B» is the most applied ventilation system in mining operations. It is characterised with an open contact with gob area of previous higher positioned working panel, along the entire active length of the gas channel (20-120 m), and major influence of barometric pressure on gas release and fire occurences [5] [6].

Longwall face ventilation air run down the longwall face to the middle hallway, where air distribution was carried out (figure 4):

- part of the air was driven down the middle hallway between the longwall face and short connecting incline, while
- the remaining air was driven to the top of the longwall face through the gas channel and short connecting incline into the ventilation hallway (joint exit from the longwall face)

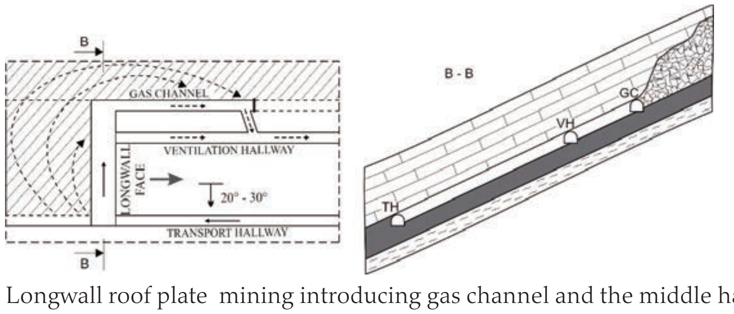

Figure 4. Longwall roof plate mining introducing gas channel and the middle hallway

Figure 5 shows the canonical scheme of the ventilation regime «B». Ventilation loop for air running down the gas channel were set through the short connecting inclines, between the middle hallway and gas channel, that were driven simultaneously with advance of the longwall face. Inclines were driven at distance of 70-100 m, depending on the coal incubation period (3-6 months) and advance of longwall face, i.e. requirement to mine through the open part of the gas channel in the period of three months. Passive part of the gas channel (part between the active part and the end of the panel) is isolated from the active part of gas channel using temporary isolation barriers, while other connections with an actively ventilated part of the mine were sealed using solid stowing barriers.

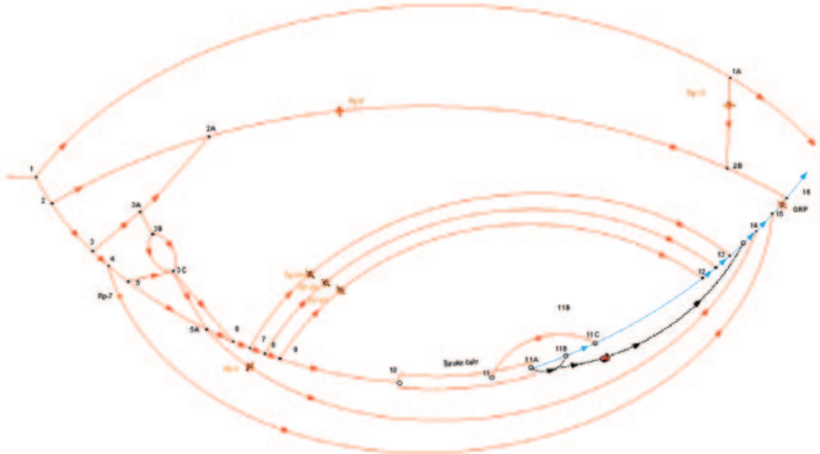

Figure 5. Canonical ventilation scheme – ventilation regime «B» [5]

Apart from air flowing from gob area of the active workings, the air was flowing in from the mined out higher-laying panel as well. Regime is characterised by a swift change of gas concentrations with variation in barometric pressure, resulting in a frequent occurence of high level of gasses in the short connecting incline.

Middle hallway, that served as a ventilation hallway during the development phase, was driven but it was necessary to leave 10 m thick safety pillar toward the transport hallway. Due to the collapsing of safety pillar and occurences of rock bursts it was widen to 25-35 m in the next panels.

Decision on separation from the gob of the higher-lying panels and introduction of separate regime of ventilation of the upper part of the longwall face was made upon 2 occurences of methane inflammation in the area of the active part of gas channel. Source of inflammation in both cases was fire in the gob of the longwall face.

3.3. VENTILATION REGIME «C»

This system was strictly applied in remediation of fires occuring in the gob of the longwall face. It is characterised by reduced inflow of air and hazardous gasses from the gob areas, and lower influence of barometric pressure on release of gasses [5] [6].Separate fan is installed in the lower third of the longwall face, with the end of the pipeline on the top of the longwall face, in order to reduce deeper penetration of fresh air from longwall face into the gob area, Figure 6. Figure 7 shows canonical ventilation scheme for the ventilation regime «C».

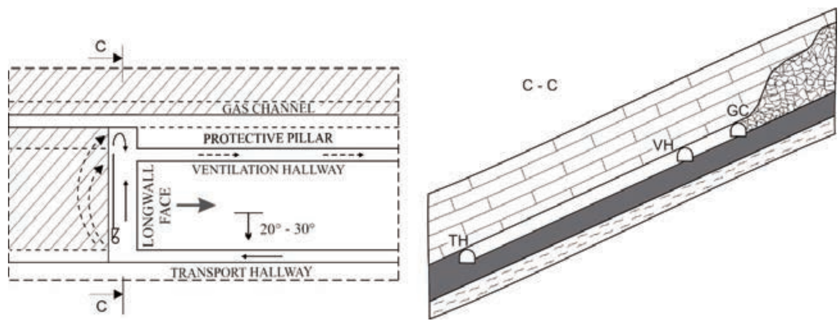

Figure 6. Longwall mining of the roof plate using separate ventilation for the upper part of the longwall face

Large diameter pipelines (ø800 mm), N = 37 kW of power, were used for the separate ventilation of the longwall face to transfer large quantities of air from entry part to the top of the face. Thus the gob was held under the lower depression, resulting in lower releases of gasses and reducing the air quantity required for the ventilation. Lack of this system can be a very long blind part, where, in the case of methane regime of operations and steep angle of longwall face, can occur swiftly increased gas content in the separately ventilated part of the face. This system is recommended in systems with low methane content, with blind part of the longwall face up to 10 m [5].

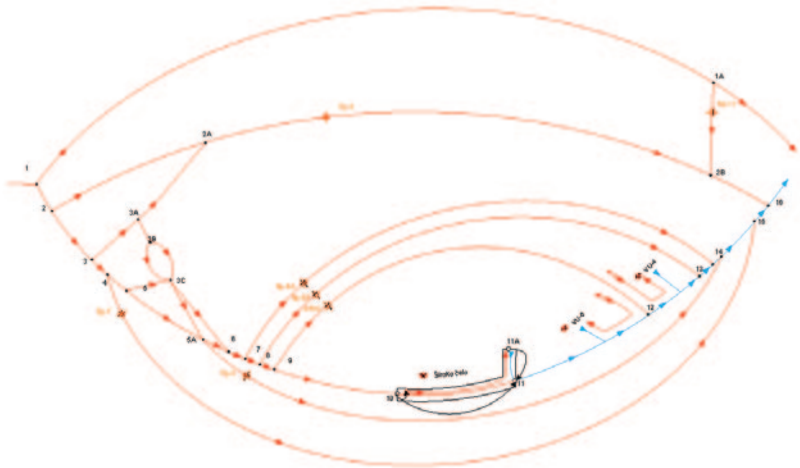

Figure 7. Cannonical ventiilation scheme – ventilation regime «C» [5]

4. MAIN PROPERTIES OF THE MODEL FOR SOLUTION TO THE ENCOUNTERED PROBLEMS IN THE LONGWALL MINING

Results of the recorded ventilation parameters and longwall face advance obtained in the 5 years period, in which three panels were mined out, and of regular chemical analyses of the mine air in that period, were basis for the one factor and two factor regression analyses of parameteres influenci$_{n}$g gasses$_{s}$ ocurrences (qCH_4 and qCO_2) and of fires (apsolute inflow of CO) for the all applied ventilation regimes [6]. Independent variables that were included are as follows: air quantity (V), advance rate of longwall face (n) and alterations in barometric pressure (B).

It was observed that relations between observed occurences drastically change with alterations in ventilation regime at longwall face. Gassess ocurrences increase in all regimes applied with drop in barometric pressurre (B). It was connected to (n) and (V) as well, but with significant differences in each of the applied ventilation systems.

Occurences of spontaneous fires are more related to applied ventilation systems, and depend on changes in air quantity (V), longwall face advance rate (n) and barometric pressure (B),

Two-factor regression analysis of gasses ocurences: $qCH_4 = f(n, V)$; a for $qCO_2 = f(n, V)$, points to a very solid connection at „A" ventilation regime, whereas that connection is weak at „B" and „C" ventilation systems.

Two-factor regression analysis determined a high influence of (V) and (n) parameters on fire occurences at „A" ventilation regime, whereas that connection between the parameters is weak at „B" and „C" ventilation systems.

Results of conducted analyses, practical observations of mentioned hazards, and some other hazards too (rock burst occurences and hazardous mine dust), gave a basic quidelines for model development (ventilation system) to be used for minimisation and elimination of certain hazards, that has to fullfill the following conditions:

- mining has to be carried out without safety pillars
- to preserve positive balance of gasses CH_4 and CO_2,
- mining without open contacts of ventilation ways with gob of the overlying panel,
- number of rooms in the endagered upper part of the panel to be reduced to minimum, to minimise rock burst occurences risk.

Giving the fact that requirements for the model development are quite variable, then solutions have to be sought that fulfill all set conditions to the most possible extent. Instead of safety pillar toward the gob, to avoid direct contact of active ventilation ways with gob of the overlying panel, a temporary safety pillar has to be left.

This pillar is projected to allow connection of the gobs of the working panels, immediately upon passage of the working face. Thus the function of this pillar in the first phase is to prevent open contact of actively ventilated rooms with the gob of the overlying panel, and upon passage of working face it collapses, connect the gobs and enable exchange of gases, CH_4 and CO_2. Thus the fire and rock burst occurrences are reduced and gasses exchange enabled, To secure the model to be fully successfull it is neccessary to explore system of development workings that will secure normal advance of development workings, and simultaneoulsy reduce the number of rooms in the endagered part of the mining panel.

5. PROPOSED LOCATION OF THE VENTILATION HALLWAY (LONGWALL FACE VENTILATION REGIME)

Taking into account the adopted experience based principles for the analysed case, and all influential factors, the most acceptable ventilation regime proved to be:

Mechanised longwall mining of the roof plate with ventilation and middle hallway (isolation safety pillar left toward the gob of the overlying panel, without the open contact toward the gob).

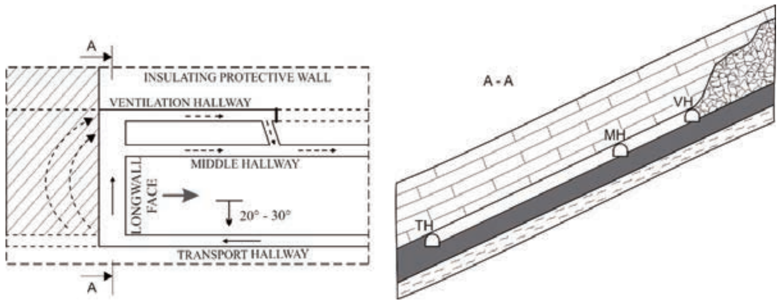

Figure 8. Longwall mining of the roof plate with ventilation and middle hallway-with isolatio safety pillar toward the gob of the overlying panel, without open contact with the gob

Proposed ventilation regime secures: exchange of gasses, ventilation without the contact of active workings with the gob of the overlying panel, reduced time required for construction of ventilation incline, and mining without safety pillar. This model reduces spontaneous mine fire and hazardous gassess occurences, and minimises the risk of rock bursts. Development operation include drivage of ventilation (later middle) hallway to secure special ventilation units during development and mining operations. Distance between ventilation-middle hallway and the gob, was altered during the mining operations in panel, therefore it is required to reconsider distance between ventilation hallway and the gob based on collected geomechanical data. Number of short connecting ventilation inclines would be reduced considerably, since they can be driven on distance of every 300-500 m. Number of rooms would be reduced significantly in the parts endagered with rock bursts. Isolation safety wall toward the gob would be set in accordance with longwall face advance. Timely and quality construction of the isolation wall would reduce inflow of air through the gob of the working panel. Thus the occurences of mine fires and gasses release (from the gob) would be reduced in the working panel. The wall would be used not only for isolation but for bearing purposes as well, in order to secure better preservation of the gas channel during the mining phase.

6. CONCLUSION

Research and analysis of endogenous fire and gasses release at mechanised longwall mining of the main seam roof plate in „Raspotočje“ mine, operation of „Zenica“ brown coal mines, was based on mining experiences, and analyses of a large number of gathered ventilation, mining and other parameters. Data analyses are performed using regression analysis method , and contemporary computer programs that enable fast and efficient check up and research into correlation of parameters [6].

Gained practical mining experience indicated a number of shortcomings of the applied mining system, that coupled with unfavourable natural conditions resulted in mining bearing high level of risks. Beside two methane inflammation, mining was followed by a frequent spontaneous fire and rock burst occurences, ranging from low to high intensity [10].

Test results and practical knowledge in mining of the analysed working panels, lead to the following conclusions:

1. Mining in the working panels in given conditions (high methane content in coal and accompaning seams with high free methane accumulations) has to be preceeded by one of the degassing activities

[3] [4], or to start with mining phase half the year upon completion of development operations (to secure time required for the partial degassing of the working panel).

2. Mining in the working panels has to be performed without safety pillars, to reduce risks of rock burst occurences, to secure exchange of gasses (CH_4 and CO_2), and to maintain the CH_4 concetration within the permitted limit [5] [9].
3. Development operations in the working panels has to be brought into conformity with advance rate of the active working panels. Alteration in geological conditions in the block (microtectonics) and strong outbursts of accumulated free methane, can significantly slow down or completely disable normal execution of the development operations.
4. Number of rooms driven in the panel has to be reduced to minimum, and the position of hallways by the gob has to be carefully determined (location of the ventilation-middle hallway and short connecting inclines.
5. Development of the auxilliary rooms underneath the transport hallway (in the area of the next panel) during the mining in the working panel has to be reduced to minimum, and these rooms has to be sealed when longwall face approaches . Experience and analyses based results indicate that the most frequent occurences of spontaneous fires are connected with these rooms (sumps, temporary sumps, storage chambers, etc)
6. Accompaning systems (transport, dewatering, material sypply delivery) have to be brought in conformity with the applied mining method to reduce influences of natural hazards.

Possibility of complete one web mining of the main coal seam is to be explored. Coal seam – floor section as oppose to coal seam – roof section, has low propensity to rock bursts, therefore driving the opening hallways in the floor plate and transfering the longwall equipment into the floor plate would reduce rock bursts occurrence risk significantly [9].

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ANALYSIS OF THE TEMPERATURE INFLUENCE OF FRESH ASPHALT MIXTURE ON THE NUMBER OF ROLLER CROSSINGS

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SUMMARY

In the construction of new roads in BiH, reconstruction or rehabilitation of existing roads, construction or replacement of asphalt layers is a mandatory part of construction work. One of the most important characteristics of the asphalt mixture during installation is the temperature of the asphalt mixture. Legal regulations, ie technical conditions, define the minimum temperature of the asphalt mixture as well as the limit air temperatures during the installation of asphalt, but special emphasis is given to the optimal temperature of the asphalt mixture. The optimal temperature of the asphalt mixture in this paper was analyzed through the compaction property of the asphalt mixture. Therefore, working on several specific examples, observes the optimal temperature of the asphalt mixture so as to obtain the best possible compaction of the installed asphalt. During the research, numerous other data that are important for the research were measured, such as the number of roller crossings, roller weight, air temperature, etc.

Keywords: asphalt mixture temperature, installation, asphalt compaction, roller

1. INTRODUCTION

The process of building asphalt pavement structures was based on tradition, art and a number of different implicit methods based on experience, which were applied in construction practice. Also, asphalt was primarily considered as a building material, and only later was the method of construction of asphalt pavement structures studied in more detail. [1] [2]. The temperature of the asphalt mixture during the installation of asphalt became especially important because it directly affected the quality of work performed, and was limited by the temperature of the asphalt mixture in the asphalt base, the length of transport of the asphalt mixture, vehicle quality, air temperature, etc.

If shrinkage due to cooling is prevented, then tensile stress increases in the asphalt material with decreasing temperature, which can lead to breakage (appearance of microcracks in the bonding matrix) if maximum tensile strength is reached, which is especially pronounced during asphalt rolling. Simply put, the stress in the asphalt sample gradually increases in parallel with the temperature drop, until the sample breaks. [3]

The research in this paper is based on measuring the number of roller passes and analyzing the influence of air temperature on the number of roller crossings, which affects the compaction of the installed asphalt mixture.

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2. ANALYSIS OF THE INFLUENCE OF THE TEMPERATURE OF FRESH ASPHALT MIXTURE ON THE NUMBER OF ROLLER CROSSINGS

If we observe the influence of asphalt temperature on the number of roller crossings, we will use the number of roller crossings and the installation temperature of the asphalt mixture as input data. [4]

As the compaction of asphalt was performed with several rollers, in relation to their technical characteristics (static weight and roller surface), the number of roller crossings was reduced to the nominal number of crossings of the relevant roller. The "Bomag 120" roller was used as the author.

- The "Bomag 120" roller puts a load on the substrate of 11.30 kg / cm. [5] As this roller was used as authoritative, it was calculated with a factor of k1 = 1.00.
- The "Hamm HD75" roller loaded the substrate with 23.20 kg / cm. [5] To reduce it to the nominal number of passes, the number of passes of this roller is multiplied by the factor

$$k2 = 23{,}20/11{,}30 = 2{,}05$$

- The "Hamm DV8" roller carries a load of 29.04 kg / cm on the ground. [5] To reduce it to the nominal number of transitions, the number of transitions of this roller is multiplied by the factor

$$k3 = 29{,}04/11{,}30 = 2{,}57$$

Thus, the "Hamm HD75" roller compacts with one pass as 2.05 passes of the "Bomag 120" roller, while one pass of the "Hamm DV8" roller is equivalent to 2.57 passes of the "Bomag 120" roller.

Table 1: Input data for the analysis "Asphalt installation temperature-number of crossings"

No sample	Temperature install (?C)	Type of asphalt	Number of roller cross			Nominal num roller crossing
			Bomag 120	Hamm HD75	Hamm DV8	
1.	136,30	BNS22	9	12	11	62
2.	139,90	BNS22	8	9	9	50
3.	123,80	BNS22	10	11	11	61
4.	112,80	BNS22	9	12	12	64
5.	152,90	BNS22	13	17	/	48
6.	157,80	AB16	8	7	7	40
7.	155,40	AB16	7	6	6	35
8.	154,20	AB16	7	7	6	37
9.	158,40	AB16	7	7	7	39
10.	160,40	AB16	8	7	8	43
11.	155,90	AB16	7	8	7	41
12.	160,10	AB16	9	9	7	45
13.	160,70	AB16	9	10	8	50
14.	157,40	BNS22	12	17	/	47

In accordance with the above, the nominal number of roller crossings is obtained by multiplying the number of crossings of each roller by the corresponding factor.

Eg for sample 1 the nominal number of roller crossings was obtained as:

$$n = 9 * 1.00 + 12 * 2.05 + 11 * 2.57 = 61.87 \approx 62 \text{ roller crossings.}$$

Values are rounded to the nearest whole number.

As we can see from Table 1, we have samples for two types of asphalt, whose layer thicknesses are also different, so we will divide the asphalts by type in the analysis and observe them that way.

We will first observe the samples from the BNS22 asphalt. For the input data, the installation temperature and the nominal number of roller passages are important to us.

Table 2: Input data for the analysis "Asphalt installation temperature-number of crossings-BNS22"

No sample	Temperature install (?C)	Type of asphalt	Nominal No roller crossing
1.	136,30	BNS22	62
2.	139,90	BNS22	50
3.	123,80	BNS22	61
4.	112,80	BNS22	64
5.	152,90	BNS22	48
14.	157,40	BNS22	47

In accordance with the input data, we obtained the following diagram describing the influence of the asphalt installation temperature on the number of roller crossings.

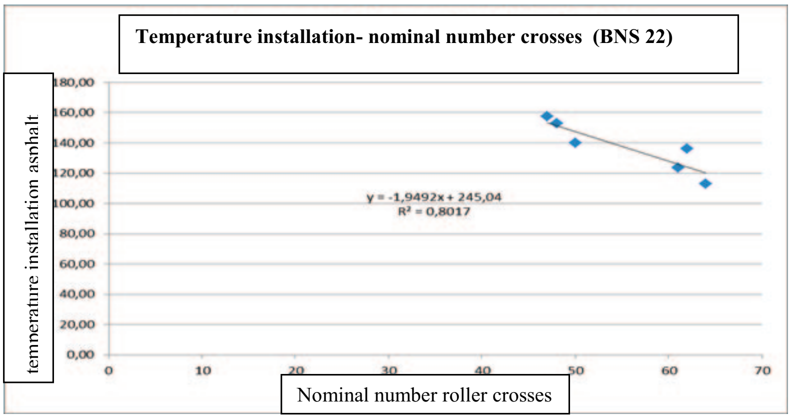

Figure 1. Dependence diagram "Asphalt installation temperature-number of crossings-BNS22"

For AB16 asphalt we use the following input data:

Table 3: Input data for the analysis "Asphalt installation temperature-number of crossings-AB16"

No sample	Temperature installation (ºC)	type of asphalt	Nominal No. crosses roller
7.	155,40	AB16	35
8.	154,20	AB16	37
9.	158,40	AB16	39
10.	160,40	AB16	43
11.	155,90	AB16	41
12.	160,10	AB16	45
13.	160,70	AB16	50

In accordance with the input data, we obtained the following diagram describing the influence of the asphalt installation temperature on the number of roller crossings.

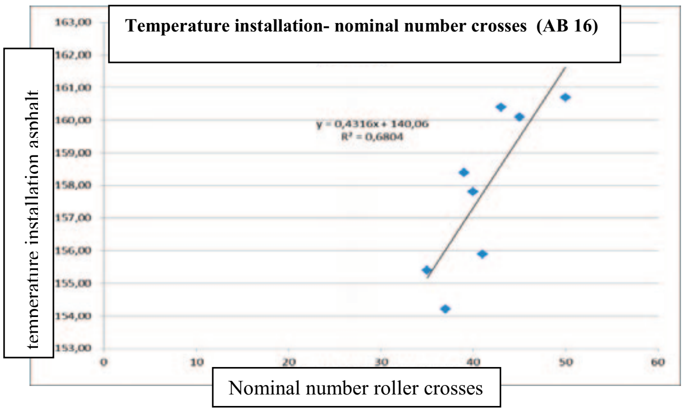

Figure 2. Dependence diagram "Asphalt installation temperature-number of crossings-AB16"

From the diagram in Figure 1, we can see how lowering the temperature of the asphalt increases the number of roller crossings in order to achieve the required compaction. We notice that the number of crossings increases significantly at asphalt temperatures lower than 140° C. From the diagram in Figure 2, we can see that the too high installation temperature requires a larger number of roller passages, because rolling is done until the asphalt temperature drops to a certain value. We see that by increasing the installation temperature above 157° C, the number of roller transitions increases again to achieve approximately equal compaction.

From the above, we can conclude that the optimal temperature of asphalt installation from the aspect of the number of roller crossings is 150-157° C.

We will confirm the conclusion with the following diagram, which combines all the samples. The data from Table 1. were used as input data.

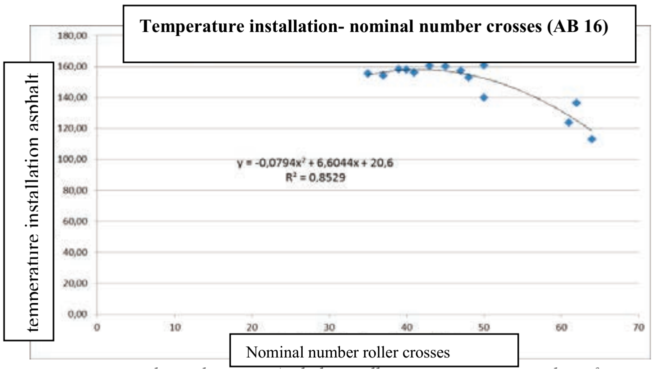

Figure 3. Dependence diagram "Asphalt installation temperature-number of crossings"

From the diagram in Figure 3, it is clear that the minimum number of roller passes is required at an installation temperature of 150-157°C. Any increase or decrease in installation temperature requires a larger number of roller passages.

3. ANALYSIS OF THE INFLUENCE OF AIR TEMPERATURE ON THE NUMBER OF CROSSINGS ROLLERS

If we observe the influence of air temperature on the number of roller crossings, we will use the number of roller crossings and air temperature when installing the asphalt mixture as input data.

Table 4: Input data for the analysis "Air temperature - number of roller crossings"

No sample	Temperature air (?C)	Type of asphalt	Nominal no crosses roller
1.	6	BNS22	62
2.	15	BNS22	50
3.	8	BNS22	61
4.	5	BNS22	64
5.	20	BNS22	48
6.	29	AB16	40
7.	20	AB16	35
8.	23	AB16	37
9.	22	AB16	39
10.	29	AB16	43
11.	26	AB16	41
12.	32	AB16	45
13.	31	AB16	50
14.	22	BNS22	47

As we can see from Table 4, we have samples for two types of asphalt, so we will divide the asphalts by type in the analysis and observe them that way.

We will first observe the samples from the BNS22 asphalt. For the input data, as already mentioned, the air temperature and the nominal number of roller crossings are important to us.

Table 5: Input data for the analysis "Air temperature-number of roller crossings - BNS22"

No sample	Temperature air (?C)	type of asphalt	Nominal no crosses roller
1.	6	BNS22	62
2.	15	BNS22	50
3.	8	BNS22	61
4.	5	BNS22	64
5.	20	BNS22	48
14.	22	BNS22	47

In accordance with the input data, we obtained the following diagram describing the influence of outside temperature on the number of roller crossings.

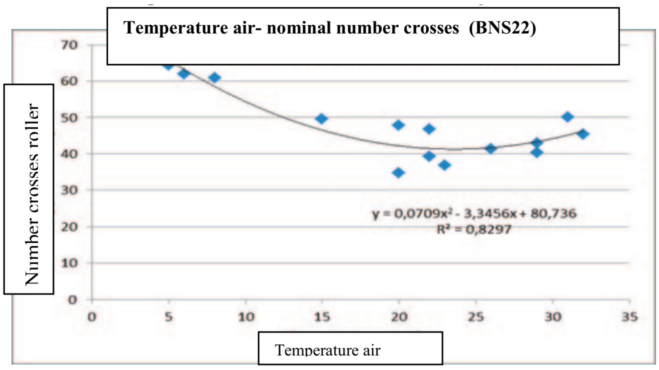

Figure 4. Dependence diagram "Air temperature-number of roller crossings - BNS22"

For AB16 asphalt we use the following input data:

Table 6: Input data for the analysis "Air temperature - number of roller passages - AB16"

No Sample	Temperature air (ºC)	Type of asphalt	Nominal No crosses roller
6.	29	AB16	40
7.	20	AB16	35
8.	23	AB16	37
9.	22	AB16	39
10.	29	AB16	43
11.	26	AB16	41
12.	32	AB16	45
13.	31	AB16	50

In accordance with the input data, we obtained the following diagram describing the influence of outside temperature on the number of roller crossings.

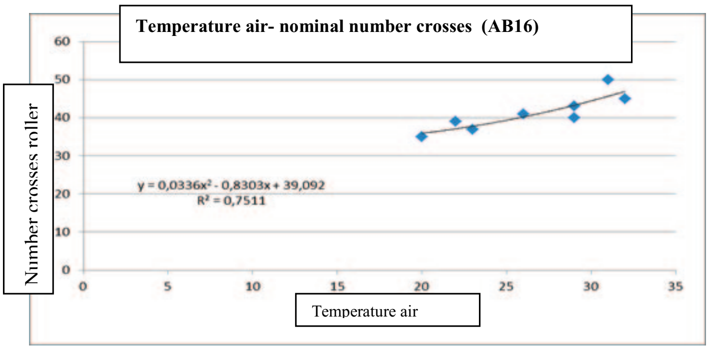

Figure 5. Dependence diagram "Air temperature - number of roller passages - AB16"

From the aspect of asphalt cooling in the analysis of outdoor temperature and asphalt temperature, it was found that the optimal air temperature for asphalt installation is min 15 °C. [6]

Also, from the diagrams in Figures 4 and 5, we can see that the minimum number of roller crossings for both types of asphalt was at an outdoor temperature of 20-23 °C. Each air temperature above 23 °C and below 17° required a significantly higher number of roller crossings. [7]

We conclude that the optimal air temperature for asphalt installation from the aspect of the number of roller crossings is 17-23 °C. [8]

We will confirm the conclusion with the following diagram, which combines all the samples. The data from Table 4 were used as input data.

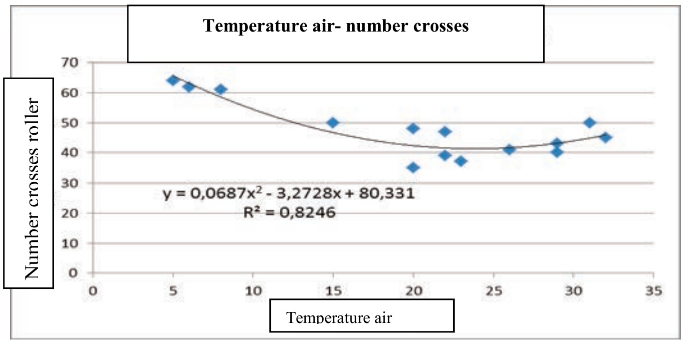

Figure 6. Dependence diagram "Air temperature-number of roller crossings"

From the diagram in Figure 6, it is clear that the minimum number of roller passes is required at an outdoor temperature of 17-23 °C. Each higher or lower air temperature requires a larger number of roller passages. [8]

4. CONCLUSION

In this paper, research and analysis of the influence of air temperature on the number of roller crossings during the installation of asphalt mass are presented. It can be concluded that by increasing the outside temperature, and by adjusting the outside temperature and the temperature of the asphalt, we can create optimal conditions for compacting asphalt (as few roller passes as possible, as short a rolling time as possible to cool the asphalt). Also, too high outdoor temperatures adversely affect the optimal compaction, because it takes more time for the asphalt to cool, and rolling is done until the asphalt reaches the appropriate (low) temperature. As we have limited values of upper and lower asphalt production temperatures, we can create optimal conditions by adjusting the outdoor temperature for the given circumstances (production temperature, temperature loss in transport). To ensure satisfactory quality of installed asphalt, we must take into account the outdoor air temperature. Therefore, if we respect the maximum temperature of asphalt production (175 °C), the greater the distance of the installation site from the asphalt base, we must have a higher air temperature to achieve the appropriate compaction and quality of the installed asphalt.

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USE OF CARBON FIBER IN CONCRETE STRUCTURES – STATE OF THE ART

Danijel Ružić,[1.] Kemal Zahirovic[2.]

SUMMARY

The development of material production technology has led to the application of new materials in construction. Because concrete is currently the most commonly used construction material, which in addition to numerous advantages (water resistance, low maintenance costs, easy workability, low cost, etc.) also has certain disadvantages (low tensile strength and brittle fracture behavior). Reinforcement of concrete using discrete fibers, randomly distributed, is an acceptable solution for improving the ductility of concrete. Carbon fiber reinforcement (CFRP) has been widely studied in the last two decades, as it represents a suitable alternative for the reinforcement of existing (endangered) RC structures. The advantages of this material are reflected in the relatively simple application, increased performance of the RC structure, low weight of the elements, etc. The paper also analyzes the existing cases of application of this material, as well as the presentation of previous research in the field of structural reinforcement using carbon fibers.

Key words: Carbon fiber, Carbon fiber-reinforced polymer (CFRP), reinforcement concrete structures, reinforcement of existing structures, structural behavior of construCTIONS

1. INTRODUCTION

Reinforced concrete (RC) is the most important and most used composite material in construction, which achieved its development through the development of its component materials concrete and concrete steel. Namely, high-strength concrete (up to 200 MPa) with additives that improve properties in terms of weather deformation and high-strength steel, especially prestressing steel, are used today.

But with the advantages, including high load capacity and relatively simple processing at low cost, apart from shaping, there is also a major disadvantage; reinforced concrete is prone to corrosion. Concrete is highly alkaline and forms what is known as a passive layer on the reinforcing steel, protecting it from corrosion. Substances that enter the concrete from the outside (carbonation) can reduce the alkalinity over time (depassivization), so that the reinforcing steel loses its protection, and the steel reinforcement begins to corrode. This leads to cracking of the concrete, reducing the durability of the structure as a whole and its failure in extreme cases. This problem is particularly present in older RC structures, which were built before the introduction of the service life and limit state of durability in the analysis of the structure's usability (built from the middle of the last century to the eighties of the last century). The standards that are in force today do not cover in sufficient detail the aspect of durability of concrete in terms of the calculation model.

Improving the properties of concrete structures in terms of load-bearing capacity, usability and durability has led to the development and application of new materials. Thus, in the nineties of the last century, research began with the aim of developing the application of technical textiles to achieve better properties

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of the edge parts of concrete elements. Here we can single out the research carried out in Dresden and Aachen (Germany) funded by two institutions, the DFG Collaborative Research Centers SFB 528 and SFB 532. The previously mentioned research showed that the use of textiles as part of the concrete protective layer enables the reduction of the thickness of the concrete protective layer, which is particularly interesting. in aggressive environments (industrial zones and marine areas), where greater thicknesses of the concrete protective layer are necessary.

The use of fiber-reinforced polymer (FRP) materials in civil infrastructure for the rehabilitation and strengthening of reinforced concrete structures as well as for new constructions has become a common practice. The most effective technique for improving the shear strength of the damaged parts of the RC structure is the joining of fiber reinforced plastic (FRP) plates or sheets as cladding. External reinforcement with highly reinforced fiber-reinforced plastics (FRP) on concrete structural elements has gained great popularity in recent years, especially in rehabilitation works but also in new construction. Comprehensive experimental research conducted in the past has shown that this strengthening method has several advantages over traditional methods, particularly corrosion resistance, high stiffness-to-weight ratio, improved durability and flexibility.

According to existing standards, the minimum requirement for a concrete protective layer is 35 mm for external building elements, which leads to a total thickness of about 100 mm for facade panels. If the reinforcing steel is now replaced by non-metallic reinforcement, for example mesh reinforcement made of glass or carbon fiber, a thickness of only a few millimeters is sufficient to ensure durability and a strong connection between the concrete and the reinforcement. It is known that TRC enables the production of facade panels between 20 and 30 mm thick, which saves up to 80% in concrete. This has a direct impact on transport costs for precast concrete components, which can be reduced by up to 80%. Much smaller mounting elements are also sufficient for installation, which further saves costs. TRC is already used today in new construction, and it also serves the purpose of strengthening existing structures. In addition to the fact that they are not susceptible to corrosion, glass and carbon fiber reinforcements are particularly distinguished by their high strength, which is up to six times higher compared to reinforcing steel.

The use of carbon fiber reinforced polymers (CFRP) can now be considered common practice in the field of strengthening and rehabilitation of reinforced concrete structures. The effectiveness of this technique has been widely documented by theoretical and experimental research and applications on real structures. CFRP reinforcement provides additional flexural or shear reinforcement, and the reliability of this application of materials depends on how well they are bonded and can transfer stress from the concrete component to the CFRP laminate. Commercially available FRP strengthening materials are made of continuous aramid (AFRP), carbon (CFRP), and glass (GFRP) fibers.

This paper presents an overview of previous research in the field of carbon fiber application in constructions, and on this occasion a systematic division of the area was made into:

- Properties of carbon fiber (CFRP),
- Method of application in constructions,
- Experiences of the exploitatiON AGE.

2. PROPERTIES OF CARBON FIBERS (CFRP)

The previously mentioned researches have shown that the use of textiles as covering of concrete elements enables the reduction of the thickness of the concrete protective layer, which is particularly interesting in aggressive environments (industrial zones and marine areas), where greater thicknesses of the concrete protective layer are necessary. The main role of strengthening structures with carbon fibers is expected in the area of tensile load capacity. Of course, in addition to the tensile load capacity in reinforced concrete structures, there is the question of the contribution of carbon reinforcements in terms of durability and even ductility of the structures. In the presented works, tests were carried out with the aim of determining the final properties of carbon fibers and carbon tape reinforcement in terms of tensile load capacity, appearance and distribution of cracks in concrete, as well as the final load capacity of the structures themselves.

Research carried out in paper [1] provides guidelines for strengthening RC beams using FRP materials, where the types and methods of FRP strengthening are generally described. Research was conducted on 38 beam samples with the aim of providing the shear capacity. The samples are variably reinforced (the main load-bearing reinforcement, the spacing of the shear reinforcement and the method of strengthening with CFRP were used differently). The obtained research results showed that beams without reinforcement or with reinforcement in the support area were not satisfactory in terms of crack opening, beams with plates over the entire span had an explosive failure without warning, while all the remaining samples reinforced with CFRP strips failed in shear and where until the strips come off. All reinforced beams showed an increase in shear resistance from 19% to as much as 109% (depending on the type of reinforcement). Conclusions are given regarding the application of commercially available materials (SIKA) and an account of the increase in beam strength with regard to the method of application of CFRP reinforcement is given.

A general report by the **ACI Board (2002)** provides current knowledge on the application of FRP, as well as examination of the flexibility, shear strength, bond behavior and durability of all materials in use. Also, further in the paper it is emphasized that the common link between all FRP products described in this paper is the use of continuous fibers (glass, aramid, carbon, etc.) embedded in a resin matrix, an adhesive that allows the fibers to act together as a single element. The resins used are thermoset (polyester, vinyl ester, etc.) or thermoplastic (nylon, polyethylene terephthalate, etc.). FRP composites differ from the short fibers that are widely used today to reinforce non-structural cementitious products known as fiber-reinforced concrete (FRC). The production methods of combining continuous fibers together with the resin matrix allow the adaptation of the FRP material so that the optimized reinforcement of the concrete structure is achieved. The primary interest of the concrete industry in FRP reinforcement, as stated in the paper [2], is the fact that it does not usually cause durability problems such as those associated with corrosion of steel reinforcement. Depending on the constituents of the FRP composite, other deterioration phenomena may occur as explained in the report. It was emphasized that concrete elements can benefit from the following characteristics of FRP reinforcement: light weight, high specific strength and modulus, durability, corrosion resistance, chemical and environmental resistance, electromagnetic permeability and impact resistance.

Paper [2] states that there are three sources for commercial carbon fibers: resin as a by-product of petroleum distillation, PAN (polyacrylonitrile), and rayon. The properties of carbon fibers are controlled by the molecular structure and degree of freedom from damage. Carbon fiber formation requires processing temperatures above 1830F (1000°C). At this temperature, most synthetic fibers will melt and evaporate, acrylic will not, and its molecular structure is retained during carbonization at high temperatures. There are two types of carbon fiber [2]: high modulus **Type I** and high strength **Type II**. The difference in properties between types I and II is the result of the difference in the microstructure of the fibers. These properties are derived from the arrangement of the graphene (hexagonal) layer networks present in graphite. If these layers are present in three-dimensional bundles, the material is defined as graphite. If the bond between the layers is weak and two-dimensional layers occur, the resulting material is defined as carbon. Carbon fibers are two-dimensional.

The paper [2] also presents a table showing the typical physical and mechanical properties of commercially available composite fibers for reinforcement. It can be noted here that the characteristics of carbon fiber products are significantly higher than those of glass or aramid fibers. The differences in tensile modulus and tensile strength are especially pronounced, where carbon fibers are up to 50% more resistant than glass or aramid fibers.

In paper [3], experimental research was carried out on RC beams with the purpose of formulating the behavior of RC beams before and after the installation of carbon fibers in order to increase the strength of damaged beams. Tests were carried out on the load to failure of concrete beams of concrete grades M15, M25 and M35. Materials used to strengthen the beams, i.e. carbon fibers, are initially loaded at 60% and 90% of the ultimate strength of the control beam. Experimental results showed that the technique of strengthening RC beams using carbon fibers will increase the ultimate strength of the beams by up to 30%. The average increase in strength of the beam with carbon fibers is greater than that of the control beam. Some of the conclusions drawn are that the load capacity can be increased by 35 to 45% by applying carbon fiber reinforcement, but also that the development of cracks before and after the installation of carbon fiber reinforcement did not change. All test samples showed large damage at the ultimate load as well as a significant increase in ductility.

Bašanović L. (2019) worked on the area where the concrete was reinforced with fibers, i.e. concrete with a proportion of fibers that are irregularly and randomly distributed in the concrete mixture with the aim of improving the physical and mechanical characteristics of concrete. The paper gives a brief overview of the application of fibers in construction, as well as the application of additives in concrete in the form of micro fibers. In the work itself, no experimental research was applied, leaving the possibility for further research in that direction.

Böhm R., Thieme M., WD, Wolz DS (2018) this paper describes the necessary production steps for the development of carbon fiber (CF) reinforcement: (1) production of cost-effective CF using new carbon fiber lines and (2) fabrication of CF-ribs with different geometry profiles. PAN/lignin-based CF was found to be currently the most promising material to meet future market demands. However, significant research needs to be undertaken to improve the properties of lignin-based CF, i.e. PAN/lignin. In the work, the authors also deal with the creation of a countertype of the production technology of carbon fiber reinforcement. It is emphasized that 10% of the world's total energy consumption is needed for the construction and dismantling of buildings, and that current buildings only have a limited lifespan of about 40 to 80 years.

The use of carbon fibers for reinforcement, as stated in this paper [8], requires that the carbon fibers must reach the minimum mechanical properties of steel. The minimum stiffness requirements of CFRP-based strengthening systems can be calculated by a simple rule of thumb that takes the typical fiber volume for either thermoset matrix systems (typically around 60%) or thermoplastic matrices (typically around 45%). The authors state that in order to achieve the required tensile strength value of around 500 MPa for CRFP reinforcing steel, carbon fibers must have a tensile strength of approximately 830 MPa for thermoset matrix systems. Also, the authors emphasize that the stiffness requirements for CF resulting from the direct replacement of steel with CFRP defined by the minimum Young's modulus are already fulfilled by both industrially available and experimental-scientific carbon fibers.

In order to evaluate the mechanical properties of the reinforcement types and the influence of the production process on the properties, comparative tests were performed. The comparison was made using established strengths because stress measurements to determine Young's modulus are highly questionable due to the different external geometries of different types of reinforcement.

At the end, a number of conclusions were drawn, and it was said that the new carbon fiber production process enables the development of cheaper carbon fibers. Five different production processes have been developed for the realization of different surface geometry profiles. In particular, the so-called spiral pultrusion - as a production process for the production of reinforced structures with surface profiling in one mold step - has been identified as efficient enough to be successfully integrated into the industrial process of rebar production. At the same time, the helical pultrusion process still offers design freedom with respect to surface contours and fiber orientation, allowing for even greater potential to optimize reinforcement bar design.

In the paper [9], the author conducted an experimental study with the use of carbon fibers for reinforcement with the aim of obtaining the most correct application procedure of reinforcement, which would lead to an increase in resistance to bending, shearing and stiffness. This paper presents the procedure for increasing/decreasing the percentage of substrates reinforced by different types and methods of application of CFRP materials. Two commercially available CRFP materials produced by SIKA were used in the work, and tests were carried out on five exemplary models of RC beams. The data collected for the deflection shows a 300% improvement in the utilization of the reinforced elements between the fifth and first beams at different loads. It is similar to all beams compared to each other at specific loads. In addition, when conducting the test, the increase in member stiffness was one of the most important factors that differed from one sample to another. All samples showed three stages: re-opening of cracks, cracking and crushing. Crack phases are improved with the application of CFRP laminates at various locations. The more anchorages and wraps used, the better the specimen performed in terms of shear, bond, flexural strength and time to complete failure of the reinforced elements. In the end, it was concluded that the use of CFRP material is one of the most powerful techniques for strengthening RC elements. Reinforcement of concrete with CFRP leads to an increase in load capacity as well as an increase in stiffness. When anchorage is considered, better performance and usability of the structure is considered. The strength and stiffness of the members increase with increased use of CFRP laminates, thus avoiding crushing or complete failure of members without warning. In this work, it was concluded that the application of CFRP laminates whenever necessary, taking into account anchorage and stiffness, actually leads to an increase in beam strength and provides additional load-bearing capacity.

Cheng-Tzu Thomas Hsu et al. (2006) investigate the shear behavior between concrete and CFRP polymer with carbon fibers in their work. A total of 27 samples were tested in this research. Test variables include the maximum compressive strength of the concrete, from 4000 to 12000 psi. This test setup was shown to be able to investigate the direct shear condition between CFRP laminates and concrete. Based on the test results so far, empirical formulas have been developed for calculating the compressive strength and slip ratio of different concrete compressive strengths. Such a relationship allows further understanding of the transfer mechanism of CFRP laminates and concrete. The paper also discusses the influence of salt water on the ultimate shear strength of the CFRP strengthening system. The paper describes in detail the characteristics of the materials used in the test with their physical and mechanical characteristics, and also describes the correct procedure for installing CFRP strips. In order to further understand the direct shear strengthening behavior with CFRP, a new test method established by Bian, Hsu, and Wang 1997 was also used; Hsu, Bian, and Jia 1997 This new experimental method not only measures the compressive strength of CFRP reinforcement, but also reveals the compressive strength and slip relationship of the system. Knowing the exact ratio is crucial for determining the bond development length of CFRP reinforced beams. Nine samples were tested for the effect of salt water on the ultimate strength of the CFRP connection. Seawater has been observed to have some effect on the epoxy curing process. It was found that the final push-off strength of the specimens immersed in seawater was 30% lower than that of the control specimens. Based on the conducted research, the following conclusions were drawn: A new direct direct shear test procedure was proposed, which is capable of studying the punching and sliding strength of CFRP strips and concrete; The present test results show that the ultimate direct shear or compressive strength of the CFRP reinforcement system increases almost linearly with the maximum compressive strength f c of concrete and the tensile splitting strength f sp of concrete; The use of a binder or epoxy resin in seawater or saltwater affects the ultimate shear strength and effectiveness of the epoxy resin CFRP strengthening system.

In [12] , an overview of previous research in the field of fiber reinforcement is given. As stated in the paper, reinforcing concrete using discrete fibers, randomly distributed, is an acceptable solution for improving the ductility of concrete. The addition of fibers in concrete affects most properties. These improvements in material properties opened the way for more research in this area to explore its advancement into untapped areas. Experimental results quantitatively reveal its improvement in various parameters such as tensile strength, flexural strength, toughness, ductility, corrosion resistance, resistance to cyclic and dynamic loads, crack resistance, etc.

Grujić B. (2016) addressed the topic in her doctoral dissertation modeling of the physical and mechanical properties of fiber-reinforced concrete with application in constructions. The subject dissertation investigated and analyzed the possibilities of obtaining improved physical and mechanical characteristics of micro-reinforced concrete with variations in the types and amount of steel fibers used in the mass of concrete. The selected steel fibers, as well as the amount of fibers used, greatly influenced the quality of the physical and mechanical characteristics of micro-reinforced concrete. At the same time, the paper presents a rheological-dynamic analysis of the subject concrete on a standard cylinder-shaped sample. The procedure for making samples of fiber-reinforced concrete is detailed, which is particularly significant from the aspect of the influence of the fiber shape factor, the time of making the mixture, and the coarseness of the aggregates and the fiber content on the consistency of the concrete. Microreinforcement mainly contributes to a significant increase in the ductility (toughness) of concrete, both when using steel and when using other artificial fibers: polymer, carbon, glass and others. The eventual increase in compressive strength due to the addition of fibers depends on a number of parameters, the most important of which are: type of fibers, amount of fibers, shape factor (L/D) and composition of the concrete matrix. The obtained results showed that there is an increase in the compressive strength of concrete elements reinforced with microfibers compared to standard concrete samples, approximately 3.7 - 9.4% for microfibers with curved ends. Flat microfibers have a slight increase in compressive strength (approx. 0.5-1.5%). A similar behavior occurs in the tensile strength test, where the increase is expressed from 56 – 100 % for samples reinforced with microfibers with curved ends. Other samples (straight microfibers) have an increase in tensile strength of approx. 30%.

In the paper [14], FRP reinforcement for concrete structures, a review of the available literature and current knowledge in the field of fiber-reinforced polymers (FRP), which are used to strengthen concrete structures, was carried out, especially on structures subject to aggressive environments or the influence of electromagnetic fields. This paper also reviews the production process, material properties, field of application and specifics of designing concrete elements reinforced with FRP composites. With a primary focus on internal reinforcement, the paper discusses recent practices of FRP application on RC elements in construction. The paper contains descriptions with a table showing the physical and mechanical characteristics of various FRP materials. Application supports are described, but it is said that there is not enough reliable experimental data on the long-term degradation of the mechanical properties of FRP materials. Therefore, common design practice is based on increased values of safety factors, which lead to higher costs of FRP-reinforced elements and make such structures economically inefficient. Also one of the main disadvantages, which is mentioned in this paper, is that most of the materials used for the production of FRP are not resistant to specific environmental conditions. For example, fiberglass is not resistant to alkalis; UV radiation is harmful to the mechanical properties of most polymer resins. The authors suggested that further research should be directed towards experimental investigations of long-term mechanical processes that take place in concrete elements with FRP reinforcement, and the development of a standard form of internal FRP bars and anchoring measures for external reinforcement, but also towards the development of design procedures for the application of combined internal and external reinforcements .

Katar M. Ihab (2017) reviews the latest research in the field of carbon composites in RC structures. The paper contains comparisons of a group of building materials that are mainly used for finishing from certain aspects and properties such as: stiffness and strength of materials in relation to weight, stiffness and strength of materials of the same thickness, weight/density, processing, thermal expansion, conduction heat, temperature resistance, long-term effect and implementation of the production process. The author provides a historical overview of the development of carbon fiber applications, with graphic representations of fiber textures. The results obtained during the research were collected and presented in tables. Analyzing the results, the study found that CFC received the second highest score among other materials after steel, although it had high scores among other materials, especially in specific properties: stiffness / strength to weight and thickness ratio, thermal expansion. On the other hand, CFC obtained lower results in other properties, such as: processing, temperature resistance and carrying out the production process, especially compared to steel and wood. Meanwhile, CFC scored average among most other materials in properties: weight/density, thermal conductivity, and long-term performance. As a result, CFC was assigned the second rank among other construction materials, but still by a small margin after steel, and not as expected before the experiment itself.

Navya HA, Patil Nayana N. (2018) investigated the behavior of M25 concrete reinforced with carbon fiber doses of 0%, 0.75%, 1.00% and 1.25% of the concrete mass, with the aim of determining the strength and durability characteristics. The mechanical properties, compressive and tensile strength, as well as bending resistance, were studied. The test samples were also subjected to the effects of acids and sulfates and were tested for their durability. The results show that there is an increase in compressive strength, tensile separation and flexural strength of carbon fiber reinforced concrete. The inclusion of 1% carbon fiber showed the maximum increase in strength and can be considered as the optimal dose. Compared with conventional concrete, the crack width also decreased in carbon fiber reinforced concrete. Some of the most important conclusions of this research are:

- As the carbon fiber dosage increases from 0% to 0.75%, 1% and 1.25%, workability decreases. However, the slump test for conventional concrete and carbon fiber reinforced concrete with 0.75% carbon fiber was found to have the same values,
- The compressive strength of M25 concrete for different amounts of carbon fibers of 0.75%, 1.00% and 1.25% compared to conventional concrete increased by 46.80%, 59.90% and 32.40%. The maximum percentage increase in compressive strength was achieved at 1.0% fiber dosage, and it was found to reduce by 1.25% the fiber content,
- It was observed that there was an increase in the tensile strength of CFRC for different dosages of 0.75%, 1% and 1.25% and they were found to be 28.1%, 56.30% and 9.40% more than with conventional concrete. The maximum percentage increase in tensile strength was achieved at 1.0% fiber dosage, and it was found to reduce more than 1%,
- Compared with conventional concrete, the flexural strengths for M25 grade CFRC for different carbon fiber percentages of 0.75%, 1.00% and 1.25% increased by 88.50%, 107.69% and 78.46% . The maximum increase in flexural strength was achieved at 1.0% fiber dosage, and it was found to reduce more than 1%,
- Carbon fiber reinforced concrete is more resistant to acid and sulfate attacks compared to conventional concrete. The highest resistance was observed in the case of CFRC with 1.0% carbon fiber, as indicated by the lowest percentage losses in weight and compressive strength.

3. METHODS OF APPLICATION IN CONSTRUCTIONS

As stated by the authors in [2], in the 1960s, corrosion problems began to appear on RC bridges and highway structures. Road salts in colder climates or sea salts in coastal areas accelerated the corrosion of reinforcing steel. Corrosion spreads and causes the concrete structure to break. The first solution for rehabilitation was the installation of a galvanized layer applied to the reinforcing bars. This solution soon fell out of favor for various reasons, but mainly due to the electrolytic reaction between the steel and the zinc-based coating, which led to a loss of corrosion protection.

In the late 1960s, several companies developed electrostatic-spray with fusion (powder) on steel oil and gas pipelines. In the early 1970s the US Federal Highway Administration funded research to evaluate over 50 types of coatings for steel reinforcing bars. This led to the current use of epoxy-coated steel rebars.

It is emphasized that research into the use of resin in concrete began in the late 1960s with a program at the Bureau of Polymer Impregnated Concrete Records. Unfortunately, the steel reinforcement could not use polymer concrete due to incompatible thermal properties. This fact led Marshall-Vega (later renamed Vega Technologies and currently reformed under the name Marshall-Vega Corporation) to produce glass rod for FRP reinforcement. The experiment succeeded and the resulting composite rebar became the polymer concrete reinforcement of choice. Despite earlier research into the application of FRP reinforcement in concrete, the commercial application of this product in conventional concrete was not recognized until the late 1970s. Then research began in earnest to determine if composites were a significant improvement over epoxy-coated steel. During the early 1980s, another pultrusion company, International Grating, Inc., recognized the product's potential and entered the FRP reinforcement industry.

The same authors [2] talk about how in 1986 the first highway bridge using composite reinforcement was built in Germany. Since then, bridges have been built across Europe, and more recently in North America and Japan. The US and Canadian governments are currently investing significant sums in product evaluation and further development. The biggest markets seem to be in the transportation industry. The paper states that at the end of 1993, there were nine companies actively selling commercial FRP rebars.

Structures such as bridges and columns built entirely from FRP composites have shown exceptional durability and effective resistance to the effects of environmental exposure. Modern FRP bonding has been established worldwide as an effective method applicable to many types of concrete structural elements such as; columns, beams, slabs and walls. It was there in 1991 that the first on-site repair with FRP joints was carried out. Since then, strengthening with externally bonded FRP composites has been studied worldwide. This sudden increase in the use of FRP composites was achieved after the 1995 Hyogoken Nanbu earthquake in Japan. By 1997, more than 1,500 concrete structures worldwide had been strengthened with FRP cladding joints. In Figure 1. the on-site application of CFRP is presented. Another form of application is the use of FRP bars instead of steel reinforcing bars or prestressing strips in concrete structures.

Figure 1. Strengthening of the RC structure using carbon (CFRP) strips [4]

Alferjani MBS, Abdul Samad AA, Elrawaff S. Blkasem, Mohamad N. (2013) in their paper gave an overview of 10 papers on the topic of using carbon fibers reinforced with polymers as reinforcement of RC beams in the shear area. Here, the area of application of carbon fiber is emphasized, as well as the history of application. In the paper itself, an overview table of data obtained by experimental and numerical research with the results of structural failure at ultimate load is provided. The conclusions were drawn that a greater number of layers of CFRP reinforcement on the beams is unnecessary and that the use of FRP material in structures enables variations in the safety factor depending on the environmental conditions and the type of stress.

Khalifa et al. (1999) conducted a test on three RC T-beams to investigate the anchorage performance of surface-mounted FRP reinforcement. The first beam was the reference beam, the second one was strengthened with CFRP without end anchorage, and the last one was strengthened with CFRP with end anchorage. The anchoring system, called the U-anchor, uses a GFRP bar placed in a groove on the edge of the beam and serves as an end anchor. They found that shear strength increased when reinforced with CFRP, but failure was governed by CFRP debonding when CFRP was used without end anchorage. However, in the specimen where anchorage was used, the shear capacity of the member increased quite a bit and ultimately no FRP debonding was observed.

Adhikary et al (2004) performed tests on eight RC beams reinforced in shear with CFRP strips using two different wrapping schemes; U-wrap and two sides of the beam. They investigated the effectiveness of cross layers over each other, vertical and horizontal; the main parameter, the fiber alignment direction (90°, 0° and 90° + 0°) and the number of layers (1 and 2). They observed that the maximum shear strength was obtained for a beam with full U-wrapped strips having vertically aligned fibers. Horizontally aligned fibers also showed enhanced shear strength compared to beams without CFRP. On the other hand, they found that the lowest concrete stress was for the same load range among all beams. A fully U-wrapped beam of one layer of CFRP with vertically aligned fibers achieved a maximum 119% increase in shear strength. Also, they compared with experimental values using models to predict the contribution of web shear to the shear resistance of CFRP bonded beams.

Al-Amery (2006) tested six RC beams with different combinations of CFRP strips and strips in addition to an unstrengthened beam serving as a control test. CFRP strip for flexural strengthening and with CFRP straps for shear strengthening or with a pair of CFRP sheets and straps, for total strengthening. They determined that the CFRP strips used significantly reduce the slippage of the joint between the CFRP strips and the concrete section. CFRP straps used to anchor CFRP strips increase the bending strength by up to 95%. only a 15% increase was achieved using CFRP strips alone. The results of the tests and observations showed that a significant improvement in the strength of the beam was achieved by joining CFRP strips and straps. That is, a more ductile behavior was obtained because debonding was prevented.

Anil (2006) improved shear capacity of RC T beams using unidirectional CFRP composites and compared experimental and analytical used ACI Committee report. From the results, it was observed that the stiffness of the beams is very close. It was also observed that the strength and stiffness of the specimens improved with the use of unidirectional CFRP. On the other hand, the analytical tensile load capacity showed a 20% lower value than the experimental bearing load, thanks to the successful anchoring performance.

Bencardino et al (2007) presented an experimental and analytical investigation of the shear strengthening of reinforced concrete rectangular beams wrapped with carbon strips (CFRP). The main variables include external anchorages with different lengths in the form of U-shaped steel ties. The results showed that the anchorage system improves the strength and deformability properties of the CFRP beam. Also, the anchoring system changes the mode of failure of the reinforced RC beam under a predominant shear force, without increasing the load capacity, to a more ductile failure with a significant increase in the load capacity almost to bending failure.

Jayaprakash et al. (2008) made an experimental investigation of the shear strengthening capacity and failure mode of precracked and uncracked RC beams externally bonded with bidirectional carbon fiber reinforced fabric (CFRP) strips. Twelve RC T-beams were produced with different internal longitudinal and shear reinforcements. These beams are subjected to two types of loading: three-point and four-point bending systems. Beams are classified into three categories, namely; control, pre-repaired and repaired and initially reinforced (ie untreated) beams. The overall increase in shear strength of the pre-rehabilitated and initially strengthened beams ranged between 13% and 61% greater than their control beams. It was found that the application of CFRP strips in pre-rehabilitated beams achieved better performance compared to originally strengthened beams. It was also observed that all reinforced beams reached premature bending failure due to the presence of an excessive amount of shear reinforcement.

Jayaprakash et al (2008) conducted tests to study the shear capacity of precracked and uncracked reinforced concrete beams with bidirectional CFRP strips on the outside. The experimental program consisted of six samples that were classified into two categories; named BT and BS, each category had eight beams, four control beams, six precracked/repaired beams and six initially strengthened specimens. Based on the results, it was observed that the outer CFRP strips act as shear reinforcement similar to steel forks. They also showed that increasing the ratio of longitudinal tension reinforcement and the spacing of CFRP strips affect the shear capacity. This research showed that the orientation of CFRP strips not only affects the cracking pattern but also affects the shear capacity.

Godat et al. (2010) studied to gain a clear understanding of size effects for carbon fiber reinforced polymer (CFRP) beams. Their experimental research investigates the shear performance of rectangular reinforced concrete beams strengthened with CFRP U-strips, as well as one fully wrapped with CFRP strips. All beams were strongly reinforced in the bending region, no steel ties were placed in the right shear range of interest, but in the left shear range they were placed to ensure that failure would occur in the shear range. From these results, they noted that CFRP's larger beam size improves shear capacity less. They investigated the crack behavior of these samples. Their research presented a comparison of test results and predictions from design guidelines.

Bukhaari et al. (2010) studied the shear strengthening of reinforced concrete beams with carbon fiber reinforced polymer (CFRP) sheets. Seven, two span continuous reinforced concrete rectangular beams.

One beam was not strengthened (control beam), and the remaining six were strengthened with different arrangements of CFRP sheets. They studied fiber orientation (0/90 and 45/135) as the main variable. Tests have shown that it is useful to orient the fibers on the CFRP strip at 45° so that they are approximately perpendicular to the shear cracks.

HK Lee, SH Cheong, SK Ha, and CG Lee (2011) investigated the behavior and performance of deep reinforced concrete shear beams strengthened with CFRP strips. A total of fourteen reinforced concrete deep T-section beams are assumed to have shear failure. The specimens are reinforced with longitudinal steel and forks near the mid-span. They also studied variables such as reinforcement length, fiber direction combination of CFRP sheets and anchorage using U-wrapped CFRP strips, these variables have a significant influence on the shear performance of reinforced deep beams. Their tested experimental results of T-section beams were considered deep beams, since the ratio of shear span to effective depth (a / d) is 1.22. On the other hand, the crack patterns and the behavior of the examined deep beams during the four-point load tests were observed.

Alferjani MBS, et al. (2014) also compared works in the field of strengthening RC beams using carbon fibers. This paper reviews 10 articles and attempts to address an important issue encountered in the shear strengthening of carbon fiber reinforced beams. One of the conclusions drawn is that there are no design guidelines for the optimization and selection of CFRP tape/laminate thickness for strengthening RC beams. A beam strengthened with multiple layers of CFRP laminates unnecessarily increased the strengthening time and costs compared to a single layer CFRP laminate.

The importance of the study for beam strengthening using CFRP laminates in the strengthening system provides an economical and versatile solution for extending the service life of reinforced concrete structures. From the reviewed literature, it is evident that epoxy resin is suitable for strengthening the removal.

Ascione L., Mancusi G. and Spadea S. (2010) presented experimental results on ten (10) samples of RC beams in order to highlight the special properties of concrete elements reinforced with CFRP bars and stirrups. A comparison between experimental evidence and theoretical predictions is also presented. In this regard, predictions are established using force balance and strain compatibility equations, assuming the following hypotheses:

- preservation of the cross-section plane;
- perfect reinforced concrete connection;
- no shearing deformations.

Further, for the comparison of experimental values, partial factors and environmental factors are assumed to be unique. Finally, research conclusions are given stating that the prediction formula proposed by CNR-DT 203 for estimating shear deflection loads appears to greatly overestimate the actual strength of FRP-reinforced members. The moment-curvature relationship of FRP-reinforced elements is basically linear in both pre-cracking and post-cracking stages, no corresponding contribution in terms of stress reduction or better stiffness appears. Experimental results significantly assess the reliability of prediction formulas, both for deviations and for crack width. Neglecting the effects of reduced stresses in the calculation of bending or member failure leads to good agreement with experimental data. The experimentally observed cracking phenomenon is more important than the theoretical one. The authors state that more experimental research is needed in the future in order to better predict the actual mode of failure.

Dan S., Bob C. (2018) and colleagues conducted experimental research with the aim of analyzing the behavior of modern and effective solutions - for the rehabilitation of reinforced concrete frame structures. In the paper, experimental researches of carbon fiber-reinforced polymer (CFRP) were carried out, which are used as a solution for strengthening frame constructions made of reinforced concrete (RC), which are assumed to be existing structures, and which were tested as non-reinforced and as (CFRP) reinforced structures. The experiment focused on RC frames, and single RC frames were constructed, all according to the Romanian codes from 1970, according to which dimensioning for seismic influences is inadequate for today's influences on the structure. Experimental investigations carried out on the RC frame structure highlighted some main aspects of the CFRP strengthening system: a slight increase in the resistance capacity by 5% at the ultimate load-bearing stage and a reduction of the top displacement by 27% at the serviceability stage.

Song and Hwang, (2004) investigated the mechanical properties of concrete reinforced with high-strength steel fibers, with different fiber content (volume fraction 0 - 2%). The researchers proposed different relationships from the test results to predict the compressive strength, split tensile strength and modulus of rupture of high strength steel fiber reinforced concrete, knowing the values of high strength concrete and the volume of added fibers. It was seen that the compressive strength increases with the fiber content up to V f = 1.5%, as shown in Figure 1. The change in tensile strength and modulus of rupture followed a linear relationship with the fiber volume fraction. Both strength parameters increased with fiber content (Figure 2).

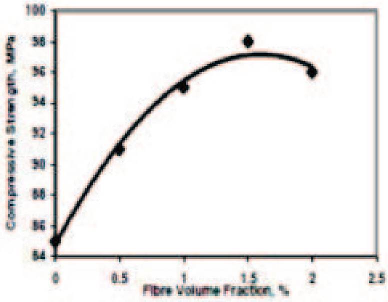

Figure 2 . Relationship between compressive strength and fiber volume fraction [13]

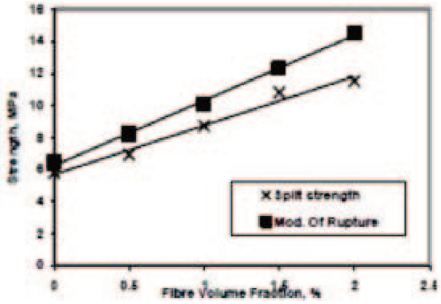

Figure 3. The relationship between tensile tensile strength, splitting modulus and fiber volume [13]

Balendran et al (2002) conducted a series of experiments to investigate the effectiveness of fiber inclusion in improving the mechanical properties of concrete with respect to concrete type and sample size. Light aggregates and limestone concrete with and without steel fibers were used in the research. It was observed that the small volume of steel fibers increased the cylinder splitting tensile strength and the modulus of rupture, although it had little effect on the compressive strength itself. The effectiveness of fiber reinforcement depends on the properties of the concrete matrix in question. With the same fiber type and volume, the improvement in tensile strength and flexural strength was much greater for lightweight concrete than for normal weight concrete. There was no effect of size on the tensile breaking strength of normal weight prism and lightweight aggregate plain concrete. In the case of fiber-reinforced concrete (both normal and lightweight), the size effect was not significant when the sample size exceeded the critical (transitional) size of 150 mm. The toughness indices of fiber-reinforced concrete are not very sensitive to sample size. On the other hand, for normal weight fiber-reinforced concrete, the toughness indices became smaller when the sample size increased. Therefore, the effect of size on toughness should be considered when designing the ductile behavior of fiber-reinforced structures. Further research is needed to examine the effect of size on toughness.

Bencardino et al (2008) presented compression test results on cube and cylindrical specimens of plain and fiber reinforced concrete with fiber volume of 1%, 1.6% and 3%. The compressive strength was evaluated experimentally and the corresponding stress-strain curves were also recorded on the cylindrical specimens to highlight the role of the fibers in the response after reaching the ultimate load. The results of the strength test show that the shape of the test specimen affects the value of the compressive strength. The results of the experiment also emphasized that with proper mix design, consistent quality fiber concrete can be produced in the field. Therefore, the results confirm that the most significant contribution of fibers in concrete is a significant improvement in behavior after reaching the ultimate load capacity, both in compression and tension. This is mainly because the fibers continue to resist crack growth and propagation after the first crack appears and allow the concrete to withstand very high stresses, five to six times the value of ordinary concrete. Increasing the fiber content improves post-peak behavior and an extended softening branch is observed. SFRC samples with fiber contents of 1.6 and 3% show, at a strain of 0.01, residual stresses of about 74 and 78% of their peak stresses. At these amounts of fibers, the ultimate stress at failure reaches values three to five times the ultimate stress values.

Job Thomas and Ananth Ramaswamy (2007) presented their results with an experimental program and analytical evaluation of the influence of fiber addition on the mechanical properties of concrete. The obtained models are based on the regression analysis of 60 test data for different mechanical properties of concrete reinforced with steel fibers. The study showed that fiber-matrix interaction significantly contributes to the improvement of mechanical properties caused by fiber incorporation. Therefore, the following conclusions were drawn from their study: there was only a small increase in compressive strength, modulus of elasticity and Poisson's ratio (less than 10%) in various types of concrete due to the addition of steel fibers; The maximum increase in tensile strength, namely tensile strength and modulus of rupture due to the addition of steel fibers has been shown to be about 40% in various types of concrete and is the basic justification for the use of fibers in concrete. The post-cracking response is significantly improved by fiber dosages throughout the different classes of concrete. It has been shown that the highest stress increase corresponding to the peak compressive strength in various types of concrete is about 30%. Enhanced peak stress capability is another significant benefit derived from the use of fibers. The proposed strength prediction models can be used to estimate the strength properties of SFRC based on the concrete and fiber class index $(RI = V_f L_f / \varphi_f)$.

Cucchiara et al (2004) investigated that the inclusion of fibers in an adequate percentage can change the brittle failure mode that characterizes shear failure to a ductile failure mechanism, thus increasing energy dissipation. It also confirmed the possibility to achieve similar performance by using reinforcing fibers instead of increasing the amount of transverse reinforcement in the form of forks. Testing was done for two ranges of shear to effective depth ratios to study the behavior in two different failure modes. In the case of FRC beams, a more progressive cracking process with reduced crack widths was observed. It was also possible to compare the ultimate strength by using steel fibers as shear reinforcement in the appropriate amount instead of the forks, although the simultaneous use is more appropriate because the forks allow a greater ability to deform beyond the elastic limit. Additionally, the presence of fibers has been shown to be more effective in beams where failure in the absence of adequate reinforcement governs the effect of the beam.

Rao and Rama Sheshu (2006) tried to investigate the useful possibilities of steel fibers in reinforced concrete elements with either longitudinal reinforcement or transverse reinforcement, as well as the potential of these fibers in concrete to act as longitudinal reinforcement or transverse reinforcement in a beam where either reinforcement was not present. They conducted tests of the torsional behavior of RC beams as torsional stiffness; torsional stiffness and torsional toughness of members play a vital role in the analysis of buildings exposed to seismic and wind loads. The pure state of shear stress due to torsional loading induces the main diagonal tensile stress which is mainly responsible for failure of ordinary concrete member under pure torsion. It was seen that the addition of steel fibers at about 1.2% improves the torsional toughness of the non-fiber beam by about 200% and the torsional stiffness by 148%. This indicates that a single type of reinforcement does not help to improve the torsional strength of beams beyond the occurrence of the first visible cracking moment. Therefore, it can be concluded that the ultimate stress strength of beams with one type of reinforcement can be limited to the torsional strength of unreinforced fibrous or non-fibrous elements. The presence of fibers forces the beam to behave ductile to some extent delaying the crack progress. However, the fibers present in the matrix improve the torsional toughness and torsional stiffness of the members. The steel fibers thus improve the breaking moment of the members to a noticeable extent, which improves the performance of the member in aggressive environments.

Rao and Rama Seshu, (2005) developed an analytical model to predict the moment response and SFRC members subjected to pure torsional loads by considering the softening effect of concrete. The addition of steel fibers only slightly improves the maximum torque capacity, but improves torque and torsional toughness to a greater extent. The durability of FRC was confirmed by experiments that corrosion in SFRC is less, compared to steel bars. The study clearly indicated that the main factor that facilitates corrosion is the rupture of the tight bond of the fiber-cement matrix, which is caused by the slippage of the fibers that follow the crack opening. The flexural strength of cracked specimens also increases due to corrosion as the surface roughness causes difficulty in sliding which ultimately results in an increase in strength.

Rapoport et al. (2002) investigated the relationship between permeability and crack width in steel fiber-reinforced cracked concrete. They also examined the influence of steel fiber reinforcement on the permeability of concrete. This indicates that fiber-reinforced concrete is subject to inelastic (irreversible) deformation compared to unreinforced concrete. From the test results, it can be seen that at higher levels of cracking, the reinforcing steel fibers clearly reduce the permeability, which is most likely a consequence of the stitching and multiple cracking that the steel fibers have. It is possible that a higher volume of fibers will further reduce the permeability of cracked concrete. However, at some fiber volume, an optimal level can be reached, above which more fibers will increase throughput.

Nataraja et al. (2005) investigated the resistance of OPC and SFRC to the influence of compressive strength for a relative comparison of the response of these mixtures. The addition of steel fibers has been shown to significantly improve the impact resistance of concrete and is therefore a suitable material for structures exposed to impact loads. A drop weight impact test, also known as a repeated impact test, was performed to assess impact resistance. The number of blows for first cracking as well as the number of blows for ultimate failure increased with increasing fiber volume fraction for all mixtures. Plain concrete specimens failed to brittle failure because they do not possess significant cracking resistance, and the concrete only resisted a few additional impacts after cracking. In the case of ordinary concrete mixes of 30

MPa, the specimens failed immediately after the opening of the first crack, and the broken pieces touched the positioned gears of the device with the addition of several impacts. In the case of the 50 MPa mix, the specimens resisted several additional impacts before contacting the girders. However, in the case of SFRC, this increase in crack resistance is approximately 40% to 60%, depending on the fiber content. The increase in the number of blows for the first crack is significantly higher in the case of SFRC.

Singh and Kaushik (2003) presented a study on the fatigue strength of steel fiber reinforced concrete. Tests of steel fiber-reinforced concrete are mainly limited to determining the limit of its bending capacity for different fiber types/volume fractions/ratio. The obtained fatigue equation can be used to obtain the flexural fatigue strength of steel fiber reinforced concrete for the desired level of resistance. However, more research is needed to determine how fiber type and ratio affect results.

Song et al (2005) investigated the impact resistance differences of high strength steel fiber reinforced concrete (HSFRC) and high strength concrete (HSC). Studies already show that hook-ended steel fibers have good potential for enabling concrete to withstand higher impact loads and that the fibers provide at least a five-fold increase in impact resistance, compared to straight steel fibers. It was also reported that steel fiber concrete is six times better at receiving impact loads than "ordinary" concrete.

Khaloo and Afshari, (2005) investigated the effect of length and volume of steel fibers on the energy absorption of concrete slabs of different concrete strengths by testing 28 small reinforced concrete slabs (SFRC) under load. The presence of steel fibers in the concrete did not significantly affect the final strength of the panels, which manifests itself with the appearance of large cracks. The fibers did not significantly affect the flexible characteristics of the panels before cracking. Also, the increase in fiber content reduced the growth rate of total energy absorption. The panels did not show any visible cracks before reaching ultimate strength. Ordinary panels failed under load without any significant warning. The addition of fibers does not significantly increase the ultimate flexural strength of SFRC panels. However, the panels' ability to absorb energy has improved. The energy absorption of panels with a fiber volume of 0.5% was about 12 times higher than a regular concrete panel. Panels with 1.0% fiber volume experienced energy absorption about 2 times higher than the 0.5% fiber panel. For this reason, experiments recommend the use of fiber volumes in the range of 0.75 - 1.75. Longer fibers (ie fibers with a higher aspect ratio) provided greater energy absorption. The absorption of the 35 mm fiber was about 1.2 times that of the 25 mm fiber. Increasing the strength of fiber concrete increases the energy absorption capacity. The test also concludes that fibers have a similar effect on the behavior of slabs of different concrete strengths.

Altun et al. (2007) investigated the influence of steel fibers on different classes of concrete. By searching the relevant literature, it can be noted that the optimal SF dosage for SFARC beams should be within 1 - 2.5% of the absolute volume. A dose of SF less than 1% becomes ineffective, and doses greater than 2.5% also become ineffective mainly due to physical difficulties in ensuring a homogeneous distribution of fibers within the concrete, which causes a significant drop in compressive strength when compared to ordinary concrete for the same class. Here, it is believed that SFARC beams having steel fibers in a dose of about 30 kg/m^3 should be suitable or even a common practice, because firstly, crack formation, crack size and crack propagation in beams according to bending moments are significantly better, secondly, the ultimate capacity a little better in bending, and thirdly and most importantly, the toughness is much higher than in an RC beam with the same usual reinforcement, but without steel fibers. Therefore, it is clearly stated that a dosage of steel fibers of 30 kg/m^3 is recommended.

Hatami F. et al. (2013) conducted research on an eccentrically loaded RC column with and without reinforcement. Stiffness, failure force and energy absorption of RC columns were tested. The results showed that by increasing the axial load, the deep cracks in the RC column without the CFRP layer were widened, and the bending stiffness was reduced until failure. After fracture, the tensile force in the longitudinal reinforcement increased slightly. Also, the behavior of RC columns without CFRP reinforcement showed that the maximum curve was in the middle of the column and that the maximum transverse stresses were recorded in the compression zone. The authors determined that the presence of CFRP reinforcement can increase the axial load capacity of the RC column by up to 108%, while an increase in the longitudinal displacement of 13% was measured for the CFRP sample compared to the unreinforced (control).

Also, along the way secant stiffness sample In 300 it was is 14.83 kN/mm, a for S 300-DD pattern about 34.5 kN/mm what is showed increase of 132.64% in relationship on unreinforced sample ar elative absorption energy sample In 300 it was is equal to 1273 kN/mm, a for S 300- DD pattern is was about 2977 kN/mm , what is increase of 133.9% in relationship on unhardened sample .

Jingjing Zhang and colleagues (2017) addressed the area of durability of RC structures reinforced with carbon fibers, as well as determining the mechanical resistance of the fibers themselves. The authors proposed a test in which chopped carbon, aramid and hybrid carbon-aramid fibers (1:1) were added to concrete in order to fully utilize the high strength and toughness of the fibers and improve the properties of concrete. The influence of different types and mixing ratio of fibers on the mechanical properties and durability of concrete was analyzed using axial compressive strength, static modulus of elasticity and carbonization tests. A calculation model, based on axial compressive strength, carbon dioxide concentration and carbonation time of concrete, was developed with existing models for calculating the depth of carbonation and verified using experimental data. The results showed that the compressive strength, elastic modulus and anti-carbonation capacity of fiber-reinforced concrete initially increased and then decreased with increasing fiber content and were better than plain concrete. The compressive strength and modulus of elasticity of hybrid fiber-reinforced concrete with a mixing ratio of 1% increase the most among all tested concrete samples, and the anti-carbonation capacity of carbon fiber-reinforced concrete with a mixing ratio of 1% is the best, and the proposed method can be taken to evaluate the practical properties and durability of fiber concrete in practical engineering.

In paper [18] , the application of CFRP strips on RC structures was investigated . From the results shown, it can be seen that the reinforced beams were stronger compared to the corresponding control beams. This difference becomes more pronounced in the shooting zone. The load/displacement curve is more pronounced for group 2 beams where the initial cracking load of beam CB-2 is lower than CB-1. It can also be noted that the load deflection curves of reinforced beams with end anchorages are stiffer compared to beams without end anchorages. The curves for the reinforced beams showed an identical response in the initial stage of loading, but as the load increased the beams without end anchorages began to deviate more due to excessive cracking at the ends and the tendency of the envelope to overturn. This effect is more pronounced in beams of group 2. From the tests of RC beams reinforced with CFRP sheaths along their length with and without anchorages, the following conclusions can be drawn: RC beams wrapped with CFRP at the bottom and extended on the sides intended for anchorages and without ends, improved structural performance of beams in terms of stiffness, load capacity and ductility; CFRP-strengthened RC beams were found to have the occurrence of fine cracks uniformly distributed along the span and a higher first crack load as high as 20.9% in WSAB-1 and 79.5% in WSAB-2 compared to their control beams; The ultimate bearing capacity of reinforced beams increased to a maximum of 16.7% compared to the corresponding control beam in Group 1, while in Group 2 it increased to a maximum of 20.8%; Reinforced beams tested at a lower a/d ratio were found to fail in shear and compression, even though they had adequate shear reinforcement. The difference in failure modes in reinforced beams with a smaller a/d ratio is due to the presence of end anchorages. However, beams tested with a higher a/d ratio fail to flex as expected. It has been observed that the use of CFRP sheaths results in a significant increase in the stiffness and ductility of RC beams, along with an increase in the load-carrying capacity of reinforced RC beams with end anchorages.

Koutas N. Lampros, Tetta Zoi et al. (2019) gave a state-of-the-art overview of the reinforcement of concrete structures with textiles. The paper states that there has been a significant improvement and upgrading of the textile materials themselves, where the characteristics have been improved from the aspect of detheorization, suffering from the effects of the environment, from the consequences of non-maintenance, etc. The advantages (high strength-to-weight ratio, corrosion resistance, easy and quick application, etc.) as well as the disadvantages of using FRP (textile) materials (high prices of epoxy resins, poor performance at high temperatures, poor adhesion to the substrate, etc.) are listed.), and new methods of strengthening with textiles are described, where in fact a new material is applied - textile reinforced concrete (TRM). This paper provides an overview of the use of the TRM system for strengthening, where certain segments of the load-bearing structure are analyzed in particular. The paper also provides an overview of methods for strengthening RC beams or plates from the aspect of bending and from the aspect of shearing, where the methods are described, general behavior due to strengthening, modes of failure, influence of parameters on the effect of strengthening, aspects of dimensioning, strengthening against shearing, improvement of concrete with the aim of increasing load and bearing capacity (deformation), seismic strengthening of RC structures, etc.

Litvinov A. (2010) dealt with the area of application of carbon fibers in his master's thesis in constructions. The purpose of this research was to investigate production technologies, properties of carbon fibers, construction methods, and to compare carbon fibers and structures, using carbon fibers with traditional materials and structures, and to discover the reasons for their application in construction. In addition to all that, the task was to create a calculation tool in Excel to evaluate the effect of carbon fiber reinforcement on the structure. In the paper itself, an overview of the use of carbon fibers in construction is given, so examples of the use of carbon fibers in the form of reinforcement of concrete elements, production of prefabricated elements (wall panels), use in bridge construction, etc. are given. At the end, a calculation was given in the software (Excel) with the aim of determining the effect of reinforcement. The calculations, which were made by the reinforcement manual in accordance with SP 52-101-2003, show a high efficiency of carbon fiber reinforcement. The main drawback of this work is the absence of experimental research on real samples and comparison with software.

Luiz Antônio Melgaço Nunes Branco and colleagues (2014) performed experimental research on concrete samples loaded under pressure reinforced with carbon tapes, with the help of the ABNT NBR 5739 standard and analysis of variance (ANOVA) - Brazilian standard. The obtained test results are shown in the tables, where it can be seen that the average strength values increased in all three types of tested concrete with the addition of laminated carbon fiber composite as reinforcement. The addition of carbon fibers enabled low-strength concrete to have an efficiency factor of 258.60, or 62.10 and 2.70 for medium- and high-strength concrete, which shows that the most effective reinforcement is for low-strength concrete, justified by lower tensile strength. The pictures show the brittle behavior of the reinforced concrete samples, where it is evident that the failure occurred in the carbon reinforcement, while the concrete section remained relatively undamaged. From the obtained results, it was established that the increase in strength was caused by the use of composite material in three classes of resistance. However, it was also observed that the efficiency factor of carbon fiber reinforcement is significantly higher at the lowest specific strength. As the resistance of reinforced concrete increases, the percentage of increase in performance of the applied system decreases (258.6% for low strength and 22.7% for high performance).

Mohammad Reza M., et al. (2009) conducted torsional load research on three groups of rectangular beams, consisting of sixteen samples with different amounts of torsional reinforcement. The aim of this research was to assess the impact different ratios of torsional reinforcement on the torsional behavior of reinforced beams. Wrapping of carbon fiber reinforced concrete (CFRP) beams consisted of various configurations, including anchored U-wrap, full wrap, and strip wrap. This study showed that the contributions of CFRP to the torsional strength of reinforced beams, which have an identical volumetric ratio of CFRP reinforcement, are quite dependent on the total amount of torsional reinforcements. CFRP contributions will increase as the torsional reinforcement of the steel increases. The experimental results show that increasing the steel reinforcement by 37% and 94% increases the CFRP contribution to the torsional strength up to 54% and 91% for reinforced beams with one CFRP layer; and up to 60% and 111% for reinforced beams with two CFRP layers, respectively. In this work, the influence of the number of CFRP layers was also investigated. It was found that the increase in the CFRP contribution to the torsional strength of beams strengthened with one layer and two layers of CFRP strips is close for different ratios of steel reinforcement compared to the increase in the total amount of steel reinforcement.

Mosallam AS (2000) investigated the strength and ductility of reinforced concrete frame joints strengthened with quasi-isotropic laminates with the aim of establishing new techniques for strengthening damaged parts of the frame. Six cyclic tests were performed on samples reduced twice, simulating the beam-column connection of a typical RC structure. The test results showed that the use of complex layers led to a significant increase in the stiffness, strength and ductility of these connections. The ductility and hardness of the reinforced samples increased by 42% and 53%, respectively, compared to the control samples.

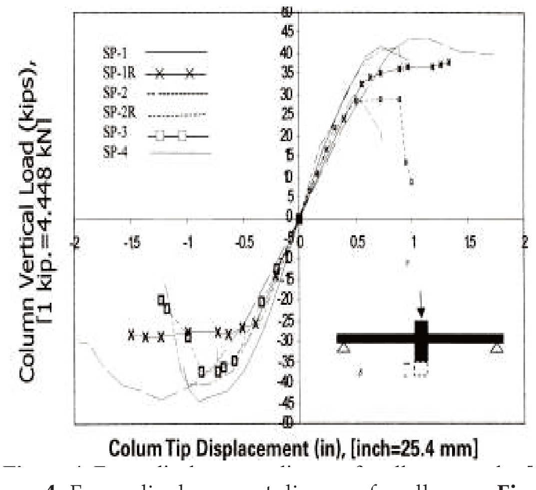

Figure 4. Force-displacement diagram for all test samples [26]

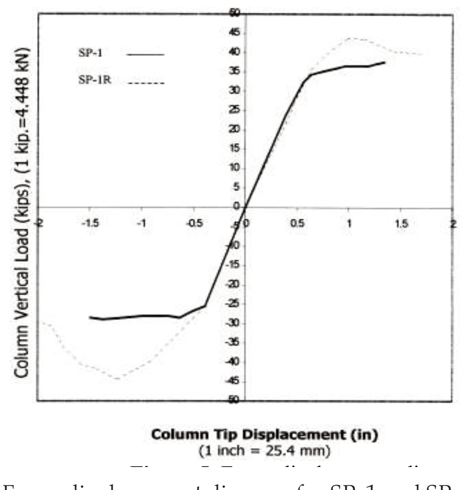

Figure 5. Force-displacement diagram for SP-1 and SP-1R [26]

From the presented diagrams and research results (Figure 4 and Figure 5), it can be seen that the use of the composite system not only restored the original capacity of the damaged sample, but also improved the ultimate load capacity by an additional 53% in the stressed zone and about 18% in the tightening. Also, the displacement at the ultimate load increased by about 29% in tension and 36% in the compression zone compared to the ultimate displacement before the damaged control specimen SP-1. Experimental results showed that the use of quasi-isotropic polymer composite laminates increases both the rotational stiffness and the ultimate strength of the reinforced concrete instant connections. The use of quasi-isotropic glass ± epoxy composites contributed to a significant increase in stiffness, strength and ductility of the repaired joint. For example, a 42% increase in the ductility of the repaired specimen (SP-1R) was achieved (3.40 vs. 2.4 for the control specimen SP-1). Cohesive failure was achieved in all tests.

Norris T., H., Ehsani RM (1997) presented the results of an experimental and analytical investigation of the behavior of damaged RC beams reinforced with carbon strips (CFRP). Carbon strips are glued with epoxy glue in the tension zone of the RC beam with the purpose of improving the shear and bending strength of the beam. In this paper, the authors consider the application of carbon strips wrapped around AB beams with the aim of increasing shear and bending resistance. These strips are made of high-strength carbon fibers, and since carbon-epoxy composite systems are not susceptible to corrosion, the problem of corrosion in reinforced concrete is automatically solved with the use of this material.

Analysis of the behavior of the bending resistance of the tested beams revealed the occurrence of deformations only after a load of 31 kN. By comparing the result of the reinforced beam with the control beam, it can be seen that the load at which the stress in the reinforcement was allowed was higher in the reinforced beam, which means that the internal forces are distributed to the carbon fibers and the reinforcement. Carbon fiber reinforced beams show higher ultimate strength than control beams at increasing loads up to 138 kN before sudden failure. This type of failure occurs due to the CFRP strips tearing off from the concrete.

The shear resistance of the tested beams investigated in this paper shows that with the application of CFRP reinforcement, the shear strength increases, regardless of the orientation of the carbon reinforcement. There is also a direct relationship visible between the increase value and brittle failure. In both cases (shear and bending) when the carbon reinforcements were installed perpendicular to the cracks, the results gave higher stiffness with brittle failure. When two layers of carbon reinforcement were used, where the second layer was installed perpendicular to the first, they usually showed the appearance of cracks and the failure still occurred suddenly, because the carbon reinforcements (part of them) were perpendicular to the cracks. The authors' conclusions in this paper show that CFRP reinforcements can increase the bending and shearing capacity of existing AB beams. The strength gain value as well as the fracture behavior model is related to the orientation of the carbon reinforcement. When CFRP reinforcements were placed perpendicular to the cracks, a large increase in stiffness and strength was observed, and brittle failure occurred due to cracking of the concrete as a result of stress concentration near the ends of the CFRP. When carbon reinforcements were placed parallel to the cracks, a smaller increase in shear strength and stiffness was observed.

4. CONCLUSION

Considering that the use of carbon fibers is increasingly prevalent in constructions, this paper systematizes previous research in this area with the aim of emphasizing the advantages and disadvantages of using carbon fibers. The most common application of carbon fibers in practice so far has been for the purpose of additional strengthening of structures, especially in the tension zone, where the application of carbon fiber reinforcement has proven to be extremely useful in terms of improving tensile load capacity, etc. In this paper, an overview of the author's research is given, and in one place, aspects with the possibilities of applying carbon reinforcements in constructions can be seen.

Based on the extensive analysis, it can be concluded that when reinforcing RC elements with carbon fibers, the main attention should be paid to the following factors:

1. Long-term degradation of mechanical properties. Depending on the type of CFRP reinforcement, the long-term strength can be reduced two to three times (compared to the short-term value). The maximum reduction in strength is associated with CFRP; however, other fibers are also sensitive to the effect of time. Moreover, creep is characteristic of most polymer resins applied in CFRP,
2. Correct selection of CFRP material due to environmental influences. Most of the materials used to make CFRP are not resistant to special environmental influences,

3. The characteristics of the adhesion bond is the leading criterion for the analysis of deformation states. In most cases, the design of CFRP RC elements is based on the application of increased safety factor values. However, recent research by the authors [8] revealed that such a methodology is too rough because deformations mainly depend on the bond properties of CFRP composites. Therefore, it can be said that, in order to increase the effectiveness of the application of structural composites, the sizing practice had to be based on experimental tests related to the joint properties of certain FRP materials,
4. Finally, carbon fiber composites are widely used as an addition to building structures. The main advantages of using this material are the achievement of higher tensile strengths and low weight, but there are still problems with cost.

Note: The work was done as part of the research carried out as part of the doctoral studies in the narrower scientific field of Civil Construction.

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INFRASTRUCTURE CATEGORISATION FOR RE-USE OF COAL MINE „LIPNICA"

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SUMMARY

Summary: In the northern synclinorium of Kreka coal basin, lignite mine ″Lipnica″ was opened in 1950 in Lipnica settlement nearby Tuzla city. During the exploitation period, mine produced 40 million tons of lignite. By extracting the coal reserves, mine was shut down in 1991. Underground caves were filled by classical methods present in underground mining and all entries are sealed. The ground infrastructure remained preserved and has had usable values in three periods when infrastructure was used for various purposes. Conversion for re-use of mining infrastructure is not defined by any categorisation for infrastructure conversion use as the planning element of sustainable development for active or shutdown mines. The article defines categories of sustainable development based on utilizing mine infrastructure on the sample of ″Lipnica″ mine. The universality of categorisation points out the possible options of optimal re-use of coal mine infrastructure, indicating implementation of a mining company sustainable development concept, either in government or private ownership.

Keywords: categorisation, sustainable development, business incubator, Lipnica mine

1. INTRODUCTION

Lipnica mine was located in the northern synclinorium of the Kreka coal basin, where organized and planned lignite exploitation began in 1950. The determined reserves of lignite at the Lipnica location were:

- The second roof coal seam: 25,490,942 t,
- The first roof coal seam: 6,181,470 t,
- The main coal seam: 6,282,864 t,
- The footwall coal seam: 6,479,412 t.

Total: 44,434,688 t

A number of infrastructural facilities for mining purposes were built in the area of the mine yard at the very beginning of exploitation: the administrative building, the building for the storing of mine materials, the cloakroom and the miner's bathroom and restaurant were built in the 1960s, while the infrastructure for the rescue squad and the fire department were built in 1977. In 1980, a machine workshop, a temporary warehouse and a workshop, a warehouse for finished products, a warehouse for reproduction materials, a renovated restaurant and a porter's office were built in the compound, and a year later an electrical

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workshop, an administrative building and an internal gas station were built. In 1985 and 1986, the transport power station "BEKORIT - KULI" was built, the call hall and mining bathrooms were reconstructed - by removing the chains on which the miners held the wardrobe, and tin cabinets were purchased for this purpose, the boiler room for central heating, ambulant within the compound, car repairing workshop and the archive for the mine.

The mentioned facilities were built in the safety pillar for the Joševica stream. All buildings are connected by modern asphalt roads from several sides.

The buildings were supplied with water from two independent sources: connection to the water supply network of the city of Tuzla and local supply through a deep well and a reservoir with a volume of 11,000 litters.

With the system of sewage network and septic tanks, with the help of the Joševica stream, the issue of waste water was solved.

Electricity supply was solved by the construction of a transformer station and the branching of the low-voltage network, so that all buildings in the vicinity of the Mine were electrified.

The mine had a telephone network installed in all facilities, and it was connected to an automatic telephone switchboard in the Lipnica settlement.

"Lipnica" mine ensured the construction of a modern swimming pool in the mine area, which still exists today, but is not in operation due to neglect.

As part of the "Lipnica" Mine, several facilities were opened that were indirectly related to coal production. First, the "Mining Overhaul and Maintenance" facility was formed, and then in 1956, the "Exploratory Drilling Section" (later renamed to OOUR "Drilling and Drainage").

After the economically interesting coal reserves were exhausted, "Lipnica" mine stopped coal production in 1991. The mining equipment was pulled out of the underground rooms, the old mine works and the entire mine were closed, and most of the openings, i.e. entrances to the pit are sealed. The coal separation facility was dismantled, and the entire area of the mine yard was closed. This ended coal production in the "Lipnica" mine, in which 40 million tons of lignite were produced from the beginning of exploitation to its closure, out of the planned 44.4 million tons.

Figure 1. Administration building of the Lipnica mine - today the administration of the incubation centre

Figure 2. Mine „Lipnica" 1949. - 1991. Year, produced 40.000.000 tons of coal

2. THE FIRST AND THE SECOND RE-USE OF „LIPNICA" MINE INFRASTRUCTURE

After of exhaustion of economically interesting of coal reserves, mine „Lipnica" stopped its production of coal, as planned in the year 1991. The workforce of "Lipnica" mine has been deployed to other mines and facilities within the "Kreka" mine company. The mine facilities were not demolished or devastated, and the accompanying infrastructure was not destroyed either.

After the cessation of coal production and the closure of the mine, part of the infrastructure facilities and installations began to be adapted for the installation of the machine factory "Perilica", in cooperation with the company "Končar" from Zagreb. Cooperation with the company "Konačar" was the first concrete step in the process of finding a solution for the sustainability of the mine infrastructure, which was preserved and planned.

War actions in the territory of the former Yugoslavia, that is, the Republic of Bosnia and Herzegovina, in 1992 stopped the aforementioned activities, in connection with the use of the "Lipnica" complex in Lipnica local community. In a very short time, the entire mine compound was occupied by a military formation and a barracks was formed, and the military units remained in that area until 2001. In the period from 1992 to 2001, the aforementioned facilities were used by the Army of the Republic of Bosnia and Herzegovina - until 1996, and after that, until mid-2001, by the FBiH Army[1.]. The reduction in the number of members of the armed forces led to the abolition of the army barracks of the Federation of Bosnia and Herzegovina in Lipnica. The buildings and land in the "Lipnica" complex were once again made available to the "Kreka" Mine company as the owner of the property.

3. THE THIRD RE-USE OF „LIPNICA" MINE INFRASTRUCTURE

In order to utilise the existing mine infrastructure, and already started ideas on re-use of place in sections for coal production, mine „Kreka", as owner of the complex, made agreement with the Municipality of Tuzla (today City of Tuzla), that the municipality establish a company, which will use space under favourable price of yard rent, by large number of small scale start-up companies. On that way, Municipality of Tuzla, established a public company JP RPC Tuzla - Public Company Tuzla, INCUBATOR „Lipnica", d. o. o., to whom mine „Kreka" gave miner' yard complex for rent in the year of 2004. Since the establishing the incubator, in private business were involved over 30 companies, and during the year of 2020, in the Incubator was located 31 private companies. Working activities in companies are production of cleaning items for disinfection, disensects and deratisation, production of wormwood, business management consulting, packing of liquid food products, production and selling of industrial oils and ubricants,

Figure 3. PC RBC TUZLA INCUBATOR „LIPNICA" D.O.O.

production of shoes, re-fixing and service of electric engines, production, packing and distribution of agricultural products, terminal reserve of stocks of material, purchase and installation of equipment for production and reparation of reserve parts for pumps and electro materials, production of eco-wormwood and eco-wood of the beech, administrative jobs, service of light weapons, production of natural juices,

[1.] In 2005, the Army of the Federation of Bosnia and Herzegovina was transformed into the Armed Forces of Bosnia and Herzegovina make-up, printing, packing and distribution of adhesive paints, filling – up of gas for houses and companies, certificating of small and middle growing companies, space for warehouses, productions of tea and natural oil, mechanical treatment, printing, production of mushrooms so-called „bukovace“, production of kids-toys by persons with special needs, production of ALUMINUM and PLASTIC carpentry, production of mechanical parts and pharmacy.

4. ELEMENTS OF CATEGORIZATION FOR RE-USE OF INFRASTRUCTURE AT “LIPNICA” MINE

The infrastructure of the ˝Lipnica˝ mine has a number of benefits that affect the conversion of facilities. It is important that the existing infrastructure has retained elements of functionality, except for a smaller number of devastated buildings or dilapidated buildings that can no longer have a useful value.

4.1 EXTERNAL INFRASTRUCTURE

The second grouping of functionality elements is non-mining, i.e. external infrastructure that is preserved and functional (asphalt road and connection with other parts of Bosnia and Herzegovina, railway).

The ˝Lipnica˝ complex is located northwest of Tuzla, 11 km from the city centre. It is connected by an asphalt road with a width of 7 m to the main road Tuzla-Doboj in Bukinje, and via this road to the main road Tuzla-Sarajevo, Tuzla-Orašje, Tuzla-Zvornik and Tuzla-Bijeljina.

The normal gauge railway Lipnica - Bukinje was put into service in 1953 and had the primary function of transporting coal from the Lipnica mine to the Tuzla thermal power plant and to other consumers. The railway was used for passenger traffic for a very short time during the war, when road traffic was difficult primarily due to lack of fuel. With the end of the war, railway traffic was suspended, and the tracks remained for possible future use.

The swimming pool is not in operation, and it is not in such a dilapidated state that it cannot be revitalized and renovated.

4.2 INTERNAL INFRASTRUCTURE

The ˝Lipnica˝ complex contains 26 built buildings with an area of 14 to 3,100 m^2, i.e. a total area of about 11,000 m^2, as well as a land area of about 160,000 m^2. The mine compound as a basis for assessing functional sustainability is well preserved, the buildings are of solid construction (the buildings have full or partial functionality), the internal road infrastructure (asphalted roads and driveways) and all accompanying elements of the infrastructure are functional (water supply, electricity, sewage network, telephones, surveillance, central heating). The regulated stream Joševica flows through the complex, which can be treated as a significant water resource.

Figure 4. River Joševica, flows through compound PC RBC TUZLA INCUBATOR „LIPNICA“ D.O.O, the trough has been completely renovated - concreted along its entire length

Based on the assessment of the condition of the facilities carried out by the Kreka mine in 2009, the elements of infrastructure categorization can be defined in terms of the new use (re-use) value of all infrastructure facilities. A total of 26 buildings of varying degrees of utilization and devastation were recorded, as well as the connection of the buildings with asphalt roads, water, sewerage network, and in some parts there is a communication system within the complex with a branched telephone network that is connected to the telephone exchange in the Lipnica settlement. Electricity is supplied through existing transformer stations and separate low-voltage electrical installations, with the fact that a number of buildings do not have an adequate electrical network due to outdated installations and devastation. Heating of buildings is solved by two built kettle's which using coal as power.

4.3 CATEGORIZATION OF CONVERSION OF INTERNAL AND EXTERNAL INFRASTRUCTURE

In the context of creating a categorization of mine infrastructure repurposing, all infrastructure facilities were analysed from different aspects in order to define the categorization of infrastructure repurposing. In the earlier period, a basic and group division of buildings was made into buildings that can be used without additional works, buildings that require work and buildings that are not profitable for renovation. Given that the mine compound in Lipnica has illustrated the possibility of repurposing mining facilities into production-business facilities, the same mine compound complex can serve as a model for defining the repurposing of other mining facilities that have infrastructure that has no use value, and which is rapidly deteriorating due to lack of maintenance and devastation.

In order to assess the possibility of reuse of facilities and the sustainable development of mining facilities, a categorization of the conversion of internal and external infrastructure was developed. Categorization is based on a detailed assessment of each object in terms of its physical and legal condition, the possibility of connection with external infrastructure, and an assessment of the possible conversion of the object in the segment of business activities and financial investments in infrastructure. The basic categories are:

- objects that can be used without investment,
- objects that need basic repair,
- objects that need basic infrastructure,
- objects that do not need investment in access roads,
- objects that require investment in renovation up to 5000 KM (2500 EUR),
- objects that require investment in renovation up to 50,000 KM (25000 EUR),
- facilities that require investment in renovation of more than 50,000 KM (25000 EUR),
- facilities that cannot be repurposed, objects that must be demolished/removed.

Each of the categories also contains subcategories that more precisely determine the categorization of the object and the possibility of conversion. The categorization of conversion has not been treated in the professional literature, so this paper is a scientific and professional contribution to the evaluation of the possibility of using the mine infrastructure and conversion of buildings into usable value. To illustrate the categorization, table 1 shows examples of categorization based on the assessment of the infrastructure of the former Lipnica lignite mine near Tuzla.

5. SUMMARY

The previous work of JP RPC TUZLA INCUBATOR "LIPNICA" doo Tuzla, in the period from 2004 to 2020, provided favourable conditions for the settling and business operations of 110 companies, in which 679 workers were employed. Out of the total number of companies that have used the services of RPC so far, 81 companies are incubated business entities, which started their business in this incubator and within which they had a very important form of support for a successful start. Today, in 2021, 31 private companies are located in JP RPC TUZLA - INCUBATOR "LIPNICA" doo Tuzla. The industrial yard - Lipnica complex is in extremely good condition, and the Incubator Centre itself has a tendency to expand and welcome new workers.

In order to use the infrastructure of mining facilities, of which there are a significant number in the area of the Kreka basin, an analysis of the possibility of repurposing facilities and the possibility of sustainable development of mining companies is necessary. Sustainable development is based on the repurposing of facilities and changing the business policy of the owner of the mine infrastructure. The assessment of the possibility or success of the conversion of facilities can be determined by categorizing the conversion of

Table 1. Example of categorization of infrastructure buildings on location of former mine „Lipnica“

Building	Building condition	Highness and square area	A year of build	Overall condition of building	Electric energy	Condition of electric parameter	Water installation	Condition of water installations	Heating	Kind of heating	Condition of heating installation	PTT	Condition of PTT installations	Roof	Re-use of the infrastructure	Category
Reception gate	Building made of bricks	Ground floor building, 18 m^2,	1980	Untouched Re-built	Yes	220 V 15 kW	No	Not in function No installation	Yes	Central heating Coal	Good in function	Yes	Good in function	Good Re-built, Downed 10°	Reception gate, Entry control, control of parking	Building without investment
Headquarter building wing 1	Building made of bricks	Basement + ground floor, the first floor + Base area 407 m^2,	1960	Untouched Re-built	Yes	220 V 60 kW	Yes	Good	Yes	Central heating Coal	Good	Yes	Good	Leaked by rain Sanitized	Office space, except basement which can be warehouse or manufacturing area	Building requests investment till 5000 KM
Headquarter building wing 2	Building made of bricks	Basement + ground floor, the first floor + Base area 97 m^2,	1960	Untouched Re-built	Yes	220 V 60 kW	Yes	Good	Yes	Central heating Coal	Good	Yes	Good	Bad Leaked by rain Sanitized	Office space, Except basement which can be warehouse or manufacturing area	Building requests investment till 5000 KM
Wardrobe and bathroom	Building made of bricks	Ground floor, + floor Ground floor: 1.394,25m^2 Floor 151,2 m^2	1970	Building re-built Roof and front part of facade	Yes	220 V 80 kW	Yes	Good	Yes	Central heating Coal	Good	No	Not in function Developed Installation in good form	Leaked by rain Sanitized	Manufacturing section	Building requests investment till 50000 KM
Restaurant	Assembly facility	Ground floor 428,22 m^2	1980	Re-built	Yes	220 V 50 kW	Yes	Good	Yes	Central heating Coal	Good	Yes	Good	Plate of salonite Not leaking	Manufacturing section, office	Building without investment
Ambulance	Building made of bricks	Ground floor 412,65 m^2	1986	Re-built	Yes	220 V 30 kW	Yes	Good	Yes	Central heating Coal	Good	Yes	Good	Sanitized totally	Manufacturing space, store	Building without investment
Rescue squad building	Building made of bricks	Ground floor 2.100 m^2 + 72 m^2 + 16 m^2	1977	By facade, building overflowed and devastated	Yes Off connection	220 V 30 kW	Yes	Good Not in function	Yes	Central heating Coal	Not in function	Yes	Not in function	Plate of salonite Not leaking	Not in use (Property of mine „The Kreka“)	Building need basic infrastructure
Storehouse	Building made of bricks	Ground floor 200 m^2	1980	Re-built and in good ondition	Yes	220 V	Yes	Good	Yes	Central heating coal	Good	Yes	In function	Plate of salonite Not leaking	Manufacturing area, storehouse	Building requests investment till 50000 KM

the internal and external infrastructure of the mine compound area. Categorization includes the spectrum of the status of objects, on the basis of which objects can be categorized and their new use value can be defined.

Stopping the operation of mining production facilities, regardless of the causes, implies the abandonment of the mine infrastructure, which is functional and can be repurposed. In the context of the conversion of facilities, it is necessary to foresee the conversion of future mines in the preparation of project documentation, especially in the Main Mining Project, with the aim of sustainable development of the mine, with all its facilities and infrastructure. Conversion activities must be carried out on time while the infrastructure has a useful value, and thus a more favourable financial valorisation, since over time the buildings deteriorate and lose value. When evaluating the possibilities of repurposing facilities, it is necessary to use the categorization of facilities in order to get the clearest possible picture of the possibilities of repurposing mining infrastructure.

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THE INFLUENCE OF BINDERS ON THE PROPERTIES OF DOLOMITE PELLETS FOR CALCIFICATION OF AGRICULTURAL LANDS

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SUMMARY

The research relates to determining the dependence of the strength and hardness, as well as the pH factor and solubility in water, of pellets made based on dolomite powder on the type and concentration of the binder. During the research, the technological parameters of the pelletizing device (speed and inclination of the disk, dosing speed of powder and binder suspension, and mixing time) were kept constant. Variations were made in the concentration of starch and molasses binder at values of 1.0, 2.5, and 4.0% and the control sample of 2.0 %. The purpose of pellets made of dolomite powder is to correct the pH factor of agricultural soil. The obtained results indicate that the production of pellets for the stated purpose requires an optimal choice of binder and its concentration due to the influence on the qualitative indicators of the pellet itself from dolomite powder. The research results are a partial extract of the Study: "Investigation of the optimal technology for the production of dolomite pellets from the "Očura" deposit for agriculture, Phase I".

Keywords: pellet, agriculture, binder concentration, strength, hardness, pH of pellets.

1. INTRODUCTION

In the conditions of climatic changes present in this period of civilization, as well as in intensive agricultural production with the use of artificial fertilizers, as a result, agrarian soils had a significant change in the pH factor in the direction of their acidification. Thus, managing acidic and highly acidic soils is one of the most critical issues of modern agriculture. Unfavourable chemical characteristics of acidic soils limit the growth and development of cultural plants, often to the point of justifying the use of these soils for the cultivation of various crops. If the pH value of soil falls below the optimal values, correction of the soil reaction is required, which is best carried out by calcification. Calcification increases production efficiency, has a favourable effect on soil biogenicity, and microbiological activity, and is ecologically completely acceptable for conventional and organic agriculture.

Calcification is carried out by different means; lime is the most commonly used. However, the tendency to protect the environment and the growing awareness of the impact of combustion products and carbon monoxide as a mandatory product on the air and consequently climate change significantly calls into question the justification of received lime from the aspect of an environmentally acceptable material. It is a well-known fact that in the production of lime (whether it is obtained from limestone or dolomite), during the thermal decomposition (baking) of Ca and Mg carbonates, up to 45% of CO_2 is released, which mostly ends up in the atmosphere. For this reason, the solution for the repair of agricultural materials is increasingly sought after in application, finely ground granulates of carbonate rocks, primarily limestone and dolomite.

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These rocks have a basic pH factor, so by introducing them into the soil, they repair the pH of acidic soils. However, the impact itself also depends on the level of comminution of the introduced soil repair products from these mentioned rocks. Increasing the fragmentation increases the activation of the mineral composition of the rock (mechatronic activation) with the chemical content in the soil, and the biological decomposition of carbonate rock grains intensifies to a considerable extent, which significantly increases the effect.

On the other hand, by reducing the grain size below 1 mm, significantly complicates the spreading technology itself, i.e. scattering and introduction of powdery particles obtained in this way. To that should be added the fact that in intensive agricultural production, machines and devices are used that have a technically solved system for spreading the necessary substances (mineral fertilizers) on the soil in accordance with the general physical condition of those products, i.e. the system disperses granules of a certain size. In order to use, almost, as a rule, the spreading technology that is already present everywhere in the world, for the agriculture of the necessary products, the aforementioned powdery particles need to be agglomerated into larger forms. This can be achieved by pelletizing or briquetting fine particles of crushed limestone or dolomite rock. Preference in these conditions should be given to the pelleting Method because the conditions for making pellets provide more useful characteristics of the product for the described purpose. They relate primarily to the favorable particle size of the crushed rock from the perspective of the requirements of the pelletizing process, because a successful process takes place if the particle size of particles smaller than 75 μm is greater than 70%.

The application of these agglomerates for the stated purposes under the present technical and technological conditions, partially stated, also requires special characteristics, i.e. properties of finished pellets. The finished product, limestone or dolomite pellets, should have the following properties:

- satisfactory impact strength due to the effect of the mechanism in the device for spreading on the ground,
- minimal wear due to abrasion, i.e. adequate pellet hardness,
- good solubility in water,
- must not have harmful substances present from the aspect of agriculture, i.e. production of healthy food.

These conditions, consequently, dictate limitations in the pellet production process for this purpose. They also dictate the tasks in the production and application of the product described in this way. In the process of pellet production, their qualitative properties are largely influenced by choice of binder, the granulometric composition of the material and the operating parameters of the device in which the process takes place. Limiting application conditions dictate the choice of binder, which directly affects the physical-mechanical characteristics of the pellets.

2. RESEARCH OBJECTIVE

The research aims to establish the influence of the selected type of binder, during the production of dolomite pellets for agriculture, on their hardness, due to the reduction of abrasive wear during their application.

3. BASICS, METHODS AND MATERIALS

Investigations of the influence on the mechanical strength of agglomerates are mainly found within the framework of various studies on the conditions and mechanisms of pellet formation. Thus, in his studies, Gluba considers the influence of the size and dynamics of the dosage of the liquid phase (binding suspension) on the growth of pellets at different operating parameters of the pelletizer. R. Ramachandran and colleagues consider a similar effect, focusing on the distribution of the binder within the pellet and the effect of this factor on the porosity of the pellet and the inhomogeneity of the layers in the pellet cross-section. It also considers consequences with an emphasis on inhomogeneity due to eniis glub, …

3.1 MECHANICAL PROPERTIES OF PELLETS

The mechanical properties of the pellets are reflected in: the impact strength of the green pellets, the strength of the pellets against uniaxial pressure and the strength of the pellets against wear. The strength of the pellets depends on the effective ratio of the grain of the material, the surface tension of the liquid phase, the porosity, the humidity of the pellets, the amount and type of binding agent, as well as the diameter of the pellets. Pellet strength increases with decreasing concentrate grain to porosity ratio and with increasing liquid surface tension and pellet diameter. The strength of green pellets is determined by falling from a certain height and the number of pellets that disintegrate after falling from a certain height or by measuring the minimum height from which all tested pellets disintegrate. The compressive strength is determined by clamping the pellet between two parallel surfaces and is expressed by force required to disintegrate the pellet.

Wear resistance is expressed by the number of small fractions formed after a certain number of revolutions of the test drum with pellets. Wear resistance is resistance to dust formation and material loss on the finished granule. It is obtained as a product of the friction of granules on granules and granules on different equipment during production, manipulation and application.

In addition to the influence of the type and amount of binder, the mechanical characteristics of the pellets are also influenced by the structure of the pellet itself, obtained as a result of the grain shape and granulometric composition of the pelletizing material, as well as the operating parameters of the device during their production. This structural influence is reflected in the porosity of the finished pellets. Porosity is a function of the mean grain size of the material (x), the pellet diameter (d) and the speed of pellet formation (v) in the pelletizing machine: $\varepsilon = f(x, d, v)$. It mainly depends on the granulometric composition of the raw material and on the mechanical forces acting during pellet formation, whereby the grain shape is not taken into account. Pelleting takes place in a three-phase concentrate-water-air system with percentage shares of 40÷45% concentrate, 10÷25% water and 35÷45% air. During pelleting, the proportion of air decreases, and the proportion of solid particles and moisture increases. It is generally considered that for further processing of green pellets, the porosity should not be less than 20%.

3.2 METHODS (RESEARCH METHODOLOGY)

The methodology or research model is based on the following principle:
- Determination of characteristic or representative (key) indicators in research;
- Determination of sample processing methods, namely:
  a. method of preparing samples for tests,
  b. selection of methods of laboratory tests,
- interpretation and processing of research results
- research conclusion

3.2.1 Representative research indicators

Of the mechanical properties when using pellets for this purpose, the most important is its strength under pressure and resistance to wear due to friction.

A representative indicator of compressive strength is expressed through the uniaxial strength of pellets. The indicator of research on the resistance of pellets to wear is represented by the coefficient of degraded grain ζ. This coefficient is calculated from the following relationship:

$$\zeta = (mdz/muz)*100\ \%, \quad (1)$$

where is :
- mdz - mass of grains that passed through the meritorious sieve after testing in the drum (g),
- muz – the mass of the sample during testing in the drum (g).

3.2.2 Sample and its preparation for testing

The basic material used in the test is dolomite powder obtained as a mixture of ground filler and as a product of dedusting during the production of technical stone from the Očura dolomite deposit (Lepoglava, Republic of Croatia), in a mass ratio of 10 : 90%. The physical and mechanical properties of the rock from the deposit are as follows: dry compressive strength 179.2 MPa, wear resistance according to Boehme 25.8 cm3/50 cm2, water absorption 0.154 wt. %, volumetric weight 2.821 t/m3, specific gravity 2.853 t/m3, degree of density 0.989, porosity 1.12 %. The moisture content of the sample was determined based on the delivered moisture. Tests were performed for the filler and the material taken from the dust collector.

The granulometric composition of the samples and input components for these tests was determined by sieve analysis on the following set of sieves: 200, 125, 90, 75 and 63 μm. Sowing was done in two ways: with a standard laboratory thresher and a ro tap system. The grain size characteristics of the input components and the research sample are given in Table 1.

Table 1. Granulometric composition of input components and dolomite sample

Size class	Sieve opening,	Sieve, filler	Sieve, duster	Sieve, sample
μm	d_i, μm	Yi_f, %	Yi_o, %	Yi_{uz}, %
(+200)	200	100,0	100,0	100,0
(-200+150)	150	98,0	100,0	98,4
(-150+125)	125	89,1	97,4	90,8
(-125+90)	90	78,6	94,8	81,9
(-90+75)	75	62,8	80,4	66,3
(-75+63)	63	40,1	49,1	41,9
(-63+0)	0	0,0	0,0	0,0

The granulometric composition of the sample is also shown in the diagram in Figure 1.

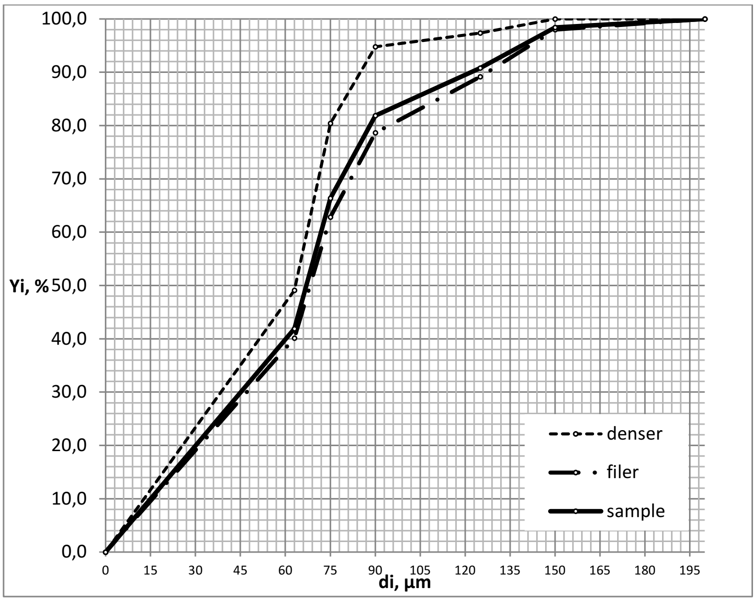

Figure 1. Granulometric composition of the dolomite sample

The following binding materials were used for the tests:

1. Sugar beet molasses. Sugar beet molasses has the following characteristics: hydrometric brix 79/80 degrees, total sugar 43/48%, organic matter without sugar 9 to 12%, nitrogen composition as protein equivalent 4%, carbohydrates: starch and other polysaccharides 4% and methoxy groups 3 up to 4%, organic acids: aconitic acid - average 2%, volatile fatty acids - average 1.3%, (formic acid, acetic acid, propionic acid, butyric acid and lactic acid), small amounts of malic acid, citric acid, etc. small amounts

of fat, resin and pigments, hydroxymethylfurfural about 40 mg/kg, humidity 22/25 %, ash 8/12 %, viscosity 5,000/15,000 CPS.

2. Starch binder produced by HG Helios Group d.o.o. Domžale Slovenia, under the commercial name, binder K. Binder K is hydrolytically and thermally processed starch. Yellow to yellow brown dust with the following properties: moisture (130°C). Max. 10%, solubility in cold water 96 – 100%, residue on sieve (1mm) max. 3%.

3.2.3 Laboratory procedures (tests)

The methodology for testing the compressive strength of pellets in this research was performed on a "mini" press, which, with accompanying equipment, is given in Figure 2.

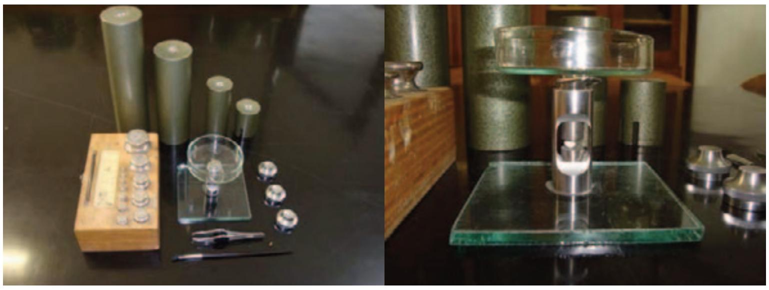

Figure 2. Laboratory "mini" press

The apparatus for testing the resistance of pellets to pressure consists of a mini laboratory press that has a tray for setting the load and a partially hollow static cylinder that has a stand for positioning the pellets. Through this cylinder passes the piston, which is connected to the tas. By placing a load on the tas via the piston, pressure is applied to the pellet, and the load is used until it breaks. The load is represented by weights of different masses: 4000, 3000, 2000, 1000, 500 and weights of 250, 100, 50, 20, 10, 5, 2 and 1 gram.

The load is applied in the following way: first, a weight of 500 grams is placed. If the pellet withstands this load, a weight of 250 is added, and then another of 250 grams. If the pellets did not burst with this load, the applied weights are removed, and a weight of 1000 grams is used as the basic weight. On top of this load, one and then two weights of 250 grams each are placed again. The procedure is repeated in the same way until the pellets burst. In this way, a continuous load of up to 250 grams is obtained on the pellet.

Granules of the size class -5+2 mm were tested. For all samples, the pressure resistance of 10 granules was tested, and the authoritative result is the arithmetic average of three series of results. Maximum and minimum results are not taken into account.

In this test, the wear resistance of the granules was performed using a rotating drum (Drum Method), which is shown in picture number 3.

Figure 3. Drum for testing the wear resistance of pellets

The apparatus consists of a drum that has six evenly spaced slats of a certain height inside. During testing, the drum is closed with a lid that is lined with rubber on its inside and has an opening in the middle through which the flow of the test can be seen. If necessary, this opening can be closed with a wooden or rubber plug.

The drum is placed on the apparatus so that its central axis of rotation is horizontal. The drum rotates at 30 revolutions per minute. The rehearsal lasts a total of five minutes. For the test, it is necessary to provide a sample with a total mass of 100 ±0.1 grams.

The granules for these tests are of the size class -5+2 mm. For testing, it is necessary to prepare sieves with circular openings of 5 and 2 mm, and seeding after the test must be done with the ro tap system. Metal spheres are separated on a 5 mm sieve, and degraded granules are classified on a 2 mm sieve. Classified granules are weighed on a scale with an accuracy of 0.1 grams.

To test the solubility and pH of pellet solutions in redistilled water, basic solutions with a concentration of 5% (w/v, 5g in 100 ml of redistilled water) were prepared. The basic solutions were prepared by weighing the appropriate amount of pellets in the form of granules on a precise analytical balance with accuracy to 4 decimal places. Accurate pH-value determination was performed on a laboratory pH-meter GLP 21 Krison, with resolutions of 0.1, 0.01 and 0.001. Electrodes: Indicator - glass electrode, reference - saturated calomel electrode. Measurements were made after the ageing of the solution/suspension for 3h, for samples that had rapid destruction of the structure. Thermostat was done at a temperature of 25°C. The measurement was performed in three repetitions.

3.2.4 Method of making agglomerates in research

Pellets for these studies were made in a laboratory plate (disk) pelletizer. Pellet growth in this pelletizer takes place in two stages. In the first stage, sprouts are formed, and their centrifugal force increases, which becomes greater than the frictional force, thus leading to a rapid increase in the mass of the pellets. When the pellets reach a certain size, they pass over the edge of the disk and leave it. The following operating parameters of the device favour the creation of sprouts and then the increase in pellet size: the angle of inclination of the disk of the plate to the horizontal (β) and the speed of rotation of the disk (no).

The angle of inclination of the plate disc to the horizontal is usually between 45 and 65°, and it is a function of the characteristics of the material that is subjected to this process, so it should be greater than the angle of the natural slope of that finely chopped material. The speed of rotation of the plate is reflected in the process through the influence of the centrifugal force on the compactness and dimension of the obtained pellet. With higher disk rotation speeds, with other constant parameters, the obtained pellet has a smaller dimension but is more compact than is the case with smaller values of this parameter. In essence, the inclination and speed of the disk dictate the path of the material along its bed in the pelletizer, thus influencing practically the formation of sprouts and the growth of pellets, as well as their compactness and the duration of the pelletizing process. In addition to the mentioned operating parameters of the device, the process of pellet formation and their final properties from the aspect of the process itself are also influenced by: the time, speed and Method of adding the binder suspension, the flow of input material into the working area of the pelletizer, and the size and characteristics of the coarseness of the input material.

In the formation of pellets for this research, the value of the static and dynamic angle of filling dolomite was determined for the previously described characteristic of the coarseness of the excavation sample. Taking into account that all the listed parameters are variable, and in terms of answering the hypothesis, they were determined as constants in the pellet production phase. Thus, the following values of the enumerated parameters were chosen during the conducted experiments:

- plate inclination angle $\beta = 52^\circ$,
- number of revolutions of the plate no = 7.5 rpm,
- material dosing speed in the pelletizer is 3.8 g/s.

The total mass of the sample for each test was 1.2 kg, and the process lasted 10 minutes. In this regard, the total planned amount of binder suspension was added during this time, which allows the dynamics of binder addition in the process to be calculated. The Method of adding the suspension was via a manual sprayer, which provided 2g of suspension per application, with a droplet diameter of 100 µm.

Working parameters set in this way during the process of obtaining pellets provide a suitable opportunity to observe the influence of the type and amount of binder on the resistance of pellets to wear, i.e. wear and tear due to the manipulation of pellets during the application phase.

4. RESEARCH RESULTS

The key influence on the binder choice is the pellets' purpose. Considering the purpose, the pellet should be soluble under the influence of moisture and should not affect the value of the pH factor of the pellet itself. For this reason, starch and molasses were chosen as binders. Another factor for analysis was the influence of the amount of added binder on the hardness of the pellets. The amount of binder undoubtedly affects the repair of all mechanical characteristics of pellets, but in terms of the possibility of application, it is limited (restricted) by economic parameters. In this connection, two groups were formed with three differently obtained pellet samples, namely:

- one group of pellets with a starch binder and with three different binder concentrations i
- the second one with molasses binder, also with three different binder concentrations,
- pellets were made for both cases with binder concentrations (ρv) of 1.0, 2.5 and 4.0%,
- a control sample was made for both binders with a concentration of 2.0%.

The results of research for pellets obtained using a starch binder are given in table 1, and the results of research for pellets obtained using molasses as a binder are given in table 2.

Table 1. Characteristics of pellets with a starch binder

Sample	Binder concentration ρ_v, %	Strength of pellet σ_p, N/pellet	Non-degraded grains (+2 mm), %	Degraded grains ζ, %
1.	4,5	90,74	96,2	3,8
2.	2,5	57,3	92,9	7,1
3.	1,0	21,5	67,6	32,4
4.	2,0	36	89,1	10,9

Table 2. Characteristics of pellets with molasses

Sample	Binder concentration ρ_v, %	Strength of pellet σ_p, N/pellet	Non-degraded grains (+2mm), %	Degradation coefficient ζ, %
1.	4,5	84,45	91,7	1,091
2.	2,5	46,9	75,3	1,328
3.	1,0	12,7	35,9	2,786
4.	2,0	44,9	67,2	1,488

Sample number 4 in both tables was the control sample.

Considering that the indicator of the pellet for this purpose is its value of the pH factor, as well as its solubility in water, the obtained samples were controlled by measuring these two parameters. The results of testing the pH factor of the obtained pellets and solubility in water are given in table 3, for pellets with starch binder and table 4. for pellets with molasses.

Table 3. Value of pH and solubility in water for pellets with starch binder

Sample	Binder concentration ρ_v, %	pH	Dissolution time in water, s	Decomposition description
1.	4,5	9,32	27,000	broken structure, without fine sediment
2.	2,5	9,83	6,000	Rapid disintegration of granules
3.	1,0	10,03	0,001	Instant decomposition
4.	2,0	9,87	0,006	Instant decomposition

Table 4. Value of pH and solubility in water for pellets with a molasses binder

Sample	Binder concentration ρ_v, %	pH	Dissolution time in water, s	Decomposition description
1.	4,5	9,71	0,001	Instant decomposition
2.	2,5	9,99	0,001	Instant decomposition
3.	1,0	10,09	0,001	Instant decomposition
4.	2,0	10,05	0,001	Instant decomposition

5. DISCUSSION OF OBTAINED RESULTS WITH CONCLUSIONS

It is a clear indication that the binder concentration has the most significant influence on the mechanical characteristics of the agglomerates obtained in this way. From the results of the conducted research, the influence of the type of binder itself on the mechanical characteristics of the binder can be clearly observed. The results given in Tables 1 and 2 give a clear idea of this statement, which can also be seen from the diagram in Figure 3.

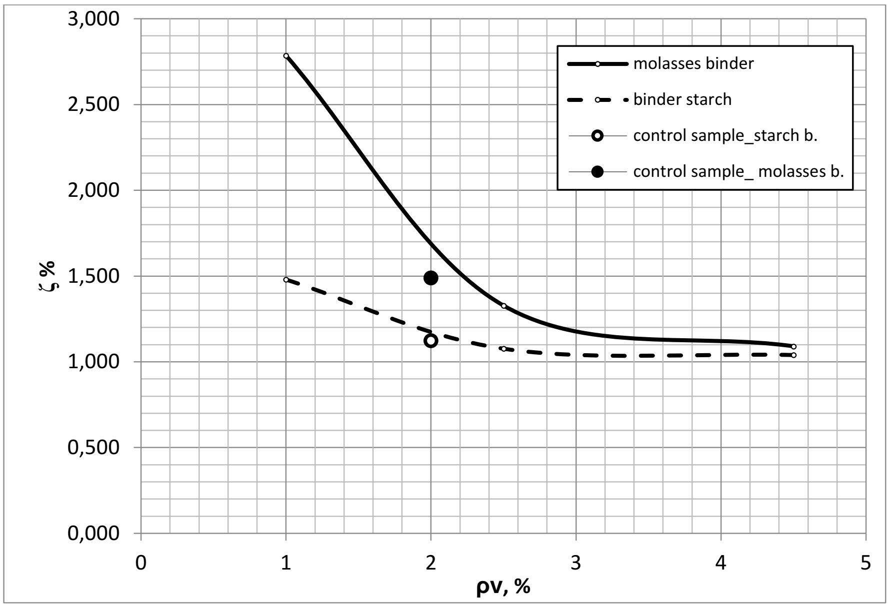

Figure 3. Strength of the obtained pellets

The strength and hardness of the material are interrelated. Usually, as the strength increases, so does the material's resistance to wear and tear, i.e. its firmness. However, the surface structure of the material has a significant impact on wear. A more pronounced roughness of the same material surface will degrade (wear out) more when exposed to wear. The diagram in Figure 4 shows the results of the wear resistance test of the obtained pellets.

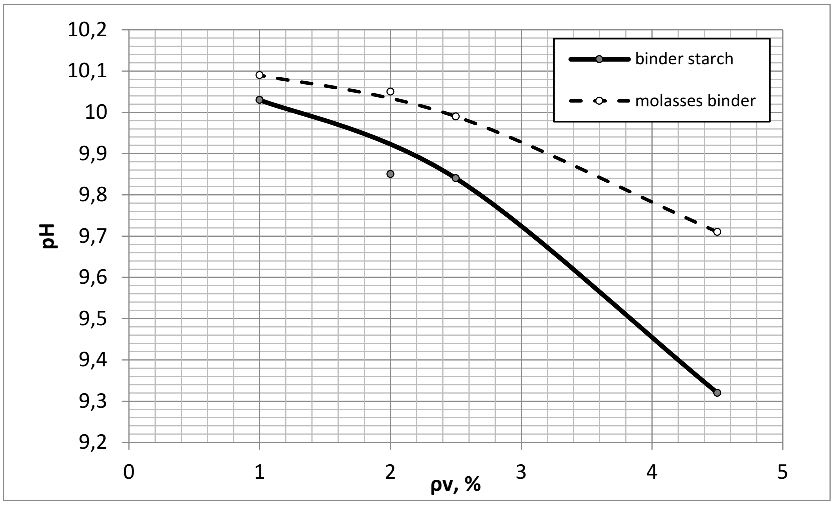

Figure 4. Coefficient of degraded grains (ζ) depending on binder concentration (ρv)

According to these results, the areas of change in the observed mechanical characteristics of the pellets can be clearly observed depending on the amount of added binder. This change is particularly pronounced in the pellet strength, which is expected. It can be said that in the observed range of binder concentration in the pellet, the dependence of power is approximately linear with the amount of added binder. However, the hardness shows a growth trend up to a certain percentage of the added binder of about 2.5%, after which the value of hardness (abrasion resistance) does not change significantly, regardless of the increase in the amount of binder.

Now let's look at the results of testing the influence of the concentration of both types of binders on the pH factor of the obtained pellets. The values of the pH factor of the pellets depending on the type and amount of binder from Table 3 are given in the diagram in Figure 5.

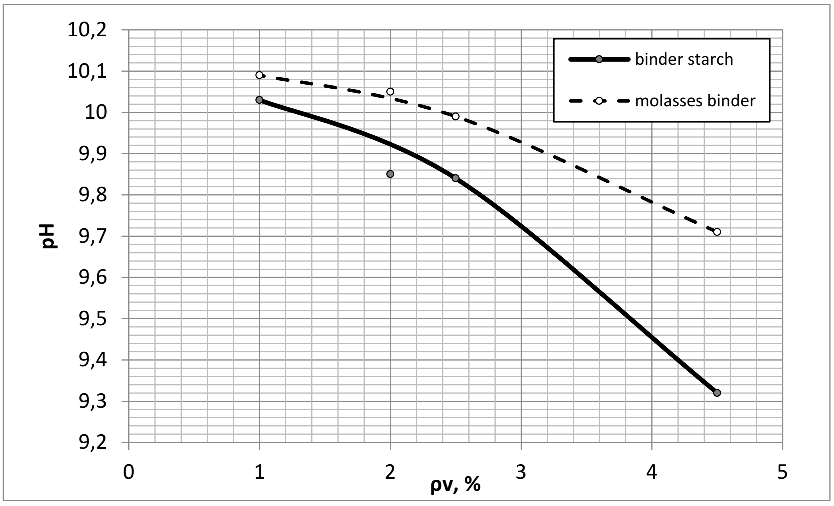

Figure 5. The value of the pH factor of the pellets depends on the type and concentration of the binder (ρv)

From the diagram, it can be seen that with an increase in the amount of added binder, the boiling point of the pH factor decreases, especially in the area above the concentration of both types of the binder of 2.5%. It is also clearly observed that the chosen type of binder has a significant impact on this indicator. Pellets using molasses as a binder give better pH factor results.

6. CONCLUSIONS

As a result of this research, the following claims can be made:

The biggest influence on the strength of the pellets is the amount and type of binder used. As the amount of binder increases, so does the strength of the pellets. In this regard, the limiting factor in the production of pellets and the choice of binder concentration is primarily the required strength value conditioned by their application and the economical parameters of such production.

On the hardness ie. the wear resistance of pellets and the type and amount of binder have a significant impact. It is reflected in the value of the wear resistance of pellets up to a certain concentration of binder in the pellet, beyond which the value of this indicator cannot be equalized by adding binder. In the case of demanding higher values of wear resistance with the optimal amount of added binder (which in this research was established with about 2.5% binder in the pellet), the only way to fix this parameter should be sought through the technological parameters of the pelletizing device (speed of rotation and tilt disc).

The type and amount of binder have a significant influence on the value of the pH factor of the pellets. More favourable results in this research were obtained for pellets in which molasses binder was applied. In the case of both types of binders, increasing the amount of added binder decreased the value of the pellet's pH factor.

Both types of binders are favourable in terms of water solubility, i.e. required results.

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ANALYSIS OF THE TEMPERATURE INFLUENCE OF ASPHALT INSTALLATION ON HOLLOWS IN THE ASPHALT LAYER

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SUMMARY

The asphalt layer is an important financial part of the total cost of road infrastructure, so it is necessary to analyze and consider all the elements that may affect the quality of the same. One of them is certainly the influence of temperature during the installation of asphalt mass. Namely, neither high nor low temperature of asphalt and the environment are favorable for its installation, so it is necessary to find optimal conditions for the installation of asphalt in order to achieve the best possible results of compaction and cavities in the asphalt layer.
The aim of this research is to find the optimal conditions for its installation by comparative analyzes of different temperatures of the asphalt mixture and the environment, in order to obtain the lowest percentage of cavities in the asphalt layer.
The legal regulations, ie technical conditions, define the minimum temperature of the asphalt mixture as well as the limit air temperature during the installation of asphalt, but special emphasis is placed on the impact of the temperature of the asphalt installation on the cavities in the asphalt layer. Therefore, the paper observes the optimal temperature of the asphalt mixture so that the smallest possible percentage of cavities in the asphalt layer is obtained.

Keywords: asphalt mixture temperature, installation, cavities

INTRODUCTION

The temperature of the asphalt mixture during the installation of asphalt became especially important because it directly affects the quality of work performed, and was limited by the temperature of the asphalt mixture in the asphalt base, the length of transport of asphalt mixture, vehicle quality, air temperature, etc. [1] [2]

If shrinkage due to cooling is prevented, then tensile stress increases in the asphalt material with decreasing temperature, which can lead to breakage (appearance of microcracks in the bonding matrix) if maximum tensile strength is reached, which is especially pronounced during asphalt rolling. Simply put, the stress in the asphalt sample gradually increases in parallel with the temperature drop, until the sample breaks. [3] [4]

The research in this paper is based on the analysis of the influence of asphalt mass temperature on the percentage of cavities in the asphalt layer.

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2. ANALYSIS OF THE INFLUENCE OF THE TEMPERATURE OF ASPHALT INSTALLATION ON HOLLOWS IN THE ASPHALT LAYER

If we observe the influence of the temperature of the asphalt mixture on the cavities in the asphalt layer, we will use the temperature of the asphalt and the cavity as input data.

Table 1. Input data for the analysis "Installation temperature - cavities"

No Sample	Temperature installation(?C)	Type of asphalt	cavities (%)
1.	136,30	BNS22	2,90
2.	139,90	BNS22	2,50
3.	123,80	BNS22	3,60
4.	112,80	BNS22	6,90
5.	152,90	BNS22	5,01
6.	157,80	AB16	5,00
7.	155,40	AB16	2,30
8.	154,20	AB16	2,50
9.	158,40	AB16	1,60
10.	160,40	AB16	2,20
11.	155,90	AB16	1,90
12.	160,10	AB16	2,80
13.	160,70	AB16	3,00
14.	157,40	BNS22	2,40

As we can see from Table 1, we have samples for two types of asphalt, so in the analysis we will divide the asphalts by types and observe them that way. We will first observe the samples from the BNS22 asphalt.

Table 2. Input data for the analysis "Asphalt temperature - cavities - BNS22"

No Sample	Temperature installation (?C)	type of asphalt	cavities (%)
1.	136,30	BNS22	2,90
2.	139,90	BNS22	2,50
3.	123,80	BNS22	3,60
4.	112,80	BNS22	6,90
5.	152,90	BNS22	5,01
14.	157,40	BNS22	2,40

In accordance with the input data, we obtained the following diagram describing the influence of asphalt temperature on the cavities.

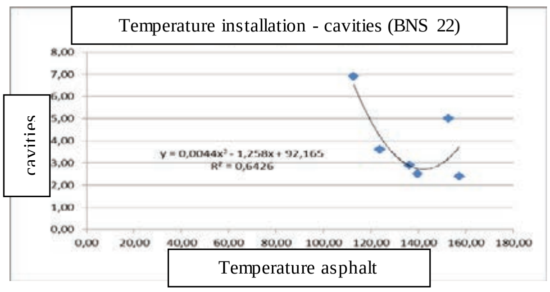

Figure 1. Dependence diagram "Asphalt temperature - cavities - BNS22"

For AB16 asphalt we use the following input data:

Table 3. Input data for the analysis "Asphalt-cavity temperature -AB16"

No sample	Temperature installation (?C)	type of asphalt	cavities (%)
6.	157,80	AB16	5,00
7.	155,40	AB16	2,30
8.	154,20	AB16	2,50
9.	158,40	AB16	1,60
10.	160,40	AB16	2,20
11.	155,90	AB16	1,90
12.	160,10	AB16	2,80
13.	160,70	AB16	3,00

In accordance with the input data, we obtained the following diagram describing the influence of asphalt temperature on the cavities.

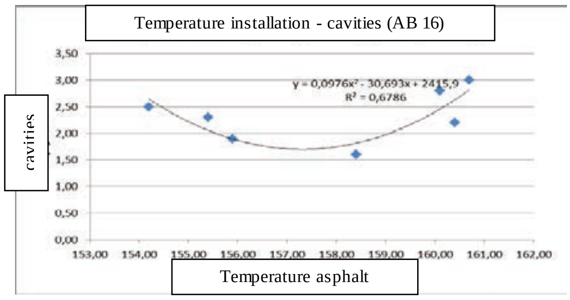

Figure 2. Dependence diagram "Asphalt temperature - cavities - AB16"

From the diagrams in Figures 1 and 2, it can be noticed that the smallest percentage of cavities in the asphalt layer from the aspect of asphalt installation temperature for BNS22 asphalt at a temperature of about 145 ^{o}C. For AB16 asphalt, the smallest cavity is at an asphalt temperature of about 157 ^{o}C. [5]

At the temperature of the asphalt above or below the stated values, a higher percentage of cavities appears.

We conclude that the optimal asphalt temperature for asphalt installation from the aspect of cavities is 145-157 ^{o}C.

We will confirm the conclusion with the following diagram in which all samples are combined. The data from Table 1 were used as input data.

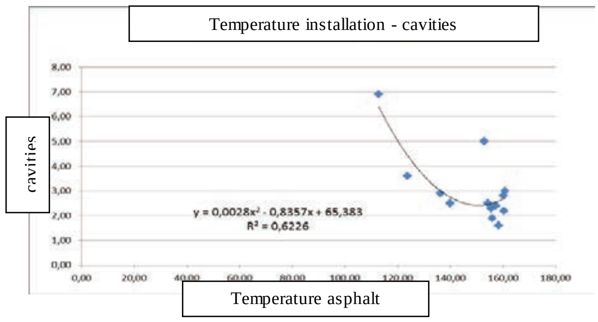

Figure 3. Dependence diagram "Asphalt temperature - cavities"

From the diagram in Figure 3, it is clear that the smallest percentage of cavities at the asphalt temperature is 145-157 ^{o}C. Each higher or lower asphalt temperature causes a higher percentage of cavities in the asphalt layer.

3. CONCLUSION

This paper presents research and analysis of the influence of asphalt mass installation temperature on the percentage of cavities in the asphalt layer. From the research in this paper, we can conclude that the control of temperature during the production and installation of asphalt is of great importance for the quality of work performed, as well as the durability of the performed layer.

It can be concluded that by increasing the outside temperature, and harmonizing the outside temperature and the temperature of the asphalt, we can create optimal conditions for compacting asphalt (as few roller passes as possible, as short a rolling time as possible to cool the asphalt). The diagram in the paper, for different types of asphalt, clearly shows the optimal limits of asphalt mass installation in which the smallest percentage of cavities.

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ANALYSIS OF THE INFLUENCE OF AIR TEMPERATURE ON THE COMPACTION OF ASPHALT DURING INSTALLATION

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SUMMARY

In this paper, the influence of air temperature on asphalt compaction during installation was investigated. The reason for this research is that during the construction of roads or the reconstruction of existing ones, the need to completely or partially replace the asphalt layers of the flexible pavement structure most often arises. The lifespan of a flexible pavement construction, i.e. road, always depends on the quality of the subgrade, but also on the quality of the final asphalt works. In order to arrive at the best possible procedure for installing the asphalt mixture, research was carried out on several concrete examples, observing the air temperature through the compaction feature of the installed asphalt, that is, through the required number of roller passes during the installation of the asphalt mixture. During the research, numerous other data that are important for the research were measured, such as the weight of the rollers, the temperature of the asphalt at the construction site, etc. On the basis of the obtained results, sorting and analysis of the same was carried out. Based on the analysis of the obtained results, conclusions were reached.

Keywords: air temperature during asphalt installation, asphalt mixture temperature, asphalt compaction

1. INTRODUCTION

The aim of the research in this paper is to investigate the influence of air temperature on asphalt compaction during installation. In order to reach the desired goal, research was carried out on the construction of flexible pavement structures.

During the production of a flexible pavement construction using the hot process, the most critical procedures during the implementation of asphalt works are the transportation of the hot asphalt mixture from the production plant to the construction site and the installation process itself. In order for flexible pavement construction to be of high quality, it is necessary to pay attention to other procedures (selection of aggregates and composition of asphalt mixture, design of asphalt mixture, production and storage of asphalt mixture). The temperature of the asphalt mixture during the installation of asphalt became particularly important because it directly affected the quality of the performed works, and was limited by the temperature of the asphalt mixture in the asphalt plant, the length of the transport of the asphalt mixture, the quality of the means of transport, the air temperature, etc. [1] [2].

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Most often, during the preparation of the asphalt mix recipe, the influence of the air temperature during the execution of the works was not taken into account. Later, in practice, it was shown that a higher air temperature has a positive effect on the installation of the asphalt mixture, and a lower air temperature in all cases was an aggravating circumstance. It is known that many materials, including asphalt, expand when heated and contract when cooled. As the temperature drops, the compaction of the asphalt material is prevented due to cooling, which causes an increase in tensile stress that can lead to failure (appearance of microcracks) if the maximum tensile strength is reached, which is especially pronounced during the rolling of the asphalt mixture. In simpler terms, the stress in the asphalt sample gradually increases in parallel with the drop in temperature, until the sample breaks. [3]

In order to obtain data on how much temperature affects the workability of the asphalt mixture, air temperature was measured on several road routes, the temperature of the asphalt mixture during installation, the number of roller passes and the compaction of the installed asphalt mixture were measured.

2. AIR TEMPERATURE AND INSTALLATION OF ASPHALT MIXTURE

The installation of the asphalt mixture consists of the following stages:

- substrate preparation,
- transport of asphalt mixture,
- spreading of asphalt mixture,
- rolling of the spread layer [4].

Transport of the asphalt mixture from the asphalt plant to the place of installation is carried out by a cargo vehicle - a dump truck. During manual installation (inaccessible places, curved surfaces), trucks with a smaller load capacity are used for transport for easier access to the location and faster transport. Installation using a paver requires greater transport capacities. It is necessary to enable the continuous operation of the paver, that is, the number and capacity of transport means should be chosen based on the capacity of production and installation. It is necessary to carefully analyze that cycle and establish how many trucks of a certain size are needed to achieve compliance with the capacity of the asphalt plant and equipment for spreading and compacting asphalt. [9]

In the entire process of production, transportation and installation of asphalt mixture, the surrounding air temperature must be taken into account. If the temperature is too low, it may happen that the installation of the asphalt mixture takes much longer, so it is necessary to harmonize the dynamics of production of fresh asphalt mixture, transport and installation, i.e. asphalt rolling. All this will significantly affect the quality of the installed asphalt layer. A drop in the temperature of the fresh asphalt mixture can also cause segregation of aggregates in the asphalt, which results in a partial or complete suspension of the asphalt mixture installation process.

Transport vehicles are open, so it is possible to cool the asphalt mixture due to the effect of wind and low temperatures. It is necessary to protect the mixture from atmospheric influences (rain, wind), but also to reduce the harmful impact on the environment, i.e. to prevent its excessive cooling and the formation of a crust on the surface. That is why vehicles must be equipped with a mechanism for covering the asphalt mixture - a tarpaulin, regardless of the season, distance from the construction site and type of asphalt mixture. [9]

The study of the cooling of the asphalt mixture during transport showed that different truck cargo boxes and covers significantly affect the cooling during transport. Thermal crates insulated with polyurethane and covered with a tarpaulin attached to the crate, so that there is no direct contact between the asphalt mass and the tarpaulin, proved to be the best. It also showed that the hot asphalt mixture cools differently in different parts of the box depending on the speed of the vehicle and other external influences. Before the start of transportation and at the construction site before installation, the temperature of the fresh asphalt mixture is controlled. [9]

Asphalt mixtures arriving at the construction site are first controlled. Its temperature is determined and its adequacy is visually checked. As already mentioned, the temperature is of particular importance for the workability of the asphalt, and its verification is an important and necessary step when receiving asphalt mixture. The temperature is measured not only in the paver but also in the vehicle, during which precise and proven instruments that quickly register the temperature (secondary thermometer) should be used [9].

Figure 1. Second remote thermometer for measuring asphalt temperature at the place of production and at the place of installation

3. RESEARCH IN SITU

As the subject of research in this work is air temperature, and in connection with the temperature of the asphalt mixture, the authors measured and recorded all the data in the field that could have an impact on the temperature change or the compaction of the embedded layer. Asphalt temperatures at the installation site were measured immediately before installation, as well as air temperature. Measurements were made with remote thermometers shown in Figure 1. During the installation of the asphalt mixture, data was recorded on the number of passes of the rollers when compacting the spread layer of asphalt. After rolling, the asphalt layer samples was taken. The samples taken were also examined in the laboratory. Data on compaction, voids and thickness of the asphalt layer were recorded and analyzed for this research.

Asphalt installation and all measurements were carried out in the area of north-eastern Bosnia and Herzegovina on real examples of asphalt works. Asphalt compaction testing was done in laboratory conditions.

Figure 2. Production temperature and temperature during installation of asphalt at location 1 – Rastošnica; air temperature 6° C

General information about the examination site 1.
Project: Reconstruction of the regional road R456 Priboj - Sapna;
The distance of the asphalt base is 35 km;
Asphalt installation date April 9, 2020.

In total, measurements were carried out at 14 locations in the area of northeastern BiH in the period December 2019 - May 2020.

Table 1. Measurement of asphalt properties

No.	The temperature of the produced asphalt	Asphalt temperature during installation	The difference in temp.	Temp. air	Date	Asphalt compaction	Nominal number of roller passes
1	162,00	136,30	25,7	**6**	09.04.	101,6	62
2	152,00	139,90	12,1	**15**	08.04.	101,9	50
3	146,00	123,80	22,2	**8**	19.12.	101,9	61
4	141,00	112,80	28,2	**5**	18.12.	98,4	64
5	167,00	152,90	14,1	**20**	18.06.	94,64	48
6	170,00	157,80	12,2	**29**	26.08.	96,1	40
7	170,00	155,40	14,6	**20**	27.08.	98,9	35
8	169,00	154,20	14,8	**23**	27.08.	98,6	37
9	170,00	158,40	11,6	**22**	27.08.	99,5	39
10	172,00	160,40	11,6	**29**	28.08.	99,1	43
11	170,00	155,90	14,1	**26**	28.08.	99,4	41
12	172,00	160,10	11,9	**32**	29.08.	98,5	45
13	171,00	160,70	10,3	**31**	29.08.	98,3	50
14	173,00	157,40	15,6	**22**	17.08.	100,20	47

4. REGRESSION ANALYSIS

When we want to investigate two or more variables or variables that are inherently related, we often use regression analysis

The regression technique allows us to quantitatively express the dependence/relationship between such variables and to define that relationship by shape and direction.

In the first step of the regression analysis, it should be determined whether there is any connection between the observed phenomena. Three forms of connection usually appear, namely:

- straight line form (linear connection), the most common form,
- curvilinear shape (non-linear connection),
- spatial form.

If there is a connection between the observed phenomena, the second step of the analysis is carried out, in which the strength of the connection between the observed phenomena is investigated. Simple regression is applied when two phenomena between which there is a connection are observed at the same time, where a certain function, i.e. a line, can be well adapted to the original values of the characteristics. If the function is a straight line, then the functional dependence is called linear regression, and it can also be a higher-order line, exponential function, hyperbolic, logarithmic [6].

A linear relationship describes a relationship between phenomena that is characterized by the fact that each unit increase in the value of one variable corresponds to an approximately equal linear change in the other variable. The quadratic function is based on the condition that the sum of the squares of the vertical deviations of the points in the scatter plot from the required regression direction is minimal. A specific indicator of the representativeness of the regression is the coefficient of determination (R). The more representative the model is, the closer the coefficient of determination is to 1, i.e. the stronger the connection between the observed phenomena.

In accordance with that, the degrees of strength of connection between observed phenomena are defined, depending on the value of the coefficient of determination, i.e. the correlation coefficient (R):

- $0.00 < |R| \leq 0.25$ - there is no connection between the observed phenomena,
- $0.26 < |R| \leq 0.50$ - weak to moderate association between observed phenomena,
- $0.51 < |R| \leq 0.75$ - good correlation between observed phenomena,
- $0.76 < |R| \leq 0.99$ - very good to excellent association between observed phenomena,
- $|R| = 1.00$ - mathematical connection. [6]

5. ANALYSIS OF TEST RESULTS

As we can see from table 1, asphalting was done in the winter period and in the summer period, which is extremely important for the analysis of the change in temperature of the asphalt mixture. In the diagram below, the dependence of the external temperature and the temperature change of the asphalt mixture is shown.

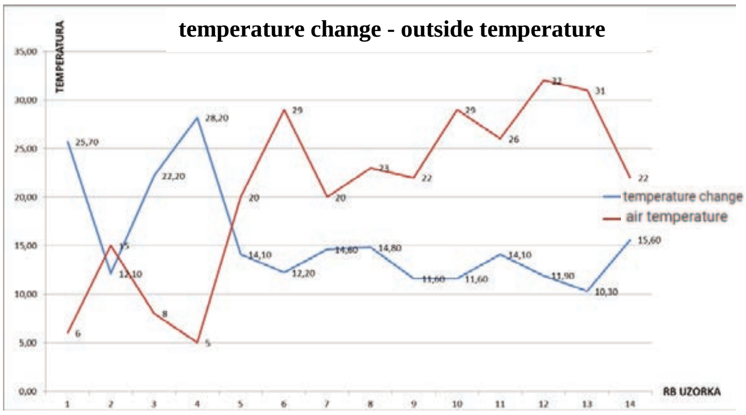

Figure 3. Dependency diagram "air temperature - asphalt temperature change"

From the previous diagram, it is clearly seen that the outside temperature significantly affects the cooling of the asphalt. We can see that the asphalt temperature changes significantly depending on the outside air temperature. Asphalt temperature changes are especially pronounced when the outside temperature is lower than 15°C.

We see that with samples no. The 1st, 3rd and 4th temperature changes were between 22 and 28° C, with outside temperatures of 5 to 8° C.

Sample code no. 2., the temperature change was 12.1° C at an external temperature of 15° C, so it can be concluded that an air temperature higher than 15° C is acceptable from the aspect of cooling the asphalt, and again everything depends on the length of the transport.

We can show the dependence of these two phenomena with the following diagram, and approximately determine the asphalt temperature loss depending on the outside temperature, and for a period of time up to max. 150 minutes (according to the author's investigation), from the moment of loading into the means of transport until the moment of installation.

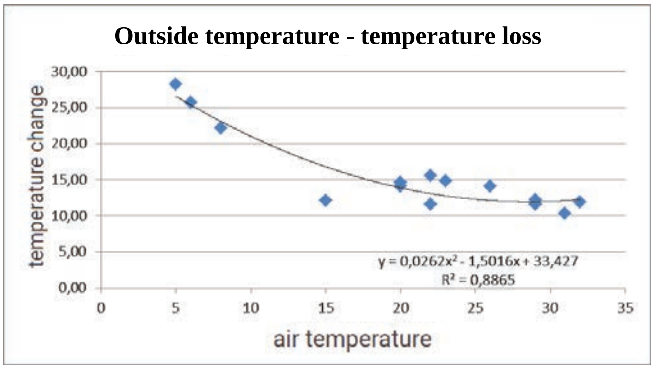

Figure 4. Dependency diagram "Outside temperature - asphalt temperature change"

By analyzing the curve in the previous diagram, it can be seen that the coefficient of determination - R has a value of 0.8865, which represents a very good connection between the observed phenomena. The authors decided on a second-order curve because it realistically describes the observed phenomenon. Through the analysis, other curves were also considered, but due to space limitations and worse results, they are not presented here.

Also, it should be pointed out that the curve clearly shows that air temperature of 30 or more degrees slightly affects the temperature change of the asphalt mixture during transportation.

6. CONCLUSION

By analyzing the curves in the diagrams in Figures 3 and 4, it can be seen that the air temperature below 15 ^{o}C significantly affects the lowering of the asphalt temperature during transport, that is, it significantly affects the asphalt installation procedure (number of roller passes, asphalt segregation, rolling and compaction of asphalt).

On the contrary, air temperature above 15 ^{o}C does not create major problems in the transport and installation of asphalt.

It is important to note that the upper limit of asphalt production temperature is usually limited to 175^{o} C, so it is necessary to take care of the outside air temperature in order to achieve a satisfactory quality of installed asphalt. From Table 1, it can be concluded that by increasing the outside temperature, and by harmonizing the outside temperature and the temperature of the asphalt, we can create optimal conditions for asphalt compaction (the fewer number of roller passes, the shorter the rolling time interval).

On the other hand, too high outside air temperature adversely affects optimal compaction, because it takes more time for the asphalt to cool down, and rolling is done until the asphalt reaches the appropriate (low) temperature.

As we have limited upper and lower temperatures for asphalt production, we can create optimal conditions by adjusting the external temperature for the given circumstances (production temperature, temperature loss during transport/distance of the construction site from the asphalt base).

In addition to the outside air temperature, the change in the temperature of the fresh asphalt mixture will be directly influenced by the distance from the asphalt base to the place of installation of asphalt, that is, the time that the fresh asphalt mixture spends exposed to natural environmental conditions.

On the basis of these researches, recommendations are made to contractors who install asphalt mixtures, that they should take special care of the air temperature during the installation of asphalt and the temperature losses of the asphalt mixture during transport from the asphalt base to the place of installation and waiting for the truck to be installed, so that the quality of the installed asphalt is optimal with minimal effort and resources during the execution of asphalt roadway.

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Professional paper

TEMPORAL AND SPATIAL ANALYSIS OF EARTHQUAKE IN STOLAC ON APRIL 22ND, 2022

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SUMMARY

The territory of Bosnia and Herzegovina represents a seismically relatively active zone, given that it is under the influence of the convergent movement of the African plate towards the Eurasian plate. Evidence of the seismicity of Bosnia and Herzegovina is the earthquakes recorded in the earthquake catalogs. The strongest earthquake recorded on the territory of Bosnia and Herzegovina in the 21st century is the earthquake that occurred in Stolac on April 22, 2022. The area of Stolac is characterized by numerous normal, oblique and overturned folds as well as reverse faults. The reverse faulting is the cause of the earthquake that occurred, which was proven by the analysis of the focal mechanism of the earthquake. Based on the GIS software of the National Institute of Geophysics and Volcanology of Italy for earthquake dislocations, the maximum possible earthquake magnitude was determined, which for the Stolac area is 7 degrees on the Richter scale. In view of this, it is necessary to design, build and reconstruct buildings so that they are resistant to earthquakes.

Keywords: earthquake, seismicity, beachball diagrams, hazard

1. INTRODUCTION

The temporal and spatial analysis of the earthquake in Stolac was performed on the basis of data taken from the archives of the Seismological Station at the Geophysical Department of the Faculty of Science, University of Zagreb, the Federal Hydrometeorological Institute of Bosnia and Herzegovina and the Euro-Mediterranean Seismological Center (EMSC). The earthquake in Stolac occurred on April 22 at 23:07 local time, magnitude Mw = 5.7 by Richter scale. According to the data of the seismological services, the earthquake was rated with intensity at the epicenter VIII° EMS and is described as a devastating earthquake [1]. The earthquake was felt throughout Bosnia and Herzegovina, Croatia, Montenegro, Albania, Italy, Kosovo, North Macedonia, Slovenia and Serbia. This earthquake was the strongest in Bosnia and Herzegovina since the one that hit Banja Luka in 1969. The earthquake caused casualty in the village Kukavac near Stolac, while major material damage was recorded in the area of Stolac, Čapljina, Mostar and surrounding villages.

2. CAUSES OF THE EARTHQUAKE IN THE AREA OF BOSNIA AND HERZEGOVINA

The area of Bosnia and Herzegovina is seismically and tectonically active due to the convergent movement of the African (Nubian) plate towards the Eurasian plate. The speed of movement is estimated at around 9-10 mm/year. based on GPS measurements (Figure 1). Approximately half of this convergent movement is consumed in active subduction zones along the Hellenic and Calabrian arcs, while the remaining part is transferred northward to the relatively stable part of the Adriatic microplate in the Ionian and Adriatic seas,

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from where it is transferred to the surrounding mountain ranges of the Apennines, the Alps and the Dinarides, and partly in the Pannonian Basin in Croatia and Hungary [1].

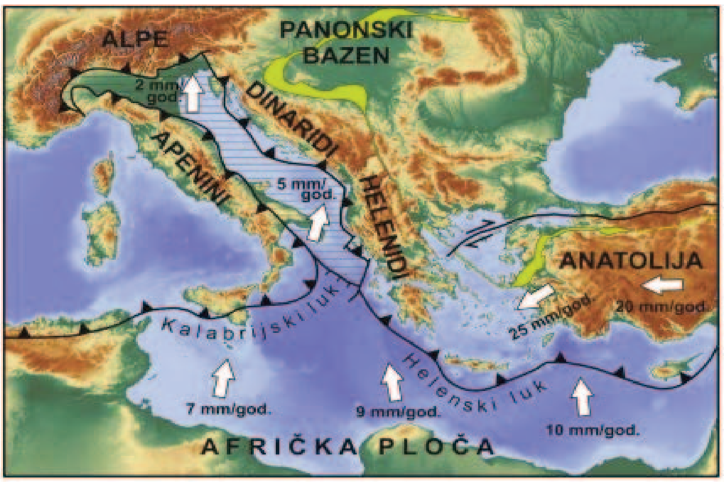

Figure 1. Geodynamic sketch of the eastern Mediterranean area and the speed of convergent movements of the African (Nubian), Anatolian and Adriatic microplates towards the Eurasian plate (tectonic boundaries taken from Handy et al., 2019 and Reilinger et al., 2006; GPS displacement vectors taken from McClusky et al., 2000 and Weber et al., 2010; Sava subduction zone, taken from Schmid et al., 2020 (marked in green)

The movement of the Adriatic microplate towards the Eurasian plate is at an estimated speed of 0.5 to 4.5 mm/year. Part is transferred by translation towards the north and simultaneous rotation in the direction opposite to the movement of the hands of the clock around the axis whose pole is located in the western Alps (Figure 2).

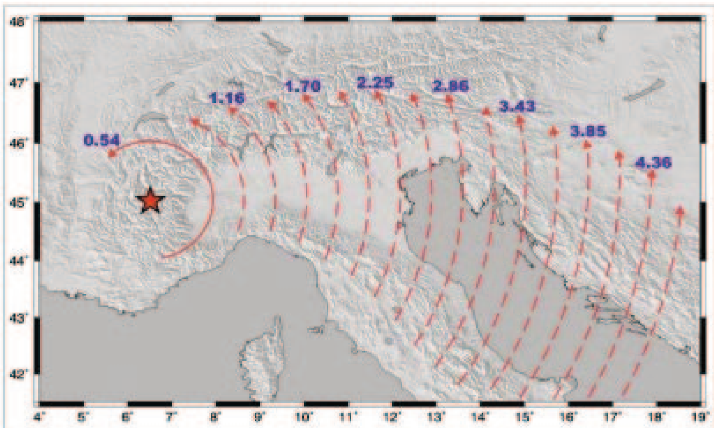

Figure 2. Schedule and speed of convergent movements (mm/year) in the stable part of the Adriatic microplate and in the surrounding mountain ranges calculated on the basis of GPS measurements (from Weber et al., 2010)

Accumulation of stresses in the rocks of the brittle part of the crust and on existing faults leads to tectonic tensions. With the increase of these tensions, the critical value of the shear resistance of rocks and existing faults is reached. By exceeding this critical value, there is a breakdown in rocks and faults and a sudden release of accumulated energy in the form of seismic waves that cause shaking on the surface [1]. That's how an earthquake occurs. Earthquakes in the territory of Bosnia and Herzegovina have been recorded and can be found in the historical archive and in the earthquake catalogs which are published by the Federal Hydrometeorological Institute of Bosnia and Herzegovina. A catalog of historical earthquakes (510 BC – 1900 AD) and earthquake catalogs for the 20th and 21st centuries are available to the public.

3. SEISMICITY OF THE TERRITORY OF BOSNIA AND HERZEGOVINA

The territory of Bosnia and Herzegovina belongs to a seismically relatively active zone. Seismotectonic active zones were formed due to the movement of segments of the Adriatic microplate. They differ in size and amount of movement and resistance of the Dinarid mass and are related to faults on the surface. In depth, the boundaries of the seismogenic zones are steeply inclined and curved due to the compressional regime of the Dinarides. Such zones are characterized by frequent earthquakes. The areas of thrust movement are related to the seismogenic zones of gentle dip. For the territory of Bosnia and Herzegovina, nine seismogenic zones have been identified. Each is characterized by the appearance of one or more dominant tectonic structures [5]:

1. Tuzla: left horizontal faults (direction of extension NE-SW), right horizontal faults (direction of extension NW-SE) and normal faults.
2. Žepče-Srebrenica: normal faults (NW-SE)
3. Banja Luka: normal faults (NW-SE) and horizontal faults (NE-SW and NW-SE)
4. Zenica-Sarajevo: normal faults (NW-SE)
5. Treskavica: normal faults and thrust fault
6. Mostar: left horizontal faults (NW-SE) and thrust fault
7. Livno-Tihaljina: thrust fault and normal faults
8. Neum-Trebinje: thrust fault and normal faults
9. Bihać: normal faults

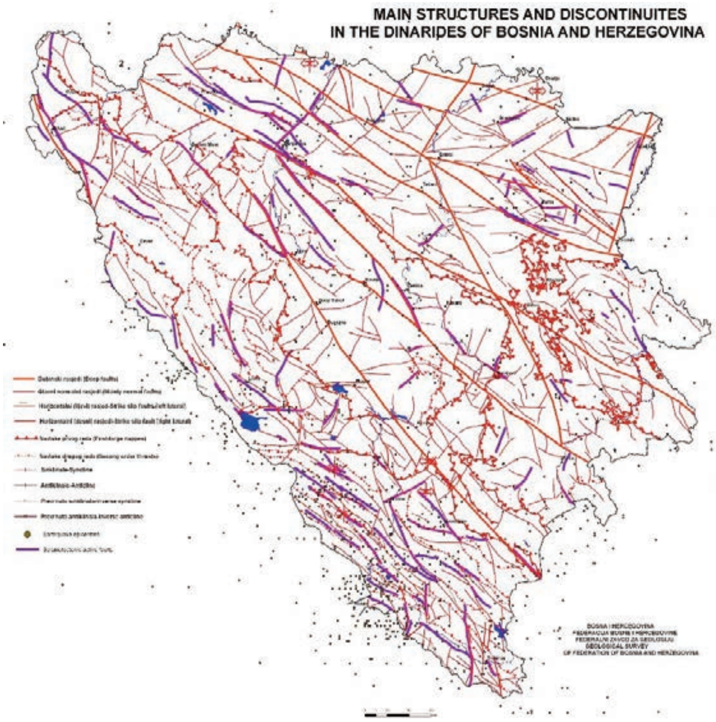

Figure 3. Main structures and discontinuities in the Dinarides of Bosnia and Herzegovina (Hrvatović, 2000)

Since 1900 (since earthquakes have been recorded instrumentally in this area), 1,084 earthquakes have been recorded whose magnitude was over 3.0 on the Richter scale or intensity greater than V° on the Mercalli scale. Numerous earthquakes with a magnitude over 5.0 on the Richter scale were also recorded. These are earthquakes that caused material damage or human casualties. In addition to natural earthquakes, which are a frequent occurrence, human caused earthquakes also occur in the region as a result of the construction of hydroaccumulation-dams, which were registered at the Bočac, Grabovica, Grančarevo, Rama and other dams.

3.1 FORECASTING SEISMIC ACTIVITY

To forecast seismic events for any territory, including Bosnia and Herzegovina, instrumental data from earthquake catalogs are used together with the application of mathematical-physical models of seismicity. In addition to the catalog of earthquakes, earthquake hazard maps for different return periods are created for the forecasting of seismicity or earthquake hazard for the territory of Bosnia and Herzegovina. Seismic hazard or earthquake hazard represents the potentially devastating effects of an earthquake (such as ground shaking, soil liquefaction, landslides, etc.) at the observed location. It is expressed by the statistical probability of exceeding the selected parameter in a given period. For the parameter that describes the earthquake action, the expected horizontal acceleration of the ground (ag, expressed in units of the acceleration of the Earth's gravity (1 g = 9.81 m/s^2), is simply associated with the forces acting on the building during the earthquake), and in different earthquake intensity scales are most often used in the public (the already mentioned MCS and Richter scale, with energy measurement at the focal point. Earthquake hazard maps for Bosnia and Herzegovina are an integral part of the National Supplement to Eurocode-8 of the Institute for Standardization of Bosnia and Herzegovina. The maps were made for different return periods, of 95 and 475 years (Figure 4).

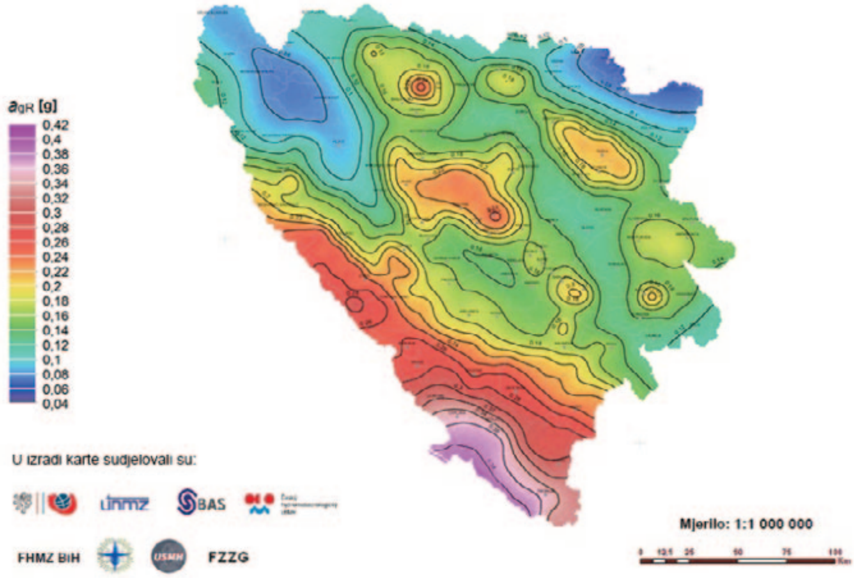

Figure 4. Earthquake hazard map for the territory of Bosnia and Herzegovina for the return period of 475 years (Institute for Standardization of Bosnia and Herzegovina)

According to this map, Bosnia and Herzegovina represents a very seismic active area. The earthquakes hazard is located in the area of Herzegovina, such as Neum, Trebinje, Nevesinje, Mostar, Posušje, Tomislavgrad and Livno. The hazard is also located in the area of Zenica, Bugojno and Banja Luka.

4. GEOLOGICAL SETTING OF STOLAC AREA

In the wider area of Stolac, rocks of Cretaceous, Paleogene and Neogene age were deposited [8].

4.1 TERRAIN EVOLUTION

When comes to evolution of the terrain, there are three distinct periods; continuous sedimentation during the Cretaceous, then very variable sedimentation conditions as a result of orogenic movements in the Paleogene, and lacustrine sedimentation during the Neogene. In the Lower Cretaceous, uniform sediments were formed in larger areas, consisted of limestone and subordinate layers of dolomite and dolomitic limestone. At the end of the Senonian, the sedimentation caused by the Laramide orogenic movements is interrupted. Mechanical-chemical wear occurs. This is when the first deposition of material and the formation of bauxite of Old Paleogene age, which are represented in the area of Stolac, actually begin. Sedimentation again covers larger areas at the beginning of the Late Paleocene. It is particularly prominent at the end of the Paleocene and in the Early Eocene, when alveolin-nummulite limestones are deposited directly on partially eroded deposits of the Late Cretaceous.

In the Middle Eocene, new tectonic movements lead to a break in the sedimentation of alveolin-nummulite limestones as terrestrial phase is formed during which the bauxite deposits are created again. After this terrestrial phase, in the Eocene there is transgression and the deposition of thick clastic deposits of the Eocene. After the deposition of clastic sediments, orogenic movements occur, which left the strongest traces on the Cretaceous and Paleogene sediments. During this orogenic phase, all the main tectonic structures in this area were created. After orogenic movements, freshwater Neogene sediments were discordantly deposited. These deposits were also covered by tectonic movements and were brought to different heights. During the older Quaternary, deposits of limnoglacial material were deposited in Mostarsko polje as a result of the melting of the ice from Velež, Čvrsnica and Prenje. Huge amounts of water and erosion in the younger Quaternary have led to the current appearance of the terrain [8].

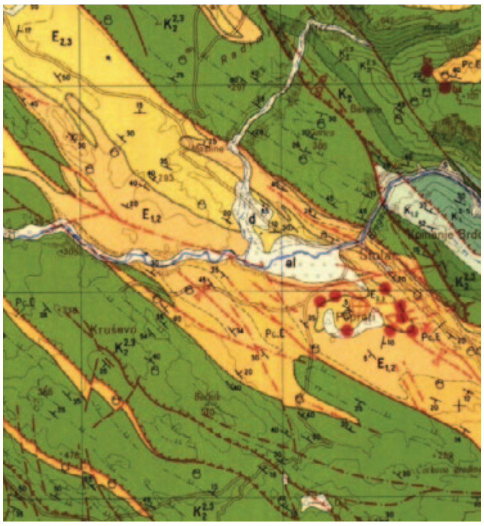

Figure 5. Geological map of the wider area of Stolac (section of the basic geological map, sheet Metković, Rajić & Papeš, 1975)

4.2 TECTONICS

From a geotectonic point of view, the analysed terrains belong to the high karst thrust fault. In this area, four tectonic units are prominent, which are pulled on top of each other from the northeast to the southwest. Those tectonic units are: Svitava-Ljubuški, Stolac-Čitluk, Blagaj and Velež tectonic units. The Stolac-Čitluk tectonic unit covers the stretch from Rasno in the northwest to Ranča hill in the southeast. The northern border of this tectonic unit is formed by a dislocation that stretches from the southeastern edge of Mostarsko Blato through Žitomislić and Hodovo to Stolac. In the northwestern part of the area, along this dislocation, chondrodonta limestones are overlain by rudist limestones. In the area of Hodovo, this dislocation is covered by freshwater Neogene sediments. Moving towards the southeast, it can be seen on Radimlja, Bregava and Komanje hill near Stolac, where Albian-Cenomanian and chondrodonta limestones and dolomites are overlain by rudist limestones and partly Paleogene sediments. From a tectonic point of view, this unit is characterized by numerous normal, oblique and overturned folds as well as reverse faults. The Klobuk thrust fault out the most, whose forehead stretches from Klobuk on the western border of the analysed area through Ljubuški, Čapljina and Svitava on the southeast side of the area. South of Stolac, a reverse fault was identified. Northwest of this fault, there is a Tertiary syncline. In the bottom of the syncline consisted of alveolinic-nummulitic limestones and classites. In the limbs of syncline are Liburnian layers. Numerous deposits of bauxite occur on the border of Cretaceous and Paleogene limestones, as well as on the border of alveolin-nummulite limestones and Eocene clastites. It is also worth mentioning the reverse fault along which the rudist limestones were pushed onto the clastic deposits of the Eocene. It is visible on the ground and can be followed from Dubrava to Stolac and further to the southeast [8].

5. EARTHQUAKE IN STOLAC

The series of earthquakes in Stolac began with an earthquake that occurred on April 22, 2022 at 23:07 local time (SEV) at a depth of 10 km. The earthquake had a local magnitude of ML=5.7 and moment magnitude of Mw=6.1. The epicenter of the earthquake was in Strupići. Figure 6 shows a preliminary map of the epicenters of recorded earthquakes that occurred until April 24, 2022 at 1:00 p.m. (SEV). Note: all data are presented according to moment magnitude (Mw). In the period from April 22 at 11:07 p.m. to April 24, 2022 at 1:00 p.m., about a hundred earthquakes with a moment magnitude greater than 2.0 were recorded.

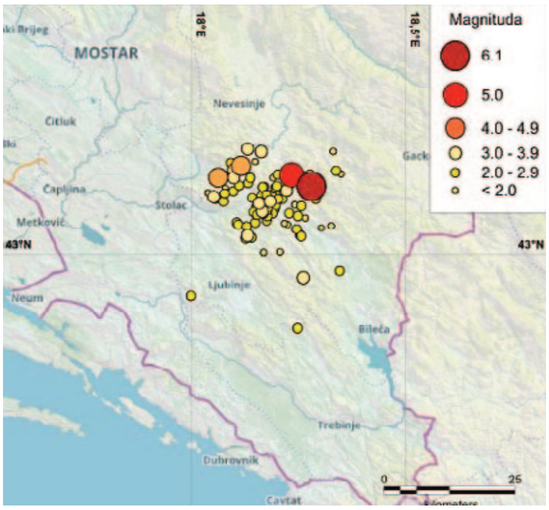

Figure 6. Map of epicenters of the earthquake near Stolac during from April 22, 2022 at 11:07 p.m. to April 24, 2022 at 1:00 p.m. according to the moment magnitude (Archive of the Seismological Station at the Geophysical Department of the Faculty of Science, University of Zagreb).

Table 1 shows the number of earthquakes by magnitude (M>3.0), and Figure 7 shows the temporal distribution of magnitudes of earthquakes with a magnitude greater than 1.9.

Table 1. Earthquakes of moment magnitude greater than 3.0 (Archives of the Seismological Station at the Geophysical Department of the Faculty of Science, University of Zagreb).

Magnitude	Date
3.4	28.08.2022
3.2	30.08.2022
3.8	01.09.2022

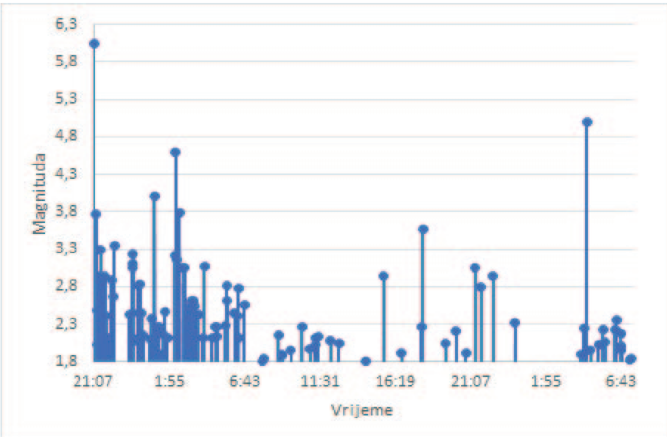

Figure 7. Temporal distribution of magnitudes of earthquakes with moment magnitude greater than 1.9 (Archives of the Seismological Station at the Geophysical Department of the Faculty of Science, University of Zagreb).

The area of Stolac is a very seismically active area. Strong earthquakes often occur, such as the series of earthquakes from August 28 to September 1, the strength of which is shown in table 2:

Table 2. Recorded earthquakes in the Stolac area in the period from August 28 to September 1, 2022. (Archives of the Seismological Station at the Geophysical Department of the Faculty of Science, University of Zagreb).

Magnitude	Date
3.4	28.08.2022
3.2	30.08.2022
3.8	01.09.2022

6. FOCAL MECHANISM OF THE EARTHQUAKE IN STOLAC 22.APRIL 2022. YEAR, MW=5.7

The analysis of the focal mechanism of the earthquake (or fault plane solutions, FPS) is based on the character of the first occurrences of P-waves that are read on the seismogram of the recorded earthquake at a certain seismological station. Since FPS diagrams resemble beachballs, they are also called "beachball diagrams". This type of diagram is needed in order to determine the type of fault that caused the earthquake, once the location and magnitude of the earthquake are known (Figure 8).The diagram shows two planes perpendicular to each other (so-called nodal planes). One of the nodal planes shows the fault on which the earthquake occurred, from whose first occurrences of P-waves the beachball diagram is constructed. In order to find out which of the two nodal planes shows the seismogenic fault, it is necessary to collect and analyze additional seismotectonic data.

Two perpendicular nodal planes in space enclose four quadrants that differ from each other in color. Opposite quadrants are usually colored black or white. The quadrants marked in black represent the tension or dilatation quadrants where the first occurrences of P-waves were recorded. They represent the displacement of material particles by the movements of P-waves "from" the focal point of the earthquake. While the white-marked quadrants represent the compression quadrants where the first P-wave tension occurrences were recorded. Here the movement of material particles are "towards" the focus of the earthquake [10].

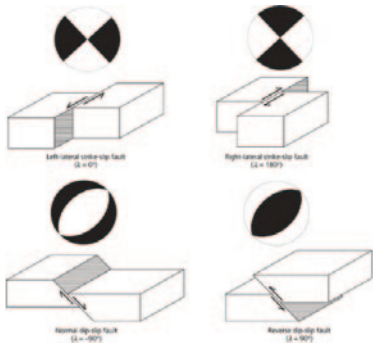

Figure 8. Focal mechanisms of earthquakes for basic types of fault geometry (Stein & Wysession, 2003)

The seismological stations located in the black quadrants recorded the first occurrences of P-waves, which are marked on the seismogram by an upward curve jump. In the white quadrants are located the stations that recorded the first arrivals of P-waves with a downward jump of the curve. To calculate the focal mechanism, seismologists use data from seismograms from as many seismological stations as possible. For the earthquake that occurred in Stolac on 22 nd April, 2022, magnitude 5.7 on the Richter scale, the focal mechanisms of the earthquake from different sources were determined (Figure 9), namely: GCMT – Global Centroid Moment-Tensor, GFZ – Geo Forschungs Zentrum Potsdam, INGV – Instituto Nazionale di Geofisica e Vulcanologia, IPGP – Institut de Physiquedu Globe de Paris, OCA – Observatoire de la Côte d'Azur, USGS – United States Geological Survey. [11]

Based on these data, it is concluded that the type of causative fault in the area of Stolac is a reverse type of fault.

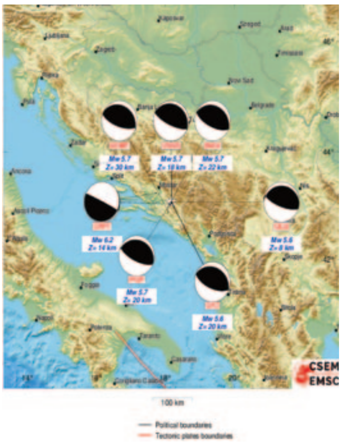

Figure 9. Focal mechanism of earthquakes from different sources (EMSC, 2022).

7. EFFECTS OF THE EARTHQUAKE

In the village of Kukavac near Stolac, one woman died as a result of injuries sustained when a rock fell on a house, while several people were injured. The earthquake caused damage to buildings throughout Stolac, Čapljina and Mostar. So far, the municipality of Stolac has received 463 reports of material damage from the recent earthquake. Coal production was interrupted in the "Sretno" pit in Breza. Due to the sudden release of pressure in the pit premises, several miners were slightly injured, and four of them requested medical assistance. Also, railway traffic from Podgorica to the northern part of Montenegro, on the Belgrade - Bar line, was interrupted.

After the earthquake in Stolac on April 22, 2022, the Euro-Mediterranean Seismological Center [11] received reports from witnesses about the ground shaking through its "I felt this earthquake" application (Figure 10).

Figure 10. Locations of citizen reports and estimated intensities reported through the "I felt this earthquake" application of the Euro-Mediterranean Seismological Center (EMSC, 2022)

8. EARTHQUAKE AREA OF STOLAC AND EARTHQUAKED HAZARD

So far, the strongest earthquake on the territory of Bosnia and Herzegovina in the 21st century was measured in Stolac on April 22, 2022. Based on data from the earthquake catalog of Bosnia and Herzegovina, the area of Stolac is extremely seismically active.Based on the GIS software of the National Institute of Geophysics and Volcanology of Italy for seismic faulting, the area of Stolac is covered by the Metković seismogenic zone, marked with the HRCS009 code (Figure 11). The orange polygon in the picture indicates a seismogenic source. Seismogenic faults can be expected within this area, which can cause an earthquake of the maximum expected magnitude of 7.

Figure 11. Seismogenic source or seismogenic zone HRCS009.
The orange polygon indicates the spatial position of that seismogenic source.

The entire area of Stolac municipality is an area that can be threatened. An earthquake is a phenomenon that affects large areas and can be felt kilometers from the epicenter. Therefore, the entire municipality is threatened by this natural phenomenon. The conditions for the construction of buildings in the function of protection from earthquake hazard, as well as other conditions for the organization of the settlement and construction, need to be regulated at the municipal level. Aseismic planning should be carried out in accordance with existing seismic maps. The planning, construction and reconstruction of buildings must be carried out in such a way that they are resistant to earthquakes, and seismic, geomechanical and geophysical research must be carried out for their location. [12].

9. CONCLUSION

Analyzing data of earthquakes in Stolac, which occurred before and after the devastating earthquake on April 22, 2022. year, the conclusion is that the area of Stolac is very seismically active. Earth tremors can be expected in this area, which can affect the life of the population. Due to the established lithological composition, the greatest danger is rockfall, which was precisely the main cause of casualty in the devastating earthquake of 2022. The main seismogenic source for the main and most earthquakes in this area is reverse faulting, which was also confirmed by solutions of the focal mechanism. Although devastating earthquakes have been recorded on the territory of Bosnia and Herzegovina in the past, the population is still not aware of the seismic danger, which is the hazard that Bosnia and Herzegovina faces. Earthquakes, such as those in Stolac in April of 2022.year, are proof that the territory of Bosnia and Herzegovina is located in a seismically active area. Precisely because of the potential devastating earthquakes that can be expected, it is necessary to implement protection measures, first of all, education of the population throughout Bosnia and Herzegovina (especially in seismically dangerous areas) and the planning, construction and reconstruction of buildings that should withstand the maximum expected earthquake magnitude.

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ENGINEERING GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS OF THE LANDSLIDE IN THE SETTLEMENT OF ZUKIĆI, MUNICIPALITY OF KALESIJA

Jasenko Čomić[1.], Aida Hrustić[2.]

SUMMARY:

In the settlement of Zukići in May 2014, during a natural disaster, a larger landslide was activated, which affected damages on the part of the local asphalt road, what has resulted as the interruption of road communication through the settlement.

After the activation of the landslide, no planned remedial measures were carried out on this part of the terrain, except for local interventions to remove formed colluvial material in order to relieve the slope, level the terrain, as well as backfill to achieve the appropriate level of the road in order to make it passable.

Based on the conducted geotechnical investigations, defined geological engineering and hydrogeological composition and properties of the terrain, the terrain was rezoned according to the degree of stability, important recommendations and conditions for landslide rehabilitation were presented.

Key words: landslide, geotechnical research, landslide rehabilitation

1. INTRODUCTION

Due to the large impact on space, people and property, the management of geohazards is only possible with a systematic approach to study, cadastral processing and taking appropriate measures. In the area of the Tuzla basin, there are a number of examples where engineering geological mapping was carried out, based on which landslides of different degrees of activity were identified and subjective assumptions were given about where landslides could occur in the future, without simultaneously carrying out research and tests, i.e. geotechnical engineering missions. In addition, there is no unique cadastre of landslides for the TK area, because the data for individual cities/municipalities differ significantly in terms of content, the way they are presented on cadastral sheets and overview topographic maps.

Therefore, for the development of an appropriate method of hazard and landslide risk assessment, the introduction of modern hazard assessment methods based on deterministic methods should definitely be adopted. This research approach, based on the results of detailed research, allows the application of a mathematical model and the calculation of stability factors to rezone the terrain according to the degree of stability and to separate the zones: unstable, conditionally stable and stable terrain.

It is necessary that the approach to the study of landslides should be carried out through appropriate regulations as binding and on the basis of a unique methodology for the creation of landslide cadastres, in the area of all local communities of the Tuzla Basin, so that in the next phase of the elaboration of spatial and urban plans, an objective landslide hazard map and a surface categorization map can be made according to the intended use of the space. In this way, it will be possible to realize the constant monitoring of landslides and the integration of landslide cadastres into a unique information system at higher levels.

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2. GENERAL CHARACTERISTICS

In the settlement of Zukići, in May 2014, due to heavy rainfall that was far above the average values, it caused the activation of a landslide, which affected the damage of part of the local asphalt road, which was reflected in the interruption of road communication through the settlement. The researched location is placed on the slope part of the terrain extending east-west, spatially located at the geographical coordinates of 44°27′7.55″ north latitude and 18°56′24.62″ east longitude according to Greenwich. The location of the local road vertically intersects the extension of a large amphitheatre-shaped slope, which is characterized by varying degrees of stability.

At the entrance to settlement Zukići from the direction of the elementary school "Memici" in Zukići and the local mosque, there are other separate and partially connected larger or smaller landslides, which affect the damage to residential buildings in different parts of the settlement, representing special spatial entities for research and rehabilitation. The research covered a part of the slope that represents one morphogenetic unit (part of the subject local road in length of about 100 m), with an average absolute terrain elevation of about 374 m.a.s.l. The configuration of the researched terrain was observed and displayed using a digital orthophoto plan and a digital 3D terrain model (Figure 1).

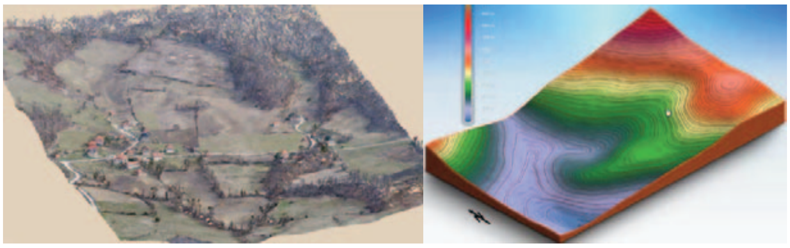

Figure 1. Orthophoto plan and digital 3D terrain model in the researched area

Based on the conducted research, the structure of the natural slope and the position of the sliding surface up to 3.40 m from the ground surface, this landslide belongs to the asequent type, which indicates a shallow rotational type landslide with an approximately circular cylindrical sliding surface, conditioned by different physical-mechanical properties and a simple relocation mechanism [1]. Through engineering geological prospecting, observed were terrain forms wich occured as a result of natural and anthropogenic processes. Shear tension deformations (extension cracks perpendicular to the direction of movement of the sliding mass) were observed on the route of the road. Above the road (figure 2), on the sliding body can be seen recognizable relief inverse forms which are manifested in the form of transverse cracks with a jump of up to 1.0 m, as well as bell-shaped protrusions caused by the multiphase gravitational movement of colluvial material.

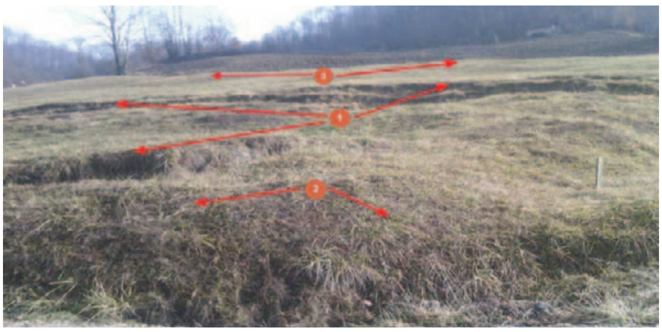

Figure 2. Inverse forms of relief: 1) transverse cracks with a jump of up to 1.0 m, 2) convex and 3) polyphase accumulation of colluvial material

In a wider area, according to the geomorphological classification, relief forms created by slope processes, fluvial relief and anthropogenic relief can be distinguished. Slope relief is present in most of this area. The slope is of western exposure, typically exogenous type, hypsometrically developed towards the east to the watershed of the terrain where the terrain bends in the opposite direction with an average absolute elevation of about 435 m.a.s.l. In general, the slope is characterized by a variable slope if local deviations are excluded, in the interval of 5° (foot part of the slope), 9° (area of the local road) to the hypsometric peak part of the slope of about 12°.

Based on the morphometric characteristics, the western foot of the mapped landslide is closed by the northern occasional and southern permanent local stream, with an average absolute elevation of about 340 m.a.s.l., which represents the crossing of the slope with a gentle slope of up to 5° and the alluvial plain of the Bukovica River. The relative height difference of the hypsometric peak and foot part of the slope is about 95 m.

After the activation of the landslide, no planned remedial measures were carried out on this part of the terrain [2], except for local interventions to remove formed colluvial material in order to relieve the slope, levelling of the terrain, and filling and achieving the appropriate level of the road in order to make it passable. After the activation of the landslide in May 2014 , the owner of the plot of land no. 1022/1 K.O. Memići self-initiatively carried out shallow drainages of questionable depth, perforations, rules of "filter layer" and absorbent recipient. These rehabilitation works did not have a stabilizing effect on the endangered area.

In geological composition of this area includes are rocks of Middle Miocene age [3], represented by coarse-grained layered clastites (sandstones, marls, and subordinate conglomerates), as well as Quaternary sediments, which are mainly represented by clays.

Hydrogeological characteristics of the slope in question were analysed both for the purposes of determining general engineering geological properties of the terrain, reconstruction of the sliding mechanism, rezoning of the terrain according to the degree of stability, and for the purposes of rehabilitating landslides.

The water resistant layer in this locality is represented by marls. Geomechanical tests determined water-permeable materials in the upper horizons of the formed colluvial cover - (dusty clay with concretions of organic material and inclusions of conglomerate). In the hydrogeological sense, the colluvial cover plays the role of a hydrogeological conductor [4].

The eluvial-deluvial cover is made of calcitic clays which represent medium water-permeable media.

During the performance of exploratory geomechanical boreholes marked (B5, B7, B9), constant wetting, swelling and weakening of the physical-mechanical characteristics in the horizon up to 3.40 m from the ground surface was determined. During the geomechanical test, the occurrence of underground water acting in the soil was determined, as well as the final levels of underground water, measured after 24 and 48 h from the end of the test.

In all boreholes, a static water level was recorded that ranged from 1.50 to 4.20 m from the ground surface, which indicates that the sliding body is waterlogged, as well as that the covering materials contain a higher percentage of natural moisture, which is one of the causes formation of landslides in this area.

As part of the field investigation works, after geodetic survey, engineering geological mapping of the terrain was carried out. Based on the results obtained from geomechanical tests (9 boreholes with continuous standard penetration tests throughout the depth of the well), as well as hydrogeological and laboratory tests, a detailed engineering geological map R 1:1000 was prepared. The landslide is connected in the central and foot part of the slope with two separate arms K1 and K2. Due to the limited degree of research due to the project task, the separated arm of the K2 landslide, where there are a larger number of individual residential and auxiliary buildings, has not been geotechnical researched. As such, it represents a special spatial unit for further research and rehabilitation.

Based on the obtained results of research and testing, 3 engineering geological units have been identified in this area, one of which belongs to the geological substrate, and two to the cover group.

The geological substrate is represented by sandstones, oolitic limestone and marls.

Cover layers

The eluvial-deluvial cover (ed) was created as a consequence of the surface wear of geological substrate formations and partial washing and accumulation of decomposed material from the higher parts of the slope in the central and lower parts. The lithological composition of this cover consists of brown dusty clays, grey calcitic clays with a medium compressibility and grey clays with a friable geological substrate, which are difficult to compress. According to GN-200, this cover belongs to category III. The thickness of this cover is variable, as can be seen on the engineering geological profile (Figure 3).

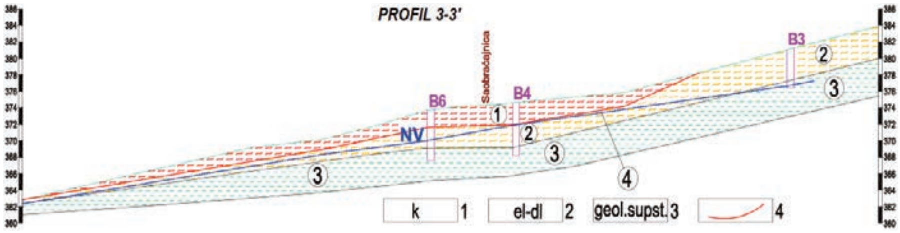

Figure 3. Engineering geological profile 3 - 3′
1. Landslide deposit, 2. Eluvial-deluvial cover, 3. Geological substrate, 4. Sliding plane.

Deposits caused by sliding (k) were formed as a consequence of the colluvial process, during which part of the eluvial-deluvial cover slide, and soil masses accumulated in the central and foot parts of the landslide. The lithological composition of the colluvial deposit consists of brown dusty clays with traces of underground water movement, concretions of organic material (semiliquid, easily squishy). The activated material in the sliding process is characterized by poor geotechnical characteristics and a higher degree of waterlogging due to the influence of underground and surface (atmospheric and waste) waters. According to GN-200, landslide belongs to category III and partly to category II.

The embankment (n) was formed in order to establish road communication in the Zukići settlement.

Detailed engineering geological map (R 1:1000) was created after geomechanical tests and hydrogeological research. A geo referenced map on a scale of 1:1000 of the wider influential part [1,5,15] of the terrain was used as a basis for its creation, which was used for rezoning the terrain according to the degree of stability (Figure 4).

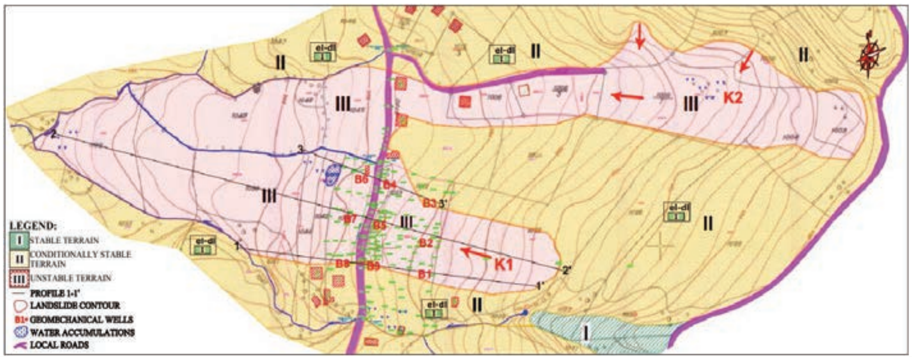

Figure 4. Land rezoning according to the degree of stability.

According to engineering geological mapping of the terrain according to the surface of the activated colluvial mass, the landslide in question belongs to a very large landslide. According to the type of process on the slope [6,7], the landslide is of a rotational type with an approximately circular cylindrical sliding surface, conditioned by different physical-mechanical properties and a simple movement mechanism. According to the volume of the sliding body, the landslide belongs to the large one. According to the morphology of the sliding body, it belongs to the amphitheatre type of landslide. According to the state of the activity it is reconciled. According to the structure of the slope and the position of the sliding surface, it belongs to the asequent landslide.

3. RESEARCH METHODS

For the purposes of defining geotechnical parameters, composition and properties of the terrain, the following research and tests were carried out:

- Geodetic surveys,
- Exploration drilling,
- Standard penetration tests,

- Laboratory tests,
- Analysis of slope stability,
- Engineering geological mapping,
- Rezoning of the terrain according to the degree of stability,
- Elaboration

As part of the geodetic survey, the situation was made in the national coordinate system on a scale of 1:500. The locations of the exploratory boreholes expressed in absolute coordinates were recorded.

As part of the exploration work, ten boreholes were drilled with a continuous standard penetration test along the entire depth of the borehole. The core was placed in appropriate boxes, mapped, determined and photographed. Disturbed and undisturbed samples were taken from completely fresh extracted core and properly preserved. Samples taken from the in-situ medium are taken at intervals in the borehole, so that all varieties of soil and rock encountered during the investigation are represented.

During the geomechanical testing, a hydrogeological observation of the phenomena of underground water acting in the soil was carried out.

Laboratory tests were performed in order to define the classification and physical mechanical characteristics of the soil [8], according to valid procedures in the accredited laboratory "GIT", Institute for Construction, Building Materials and Non-Metals, Tuzla: "BATA" accreditation is registered under number LI-04-01.

- Natural moisture is determined by the procedure: BAS ISO/TS 17892-1 Geotechnical investigations and tests - Laboratory soil testing - Part 1: Determination of moisture
- The volumetric mass of the soil is determined by the procedure: BAS ISO/TS 17892-2 Geotechnical investigations and tests - Laboratory testing of soil - Part 2: Determination of the density of fine-grained materials.
- Specific mass, at the prescribed temperature and humidity of the environment: determined by the procedure: BAS ISO/TS 17892-3.
- Atterberg's consistency limits represent the yield, plasticity and shrinkage limits, and are determined according to BAS ISO/TS 17892-12.
- The direct shear test was performed in an apparatus with controlled shear deformation, under three vertical loads. The analysis serves to determine the parameters of shear strength, cohesion c (KNm^{-2}) and angle of internal friction ϕ ($^{\circ}$). The test was carried out according to the provisions of BAS ISO/TS 17892-10 Geotechnical investigations and tests - Laboratory soil testing - Part 10: Direct shear test. Shear resistance parameters for the soil were determined in apparatuses for direct translation with a controlled deformation rate of dimensions 60 x 60 mm and a height of 25 mm. The samples were made in an undisturbed state and consolidated under vertical load σ': 50, 100, 150 KNm^{-2}, shear resistance ζ: 50, 100, 150 KNm^{-2}. Tangential stresses were increased in degrees $\Delta=\sigma'/40$ in time intervals $\Delta t1$ from 5 to 30 minutes. Shear rate 0.3048 mm/min, dynamometer constant 0.034 mm/kp. The obtained values of shear strength and parameters c and ϕ obtained on soil samples for the corresponding intervals range from 5 to 9 KNm^2 for cohesion (c) and from 18 to 19° for the angle of internal friction (ϕ). Proper sampling provided representative values of material properties (for a given realistic case). For the calculation of slope stability, input geomechanical parameters that reflect the most unfavourable conditions on the slope were adopted.

The slope stability analysis was done with recorded terrain elevations, pore pressure coefficients for natural and drained slopes according to the Bishop method, on a licensed computer program of the Faculty of Mining, Geology and Civil Engineering from Tuzla. The method is based on the analysis of the equilibrium moment of a potentially unstable segment of the soil, for which the sliding surface is assumed to be a flat or cylindrical surface. The calculation is reduced to the determination of the safety factor against sliding (Fs), which is defined as the ratio of the moment of sliding resistance (Mo) and the moment of active forces (Ma) around the potential field of rotation of the sliding body.

On the basis of the conducted research, tests and obtained results, a detailed engineering geological map was prepared on a scale of 1: 1000. The engineering geological map contains all relevant data for design and was made according to the Instructions for the preparation of engineering geological maps (Instructions of the International Association for Engineering Geology - IAEG).

Within the investigation area, the terrain was rezoned [9,10] according to the degree of stability with separate zones: stable, conditionally stable and unstable terrain, according to the area of representation (m^2) and percentage (%).

After the completion of the field work and laboratory tests, cabinet processing, analysis and interpretation of the collected data and obtained results were carried out with elaboration of the same.

4. RESULTS AND DISCUSSION

The paper presents the performed slope stability analysis for the 2-2′ profile with a length of 462.59 m and a slope of 5° to 12°. Through proper field and laboratory testing, representative samples that reflect the most unfavourable conditions prevailing on the slope were selected. For geotechnical calculation [1,11], geomechanical parameters are shown (Figure 5): volumetric weight, cohesion, tgϕ, pore pressure coefficient for each layer individually: 1) sliding mass, 2) cover and 3) geological substrate.

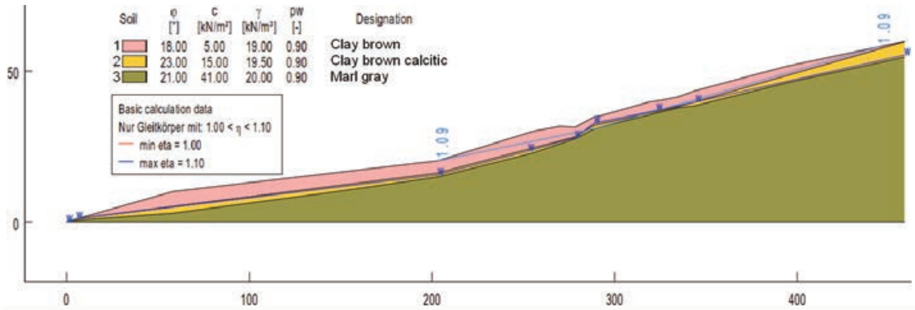

Figure 5. Model for stability analysis using the "Bishop" method, for the natural state (slope at the time of the survey).

The safety factor of the natural slope at the time of research is Fs=1.09. The natural content of water in the cover materials and formed colluvial mass has a great influence on the above mentioned parameters.

On the analysed unstable slope, through the interaction of surface - precipitation, accumulated - stagnant and underground water, they adversely affect the reduction of normal effective stresses and thus the strength of the soil. Based on the established causes, sliding mechanism and vulnerability of the investigation area, the selection of the optimal method of stabilization of the traffic road in the length of 135 m as well as the immediately influential part of the unstable slope was performed [12]. By redistributing the masses, removing the excess soil material from the central part of the sliding body, installing stone material in the body of the road in the form of stone ribs (axial distance of 6.00 m) that will be supported by a layer of geological substrate, surface and deep drainage in the case of rotary sliding is reduced moment of the weight of the sliding body [13] with respect to the centre of rotation and thereby increases the safety factor (Figure 6).

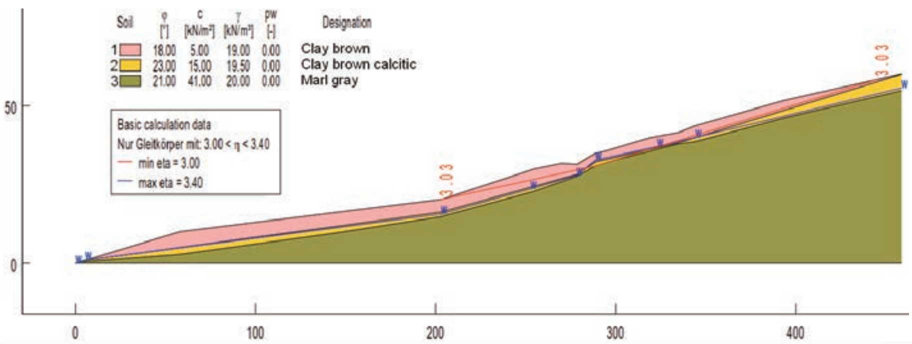

Figure 6. Model for stability analysis using the "Bishop" method (rehabilitated slope).

The safety factor with the specified measures of the rehabilitated slope is FS = 3.03 (stable). Landscaping, re-cultivation and drainage of the slope will ensure a reduction of pore pressure in the zone of the critical sliding surface.

After the rehabilitation is completed, it is necessary to establish instrumental monitoring [14] for a period of time of at least two hydrological years [15].

In the engineering geological map (Figure 4), all three categories of the terrain are spatially defined and marked according to the degree of stability:

Unstable terrain

This terrain category represents a part of the explored area that is affected by the sliding process. The unstable part of the terrain occupies an area of 45,689.67 m^2, which represents 24.48% of the total area of the investigated slope of 186,677.30 m^2. The area of unstable terrain outside the investigated area is 23,881.48 m^2, which represents 12.79% of the total area of the endangered slope. In the unstable category of terrain, which is spatially clearly defined, no new construction or legalization of existing buildings can be carried out until detailed geotechnical investigations of the entire endangered area have been carried out. In this part of the terrain, it is not allowed to cut the terrain, as well as the formation of larger embankments.

Conditionally stable terrain

This category of terrain covers an area of 114,034.01 m^2, which is 61.09% of the total area of the terrain. With the prescribed measures and performed rehabilitation, it can be coverted into a stable terrain category.

Stable terrain

This category of terrain occupies an area of 3,063.14 m^2 or 1.64% of the total area of the investigated slope and is characterized by favourable geotechnical characteristics.

5. CONCLUSION

Based on the results of conducted engineering geological and geomechanical research, the established engineering geological and hydrogeological composition and properties of the terrain, the degree of representation of exogenous geological processes, the rezoning of the terrain according to the degree of stability, recommendations and conditions for the rehabilitation of unstable and conditionally stable terrain were given. Based on the rezoning of the terrain according to the degree of stability, it is possible to adapt the purpose of land use to geological conditions, which will certainly minimize material damage.

In this locality, the projected rehabilitation measures have not even started to date, although they have been marked as urgent due to the threat to material assets. Unfortunately, in most cases, the implementation of proposed rehabilitation measures depends on the available funds of the social community. Although landslides have become a problem that we encounter very often, not enough is being done on preventive measures to prevent new ones from appearing and to reactivate existing ones.

Landslides, as geohazards, have proven to be particularly problematic due to the absence of clearly defined responsibilities of various institutions that primarily deal with them, the lack of strategies for their management, the lack of information and data (cadastres), forecast maps (hazards and risks), as well as low awareness of landslides and their consequences in the general public and various levels of government.

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