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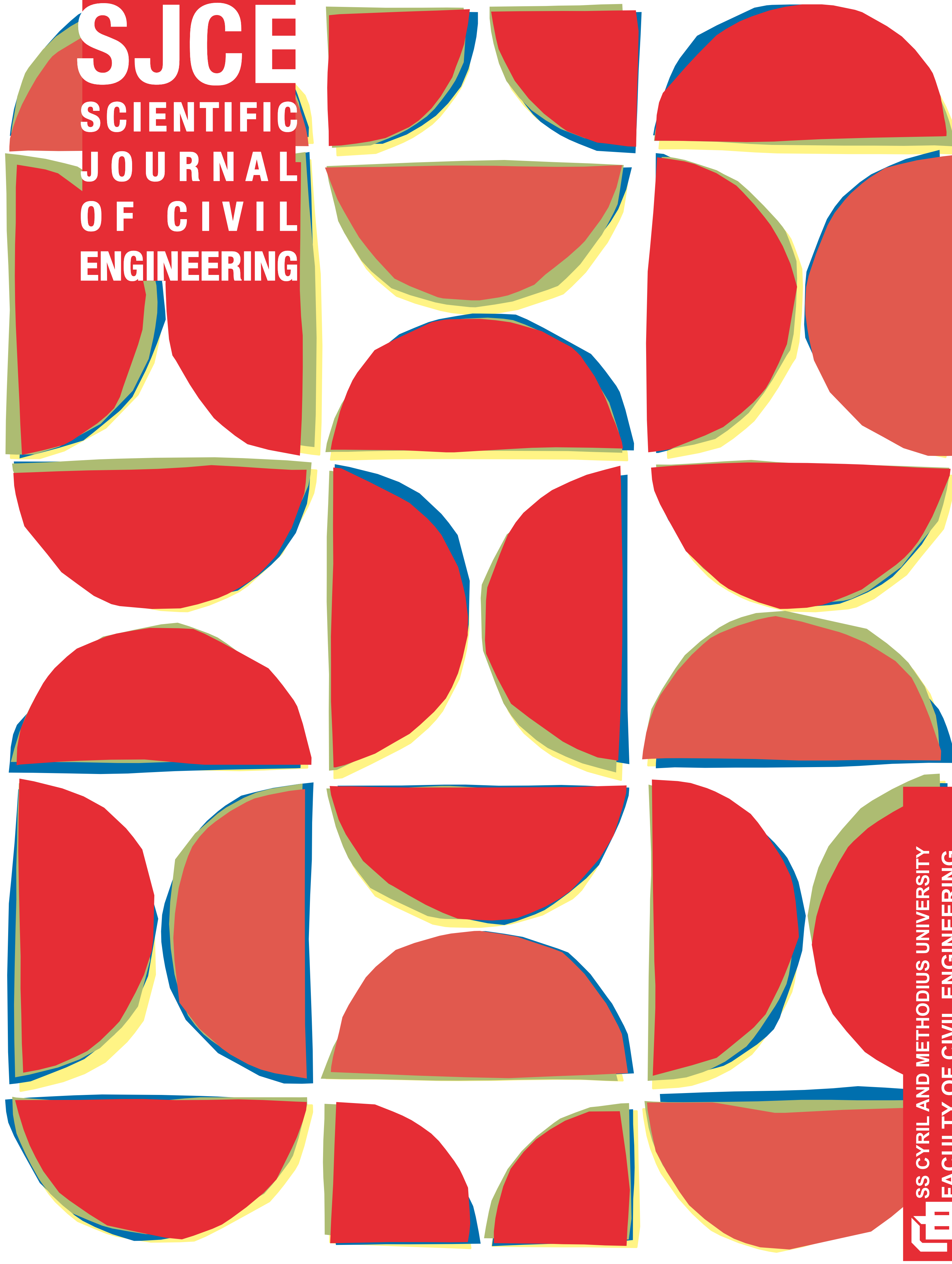
SCIENTIFIC
JOURNAL
OF CIVIL
ENGINEERING

SS CYRIL AND METHODIUS UNIVERSITY
FACULTY OF CIVIL ENGINEERING

Volume 10
Issue 1
July 2021

ISSN - 1857 - 839X

SJCE
SCIENTIFIC JOURNAL OF CIVIL ENGINEERING



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EDITORIAL - Preface to Volume 10 Issue 1 of the Scientific Journal of Civil Engineering (SJCE)

Marijana Lazarevska EDITOR - IN - CHIEF

Dear Readers,

Scientific **J**ournal of **C**ivil **E**ngineering (SJCE) is an international, peer-reviewed journal published bi-annually since December 2012. It is an open access Journal available at the web site of the Faculty of Civil Engineering in Skopje (www.gf.ukim.edu.mk).

This Journal is committed to publish and disseminate high quality and novel scientific research work in the broad field of engineering sciences. SJCE is designed to advance technical knowledge and to foster innovative engineering solutions in the field of civil engineering, geotechnics, survey and geo-spatial engineering, environmental protection, construction management etc.

Our aim is to provide the best platform for the researchers to publish their work with transparency and integrity with the open-access model, and to provide a forum for original papers on theoretical and practical aspects of civil engineering and related sub-topics.

As an editor-in-chief of the Scientific Journal of Civil Engineering, it is my great pleasure to present you the First Issue of Volume 10, an open-subject issue that contains ten scientific-research papers that have passed the general review process of this journal.

These papers cover various advanced scientific topics The first paper describes in detail the new technology for digital terrain and surface modelling applied for scanning of the entire territory of RNM. The second paper presents conclusions from the establishment of the national spatial data infrastructure in our country. The third paper explains the systematic approach of the tunnel risk management as a general concept that should include all the available information in order to obtain a quality tunnel designs. The assessment of the impact of bend type on flow characteristics is given in the fifth paper. The sixth paper deals with the general principles of chemical soil treatment, whereby a special attention is paid to the application of lime as a chemical stabilizer. The seventh paper shows the results of the experimental research and analysed factors of influence on the shear strength of rock joints. The next two papers present two different views of the urban planning, one through the eyes of civil engineers who analyses the traffic load and the most acceptable traffic solution, and the other through the eyes of an architects who examine the aspects or urban plans that affect the changes of real estate values. The final paper points out the importance of Eurocodes as design principles and presents the results from the experimental investigations of RC elements exposed to long-term sustained loads of different intensity.

I sincerely hope that all papers published in this issue will encourage further researches on the fields.

I thank all the authors for contributing to this Issue and all the reviewers for providing detailed and timely evaluations of the submitted manuscripts.

Furthermore, I would like to express my sincere gratitude to all editor members for their excellent work, remarkable contribution, enthusiasm and support, especially during these tough times of COVID-19 pandemic. Let's not forget that tough times never last, tough people do.

Sincerely Yours,

Assoc. Prof. Dr. Marijana Lazarevska

July, 2021

SJCE
SCIENTIFIC JOURNAL OF CIVIL ENGINEERING

Impressum

FOUNDER AND PUBLISHER

Faculty of Civil Engineering
Partizanski odredi 24, 1000
Skopje, N. Macedonia

PRINT

This Journal is printed in Mar-saz DOOEL Skopje

EDITORIAL OFFICE

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ISSN: 1857-839X

ORDERING INFO

SJCE is published bi-annually.
All articles published in the journal have been reviewed.
Edition: 100 copies

SUBSCRIPTIONS

Price of a single copy:
for Macedonia (500 den);
for abroad (10 EUR + shipping cost).

BANKING DETAILS (NORTH MACEDONIA)

Narodna banka na RNM
Account number:
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Prihodno konto 723219,
Programa 41

BANKING DETAILS (INTERNATIONAL)

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LIDAR SCANNING OF THE TERRITORY OF THE REPUBLIC OF NORTH MACEDONIA

The fast pace of modern life creates a need to apply new technologies and methods that would help us achieve our goals in a short period of time in any field of life. This is also required in the field of spatial data acquisition, i.e., the goal is to find a way to obtain accurate and detailed spatial data from the Earth's surface in a shorter period of time. In that direction, LiDAR technology is one of the most powerful and promising data acquisition technology, providing fast, precise and detailed spatial data for the Earth's surface.

The digital terrain model and the digital surface model generated by this technology are at a very high level of detail, thus makes this technology useful in various areas, primarily in crisis management, environmental protection, spatial planning, engineering geodesy, agriculture, defence and other areas where geospatial data are used.

Following the world trends, the Agency for Real Estate Cadastre has started a project that includes LiDAR scanning of the entire territory of the Republic of North Macedonia, building institutional capacities for the process of controlling LiDAR data and their archiving, as well as creating a LiDAR portal to distribute the LiDAR products to the all interested users.

Keywords: LiDAR, Point Cloud, Digital Terrain Model, Digital Surface Model

1. LIDAR TECHNOLOGY

1.1 BASIC CONCEPTS AND DEFINITIONS

LiDAR or Light detection and ranging (LiDAR) is an active remote sensing technology that uses electromagnetic energy in the optical range to detect an object (target), determine the distance between the target and the instrument (range), and deduce the physical properties of the object based on the interaction of the radiation with the target through phenomena such as scattering, absorption, reflection, and fluorescence.

The basic components of the LiDAR system are:

- LiDAR scanner,
- GPS receiver,
- IMU – Inertial Measurement Unit,
- Computer and data warehouse.

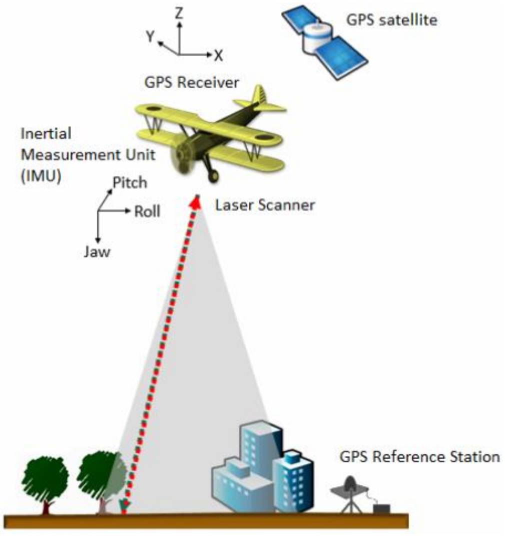

Figure 1. Components of the LiDAR system

The concept of this technology is based on accurate measurement of the time for which the laser beam is emitted from the corresponding module in the LiDAR system to the ground, then is reflected from the ground or from an object on the ground (vegetation, building, bridge, power line, etc.) and then returned to the sensor housed in the LiDAR system.

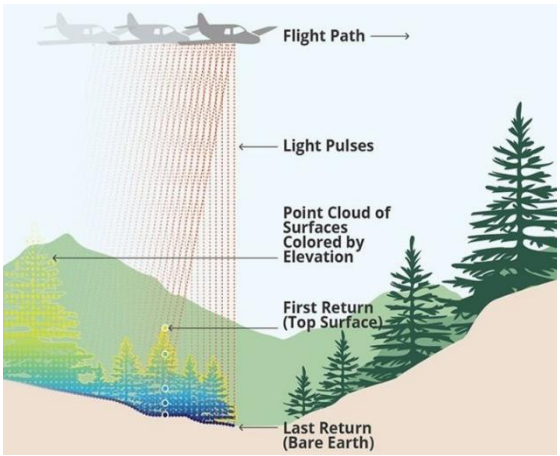

Figure 2. Basic concept of LiDAR technology

When the laser beam returns, it carries information about the object it has come in contact with, including distance and optical characteristics such as reflectivity.

A single emitted laser beam can be returned to the sensor as one or more returns. Each emitted laser pulse that encounters multiple reflective surfaces as it travels to the ground is divided into as many returns as there are reflective surfaces.

The first returned laser beam is the highest characteristic on the Earth's surface, such as a tree or the top of a building. The first return can also be from the ground and in that case only one return from the corresponding laser beam will be detected in the LiDAR system.

The multiple returns allow multiple objects to be detected at different altitudes within a single laser pulse. The middle return is used for vegetation analysis while the last return is used for terrain models of the ground.

The last return does not always have to be a return from the ground. For example, if the pulse hits a thick branch on its way to the ground, it does not actually reach the ground. In this case, the last return is not from the ground.

LiDAR has many advantages that make this technology attractive in the field of spatial data acquisition, some of those are:

- spatial data acquisition speed (around 500000 points per second),
- vertical data accuracy (+/- 10 cm),
- point density (minimum 2 points per m^2, but often 5 or more points per m^2),
- independence from the presence of vegetation to collect data of the structure of the terrain,
- independence from the period of the day when the data is collected,
- integration with other data sources (orthophoto images, vector data for parcels, buildings, roads, bridges, etc.).

1.2 LIDAR PRODUCTS

1.2.1 Point Cloud

Post-processed spatially organized LiDAR data is known as Point Cloud. The starting Point Cloud is a large collection of 3D points, which have defined X, Y and Z coordinates but can also contain attribute data for:

- Intensity,
- Number of returns,
- Return number,
- Class code according to the type of object from which the point is reflected,
- RGB value,
- GPS time,
- Scan angle,
- Scan direction.

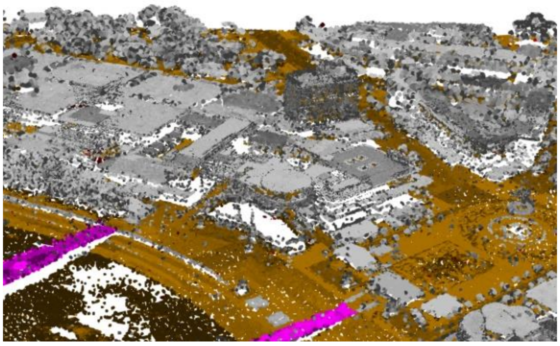

Figure 3. Point Cloud visualized by class and intensity (brown–ground points, pink–bridge points, grey–unclassified points)

1.2.2 DTM vs. DSM

LiDAR Point Cloud is commonly used as input for creating digital elevation models.
DTM (Digital Terrain Model) is a mathematically defined continuous surface in digital form that with a certain accuracy represents the terrain.
DSM (Digital Surface Model) is a mathematically defined continuous surface in digital form that with a certain accuracy represents the terrain along with natural and artificial objects located on the terrain.

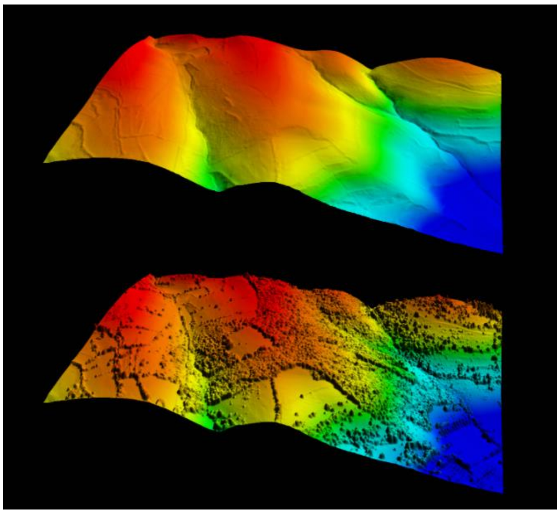

Figure 4. Digital Terrain Model vs. Digital Surface Model

1.3 APPLICATION OF LIDAR TECHNOLOGY

LiDAR technology can make a major contribution in various areas that need high-quality spatial data.

1.3.1 Engineering Geodesy and Civil Engineering

LiDAR provides a detailed view of objects of interest, identification of changes in relation to the planned and performed condition or condition recorded after a certain time. LiDAR products can also be used as a basis for designing and volume calculation.

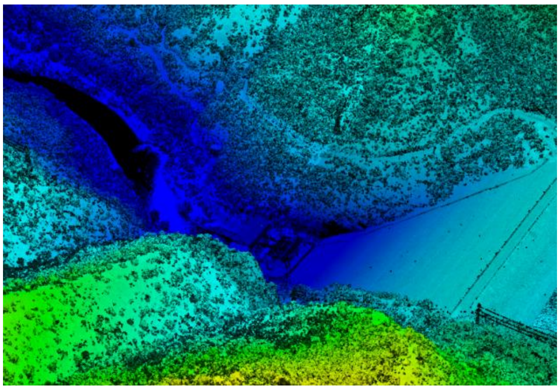

Figure 5. Point Cloud of Kozjak Dam

1.3.2 Spatial Planning

By applying the data from the LiDAR scanning and combining them with spatial data from other sources (orthophoto images, vector data for parcels, buildings, roads etc.) a 2D/3D spatial models can be created, which are an excellent basis for improving spatial planning, analysis of lighting, etc.

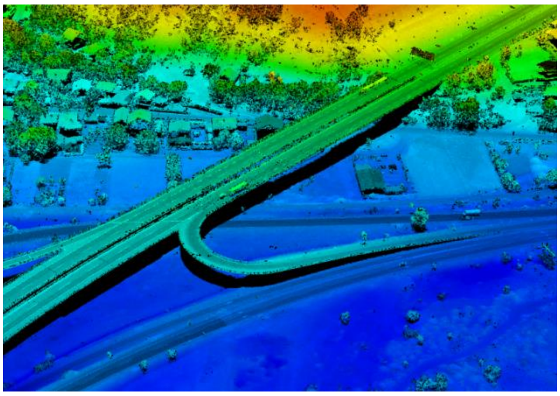

Figure 6. Point Cloud of Skopje - Northern Bypass

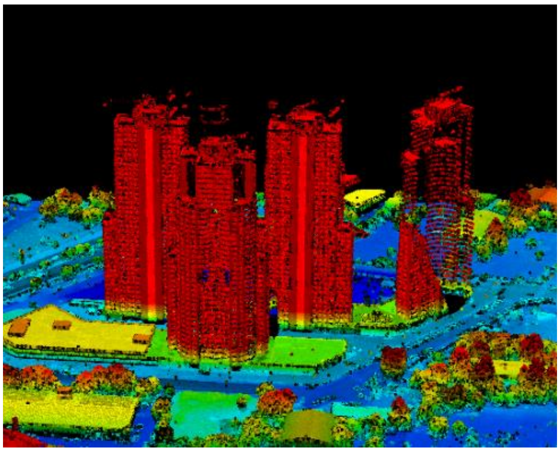

Figure 7. Point Cloud of Cevahir Towers

1.3.3 Forest and Environmental Management

LiDAR technology enables the detection of each tree, its height, volume, density, which is of great importance in forest management, classification of vegetation types and fire protection.

1.3.4 Geology

In areas with dense vegetation, it is almost impossible to accurately detect geological phenomena such as landslides. LiDAR technology can be used to assess impacts and movements in this type of areas.

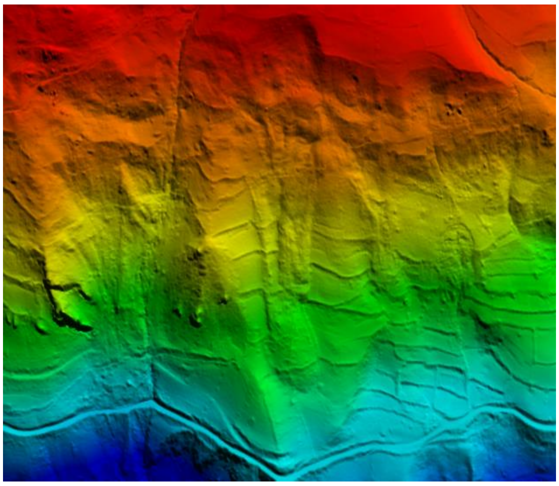

Figure 8. DTM of a landslide in Polog Region

1.3.5 Archaeology

In archaeological excavations, LiDAR technology can be applied in planning research actions, field studies and detailed site research. LiDAR data can detect debris of wall structures in the study area.

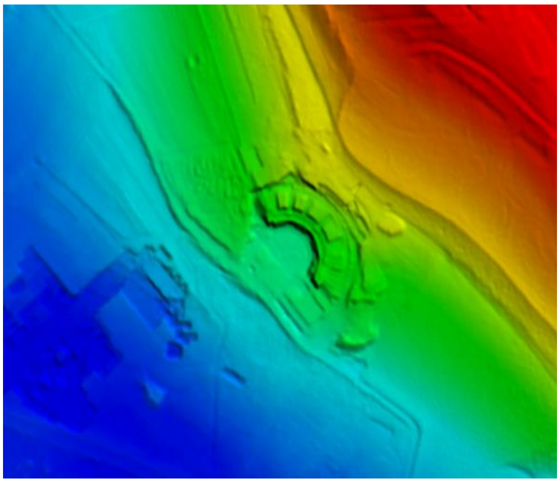

Figure 9. DTM of Skupi Archaeological Site

2. LIDAR PROJECT

The Norwegian Ministry of Foreign Affairs and the Norwegian Mapping Authority (Statens Kartverk) have concluded an agreement for financial aid for the Balkan countries (North Macedonia, Serbia, Bosnia and Herzegovina, Montenegro, Albania and Kosovo) with the goal of improving the access to updated geographic information and to basic registers, for the period October 2017 – October 2020. With this agreement, the Government of Norway, through the Norwegian Mapping Authority, supports the countries from the region in the process of obtaining updated geographic information in digital form, available via the Internet both to the public and the private sector, for the purpose of mitigating the effects of climate change. The grant agreement states that individual agreements on institutional cooperation will be concluded between the Norwegian Mapping Authority and each of the partner institutions in the region. Within this cooperation, the Agency for Real Estate Cadastre applied with the project "*LiDAR surveying of the territory of the Republic of Macedonia for the preparation of precise digital height models and other quantitative and qualitative analyses of the Earth's surface*".

For the purpose of implementing the project activities, the Agency for Real Estate Cadastre and the Norwegian Mapping Authority in March 2018 signed a cooperation agreement. The project activities include LiDAR scanning of the territory of the Republic of North Macedonia, building institutional capacities for the process of controlling the LiDAR data and their archiving, as well as creating a LiDAR portal for distribution of the LiDAR products to the end-users.

LiDAR scanning of the territory of the Republic of North Macedonia within the first phase of the project was carried out by the company MGGP Aero from Poland, based on a tripartite agreement signed with the Agency for Real Estate Cadastre and the Norwegian Mapping Authority in December 2018. Within the first phase of the LiDAR project, appropriate LiDAR products have been prepared for 11072 km^2 of the territory of the Republic of North Macedonia. That is 43% of the country. The territory covered by LiDAR scanning is divided into 14 blocks:

- Block 1, Block 4, Block 12, Block 13 and Block 14 – with point density 5ppm^2,

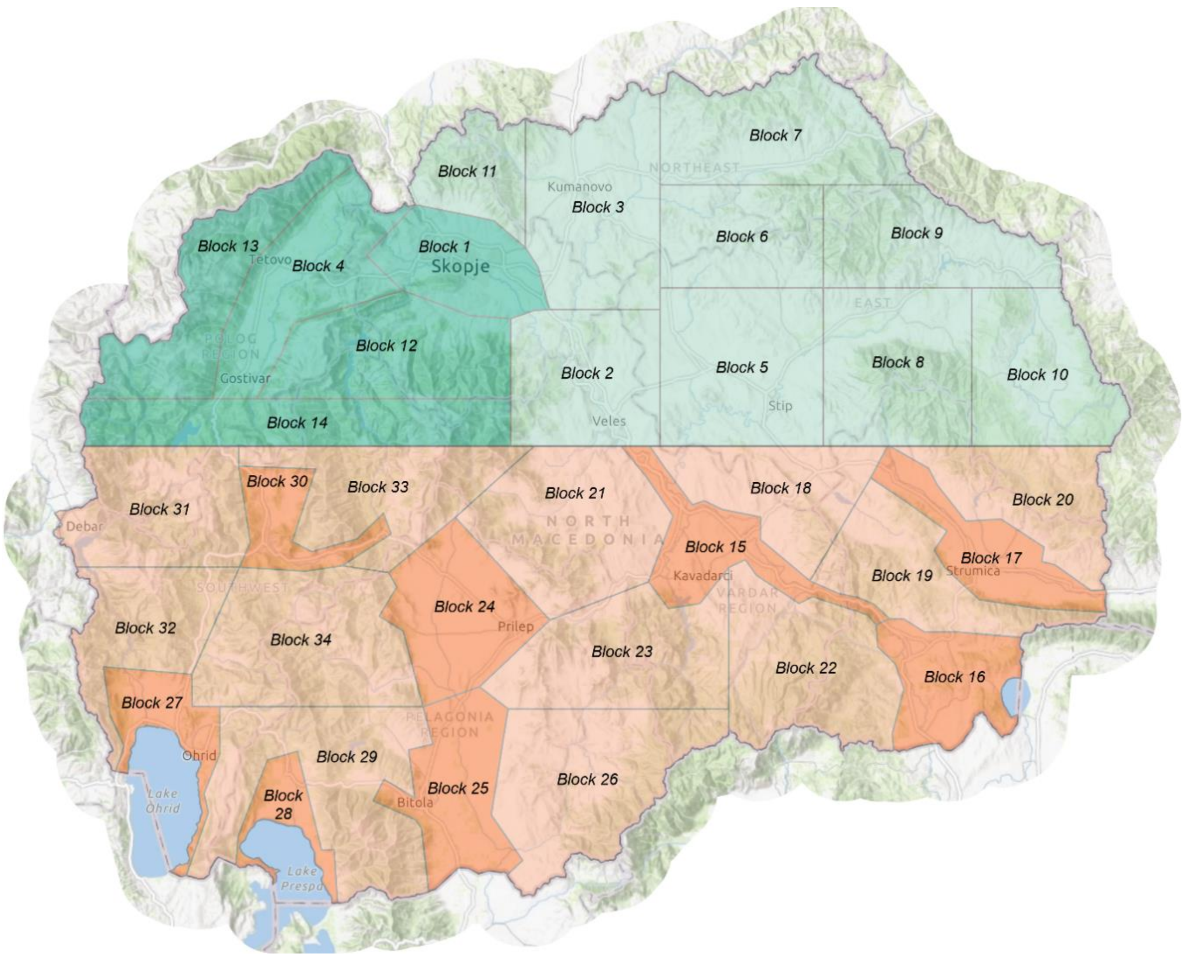

Figure 10. LiDAR project – division into blocks (green–first phase, orange–second phase)

- Block 2, Block 3, Block 5, Block 6, Block 7, Block 8, Block 9, Block 10 and Block 11 – with point density 2ppm^2.

In September 2020, the Agency for Real Estate Cadastre and the Norwegian Mapping Authority, have concluded an Annex to the Agreement on Institutional Cooperation regarding the LiDAR project in order to extend the project duration period by 31.12.2021, as well as expansion of the budget frame. In October 2020, a tripartite agreement was signed for the second phase of the project, between the Norwegian Mapping Authority, the Agency for Real Estate Cadastre and the company Primul Meridian from Romania. Within the second phase of the LiDAR project, the territory of the Republic of North Macedonia covered by LiDAR scanning is divided into 20 blocks:

- Block 15, Block 16, Block 17, Block 24, Block 25, Block 27, Block 28 and Block 30 – with point density 5ppm^2,
- Block 18, Block 19, Block 20, Block 21, Block 22, Block 23, Block 28, Block 29, Block 31, Block 32, Block 33 and Block 34 – with point density 2ppm^2.

3. LIDAR PORTAL

In order to offer a simple and fast way to provide LiDAR data to all interested users, a LiDAR portal has been created as a technical solution for distributing the data obtained with the LiDAR scanning of the terrain.

Figure 11. Home page of LiDAR portal

Customers have the ability to choose one of three offered options to specify the desired area for purchasing LiDAR products. The area for purchasing may be specified by:

- selected frame/s at a scale of 1:1000,
- uploaded polygon in shp–format into the LiDAR portal,
- drawn polygon directly into the LiDAR portal.

For the specified area, the customer has the ability to choose a type of LiDAR product that is interested in: LiDAR Point Cloud, DTM or DSM.

Figure 12. Drawn polygon as input for purchasing LiDAR products

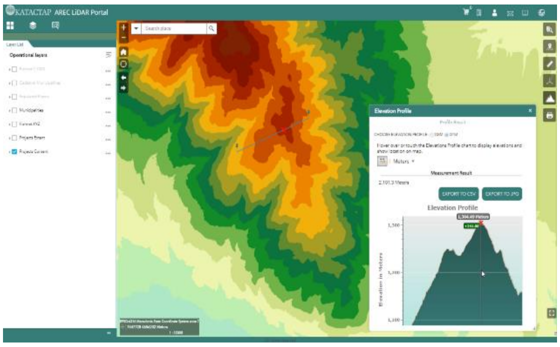

Figure 13. Elevation profile in LiDAR portal

LiDAR portal offers various possibilities for visualization and analysis of LiDAR data, tools for measuring distance and area, as well as tools for creating elevation profile of a digital terrain model or digital surface model.

4. CONCLUSION

LiDAR technology opens a new era in the field of spatial data acquisition, providing fast, accurate and detailed spatial data. This technology, as well as the technology for processing the data obtained from LiDAR scanning and for creating new products, are becoming more accessible and more advanced day by day and give a great contribution in all areas that need quality spatial data. All interested users now are able to take advantage of this powerful spatial data provided through the LiDAR project of the Agency for Real Estate Cadastre.

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ANALYSIS OF DATA SETS AND SERVICES ON THE NSDI GEOPORTAL

The paper analyzes the data sets and services published on the new NSDI geoportal. The Agency for Real Estate Cadastre as the institution which according to law is responsible for establishing, maintaining and enabling public access to the NSDI geoportal, started to modernize the existing NSDI geoportal in order to upgrade it and increase the number of data and services.

The new NSDI geoportal was launched in November 2020 and, as a cutting-edge software solution based on the ESRI platform, enables viewing and using the spatial data through web-oriented services and interoperable infrastructure. Several aspects were in focus during the creation of the geo-portal, such as the products, users, administrative aspects, business aspects, strategic goals, but also certain visions and concepts about the way the system should function in the future

Keywords: NSDI, geoportal, data set, web service, INSPIRE, harmonization

1. INTRODUCTION

The establishment of the National Spatial Data Infrastructure (NSDI) in the Republic of North Macedonia is regulated by the Law on the National Spatial Data Infrastructure of the Republic of Macedonia (*Official Gazette of the Republic of Macedonia* no.38/14 and 106/16). The law transposes the European directive INSPIRE (2007/2/EC) which regulates the establishment of spatial data infrastructure in the European Community.

The objective of the establishment of the NSDI is to facilitate the access, sharing, use and distribution of standardized spatial data and services in an efficient, effective and synchronized manner.

According to the Law on NSDI, 20 institutions are the NSDI subjects/stakeholders: Ministry of Justice, Ministry of Defense, Ministry of Interior affairs, Ministry of Economy, Ministry of Agriculture Forestry and Water Economy, Ministry of Local Self-Government, Ministry of Culture, Ministry of Transport and Communications, Ministry of Environment and

Physical Planning, Ministry of Information Society and Administration, Ministry of Political System and Inter-Community Relations, State Statistical Office, Agency for Real Estate Cadastre, Agency for Spatial Planning, Central Registry of the Republic of North Macedonia, Crisis Management Center, City of Skopje, Economic Chamber of the Republic of North Macedonia, Geological Survey of the Republic of North Macedonia and Association of Economic Chambers of the Republic of North Macedonia.

In order to facilitate the data sharing between state institutions, an Agreement for sharing of spatial data sets and services has been prepared and already signed by fifteen stakeholders. The agreement sets a legal framework which regulates the sharing of data within the NSDI geoportal.

Currently, 161 metadata, 79 web services and 82 geospatial data sets are published and available for using on the NSDI geoportal, demonstrating expansion in modern way of sharing and using data sets through web services (Figure 1).

Figure 1. Number of data sets and services on the NSDI geoportal (http://nipp.katastar.gov.mk)

2. ARCHITECTURE AND FUNCTIONALITY OF THE NSDI GEOPORTAL

The NSDI geoportal integrates spatial data sets from the NSDI subjects. It is based on ISO and OGC standards and the INSPIRE directive implementing rules, with the focus on the web network services architecture.

The geoportal is user-oriented and contains many of information and data that users need. The data sets from the NSDI geoportal can be used by the state institutions, private sector and citizens. With one click on the NSDI geoportal users can access requested information, see all data published on the portal (hydrographic network, road network, cadastral parcels, land use, buildings, geographical names, elevation model, orthophoto map and other). The geoportal contains several functionalities: search application, metadata catalogue with a tool for metadata creation, map viewing application /GIS browser, registries, administrative module and module for e-commerce services.

The portal enables connection of spatial data sets and services between the NSDI subjects, as well as with third parties interested. The NSDI geoportal is used to manage the NSDI through facilitation of access, exchange, sharing, searching and using of standardized spatial data and services. Its main components are: geospatial data sets, metadata, network services (discovery, view, download) and interoperability of data sets and spatial data services.

The architecture of the NSDI geoportal (Figure 2) is in the line with the architecture on the INSPIRE geoportal.

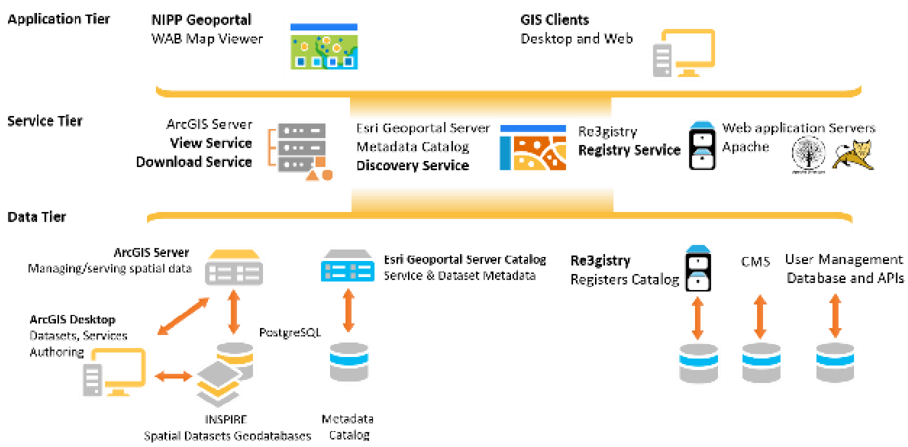

Figure 2. Architecture of the NSDI geoportal

A central functionality of the NSDI geoportal is the metadata catalogue. Metadata is presented with xml files which provide information on the content, owner, quality, type, accuracy, spatial information and updates of data set or service. This is essential information that other individuals or organizations need to know before they can use the data or service. A Metadata Catalogue Service is a mechanism for storing and accessing descriptive metadata and allows users to query data items based on desired attribute the catalogue service. NSDI geoportal contain a metadata editor (tool for creation of metadata for data set and service creation). All metadata on the NSDI geoportal are created according the Regulation of standards of metadata ("Official Gazette of the Republic of North Macedonia" no. 123/19) and Commission regulation (EC) no 1205/2008 implementing directive 2007/2/EC of the European parliament and of the Council as regards metadata.

Overview of architecture and functionality of the NSDI geoportal shows that geoportal fulfils following conditions: keeps the data where it can be maintained most effectively, the ISO and OGC standards are implemented, has the possibility to combine seamless spatial information from different NSDI subjects and share it with many users and applications, information for data set and services, transparency and availability of data and services.

3. ANALYSIS OF THE DATA SETS ON THE NSDI GEOPORTAL

Themes of data sets are described within the article 5 of the NSDI law. The NSDI law defines 32 themes of data sets: coordinate reference systems, geographic network systems (grid systems), geographic names, administrative units, addresses, cadastre parcels, traffic networks, hydrography, protected areas, elevation/height terrain model, earth surface, ortho-photogrammetry, geology, statistical units, buildings, soil, land use, human health and protection, services of public interest, environment monitoring systems, Production and industrial capacities, agricultural and aqua - cultural capacities, population density - demography, areas for management, limitation and regulation and reporting units, natural risk zones, atmospheric conditions, meteorological geographical characteristics, bio - geographic regions, habitats and biotopes, density of the species, energy sources and mineral resources. In comparison with the INSPIRE themes, Macedonian NSDI lacks two themes: oceanographic geographical feature and sea regions, since these are not applicable. The number of 67 published national data sets on the NSDI geoportal belong to 19 themes of data sets is presented in Figure 3.

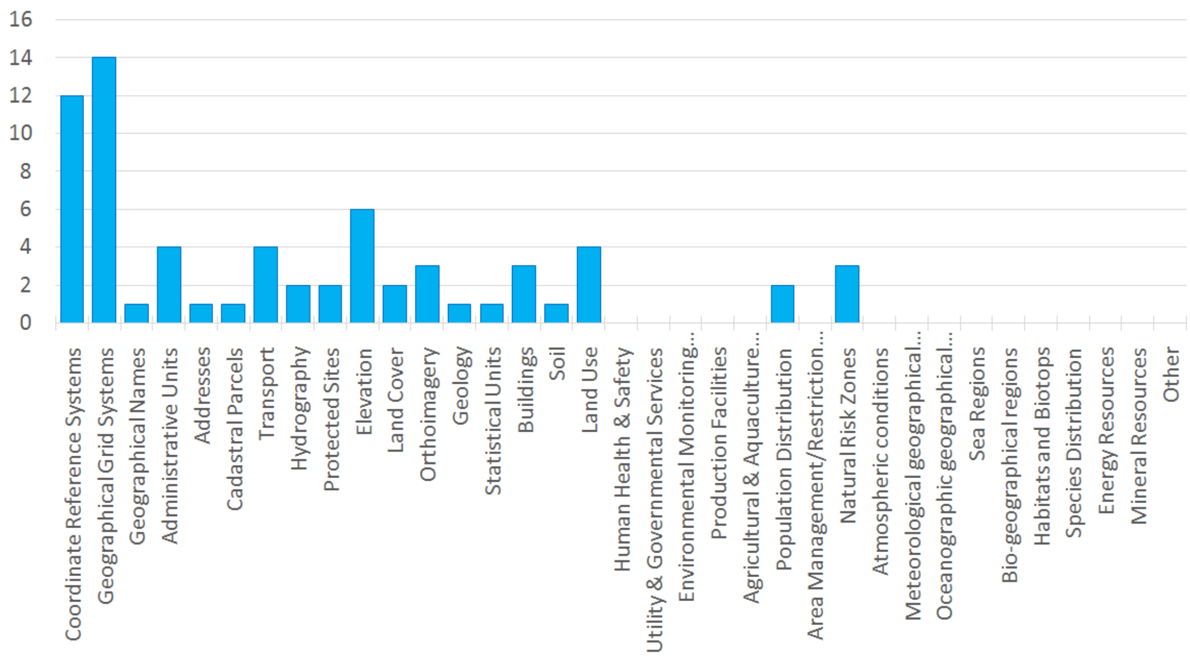

Figure 3. Number of spatial data sets per themes published on the NSDI geoportal

All of the published data sets on the NSDI geoportal are: transparent (data for governing should be easily found and available), accessible (disparate data sources can be easily combined, no matter what their origin is), scalable (data is structured so that it can be used across different scopes), reliable (data is collected only once and is maintained regularly) and accountable (metadata is formalized and set to a standard).

Having in mind that the NSDI initiative is directly related to the European INSPIRE Directive, the data from the Agency for Real Estate Cadastre are a step ahead of the data from the other NSDI subjects. They are harmonized in accordance with the relevant INSPIRE data specifications (technical guidance) and are interoperable with data sets from European countries. The number of 15 national data sets which are harmonized with INSPIRE data specifications are shown in Figure 4.

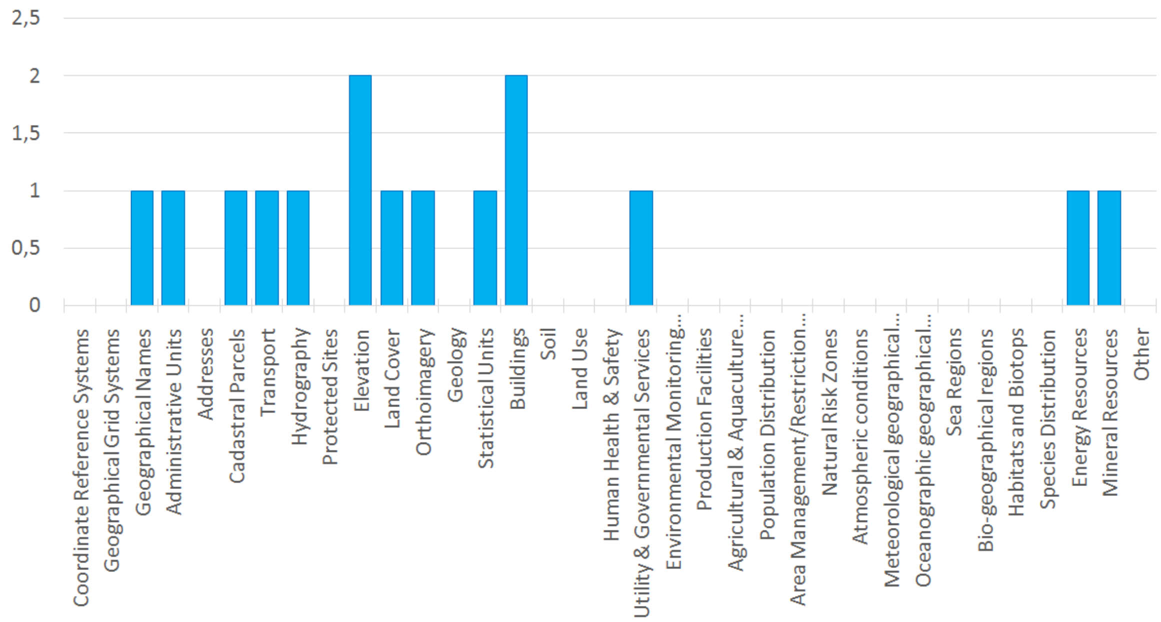

Figure 4. Spatial data sets harmonized with INSPIRE data specifications

4. ANALYSIS OF THE WEB SERVICES AT THE NSDI GEOPORTAL

Following services are available for data sets published on the NSDI geoportal: discovery, view, and download, through following protocols:

- URL address (Get Capabilities) for OGC services (OWS, WMS, WMTS, WCS, WFS, WPS, SOS, SPS, CSW, KML)
- Open Archive Initiative
- Get Capabilities URL for CS-W Discovery
- URL address (WAF HTTP/FTP)
- URL address for JSON data format DCAT
- ATOM Feed
- URL for metadata XML document
- OpenSearch XML

The term web service describes a standardized way of integrating web-based applications using OGC (Open Geospatial Consortium) and ISO (International Organization for Standardization) standards via Internet Protocol. The service allows the user to search, view and download spatial data. The analysis of the services published on the geoportal gives the following results:

WMS is the most widely accepted and popular web mapping service, which describes the communication mechanisms and allows software products to request and provide pre-assembled map images ("composite" map images, which can contain both vector and raster data) to the requesting client. Web Map Service interface standard (WMS) provides a simple HTTP interface for requesting geo-registered map images from one or more distributed geospatial databases. A WMS request defines the geographic layer(s) and area of interest to be processed. The response to the request is one or more geo-registered map images (returned as JPEG, PNG, etc.) that can be displayed in a browser application. The interface also supports the ability to specify whether the returned images should be transparent so that layers from multiple servers can be combined or not.

Web Feature Service (WFS) interface standard provides an interface allowing requests for geographical features across the web using platform-independent calls. One can think of geographical features as the "source code" behind a map, whereas the WMS interface or online tiled mapping portals like Google Maps return only an image, which end-users cannot edit or spatially analyze. The Web Feature Service (WFS) represents a change in the way geographic information is created, modified and exchanged on the Internet. Rather than sharing geographic information at the file level using File Transfer Protocol (FTP), for example, the WFS offers direct fine-grained access to geographic information at the feature and feature property level.

Download service offer transmission of a file from NSDI geoportal to the computer system using Internet point-of-view, to download a requested file and to receive it. Those services can be realized using File Transfer Protocol (FTP) which is the Internet protocol for downloading and uploading files and a number of special applications.

In relation to the web services published on the NSDI geoportal, 55% of them are view services what enable viewing the data sets using Web Map Services (WMS) and 45% is download services which include web feature service (WFS), Atom and web cover services (WCS) (Figure 5).

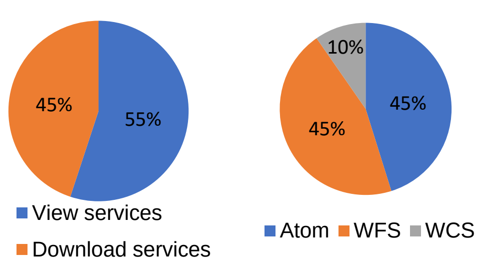

Figure 5. Graphical presentation of the coverage of the web services on the NSDI geoportal (Left: Ratio between view and download services; Right: Ratio of types of download services)

The comparison of data sets and services on the NSDI geoportal indicates that only 34% data sets are available for use by the view services, and 24% are available for use by the download services. The statistic also to point out that a lot of data sets don't have a web services, they only have metadata (Figure 6).

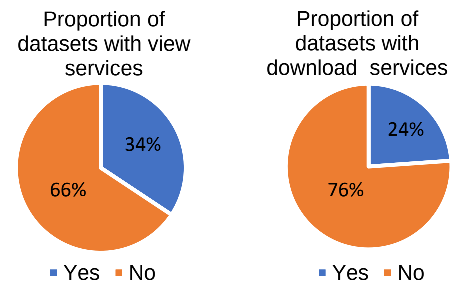

Figure 6. Percentage of representation of data sets through web services

The users of the NSDI geoportal can use 10% view services free of charge, but the download services at the moment may not be used free of charge (Figure 7). Further geoportal development considers the implementation of open data initiative which will allow part of the download services to be used free of charge.

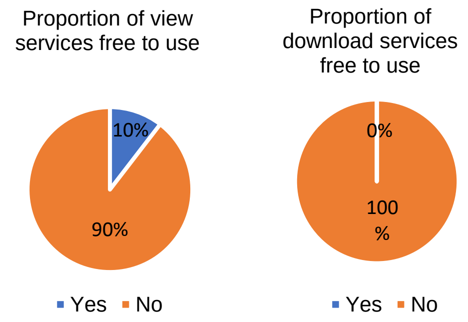

Figure 7. Percentage of web services use free of charge

5. CONCLUSION

Analysis of data sets and services published on the NSDI geoportal illustrate that the structure of the NSDI geoportal, data sets and services are well structured and they are harmonized with INSPIRE directive. At the moment the number of published data sets and web services is good, but not enough, because it is necessary to publish data sets for all themes. This means that the NSDI stakeholders are needed to take activities to enlarge the number of published data sets and services on the NSDI geoportal. More data sets and services published on the geoportal will contribute to better future on NSDI and will have positive impact to the society.

The development of NSDI geoportal contributes to significant progress in NSDI area in recent years. One major goal in this area is cooperation and support of the NSDI subjects with reference data, which are needed in their daily work. However, one of the challenges in the future is to take activities for wider promotion of the NSDI geoportal in order to increase using available data sets through web services and motivate state institutions to share datasets via NSDI geoportal.

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AN APPROACH FOR TUNNEL RISK MANAGEMENT

Modern tunnel construction is a very complex and intense process. In this process different uncertainties and risk can occur and they should be adequately managed. This paper explains the systematic approach of the tunnel risk management as a general concept that should include all the available information and preliminary data in order to obtain a quality solution.

Keywords: Tunneling, construction uncertainties, risk management, acceptable risk, decision making

1. INTRODUCTION

The importance and value of construction projects, especially in the area of tunnels require the necessity for different approaches for assessing and dealing with risks. Therefore, there is a wide application of analyzes and methods for risk assessment, as well as their management. Globally, there is an increasing trend for the application of risk-based approaches, which idea is to increase awareness of this issue in various branches of society.

Once the risks are assessed and their intensity is determined, the management follows. This process is crucial in all design and construction issues, as it should define ways to deal with unwanted events, and it is desirable to perform it in an environment of good cooperation between stakeholders.

2. TUNNEL RISK MANAGEMENT

Tunnel risk management usually includes the following:

- Analysis of the results from the hazard and risk assessment;
- Decision making for risk treatment;
- Implementation of the proposed treatment measures;
- Monitoring.

In certain literatures the hazard and risk assesment is also stated in the risk management.

2.1 ANALYSIS OF THE ASSESSED HAZARDS AND RISKS

Once the hazards and risks have been assessed, the results obtained should be analyzed in order to suggest appropriate measures for their treatment. The analysis generally consists of classifying, ranking and comparing the assessed risks in relation to predefined parameters. Classifications or ranking systems can be defined specifically for the project itself, but most often their origin is based on more detailed research on previous and current problems and experiences in this area (Table 1). One of the main goals of these analysis is to define the so-called acceptable (tolerable) level of risk. In relation to this level, the other levels (classes) can be determined and the necessary measures can be determined accordingly. The type of results has a great influence on the classification, more precisely whether it is a qualitative description or a quantitative value. Several organizations and agencies dealing with this problem have issued detailed classifications according to which the assessed risks can be analyzed.

Table 1. Example of risk classification and actions (measures), Eskesen et al (2004)

Risk classification	Example of actions that should be carried out for each class
Unacceptable	The risk shall be reduced at least to Unwanted regardless of the cost.
Unwanted	Risk mitigation measures shall be identified. The measures shall be implemented as long as the costs of the measures are not disproportionate with the risk reduction obtained (ALARP principle – As Low As Reasonably Practicable) .
Acceptable	The hazards shall be managed throughout the project. Consideration of risk mitigation is not required.
Negligible	No further consideration of risks or hazards is needed.

The most critical risks in society are those with the greatest consequences, and that represents human victims. Most often, these analyses first consider these risks and therefore in the literature there is a large number of data and values presented in relation to the number of victims for a certain period of time.

In the case of tunnel construction, the values for the occurrence of a risk are usually in relation to the entire period of construction. Depending on the parameters considered, the risks and hazards can also be interpreted in terms of the length of the tunnels or the number of tunnels if multiple cases are considered.

Acceptance limits for tunnels and construction in general range from 10^{-2} to 10^{-4}, often referred to as the ALARP zone or acceptable area. These values refer to the probability of occurrence of a human victim in a certain period which is usually taken as one year. Acceptance limits can also be used to analyse other critical risks, such as those with large economic and time losses or large environmental impacts.

In our country there are no rules and guidelines that define the acceptable level of risk in tunnels. For this purpose, limits of acceptable level of risk are proposed that could be used for any type of tunnels (Figure 1).

For the probability (frequency) of occurrence of victims, the adopted limits are based on criteria and guidelines of few European countries. They are expressed through the following equations:

- Upper limit:

$$F_1 = 10^{-2} * N^{-1} \text{ for } 1 \leq N \leq 1000 \text{ victims} \quad (1)$$

- Lower limit:

$$F_2 = 10^{-4} * N^{-1} \text{ for } 1 \leq N \leq 1000 \text{ victims} \quad (2)$$

F – probability of occurence

N – number of victims

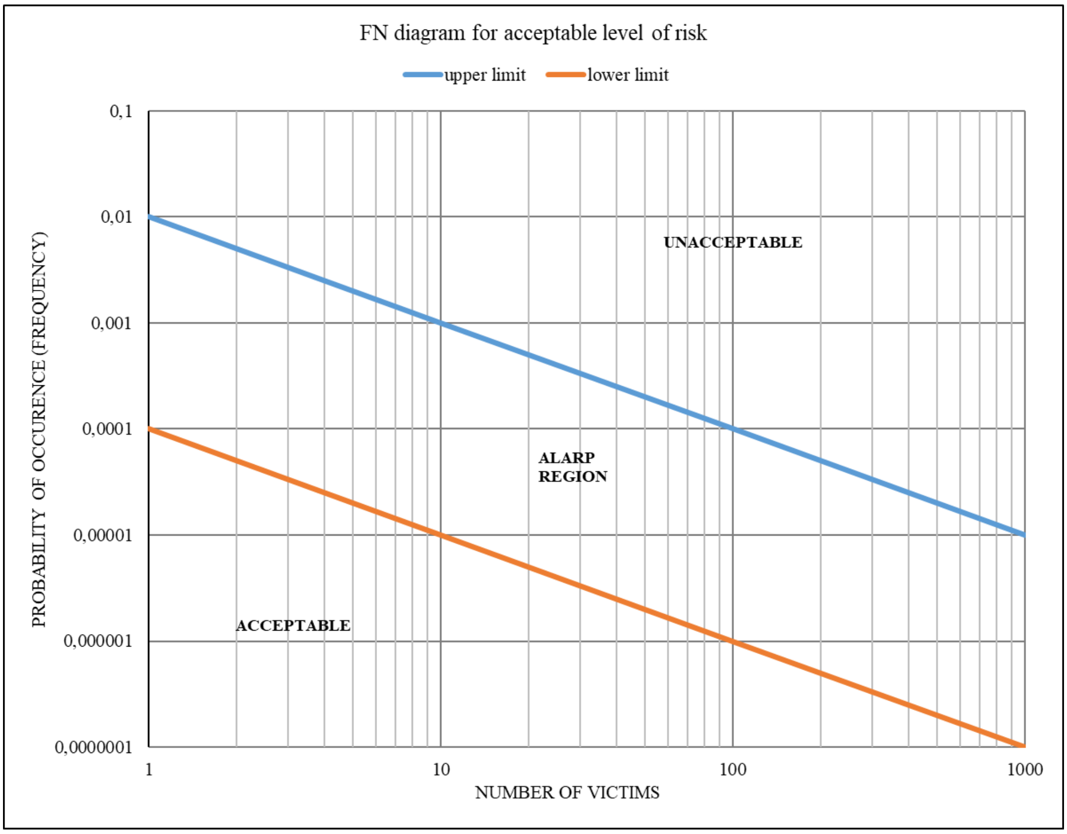

Figure 1. Proposed diagram for acceptable level of risk in relation to human victims in tunnels

The more common and frequent risks are associated with economic loss and time loss (delay). The classifications and acceptable levels for these kind of risks are usually different than the ones with human and environment consequences.

This paper presents an approach for tunnel risk management with proposed classifications and acceptable risk levels for analyzing of the assessed risk in tunnel construction.

The probability of occurrence of risks is the final value, i.e. the result of the quantitative assessment. A five-class system is proposed for ranking, which refers to the potential for risk occurrence in the entire construction period (Table 2).

Based on more detailed research, a classification is proposed that takes into account economic and time losses, which are often closely related (Table 3). The classification does not express the consequences through direct (fixed) values, but as a percentage increase of the initially defined values of the project.

Table 2. Proposed classification for the probability of occurrence of risks in tunnels

Probability of occurrence	**Interval**	
Very High	> 1/2	> 0,5
High	1/10 - 1/2	0,1 - 0,5
Moderate	1/100 - 1/10	0,01 - 0,1
Low	1/500 - 1/100	0,002 - 0,01
Very Low	< 1/500	< 0,002

Table 3. Proposed classification for economic and time consequences in tunnels

Impact on project costs and time	
Disastrous	> 80 %
Severe	10 - 80 %
Serious	1 - 10 %
Considerable	0,1 - 1 %
Insignificant	< 0,1 %

The final classification or ranking is done using a risk matrix (table 4) which is listed in the guidelines of the ITA International Tunneling and Underground Space Association). It contains 5 columns and 5 rows that correspond to frequencies (probabilities) and consequences. With this matrix there are 25 combinations between the frequencies and consequences and 4 possible outcomes. Based on those outcomes appropriate measures for the risk can be proposed.

Table 4. Risk matrix, Eskesen et al (2004)

	Consequence				
Frequency (probability)	Disastrous	Severe	Serious	Considerable	Insignificant
Very high	Unacceptable	Unacceptable	Unacceptable	Unwanted	Unwanted
High	Unacceptable	Unacceptable	Unwanted	Unwanted	Acceptable
Moderate	Unacceptable	Unwanted	Unwanted	Acceptable	Acceptable
Low	Unwanted	Unwanted	Acceptable	Acceptable	Negligible
Very Low	Unwanted	Acceptable	Acceptable	Negligible	Negligible

2.2 ANALYSIS OF RAILWAY TUNNELS

Using this approach for risk classification and ranking, an analysis of the assessed risk was made for railway tunnels. Four railway tunnels on the future railway line on the corridor 8 in Macedonia (section Kriva Palanka – border pass Deve Bair) were previously assessed in terms of hazards and risk (table 5). The assessment covers the three most critical hazards and risks that threaten the construction of these railway tunnels.

Using these quantitative results, a classification was made according to the risk matrix (table 4). For a serious and severe consequence and a very low and low probability of occurrence (< 0,002; 0,002 – 0,01) the risks are classified as acceptable. This means that the hazards shall be managed throughout the project. Consideration of risk mitigation is not required.

In terms of the diagram for probability of occurrence of human victims (figure 1), the values belong to the ALARP region, which means that the measures shall be implemented as long as the costs of the measures are not disproportionate with the risk reduction obtained.

In general, the analysis and the results can help in the management process during the construction of the tunnels, which can start after a very longer period than planned.

Table 5. Results from previous quantitative risk analysis for the railway tunnels

Hazards	**RISK (probability of occurrence)**	
	Severe	**Serious**
Ground water inflow	0,0000779 (very low)	0,0007011(very low)
Excessive deformation (swelling)	0,0001990(very low)	0,0017915 (low)
Instability of the excavation face	0,0003130 (very low)	0,001252 (low)

2.3 DECISION MAKING

The treatment of unacceptable risks can be done in many ways. Risks can be avoided, reduced (mitigated) or transferred. Some risks can be avoided by adapting to a more robust method of construction or changing the tunnel alignment. Other risks may be transferred to insurance companies. However, most of the risks must be reduced to an acceptable level.

Risk mitigation can be seen as part of quality assurance work.

The optimal methods for risk mitigation are aimed at the epistemic nature of uncertainties, which implies that risks can be reduced by obtaining additional information.

The selection of appropriate measures should be made in an environment of good cooperation between stakeholders. The following parameters usually have the largest influence on the decision making process:

- The results from the analysis of assessed hazards and risks;
- Type of project;
- The size of the project;
- Budget size;
- Design phase;
- Stakeholders and third parties;
- Possibility to implement and folow the effects of the proposed measures.

2.4 RISK MONITORING

One of the least discussed components of risk management is monitoring (figure 2). The main objectives of the monitoring are:

- Predicting future events;
- Validation of the modeled asumptions;
- Improving of the overall accuracy of the risk-related decisions;
- Beter communication between the stakeholders.

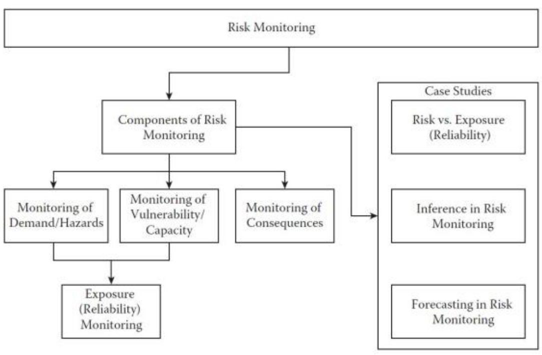

Figure 2. Risk monitoring diagram, Ettouney et al (2017)

The methods of monitoring (observation) and the location of the instruments depend on the field conditions, methods and technologies for construction and the nature of the risk events.

Risk monitoring is esspecialy important in structures such as tunnels or bridges.

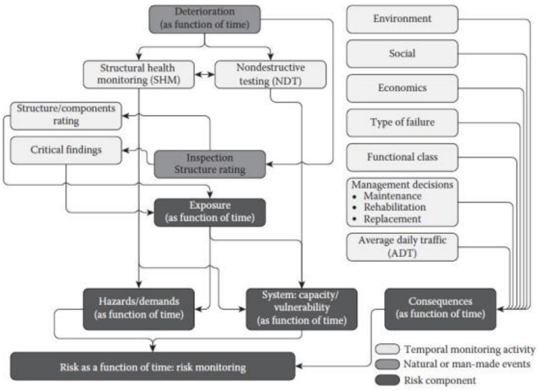

Figure 3. Risk monitoring components, Ettouney et al (2017)

3. CONCLUSIONS

Tunnel risk management is an important part in the construction phase. The success and benefits of implementing an effective risk management depend on the quality of identified risk reduction measures and the active involvement, experience and general opinion of the participants (Investor, Designers, Contractors and Supervisors). Risk management is not achieved by implementing systems and procedures individually, but through meetings where there is an understanding and appreciation of this issue.

The approach showed in this paper is based on a quantitative risk assessment which in the end gives results that can be classified or ranked. The proposed values for probabilities of occurrence and consequences may be used not only for tunneling but in other large civil engineering projects also.

The results from the railway tunnels analysis show that the three most critical risks for the projects are acceptable in terms of economic and time consequences, but in relation to human victims the risks belong in the ALARP region. This means that before (or during) the constructions of the tunnels appropriate measures should be proposed and considered.

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ASSESSMENT OF THE IMPACT OF BEND TYPE ON FLOW CHARACTERISTICS IN 180° COMPOUND BENDS

River bends are rarely simple, i.e., the bend radius is rarely constant throughout the bend length. They are rather irregular, i.e. their shape in the plan can be approximated with different bend radii. Since the bulk of previous research is concerned with simple bends, this paper aims at studying the effect of variable bend curvature on flow in an irregular bend with two bend radii. The effect is studied numerically, using a 3D finite volume based model SSIIM1, which solves Reynolds-Averaged Navier-Stokes (RANS) equations with two-equation turbulence model closure. After successful calibration against experimental data from the simple 180° bend, the model is used to simulate flow in 180° variable curvature bends with the following combinations of relative curvature: mild-mild, mild-sharp and sharp-mild. Results have shown that: (1) The maximum spanwise water surface slope always develops at the entrance to the sharp bend, (2) the greatest slope develops in the sharp-mild layout, (3) the location where the maximum velocity path cuts the bend centerline does not change with the bend layout (simple or variable curvature), (4) the bend curvature sequence affects lateral velocity distributions of the streamwise velocity only in the middle of the bend, (5) the relative curvature sequence has an adverse effect on the size of two regions with maximum bed shear stress – one around the curvature inflection point and the other close to the bend exit.

Keywords: Variable Curvature Bend, Relative Curvature, Shear Stress, SSIIM1

1. INTRODUCTION

River channels are rarely straight. Even in straight reaches the thalweg meanders. Artificial channels (irrigation, drainage and navigation canals) also have bends. They are usually forced by various constraints (design, topographic, etc.). River bends are rarely simple, i.e., they rarely have a constant radius throughout their length. They rather have variable curvature, which means that their shape in the plan view can be described with several different radii. There are many studies of flow in river bends ([5-8], [10,11], [15], [33], [25], [27], [19], [39], [2], [23], [38]). Further, the flow in bends is three-dimensional and the hydrodynamics of bends in movable bed channels is very complex due to scour and deposition in different parts of the bend. The rate of erosion in a bend depends on the flow properties, sediment composition of river/ channel banks and the channel geometry [4]. Rozovskii [31] classified bends into sharp and mild bends, depending on the value of the radius to width ratio (R / W), i.e., relative curvature (where R is the radius of the bend centreline and W is the channel width). According to Rozovskii's classification, the R / W-value in sharp bend is less than 3.0 and for mild bends, it is larger than 3.0. One of the early studies on the bend flow hydrodynamics was conducted by Shukry [34]. This study reports experimental results from 90° and 180° open channel bends in channels with rectangular cross section and different values of radius to width ratio. The effect of the bend was investigated under different Reynolds numbers, depth-to-width and radius-to-width ratios. Rozovskii [4] studied flow characteristics and boundary shear stress distribution in channel bends. Mosonyi and Gotz [22] investigated the secondary flow pattern and helical flow strength in bends. Their results showed that the secondary flow can be well described by changes in its strength. Leschziner and Rodi [17] presented a three-dimensional numerical model based on the finite-difference method for simulation of turbulent flow in strongly curved channels with rectangular cross-section. They studied the effect of the streamwise pressure gradient on the flow pattern in swift bends. Manssori [21] studied the sedimentation and scouring pattern in a 180° open channel bend using SSIIM model. The computed bed profiles clearly indicated development of a point bar and a pool in two regions within the bend – the first one was positioned close to the location defined by the central angle θ = 45° and the second one close to θ = 130°. Fazeli et al. [12] carried out an experimental study in a 90° open channel mild bend. They studied the effect of a spur dike on flow and scouring patterns and presented an equation for the estimation of a maximum scour depth. Ahmadi et al. [1] proposed a 2D depth-averaged model for the simulation of unsteady flow in open channel bends. The comparison of the simulation results and the experimental data has shown good agreement between predicted and measured water surface profiles in both sharp and mild bends. Stoesser et al. [35] studied numerically 3D flow and wall shear stress distributions in a meandering open channel by using the Large Eddy Simulation approach and a method that is based on Reynolds-Averaged Navier-Stokes (RANS) equations for which two different isotropic turbulence closures were employed. Their overall results indicated that the predicted bed-shear stresses in RANS simulations is about 15-20% higher than that in the LES, particularly in the centre of the channel at the entrance and the exit sections of the bend as well as near the inner bank at the apex. Results also provide clear evidence that the RANS code is able to predict time-averaged primary velocities which are in good agreement with measurements regardless of the turbulence model used. LES was found to be slightly superior to RANS in computing the time-averaged secondary velocities. Esfahani and Keshavarz [10] studied the effect of different curvatures inside the river meander on flow characteristics in a bend. Two physical models of river meander with different curvature were studied. Each model consisted of three sequential bends. Results showed that the effect of the centripetal force of preceding and succeeding bends on the tangential velocity, bed topography and flow structure in the middle, second bend was stronger in the model 1 with sharp multi bend than in the model 2 with mild multi bend. Blanckaert [5] studied secondary flow saturation, outer banks cells and inner bank flow separation of sharp bends with fixed banks and discussed their morphological effects. Ghobadian and Mohammadi [13] carried out a numerical study in 180° uniform and convergent open channel bends using SSIIM model. Their results have shown that the maximum velocity path near the water surface crossed the channel's centreline in the cross-section located between θ = 30° and θ = 40° for convergent channel, while in the uniform bend, the crossing was located at θ = 50°. Vaghefi et al. [37] carried out an experimental study in a 90° bend and investigated the effect of Froude number on scouring depths around a T-shaped spur dike. Their results showed that two scour holes developed due to a presence of the T shaped spur dike – one at the nose of the spur dike and the other one downstream of the dike. Liyaghat et al. [20] investigated the effect of increasing and decreasing width of a 180° bend on flow characteristics in a bend. They calibrated the model for a uniform bend and then compared flow patterns with those in divergent and convergent bends. Results showed that in the convergent bend the maximum velocity path near the water surface crossed the channel's centreline between θ = 30° and θ = 40°. In the divergent bend the crossing was close to θ =50°, whereas in the bend with uniform channel width, the crossing was located around θ =55°. Vaghefi et al. [36] conducted an experimental study on flow velocity components in a 180° sharp bend. The results showed that the maximum velocity occurred at the inner bank of bend due to strong pressure gradients at the bend entrance. The results also indicated that the values of the vertical velocity component at the bend entrance were positive near the bed and adjacent to the inner wall and negative in other areas.

This review shows that the previous work was primarily focused on simple bends and that the flow pattern in variable curvature bends was rarely studied. The previous work (e.g., [3], [18], [14], [24]) has also indicated that sharp bends' hydrodynamics differs from that in mild curvature bends. Additionally, the hydrodynamics in case of different combinations of sharp and mild bends may differ from those for single bends with moderate and sharp curvature. Thus, this paper aims at assessing the effect of different combinations of bend curvature sequences on flow and bed shear stress patterns in a 180° variable curvature bend. The effect is studied using a 3D finite volume based model SSIIM1. The model is calibrated first against the experimental data from a simple, mild bend. Consequently, three layouts of a variable curvature bend are analysed: "Mild-Mild", "Mild-Sharp" and "Sharp-Mild". The "Mild-Mild" layout has two mild bends with *R* / *W* = 5 in the first bend and *R* / *W* = 3.67 in the second one. The *R* / *W* ratios in the leading and the secondary bends of the "Mild-Sharp" layout are 5.83 and 2.83, respectively, whereas the corresponding ones of the "Sharp-Mild" bends are 2.83 and 5.83. The effect of different variable curvature bend layouts is studied by comparison of: (1) water surface profiles along the inner and outer banks, (2) maximum velocity paths close to the free-surface, (3) lateral profiles of the streamwise velocity in different cross-sections within the bend and (4) bed shear stress distributions with the corresponding ones in the simple bend.

2. MATHEMATICAL MODEL

2.1 GOVERNING EQUATIONS

The three-dimensional steady incompressible fluid flow is described by the mass conservation and Reynolds-Averaged Navier-Stokes (*RANS)* equations [28]:

$$\frac{\partial U_j}{\partial x_j} = 0, \quad j = 1, 2, 3 \qquad (1)$$

$$\frac{\partial U_i}{\partial t} + U_j \frac{\partial U_i}{\partial x_j} = \frac{1}{\rho} \frac{\partial}{\partial x_j}\left(-P\delta_{ij} - \rho\, \overline{u_i' u_j'}\right) \qquad (2)$$

In these equations time t and space coordinates x_j, $j = 1, 2, 3$ are independent variables. Dependent variables are: time averaged velocities in the three coordinate directions U_j, $j = 1, 2, 3$, the time averaged pressure *P*, and Reynolds stress terms $\rho\, \overline{u_i' u_j'}$, *i*, *j* = 1, 2, 3. The density of water is denoted by ρ and the symbol δ_{ij} is the Kronecker delta. The Kronecker δ_{ij} is equal to 1 if *i* = *j*, and it is equal to 0 otherwise. The last term, which describes the contribution of velocity fluctuations u_i' and u_j' to the flow of momentum in the *j*-direction, i.e. the Reynolds stress term, is modelled with the Boussinesque approximation [28]:

$$\rho\, \overline{u_i' u_j'} = \rho \nu_t \left(\frac{\partial U_i}{\partial x_j} + \frac{\partial U_j}{\partial x_i}\right) - \frac{2}{3}\rho\, k\, \delta_{ij} \qquad (3)$$

where ν_t is the eddy viscosity and k is the turbulence kinetic energy (*TKE*). The eddy viscosity is not a fluid property, but strongly depends on the state of turbulence and can be determined using turbulence models with different levels of complexity: zero-equation models, one-equation models or two-equation models as described in [30]. In two-equation models the eddy viscosity is a function of the turbulence kinetic energy k and turbulence dissipation rate ε as described by Kolmogorov-Prandtl equation [30]:

$$\nu_t = C_\mu' \frac{k^2}{\varepsilon} \qquad (4)$$

The turbulence kinetic energy k is defined as:

$$k = \frac{1}{2}\overline{u_i' u_j'} \qquad (5)$$

The transport equation of k is the same for the standard and *RNG* types of *k*-ε turbulence model [30]:

$$U_i \frac{\partial k}{\partial x_i} = \frac{\partial}{\partial x_i}\left(\frac{\nu_t}{\sigma_k}\frac{\partial k}{\partial x_i}\right) - \\ - \nu_t\left(\frac{\partial U_i}{\partial x_j} + \frac{\partial U_j}{\partial x_i}\right)\frac{\partial U_i}{\partial x_j} \qquad (6)$$

The general form of the transport equation for ε is described by the following expression [30]:

$$U_i \frac{\partial \varepsilon}{\partial x_i} = \frac{\partial}{\partial x_i}\left(\frac{\nu_t}{\sigma_\varepsilon}\frac{\partial \varepsilon}{\partial x_i}\right) - C_{1\varepsilon}\frac{\varepsilon}{k} P_k - \\ - C_{1\varepsilon}\frac{\varepsilon^2}{k} - \alpha\frac{\varepsilon^2}{k} \qquad (7)$$

The last term is an extra term and describes the influence of small-scale vortices (smaller than the grid size) that are developed during the *TKE* dissipation in shear flows. This is the mean strain rate, which accelerates turbulence dissipation in areas with the increased shear. The turbulence model parameters have the following values: C'_μ = 0.09, σ_k = 1.00, σ_ε = 1,30, $C_{1\varepsilon}$ = 1.44, $C_{2\varepsilon}$= 1.92. These are the default values in the SSIIM1 for the k- ε turbulence model. The extra term contains a parameter α, which is defined by the ratio η of two time scales – that of the turbulent strain and that of the mean strain. Expressions for α and η read:

$$\alpha = C'_\mu\,\eta^3\,\frac{(1-\eta/\eta_0)}{1+\beta\eta^3}, \qquad \eta = S\,\frac{k}{\varepsilon},$$
$$S = \sqrt{2\,S_{ij}^2} \qquad S_{ij} = 0.5\left(\frac{\partial\,U_i}{\partial\,x_j}+\frac{\partial\,U_j}{\partial\,x_i}\right) \tag{8}$$

In these expressions η_0 is a fixed point for homogeneously strain turbulent flows (η_0 = 4.8) and β = 0.12. Thus, when the mean strain is weak (η → 0), the extra production term is also small, but when the mean strain rate is strong (large η), the extra production term leads to an increase in turbulence dissipation. Consequently, the eddy viscosity is decreased and the momentum of the mean flow is reduced. The resulting recirculation zone size is comparable to that observed in laboratory experiments.

Boundary Conditions

The full description of the problem requires that the governing equations are supplemented with boundary conditions. Boundary conditions are defined at open and solid boundaries. Open boundaries include inflow and outflow cross-sections and the free-surface, whereas solid boundaries include channel bed and walls/banks.

Inflow and Outflow Boundaries

Dirichlet boundary condition is applied at the inflow boundary. The application of this condition is relatively straightforward for velocities. However, it is more difficult to specify the turbulence properties, i.e. k and ε. Their values are found from: 1) the distribution of the eddy viscosity coefficient (ν_t = 0.11u_*h, [16]) or the depth averaged value ν_t = 0.067u_*h) the assumption that the production and the dissipation of the *TKE* are at equilibrium near the solid boundary and 3) the assumption of the linear distribution of the *TKE* in which the free-surface value is equal to half of its bottom values [26]. For the given velocity, it is also possible to estimate the bed shear stress (τ), which can be further used to find the *TKE* value on the bed in the inflow cross-section:

$$k = \frac{\tau}{\rho\,\sqrt{C'_\mu}} \tag{9}$$

Based on the previously determined values of the eddy viscosity and *TKE* on the bed, the corresponding value of the *TKE* dissipation rate ε can be found using the equation (4). The vertical profile ε (x_3) is then calculated based on the ν_t and *k* vertical profiles.

Water Surface

The free surface is computed using a fixed-lid approach, with zero gradients for all variables except the vertical velocity which is calculated from the zero discharge condition through the free-surface, and the *TKE*, which is set to half of its bottom value, as already described. The location of the free-surface is determined from the calculated pressure field using Bernoulli equation.

Solid Boundaries - bed/wall

The wall-law for rough boundaries was used as a boundary condition for the bed and wall [32]:

$$\frac{U}{u_*} = \frac{1}{\kappa}\ln\frac{30\,y}{k_s} \tag{10}$$

Here, the bed effective roughness is denoted with the symbol k_s. The bed effective roughness after van Rijn is used in this study. Other variables in Eq. 10 include velocity U, shear velocity u_*, von Karman constant κ = 0.4 and the distance from the wall to the centre of the first computational cell *y*.

2.2 NUMERICAL MODEL

The set of six equations (1), (2), (6) and (7) is solved on a structured, curvilinear, orthogonal space grid using a 3D finite-volume model SSIIM1. Convective terms in the momentum equations can be discretized either using the power-law or the second-order upwind scheme. In this paper the power-law scheme is used. Since the velocity field, which is obtained by solving *RANS* equations (2), does not satisfy the mass conservation equation (1), the coupling of equations is achieved using the SIMPLE algorithm [26]. Velocity derivatives in transport equations for *TKE* and ε are discretized using central differences.

2.3 EXPERIMENTAL SETUP

Prior to the study of the effect of a variable curvature bend layout on the flow characteristics in the bend, the selected numerical model (SSIIM1) is calibrated against the experimental data from a 180º simple bend with a constant

Figure 1. Pirestani's experimental facility (Pirestani, 2004)

bend radius [29]. The data were collected in a laboratory flume made in the Tarbiatmodarres University Laboratory with the bend of radius *R* = 2.6 m which was inserted between two straight channels (Fig. 1). The length of the upstream channel is 7.2 m and that of the downstream channel is 5.2 m. The cross-section of the flume is a square with a 0.6 m long edge. This gives a radius to width ratio-value *R* / *W* = 4.33. A bend with this *R* / *W* value belongs to the class of mild bends according to Rozovskii [31] classification. The longitudinal channel bed slope is 0.0015.

Experiments were carried out for three discharge values *Q* = {30, 45, 60} l/s and three Froude number values *Fr* = {0.27, 0.41, 0.55}. The discharge was measured on the calibrated orifice set in the supply pipe, while longitudinal and lateral velocity components were measured using the P-EMS velocimeter. A digital point gauge with an accuracy of ±0.01 mm was used for flow depth measurements. Pirestani's data from the experiment with the constant discharge of 30 l/s and the flow depth of 0.15 m at the entrance to the channel were used for verification of the numerical model.

2.4 VERIFICATION OF THE NUMERICAL MODEL

The computational domain covered the full length of Pirestani's facility to ensure no influence of boundary conditions on the flow pattern in the bend (Fig. 2). Since

SSIIM1 grid generator can make only uniform grid, an independent mesh generator was written in Visual Basic to produce a desired non-uniform grid with higher cell density near channel walls (Fig. 3a). In addition to lateral, it is also possible to provide longitudinal cell-size variabilty. This allows one to reduce computational time by increasing the cell length in the streamwise direction along the upstream and downstream straight reaches, which are of no interest for the present study.

A special attention was paid to grid sensitivity analysis and model calibration, since the model output is strongly affected by the grid size/density, imposed boundary conditions and wall roughness. All three were varied until acceptable match has been attained between simulated and measured water surface profiles, pressures and velocities. Statistical indicators such as $RMSE$ (root-mean-square error) and MAE (mean absolute error) were used to assess the quality of the SSIIM1 model results:

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}[(U_c)_i - (U_m)_i]^2} \qquad (11)$$

$$MAE = \frac{1}{N}\sum_{i=1}^{N}[(U_c)_i - (U_m)_i] \qquad (12)$$

Here the subscript m refers to the measured variable value and *c* to the calculated one. To select the final mesh size, study of grid sensitivity analysis was done based on comparison of calculated water surface and streamwise flow velocity at different cross-sections with the measured ones. For this purpose, three mesh sizes were considered. The coarse mesh with size 71×12×7 in the streamwise, lateral and vertical directions, respectively, the fine mesh of 91×19×7 size and the finest mesh of size 351×35×15. The grid sensitivity analysis has shown that the mesh of size 91×19×7 provided satisfactory results. This means that the choice of a finer mesh has not significant effect on the quality of results when compared to the selected mesh size.

Lateral profiles of the streamwise velocity component (*U*) near the free-surface, that were obtained with the absolute roughness k_s = 0.1 mm and the downstream flow depth of 0.14 m

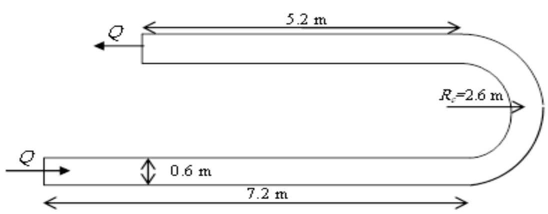


Figure 2. Dimensions of the computational domain in the plan view

are compared to measured ones in several cross-sections within the bend (Figs. 4, 5). Values of the $RMSE$ and MAE for the five cross-sections are given in Table 1. Low values of both statistics in all selected cross-sections clearly indicate that the SSIIM1 model gives physically plausible results with acceptable level of accuracy. Velocities are over predicted close to the bend entrance (θ = {10º, 40º}) by 0.028 and 0.039 m/s on the average, while the match between the two is almost perfect in the middle part (θ = {90º, 130º}), i.e. MAE = 0.015 m/s. Although a greater dispersion in measured elocities is observed at the exit (θ =170º), the shape of the lateral profile is caught satisfactorily.

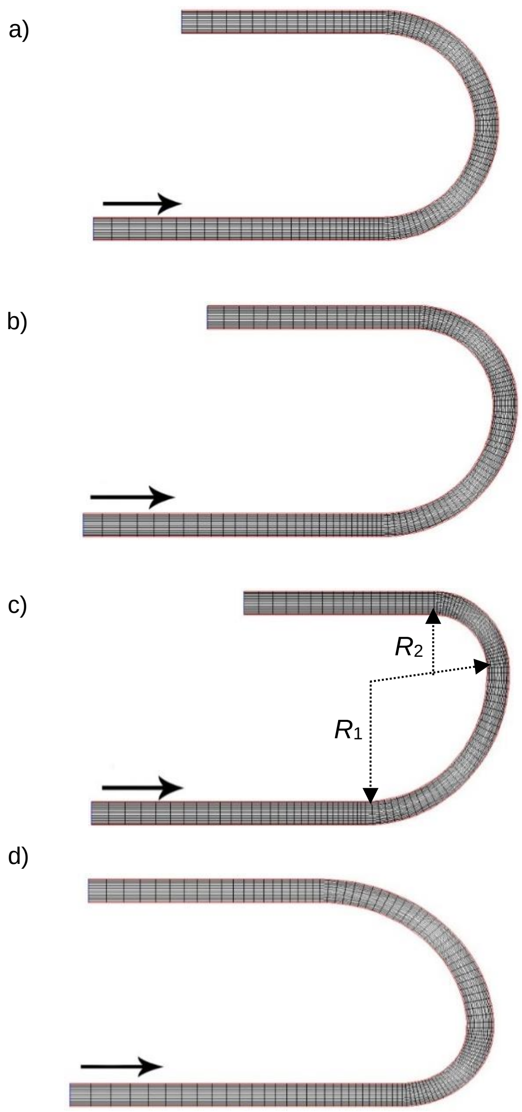


Figure 3. Schematic view of the generated mesh for: a) Simple, b) Mild-Mild, c) Mild-Sharp and d) Sharp-Mild bends

Table 1. Comparison of calculated and measured velocities in plane near water surface

Statistical indicators	Section location				
	10º	40º	90º	130º	170º
$RMSE$ [m/s]	0.0350	0.0457	0.0216	0.022	0.0315
MAE [m/s]	0.0285	0.0398	0.0152	0.0156	0.0276

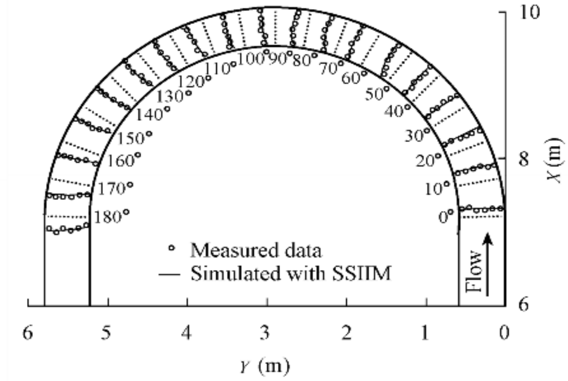
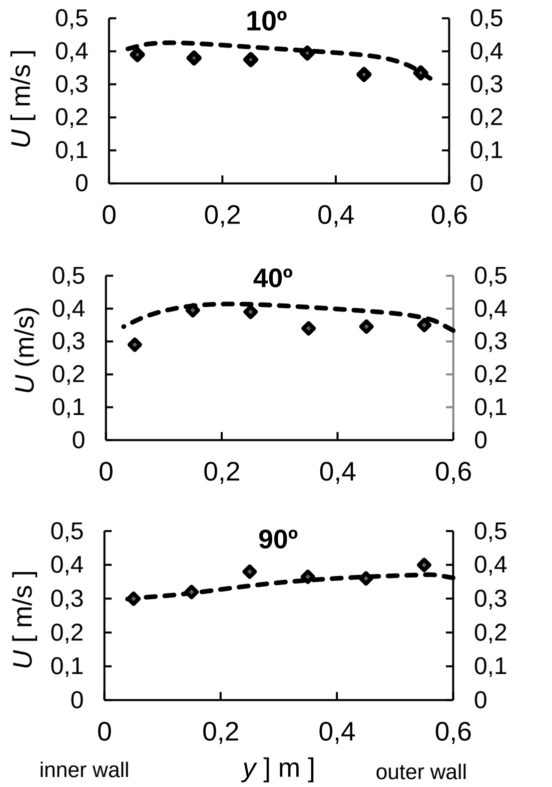


Figure 4. Comparison of streamwise velocity profiles on the plane near the water surface (circles are used for experimental data and lines for simulation results; dotted lines indicate position of profile lines), [13]



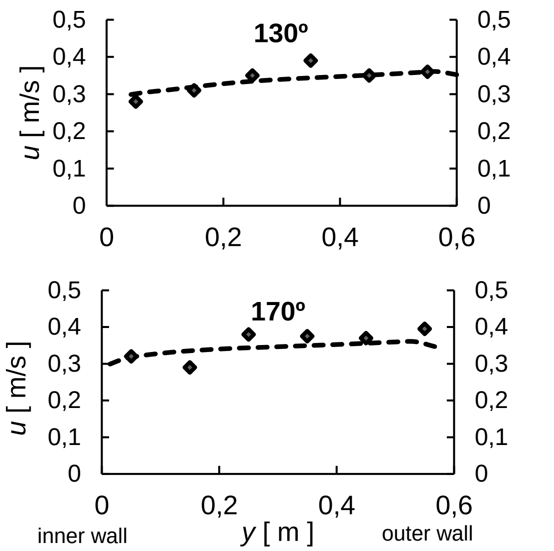


Figure 5. Comparison of calculated (dashed line) and measured (diamond-symbol) lateral profiles of the streamwise velocity within the bend (Profiles are presented on the plane near water surface)

In addition to streamwise velocity profiles on the plane near the water surface, simulated streamwise vertical profiles at the exit cross section of the bend are compared with the experimental ones. As it can be seen from Fig. 6, the model predictions only at the water surface and close to bed near the inner and the outer walls of the bend slightly deviate from the measured ones (relative error is less than 14.43%). In the middle of the cross-section (y = {0.5-0.45} m) the model predictions are relatively good (relative is error less than 11.28%).

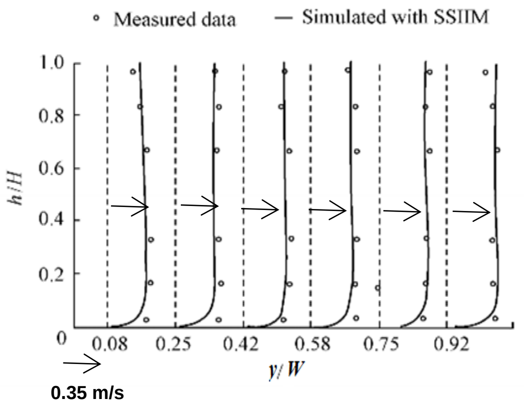


Figure 6. Comparison of the calculated vertical velocity profiles of the streamwise velocity with experimental data in the exit cross-section ω = 180º, [13]

2.5 NUMERICAL EXPERIMENTS IN A VARIABLE CURVATURE BEND

To study the effect of variable bend curvature ratio on flow characteristics in 180º bend, three hypothetical variable curvature bend layouts are considered (Table 2). They cover three possible cases: 1) case when both successive bends are of mild type, 2) case when mild bend precedes the sharp one and 3) case when sharp bend precedes the mild one. The cross-sectional channel geometry and lengths of the upstream and downstream straight channel sections are taken from Pirestani's facility (Fig. 2). Grid with the same size as the one used in the simple bend is applied in all hypothetical confluence layouts (Figs. 3b, c, d). To facilitate comparison with the simple bend case, numerical experiments were conducted with the same hydraulic data (see Section 3).

Table 2. Types of studied bends

Bend case (Fig.1)	R_1 [m]	R_1/W	R_2 [m]	R_2/W	Bend class	Bend type
a	2.6	4.33	2.6	4.33	Simple	Mild
b	3	5	2	3.33	variable curvature	Mild-Mild
c	3.5	5.83	1.7	2.83	variable curvature	Mild-Sharp
d	1.7	2.83	3.5	5.83	variable curvature	Sharp -Mild

3. RESULTS AND DISCUSSION

The effect of a variable curvature bend on flow characteristics in 180° bend is studied by comparison of: 1) the streamwise water surface profiles along the outer and inner banks, 2) maximum velocity paths, 3) lateral profiles of the streamwise velocity and 4) bed shear stress distributions for the four considered bend layouts.

3.1 EFFECT OF BEND TYPE ON STREAMWISE/SPANWISE WATER SURFACE PROFILES

Non-dimensional water surface profiles along the outer and inner banks for different bend layouts of a variable curvature bend are presented in Figure 7. Water surface profiles are non-dimensionalised using the flow depth at the outflow cross-section (h_d = 0.14 m). The distance along the channel L is non-dimensionalised using the total channel length L_t. The following can be observed. Water surface profile along the outer bank in the part of the variable curvature bend with the larger curvature (sharp bend) is always above the corresponding profile of the simple bend regardless the bend sequence (Sharp-Mild or

Mild-Sharp). This is expected since the centrifugal force in the bend with the greater curvature is stronger. As such, it produces greater water level rise along the outer bank. Conversely, the water surface profile along the inner bank of the sharp-bend reach in the variable curvature bend is always below that in the simple bend. Additionally, it is readily noticeable that the spanwise (lateral) water surface slope, which is present throughout the bend due to the difference in water surface elevations at the opposite banks, develops at a short distance upstream of the bend. This phenomenon is caused by the change in the direction of the streamwise momentum as the flow approaches and enters the bend [13].

As it can be seen from the water surface profiles of the Mild-Mild and the Mild-Sharp bends, the water surface elevations along the entrance section (L / L_t = 0.0 to 0.32) are lower than those along the same stretch of the simple bend when the compound bend begins with a bend that has greater radius than the simple bend. The situation is quite the opposite when the compound bend starts with the bend of smaller radius then the simple bend.

The spanwise water surface elevation difference from L / L_t = 0.32 (entrance to the bend) to L / L_t = 0.54 in the case *b* (Mild-Mild sequence of bends) is 8.9% smaller than the corresponding spanwise water surface elevation difference in the simple bend. From L / L_t = 0.54 till the end of the bend it becomes 50.1% greater than the corresponding spanwise water surface elevation difference in the simple bend. Such a behaviour is explained by the fact that the bend radius of a compound bend *b* is less than the radius of simple bend from L / L_t = 0.32 to L / L_t = 0.54 and it is greater from that of the simple bend from L / L_t = 0.54 to the exit of bend. Therefore, the centrifugal force in the first part of the bend in case *b* is smaller than that in the simple bend and it is greater than the corresponding one in the simple bend in the remaining part of the compound bend.

In the bend case *c* (Mild-Sharp bend), the spanwise water surface elevation difference is 17.2% less than that in the simple bend in the first part of the bend (L / L_t = 0.32 to 0.57) because the bend radius is larger than that in the simple bend. The bend radius in the second part of the compound bend is also less than that in a simple bend and it is smaller than that in the case *b*. Thus, the resulting spanwise water surface elevation difference is 72.74% larger than that in the simple bend.

In the bend case *d* (Sharp-Mild bend) the situation is the opposite to the bend case *c,* which means that the spanwise water surface elevation difference in the first part of the bend (sharp bend) is 46.24% greater than that in the simple bend while it is 22.21% less than that in the simple bend in the second part of the compound bend (mild bend).

There is almost no difference between the simple and compound bends' water surface profiles downstream of the bend (L / L_t = 0.75 to 1.00).

The maximum non-dimensional spanwise water surface elevation difference ($H_{max} = (h / h_d)_{max}$) and its location ($L_{H\max} / L_t$) are listed in Table 3 (L_t is the total length of the computational domain). It is readily noticeable that the largest difference ($\max H_{max}$) develops in the Sharp-Mild bend at the location which matches that of the simple bend.

Changes in lateral water surface slope (S_l) in simple and compound bends are compared in Figure 8. One can notice that the Sharp-Mild compound bend layout produces the steepest lateral slope S_l = 0.56% at L / L_t = 0.37. This maximal slope is 80.64% greater than the maximal slope in the simple bend ($\max S_l$ = 0.31%). However, starting from L / L_t = 0.48 the lateral slope S_l for the Sharp-Mild bend becomes approximately 69% smaller than that in the simple bend. At the exit of the bend, it coincides with that of the simple bend. In both Mild-Mild and Mild-Sharp layouts, the maximal lateral slope in the upstream, Mild bend is smaller than that in the simple bend due to a smaller curvature. It is 19.3% smaller in the Mild-Mild and 32.2% smaller in the Mild-Sharp bend. Shortly after the change in bend curvature, the lateral slope becomes greater than that in the simple bend due to larger curvature of the second bend when compared to the simple one. The maximal lateral slope in the Sharp second bend (Mild-Sharp layout) is 26.6% greater than that in the Mild second bend of the

Table 3. Maximum spanwise water surface elevation difference and its location

Bend type	$H_{max} = (h/h_d)_{max}$	L_{Hmax}/L_t	$\max S_l$ [%]
Simple	0.0001879	0.383	0.31
Mild-Mild	0.0001407	0.597	0.26
Mild-Sharp	0.0001791	0.669	0.30
Sharp-Mild	0.0003347	0.383	0.56

Mild-Mild layout. The location of the maxS_l is shifted downstream by 0.23L_t in the Mild-Sharp when compared to the Mild –Mild bend. However, the slope reduction towards the bend exit is much slower than in the Mild-Mild layout.

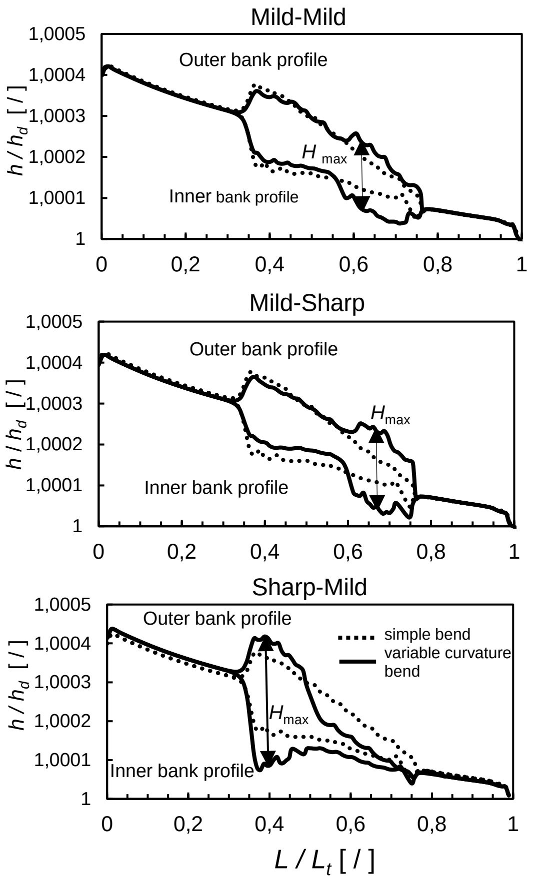


Figure 7. The effect of bend type on water surface profile (L is distance from beginning of channel, L_t is the total channel length and h / h_d is dimensionless water depth ratio

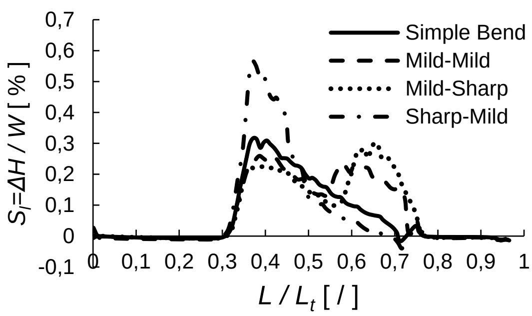


Figure 8. The effect of bend type on spanwise water surface slope

3.2 EFFECT OF BEND TYPE ON MAXIMUM VELOCITY PATH

The maximum velocity path is the line which connects points with maximum velocity magnitude at a given elevation above the bed. Figure 9 shows a schematic of the maximum velocity path near the water surface. Two points are indicated in this figure: the point L_1 where the maximum velocity path cuts the centreline and the point L_2 where it reaches the outer bank and from which it keeps following this bank. Distances of the two points from the beginning of the bend are also denoted by L_1 and L_2. These lengths are non-dimensionalised by the total length of the bend (L_b). They are presented in the Table 4. As can be seen, the maximum velocity path cuts the centreline at the shortest distance in the Mild-Sharp bend, which means that the influence of the centrifugal force in this compound bend layout increases much faster than in the other bend layouts. However, it can be concluded that the distance is practically the same regardless of the bend layout (variable curvature or simple) since the difference between the simple and variable curvature cases is 7% at maximum. On the other hand, the distance L_2 in the variable curvature bend is shortened when compared to the simple bend. The maximum shortening (27%) is recorded in the Sharp-Mild bend, which means that in this bend layout the secondary flow takes the dominance over the effect of streamwise pressure gradient much faster than in other considered layouts.

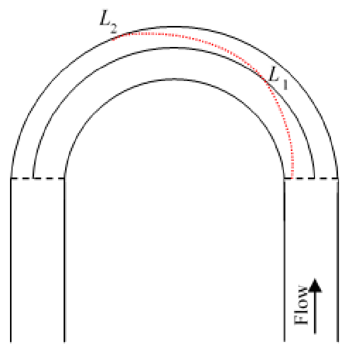


Fig. 9. Schematic image of maximum velocity path near the water surface

3.3 EFFECT OF BEND TYPE ON LATERAL PROFILES OF STREAMWISE VELOCITY

Lateral profiles of the streamwise velocity for different bend layouts are compared in Figure 10. The bend layout practically has no influence on lateral profiles at the entrance to the bend (L / L_b = 0.00) and close to its exit (L / L_b = 0.75).

Here L stands for the distance of the cross-section from the entrance to the bend measured along the path of maximum velocity, and L_b is the total length of the bend. The only visible differences are recorded near the inner and outer banks in the Sharp-Mild bend. The maximal increase in the velocity magnitude of 4.61% is recorded near the inner bank at L / L_b = 0.00 and the maximum decrease of 4.76% is recorded near the outer bank close to the bend exit (L / L_b = 0.75). Such behaviour becomes prominent for the opposite bend curvature sequence in the variable curvature bend, i.e. for the Mild-Sharp bend. Differences between the four distributions start to emerge from L / L_b = 0.25 and increase till L / L_b = 0.50. At L / L_b = 0.25 the lowest velocity magnitudes are recorded in the Sharp-Mild bend and the largest in the Mild-Sharp bend, with the remark that differences between the Simple, Mild-Mild and Mild-Sharp bends are practically negligible in this cross-section. In the middle of the bend (L / L_b = 0.50) velocity magnitudes in the Mild-Mild and Mild-Sharp bends are greater than in the Simple bend, while those in the Sharp-Mild bend remain the lowest. The maximum velocity difference of 10.18% between the Mild-Sharp and Sharp-Mild cases is recorded at y = 0.40 m (not presented).

It can be also noticed form Figs. 10a, d that in cross sections near the bend entrance and close to its exit the streamwise velocity at the channel centreline in the plane close to the water surface is not affected by the bend curvature.

Table 4. Dimensionless length of the maximum velocity path

Bend type	Radius / radii [m]	Location where maximum velocity path cuts the centreline (L_1 / L_b)	Location where maximum velocity path starts to follow the outer bank (L_2 / L_b)
Simple	2.6	0.314	0.668
Mild-Mild	3.3 and 2	0.295	0.549
Mild-Sharp	3.5 and 1.7	0.292	0.608
Sharp-Mild	1.7 and 3.5	0.309	0.486

3.4 EFFECT OF BEND TYPE ON BED SHEAR STRESS DISTRIBUTIONS

Bed shear stress is one of the key factors for estimation of sediment transport in rivers and open channels. The effect of bend type on dimensionless bed shear stress ($\tau_0 / \rho V^2$) for all cases is presented in Figure 12. Here V stands for the outflow cross-sectional average velocity and ρ is water density. Two regions with maximum bed shear stress are distinguished regardless the bend layout – one is located close to the end of the bend, and the other is located near the radius inflection point. High velocity gradients are the main reason for the development of the high bed-shear stress region close to the end of the bend. As it can be seen from Figure 11 these high velocity gradients result from the movement of the high velocity core towards the outer bank and its expansion towards the bed [13]. The area with

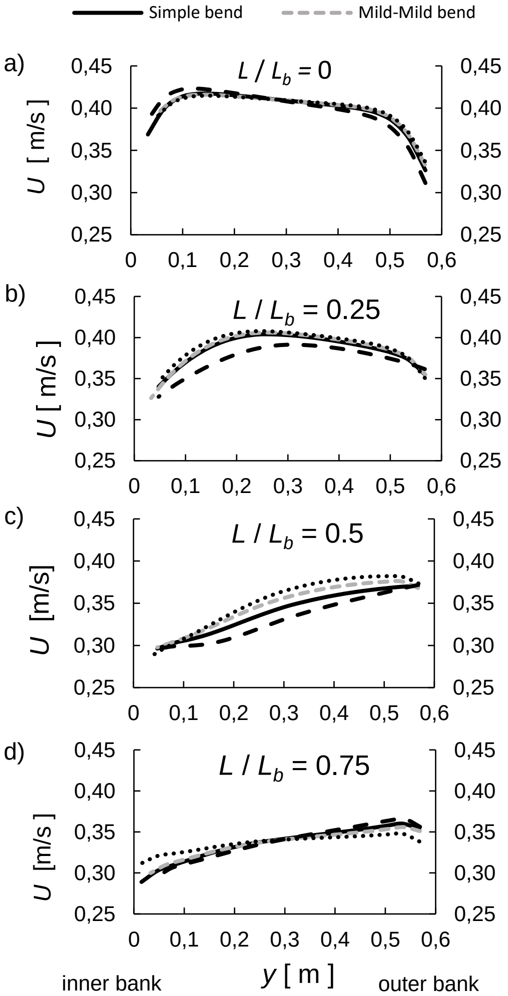


Figure. 10. Effect of bend type on lateral profiles of streamwise velocity in different cross-sections within the bend

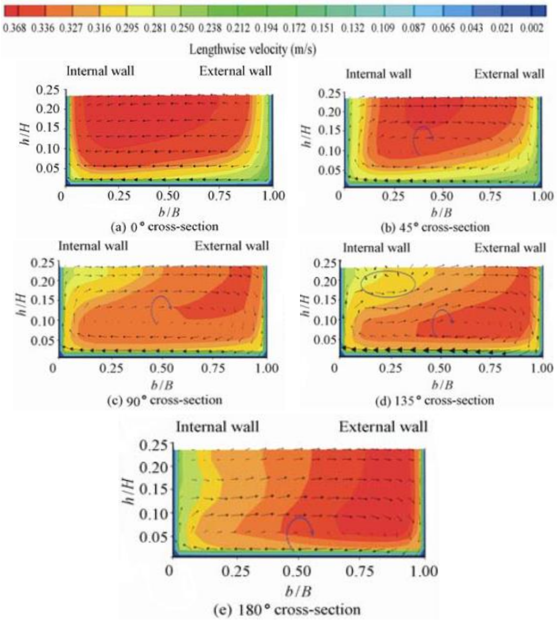


Figure. 11. Velocity distributions and secondary flows at different cross-sections in simple bend [13]

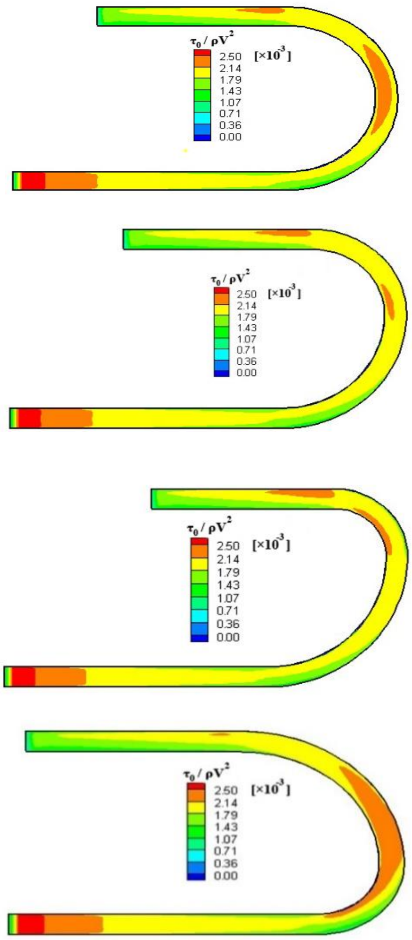


Figure 12. Effect of bend type on bed shear stress distribution: a) simple bend, b) Mild-Mild bend, c) Mild-Sharp bend and d) Sharp-Mild bend

the maximum bed shear stress close to the end of a bend is significantly reduced in the Sharp-Mild bend (by almost 90%) when compared to the simple bend. This area reduction is attributed to an increase in the second bend radius (R_2 = 3.5 m), when compared to the radius of the simple bend (R_c = 2.6 m). On the other hand, in Mild-Mild and Mild-Sharp bends decrease in the second radius, when compared to the radius of a simple bend (see Table 2), results in an increase of the $\tau_{0,\max}$ area. The percentage increase in the Mild-Mild bend is approximately 38% and in the Mild-Sharp bend, around 87%.

The size of the $\tau_{0,\max}$ area, which is located near the radius inflection point grows in Sharp-Mild bends and reduces in the Mild-Mild and Mild-Sharp bends. When compared to the size of the corresponding area in the simple bend (bend apex area), the percentage growth in the Sharp-Mild case almost reaches 180%, while percentage reductions in the Mild-Mild and Mild-Sharp bends are 81% and 76%, respectively.

4. CONCLUSIONS

The effect of bend type on flow characteristics in a 180° variable curvature channel bend was studied in this paper using the 3D finite-volume based numerical model SSIIM1, which solves Reynolds-Averaged Navier-Stokes equations. Equations are closed with the two-equation turbulence model of the k-ε type. The model was calibrated against the experimental data from the simple mild bend with θ = 180º (R_c / W = 4.33), first. Consequently, three variable curvature bend layouts with two alternating bend curvatures were analysed: Mild-Mild, Mild-Sharp and Sharp-Mild. The study has shown the following.

1. When the compound bend starts with the Mild bend of the greater radius than the simple bend, the maximal lateral slope of the water surface is less than that in the simple bend. However, the lateral slope increases right after the bend curvature transition point. The position of the cross-section with the maximal lateral slope moves downstream with the increase in downstream bend curvature.

2. When the Sharp bend precedes the Mild bend, the maximal lateral slope of the water surface is greater than that in the simple bend. However, downstream of the bend curvature

transition point, the lateral slope becomes smaller than that in a simple bend.

3. The distance from the entrance to the bend to the location where the maximum velocity path cuts the bend centreline is essentially the same regardless of the bend layout (simple or variable curvature). However, the distance to the point where the maximum velocity path starts to follow the outer bank reduces up to 27% when the sharp bend precedes the mild one.

4. The bend sequence affects lateral velocity distribution in the plane near the free-surface neither at the entrance, nor at the exit of a variable curvature bend. The differences between the four layouts exist at the bend mid-length. At this location the lowest velocity magnitudes are developed in the Sharp-Mild, and the highest velocity magnitudes in the Mild-Sharp bend. The maximal difference between the two velocity distributions is reached at $y / W = 0.40$.

5. Generally, there are two regions with the high bed shear stresses – one at the curvature inflection point and the other, close to the bend exit. When compared to the corresponding regions in the simple bend, the former one enlarges by approximately 180% when the sharp bend precedes the mild bend, and reduces by 81% and 76% as the curvature of the succeeding bend reduces. The situation with the latter one is quite the opposite – the precedence of the sharp bend causes reduction of the area size by almost 90% when compared to that in the simple bend, while precedence of the mild bend causes its growth by 38% and 87% in the Mild-Mild and Mild-Sharp bends, respectively.

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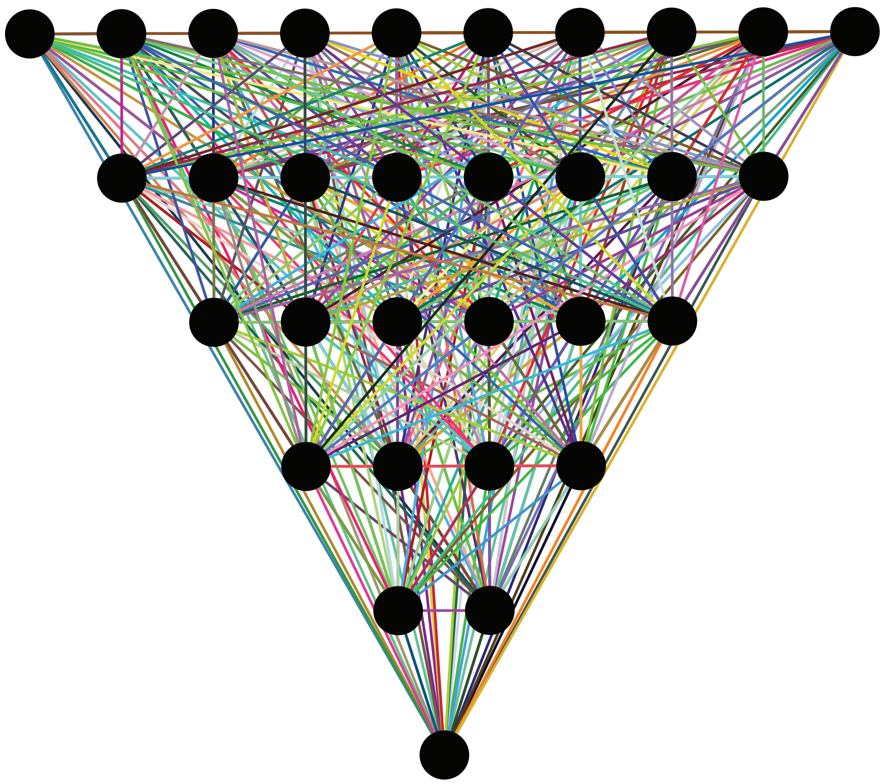

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CHEMICAL STABILIZATION OF SOIL USING LIME AS A CHEMICAL REAGENT

One of the techniques that is increasingly used in geotechnical practice in order to improve the properties of soil is chemical stabilization. It is based on modifying the properties by adding a chemical reagent, which will react with the minerals in the soil. The efficiency of the method will largely depend on numerous factors, primarily on the type of soil, type and amount of chemical reagent, as well as on the achieved chemical reactions between stabilizer (chemical material) and soil minerals (pozzolanic materials). In practice, lime, cement and fly ash are most often used as chemical stabilizers, which ultimately contribute to increasing soil strength, reducing water permeability and reducing the potential of swelling. The paper deals with the general principles of chemical soil treatment, whereby a special attention is paid to the application of lime as a chemical stabilizer, which has been shown to be particularly effective in the case of clayey soil. Lastly, the advantages and limitations of this soil stabilization technique are considered, as well as possible directions for its improvement.

Keywords: chemical stabilization, clayey soil, geotechnical properties, lime

1. INTRODUCTION

When constructing civil engineering structures, it is often not possible to avoid locations with soils of poor quality and low bearing capacity. Hence, for such soils that can't meet the minimum requirements in terms of bearing capacity and quality, it is necessary to carry out the procedure of soil stabilization and improvement. In order to choose an adequate soil stabilization technique, it is first necessary to properly consider the problem and discover the cause of its occurrence. The stabilization method that will give the best results depends on the type of soil, location and purpose of the facility to be built.

The concept of soil improvement means a set of measures (techniques) that are implemented to improve the physical and mechanical properties of the soil and that allow the safe construction of structures for various purposes

[1]. Improvement techniques can be of a temporary or permanent character. When considering temporary techniques, the effects of improving soil properties last relatively short (usually only during the construction stage, e.g., water table lowering, soil freezing), whereas in the latter case the effects last for a longer period of time (e.g. grouting, reinforcement, dynamic compaction) [2]. In general, there are about 30 different soil stabilization techniques aimed at improving soil properties and stabilization, such as material replacement, chemical stabilization, strengthening by steel reinforcement or geosynthetics, drainage, vibration compaction, consolidation, electroosmosis, medusoil and others [3]. Many techniques can be used in combination with others, and as a result, new methods can emerge. According to the more recent classification of Schaefer et al. [4], 46 soil stabilization methods have been identified. This categorization is a result of the fact that some of the methods for soil improvement can be included in one or more categories.

Improving soil properties is nowadays recognized as one of the main subdisciplines of geotechnical engineering. Soil improvement techniques have developed significantly over the past decades and are today almost routinely used in most procedures during the construction of various structures. Each of these methods has its advantages, but also shortcomings. The increase in the number of soil stabilization techniques, as well as products and engineering tools, has been expanding recently due to extensive research in this area and a large number of available technologies. A significant progress in this development has been noted at many conferences, workshops, as well as in a number of papers and reports.

2. PRINCIPLES OF CHEMICAL SOIL TREATMENT

Improving soil properties by chemical treatment is based on modifying the soil properties by adding a chemical reagent, which will react with the minerals in the soil. However, there are also mechanical additives, which are not able to change the chemical soil properties, but enable an improvement of the natural properties of the treated soil. There are numerous additives that are used to improve soil properties, but in practice, in over 80% of cases, cement, lime and fly ash are most often used [5].

In most cases, stabilizers are added to the soil using appropriate construction equipment. The method to be used depends on the location of the construction site and availability of the equipment, whereas the selection of additives mainly depends on the type of soil. One additive can affect various soil types differently. Chemical treatment is mostly used to improve the properties of fine-grained, clayey soil, because this type of soil is susceptible to high water content. There are 18 different chemical mechanisms (that is cation exchange, anion exchange, adsorption, fixation, formation of new minerals, cementation, salt conversion, modification of aqueous films, modification of capillary forces, modification of electrical surface tension of clay minerals), which can result in an improvement of the properties of clay [6]. However, inadequate use of reagents can lead to reactions such as flocculation of the particles, heat generation during chemical reactions, etc. Today, chemical stabilization is used as a routine technique worldwide (primarily stabilization with lime and cement), but research in the field of finding new reagents and their combinations is still ongoing. The principles of chemical soil treatment in order to improve the geotechnical soil properties using lime as a chemical stabilizer are explained below.

3. IMPROVING SOIL PROPERTIES BY ADDING LIME

Adding lime is a suitable technique for stabilization of fine-grained, clayey soils. Lime affects certain soil modifications, which are reflected in the decrease of water content, plasticity index, water permeability and swelling, change of grain size distribution and increase of compressive strength, bearing capacity and optimum moisture content. Due to the improvement of all these properties, a clay with the addition of lime presents a modified, much better material than the basic soil. Soils with such improved properties also provide technically preferable and often more economic solutions.

Chemical reactions that, in the presence of water, take place in the soil after lime is added and distributed (spotted), and then compacted, lead to significant, easily noticeable changes in the structure of the material and in its physical and mechanical properties. Due to its composition and electrostatic relations on the surface and inside the lattice of clay minerals, clay soil is very sensitive to changes in water content. If water is added to dried clays, which are stable and solid in that state, they will change their consistency, that is they become semi-solid, then plastic and finally liquid, which is accompanied by smaller or larger changes in volume. With increasing the clayey soil water content, its load bearing capacity decreases drastically, so that in the case of materials in liquid state consistency, the bearing capacity of the soil is practically non-existent. In addition, high water content is a great problem in execution of construction works, because the required soil compaction cannot be achieved.

In relation to the mass of dry soil, the optimum amount of lime added to the soil is in the range from 3% to 6%. The value to be added is determined based on the pH test (the Eades and Grim test) where it is necessary to reach a pH value of approximately 12.4. The optimum percentage of lime must be verified on the basis of the examination of the change in shear strength before and after the soil treatment. The effect of lime on the soil takes place through three main stages: reduction of the water content in the soil, modification of the soil and increase of the soil strength (stabilization).

A review of the effects that can be achieved by treating clayey soil in the corresponding way by adding lime is shown in Figure 1. It should be noted that some soil properties can adversely affect the course of the described reactions (highly organic soils, high pH value). The change in the structure of soil particles occurs slowly and depends on the type of clay. Clays of a medium to high plasticity are suitable for stabilization with lime, and this method can also be applied to gravelly and sandy soils if they contain a sufficient amount of clay fractions. In the case of low-plastic clays, as well as silts and pure cohesionless materials, lime alone is not an efficient solution; however, some pozzolanic materials can be added in addition to lime, such as slag, fly ash, cement and other. In geotechnics, the effect of lime is determined by laboratory testing of soil mixtures. The soil stabilization effects are established by examining the parameters of strength, load bearing capacity, compressibility and water permeability.

Lime can be used in various forms: quicklime, slaked lime and liquid lime. It is noticed that lime is not an independent stabilizer, but the stabilizing effect is achieved just after the reaction of lime with clay components of the soil. For that reason, chemical stabilization with lime is used in the case of clayey soils, whereby the mineral composition of the clay has a great influence on the intensity and rate of the reaction.

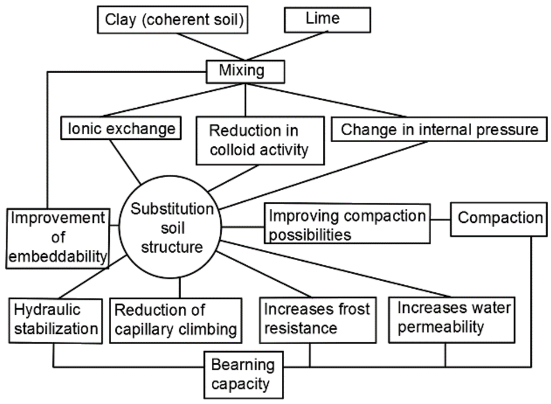

Figure 1. Effects of added lime on clayey soil (according to W. Brand) [7]

The process of soil stabilization with lime on a construction site consists of homogeneous mixing of soil with an appropriate amount of lime, where the mixture must, as a rule, have the optimum water content. The method is suitable for large surfaces, as it is relatively easy and fast to perform (e.g., in construction of roads). It is recommended that the soil and lime mixture are compacted in layers up to 50 cm thickness with corresponding technique over a certain period of time until the required properties are achieved. To make one of the layers of the road, stabilization by adding lime can be done by mixing in place. Figure 2 presents the process and the equipment that is most often used in chemical treatment of soil by adding lime during road construction.

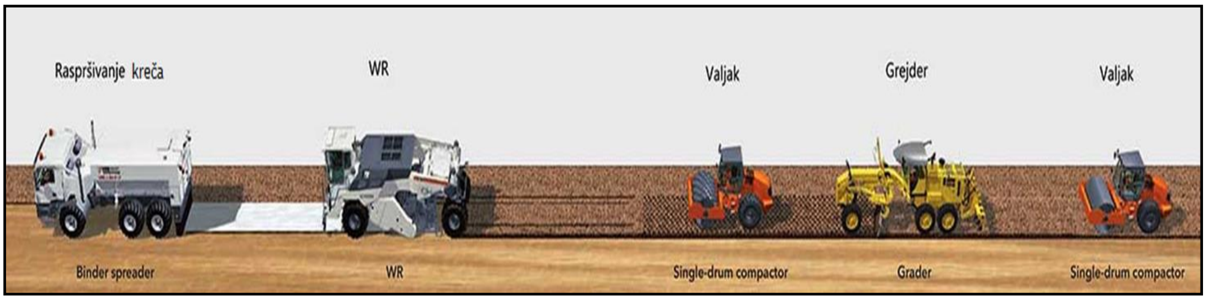

Figure 2. Chemical treatment procedure by mixing the soil with lime on a construction site (www.frekomos.hr)

Table 1. Test results for improving the geotechnical properties of fine-grained soil by adding lime and dry mixing

Author(s)	Type of soil and stabilizer(s) in chemical treatment	Results (effects) of chemical soil treatment
Bell, 1996 [8]	Kaolinite, montmorillonite, quartz, treated with lime	Decrease in the plasticity (particularly typical for montmorillonite) and increase in the dry density during compaction and compressive strength. Significant improvements in soil properties were achieved by the addition of 4% or more lime.
Susinov and Josifovski, 2013 [9]	Local soil, disturbed soil samples were obtained from excavation pit at 2.0 m depth	The mixture of lime and silty soil material has significantly improved the mechanical properties. At 2% of lime, a reduction of moisture content and the plasticity index is around 40% and 45%, respectively. The CBR has improved up to 16 times when 8% of lime is added to soil and cured 7 days and even better results are expected for longer period of time. About 4% of lime can increase the compressibility modulus up to six times. The largest increase UCS is observed in specimen with 4% lime, the stabilized soil shows 2 times greater strength compared to the unstabilized soil.
Kumar and Dutta, 2014 [10]	A mixture of soil with bentonite and lime, reinforced with sisal fibers	Examined series of bentonite with the addition of lime (2%, 4%, 6%, 8% and 10%) - Series I. Then a different percentage of phosphogypsum (0.5%, 1%, 2%, 4%, 8%) was added to the mixture of bentonite and lime (8%) - Series II. In Series III, a different percentage of sisal fibers was added to the mixture of bentonite, lime and phosphogypsum. Soil improvements in terms of compaction and unconfined compressive strength were investigated. The results show that, for the examined Series I, II and III, the improvement occurs 7 days, 14 days and 28 days after the treatment, respectively.
James and Pandian, 2015 [11]	Local soil treated by adding lime and phosphogypsum	The results indicate that the use of phosphogypsum in soil stabilization depends on the percentage share of lime. For a soil mixture with the addition of up to 3% lime, it is necessary to add more than 2% phosphogypsum, whereas for the soil with the addition of 7% lime it is necessary to add only 0.5% phosphogypsum, in order to achieve the same or even better results than those when only lime is used as a stabilizer. The effects of stabilization in terms of reduction of liquid limit, plasticity index and swelling index were investigated.
Garzon et al., 2016 [12]	Phyllite, clay, treated with lime	Numerous geotechnical properties of the treated soil were investigated. The addition of 3%, 5% and 7% lime had the effect of reducing the consistency limits, improving the compaction results and CBR values, as well as reducing the swelling potential and water permeability.
Amidi and Okeiyi, 2017 [13]	Red local clay, treated with quicklime and slaked lime	2.5%, 5%, 7.5% and 10% of both types of lime were added, and quicklime was found to reduce plasticity, whereas slaked lime resulted in a higher dry bulk density and a higher compressive strengths according to triaxial testing for unconfined compression test, in particular at higher amounts (7.5% and 10%).
Jahandari et al., 2017 [14]	Kaolin clay, application of lime and geogrid	In addition to the treatment by adding lime, geogrids have also been added. Beside the improvement of almost all geotechnical properties, a decrease in the deformability index of the treated soil was observed.
Innocent and Okonta, 2018 [15]	Local soil, treatment with pre-compression, addition of lime and fly ash	In addition to lime, fly ash was used in small percentages (up to 1%). The The pre-compression stresses were applied after 4 h, 8 h and 24 h. The results after 7 days revealed that optimum strength of 3.5 MPa was mobilised by un-precompressed specimens at 0.75% fiber content. Pre-compression with 10% UCS showed maximum strength of 2.8 MPa at 0.25% fiber content whereas 20% UCS indicated optimum strength of 3.04 MPa at 0.25% fiber content. In comparison, pre-compressed specimens exhibited lower strength values than un-precompressed specimens. The maximum strengths of specimens for both pre-compression levels occurred after 24 h of curing.
Salih and Abdalla, 2020 [16]	Low-plasticity clay, treated with lime	2.5%, 5%, 7.5% and 10% lime were added and it was found that the addition of lime contributed to the reduction of the plasticity index up to 20%, as well as that the load bearing capacity of the treated soil increased 5.5 times (from 174 kPa to 960 kPa). It was determined by the Proctor test that the optimum water content of the treated soil was reduced by 10%, while the dry density was increased by nearly 15%.

Prior to the beginning of a stabilization process, the surface of the soil layer should be prepared according to the design. The layer is then spread, but also left in a loose state in order to be mixed more easily with the lime. The thickness of the loose material should be produced in such way that, after mixing with lime and compaction process, the layer of a designed thickness can be achieved. The required amount of lime per unit surface is then spread over the pre-prepared soil. Spreading is done by a construction plant, which ensures uniformity of the stabilizer in all parts of the soil layer. After that, the soil is mixed with lime until a homogeneous mixture is obtained (which can be checked according to the color of the mixture). During mixing, a certain amount of water is added optionally in order to obtain the optimum water content of the mixture, and vice versa, if the material is too wet, it is previously exposed to sun and wind in order to reduce its water content to the required extent. Finally, the material is spread and compacted using a padfoot drum compactor (roller), grader and smooth drum compactor.

An overview of previous studies on the effects of chemical stabilization of soil using lime in order to improve the geotechnical properties of clayey soil is given in Table 1.

Recently, numerous studies have been related to the addition of "new" chemical reagents in combination with lime, such as CaCl2 or NaOH, which can further improve soil properties. Nevertheless, a rather small number of studies have been conducted regarding the effects of the mentioned combinations on the improvement of geotechnical properties of soils. Some of them are presented in papers [17,18].

4. CHEMICAL ELECTROKINETIC TREATMENT

The electrokinetic soil treatment can be carried out at greater soil depths and below existing structures; however, the effects of such treatment are not permanent. A graphical interpretation of the principal processes that take place in fine-grained clayey soil under electrokinetic soil treatment is shown in Figure 3. By passing a direct current through the soil, the process of electro-osmotic flow of water in the soil from the anode (positively charged electrode) toward the cathode (negatively charged electrode) occurs. The result of this process is a decrease in soil moisture content, which begins in the anode zone and then spreads into the surrounding soil, thus resulting in a decrease in pore pressure, an increase in effective stresses in the soil, and improved physical and mechanical properties of the soil.

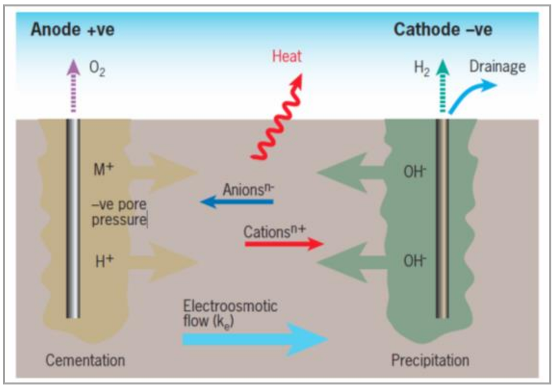

Figure 3. Principal processes active in a section of clayey soil under the electrokinetic treatment [19]

Chemical electrokinetic soil stabilization is a technique that overcomes the shortcomings of the aforementioned soil stabilization procedures. Namely, the technique of chemical electrokinetic soil stabilization is an enhancement of the technique of electrokinetic soil treatment using chemical agents (stabilizers), where the injection and movement through the soil of stabilizing agents takes place under the influence of a direct current, whereby the mechanism of stabilization itself can be explained by the principles of chemical stabilization. The combination of complex electrochemical processes under the influence of an electric field and in the presence of appropriate chemical agents should lead to a permanent improvement of the soil in terms of physico–chemical characteristics, and in particular the mechanical properties of fine-grained soil, which are of paramount importance in geotechnical engineering. Moreover, its advantage is that it can be applied to crucial geotechnical problems such as stabilization of slopes and landslides, increasing the bearing capacity of the foundation soil, reducing the soil moisture content under existing structures, as well as stabilizing the ground for construction of deep foundation excavations, tunnels, and other underground structures. Electrokinetic treatment of fine-grained soils in combination with chemical stabilizers has, to a very limited extent, been the subject of scientific research in the world over the past period [20,21]. The application of chemical stabilizers mainly on the basis of chlorides has been evaluated, mostly on soil samples tested in laboratory conditions, and without considering the time effect in terms of controlling the achievement of permanent improvement of the properties of the treated soil. All these facts indicate the necessity for detailed experimental research to better understand the nature, effects, efficiency, and scope of application of this soil stabilization technology, as well as with the aim of considering the possibility of applying new chemical agents that would contribute to the stabilization effect of a permanent character, which is of elementary importance in geotechnical engineering.

5. CONCLUDING REMARKS

The selection and application of the most adequate technique of soil stabilization can significantly contribute to the improvement of soil properties during the construction of foundations and earth structures, including embankments and cuts on roads and railways, earth dams, etc. In addition to improving soil properties, stabilization techniques may also contribute to a more economical solution. If it is not possible to perform works on the location planned for the construction of a structure, this problem can be solved in one of the following ways [22]:

1. To quit the location originally considered (replacing the planned facility to another location);
2. To adapt the facility to the existing conditions (e.g. pile foundation, construction of the facility as a very rigid or very flexible structure, etc.);
3. To remove the surficial or complete layer of the unsuitable (organic, compressible) soil;
4. To perform chemical treatment of the existing soil in order to improve its properties.

Chemical stabilization of soil contributes to the permanent improvement of physical, chemical and mechanical properties of soil, but due to the method of application it can be used only in surficial, easily accessible soil layers, which excludes its application in particularly important geotechnical problems such as slope stabilization, increasing the load bearing capacity of foundation soil, as well as reducing the water content and settlements of the soil beneath existing facilities. Owing to the high availability and accessibility of materials that can be used as reagents in improving soil properties, it is necessary to invest resources to further improve existing and discover new methods. Unlike the chemical stabilization technique, the electrokinetic treatment of soil can be carried out at greater soil depths and below existing structures. Electrokinetic chemical stabilization is a ground improvement technique in which stabilizing agents are induced into soil under a direct current. The movement of stabilizing agents into soil mass is governed by the principles of electrokinetic, while the mechanisms of stabilization can be explained by the principles of chemical stabilization. Further research may be seen in this direction.

In addition to the mentioned chemical stabilizers, in practice, bitumen is also used for chemical stabilization of soil (mixing problems may occur in the case of plastic clays, and thus the recommended optimum amount of the stabilizer ranges from 4% to 7%), chemical and synthetic materials (natural polymers, synthetic resins) or recycled materials. A combination of the electrokinetic soil treatment by the addition of chemical stabilizers would provide the possibility of using many other, less commonly used materials today (e.g. based on polymers, nanomaterials, etc.) that may be particularly suitable for the application of this soil stabilization technique.

Acknowledgements

The authors gratefully acknowledge the support of the Ministry of Education, Science and Technological Development of the Republic of Serbia in the frame of the scientific–research Project TR36028 (2011–2019).

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THE SHEAR STRENGTH OF INFILLED ROCK JOINTS

According to the Jaeger's theory, the minimum possible rock mass shear strength as a discontinuum actually corresponds to the shear strength of rock joints. Since failures in rock masses due to loads caused by civil structure and/or civil works occur mainly by exceeding their shear strength, the shear strength of rock joints has huge practical significance in rock engineering. For this reason, in this paper it was decided to analyse one of the factors which have very important influence on the shear strength of rock joints. Namely, natural rock joints are often filled with soft soil material and this infill material may have a significant and often decisive influence on the shear strength of rock joints. As a basis for the conducted analyses were used the results of direct shear tests under constant normal stress which was performed on natural or artificial specimens with horizontal infilled joints by various researchers around the word. Analyses have shown that some basic principles of mechanical behaviour of infilled rock joints during shearing can be reached. The thickness *t* and mechanical characteristic of the infill material have decisive influence on the peak and residual shear strength of infilled rock joints.

Keywords: infilled rock joints, shear strength, direct shear test, infill material thickness

1. INTRODUCTION

The space between the walls of open rock joints is often not empty or filled only with water and/or air. This space is often filled with material which can be primary or secondary origin i.e. it can be a product of physical and chemical degradation of adjacent rock blocks or it can transported material. The infill material can be such a mineral composition and mechanical characteristics that its shear strength is approximately equal to the shear strength of the intact rock. In this situation actually we talk about healed rock joints. However, from the aspect of shear strength of rock joints, special attention is required by geological situations when the space between the walls of joints is filled with soft non-cohesive (sand) or cohesive (clay) infill material. In the following text, more will be said about these situations.

Factors that affect the shear strength of natural unfilled rock joints (joint surface roughness, compressive strength at the joint surface, normal stress during shearing, scale effect, etc.) also affect the shear strength of natural infilled rock joints. However, there are two additional extremely important factors in the case of infilled rock joints. These are the infill material thickness *t* and the mechanical characteristics of the infill material. In general, the infill material thickness has a decisive or dominant influence on mechanical behaviour of infilled rock joints during shearing. However, this thickness is a relative category because it must be observed in relation to the height (amplitude) of the joint surface asperities *a*. It is concluded that in fact the relative infill material thickness i.e. the ratio *t/a* is the one that has a dominant influence on the mechanical behaviour of the infilled rock joints during shearing. Many experimental, laboratory research have proven that with increasing values of the ratio *t/a* peak and residual shear strength of the infilled rock joints decrease. The main reason for this drop in shear strength is the decreasing direct contact between the joint walls i.e. between irregularities on the joint walls (rock-rock asperity contact) with increasing infill material thickness. This actually changes the geometry of the shear plane, prevents the development of dilation and interlocking effects. Also, the presence of the infill material reduces the basic friction angle ϕ_b i.e. reduces the coefficient of friction of joint surface. This is very important for value of residual shear strength.

2. DETERMINATION OF THE SHEAR STRENGTH OF ROCK JOINTS

Determination of shear strength of rock joints is generally performed in displacement controlled direct shear test along joint under constant normal stress or constant normal stiffness. In accordance with ISRM (2013), rock specimens with a regular (rectangular or elliptical) cross-section are preferred. The length of the tested rock joint i.e. tested rock specimen (measured along the shear direction) should be at least 10 times the maximum joint wall asperity height and sufficient to encapsulate the specimen in the specimen holder. Of course, this length must be significantly greater than maximum shear displacement during test. The width of the tested rock joint i.e. tested rock specimen (measured perpendicularly to the shear direction) should have at least 48mm and this width should not change significantly over the shearing length. Minimum width of tested rock joint should be greater than 75% of its maximum width.

In accordance with ISRM (2013), in the first phase of direct shear test along joint normal stress should be applied on the rock specimen continuously at selected rate of normal stress. The rates of 0.01 MPa/s or less are recommended. In the second phase of test, after the normal displacements stabilize under the applied normal stress, shear displacement should be applied on the rock specimen continuously at selected rate of shear displacement until ultimate or residual shear stress is reached. Shear displacement rates around 0.1–0.2 mm/min are usually suitable for the whole test, although it can be slightly increased up to values around 0.5 mm/min after peak shear strength. The normal and shear forces are measured with accuracy better than ±2.0% directly by load cells, or indirectly by pressure gauges, transducers, or proving rings. Displacement transducers are used to measure the displacements. A minimum of two displacement transducers are required: one mounted parallel with the rock joint to measure the shear displacement and one mounted vertically at the centre of the specimen to measure normal displacement.

3. ANALYSIS RESULTS OF SAME EXPERIMENTAL RESEARCH

Papaliangas et al. (1993) is examined in shear displacement controlled direct shear test along joint under constant normal stress a large number of artificial sandstone prismatic specimens of dimensions 12cm/25cm/12cm. All specimens contained one horizontal infilled joint with smooth and undulated walls (average asperities height of 7 mm) and different infill material thicknesses. The specimens were formed by hardening a mixture of silver sand, dental plaster, water and additives (calcined alumina and mineral barite). In this way was obtained a material which had a density of 1.85mg/m3, uniaxial compressive strength of 3.50MPa, point load strength of 0.45MPa and Young's modulus of elasticity of 0.60GPa. Dry, non-cohesive pulverised fuel ash with almost spherical particles of glass, specific gravity of 2.39mg/m3, mean particle size of 0.001mm and shear resistance angle of 33º was used as the infill material. In this way, the obtained specimens with infilled joint which are represent prototypes of real rock block of dimensions 15 times those of the specimen and strength parameters 20 times those of the specimen. The pulverised fuel ash simulated a silt or silty-sandy infill material of natural rock joints. The results of performed direct shear tests along joint under constant normal stress are shown below.

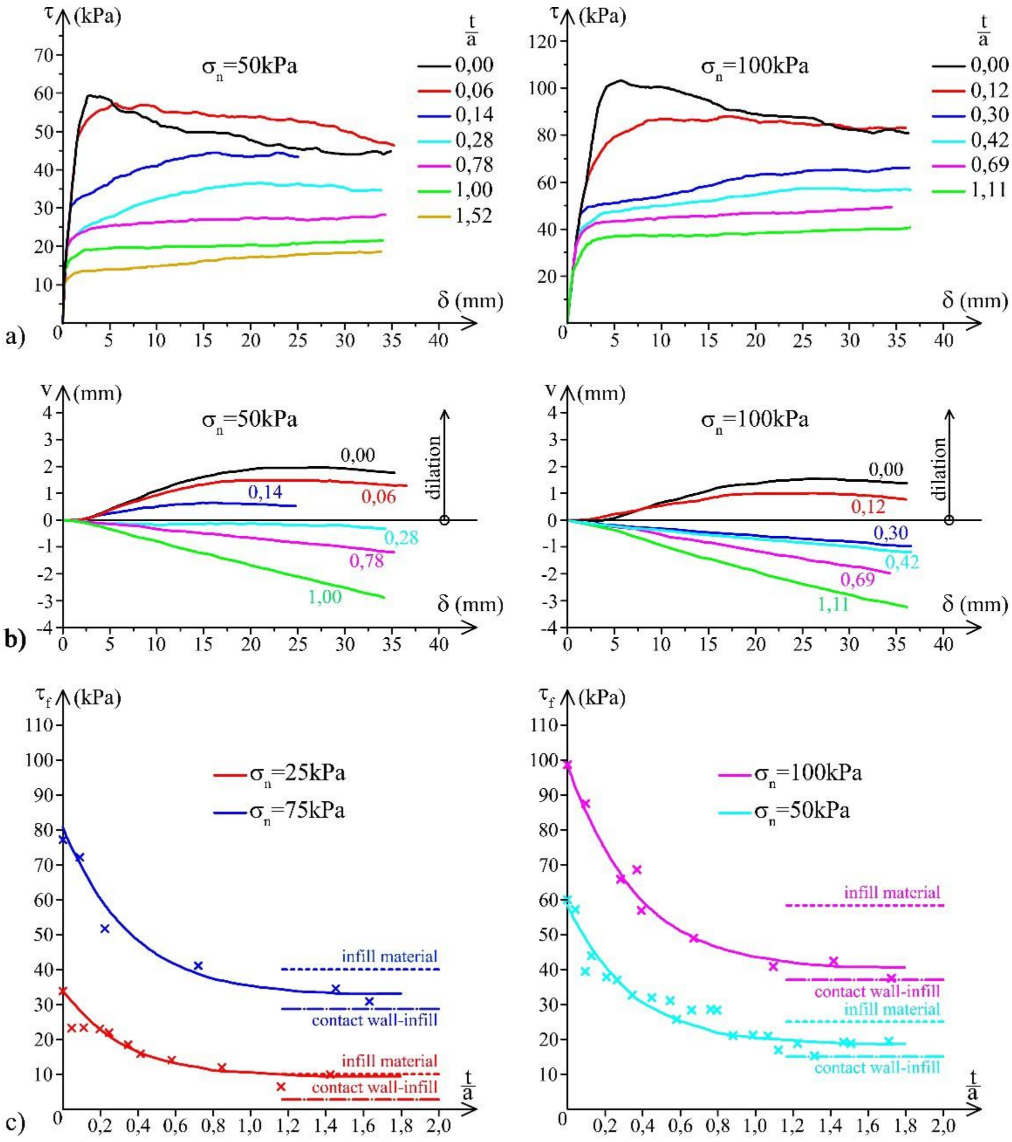

Figure 1. Results of direct shear tests along joint under constant normal stress for infilled artificial rock joints with smooth and undulated walls a) Shear stress τ vs shear displacement δ curves for different value of ratio t/a b) Vertical displacement v vs shear displacement δ curves for different value of ratio t/a c) Peak shear strength τ_f vs ratio t/a curves for different value of normal stress during shearing (Papaleangas et al., 1993)

The presented results indicate the expected decrease in the peak and residual shear strength of the examined infilled rock joints as well as a gradual change in their mechanical behaviour during shearing under constant normal stress with increasing their ratio *t/a* i.e. with increasing their relative infill material thickness. Also, it can be seen that instead of dilation a contraction of examined specimen is registered when its value of the ratio t/a ratio is greater than approximately 20%.

Special attention should be paid to the recorded values of the peak shear strength of the infilled joints at a relatively large infill material thickness i.e. in situations when the ratio *t/a*>100%. Figure 1c shows that these minimum values of the peak shear strength of the tested infilled joints were often significantly lower than the shear strength of the infill material itself at the same value of normal stress during shearing. In Figure 1c, the colored dashed lines (the color indicates the value of the normal stress during shearing) define the shear strength of the infill material itself at the corresponding value of normal stress during shearing. So, Papaliangas et al. (1993) unequivocally proved that the shear plane of does not always pass through the infill material. Actually, the contact between joint walls and infill material (contact "joint wall-infill") is often the weakest part of the infilled joint.

In order to complete the obtained results, the same author and his collaborators formed and tested several specimens of the same rock-like material (artificial sandstone) with planar and smooth horizontal joints and relatively large infill material thickness (pulverised fuel ash) in the direct shear test under constant normal stress. In this way, they experimentally defined the shear strength of the contact "joint wall-infill" for some values of normal stress during shearing. Since the joints walls of these additionally tested specimens were planar and smooth, their measured shear strengths at the same time represent the minimum possible shear strengths of the all analyzed infilled joints for the corresponding values of normal stresses during shearing. Of course, this minimum values of shear strength correspond to the dimensions of the tested specimens, physical and mechanical characteristics of rock-like material and machanical characteristics of infill material. In Figure 1c, the colored dash-dotted lines (the color indicates the value of the normal stress during shearing) define values of these minimum possible shear strengths of all tested infilled joints (shear strength of contact "planar and smooth joint wall-infill") for corresponding values of normal stresses during shearing.

Based on the obtained results of the direct shear test along joint, idealized curves of the change in the normalized shear strength τ/σ_n of the infilled rock joint with increasing relative infill material thickness i.e. with increasing values of the ratio *t/a* were formed (Figure 2). Two types of infilled joints that differ from each other only in terms of the joint surface roughness (the same type of rock and the same type of infill material) were analyzed. The first type, marked I, represents an infilled joint with very rough surface. The second type, marked II, represents an infilled joint with planar surface.

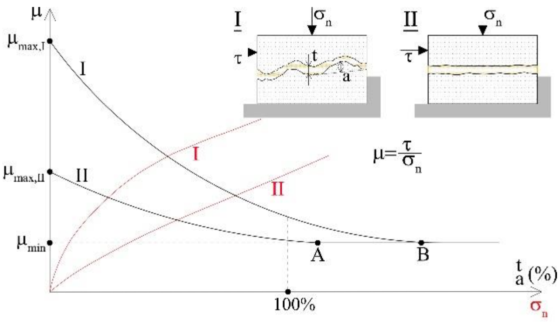

Figure 2. Idealized curve of the change in the normalized shear strength of infilled rock joint with increasing relative infill material thickness (modified after Papalianges et al., 1993)

At very small values of the ratio *t/a* (a few percent), i.e. at very small relative infill material thickness, the presence of infill material can be completely neglected. For both treated types of rock joints the shear strength is approximately equal to its maximum possible value (τ_{max} or μ_{max}), which actually corresponds to the shear strength of the identical unfilled rock joints for the same value of normal stress. In the figure above, red lines represent normalized shear strength envelopes for the identical unfilled rock joints I and II. However, with increasing infill material thickness, the shear strength of the rock joints decrease. This decrease is very pronounced in joint type I because with increasing infill material thickness dilation is becoming less pronounced. In these situations, the actual shear strength of the infilled rock joint depends on the geometric and mechanical characteristics of its surface (walls) as wall as the mechanical characteristics of the infill material.

With a further increase in the relative infill material thickness the shear strength of the joints decreases but at a decreasing speed. So, the shear strength of the joints asymptotically tends some of its final minimum value (τ_{min} or μ_{min}). It is important to note two facts that have been experimentally confirmed several times. The first fact refers to the moment of reaching the minimum shear strength of the rock joints. In rock joint type I (joint with very rough surface) at the moment when the ratio *t/a*=100%, its shear strength is still approximately 10% to 50% higher than the minimum value, depending on the level of normal stress during shearing (Goodman, 1970; Ladanyi & Archambault, 1977). This fact can be considered as the influence of the joint surface roughness and the compaction (consolidation) of the infill material during shearing. In other words, with joint type I, the minimum shear strength is reached when the infill material thickness is significantly greater than the height (amplitude) of joint surface asperities (point B). The described excess of the height of the asperities can be approximately 25% to 50% (Papaliangas et al., 1993) or even more than 100% (Barton, 1973). For joint type II at the moment when the ratio *t/a*=100% shear strength of rock joint is approximately equal to the minimum shear strength.

Second fact to note relates to the value of the minimum shear strength of the infilled rock joint. In general, this value corresponds to the shear strength of the infill material itself. Civil enginners in practice usually think this way in situations when the ratio is *t/a*≥100%. However, experimental research has shown that in these situations the shear strength of the infilled rock joint may be less than the shear strength of the infill material itself. These are usually situations with non-cohesive, fine-grained infill material with low water content and/or situations with planar and smooth joint surface. In these situations, in fact the shear plane does not pass through the infill material but passes completely or for the most part through the contact of the joint surface (joint wall) and the infill material.

Figure 3 shows the typical shear stress τ vs shear displacement δ curves for rock joint type I with different infill material thickness in displacement controlled direct shear test along joint under constant normal stress (σ_n=50kPa).

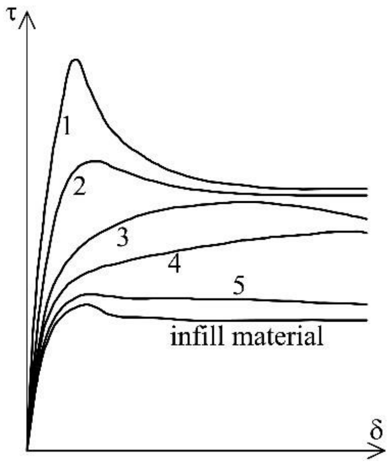

Figure 3. Typical shear stress τ vs shear displacement δ curves for joint type I with different infill material thickness in direct shear test under constant normal stress (Papalianges et al., 1993)

As previously established, when the value of the ratio *t*/*a* is very small (a few percent), then the mechanical behaviour of the rock joint type I during shearing is actually the same as the behaviour of the identical unfilled rock joint during shearing. The initial stiffness of the joint is large and its peak shear strength is reached at a relatively small shear displacement δ (curve 1).

With increasing infill material thickness *t* the contribution of dilation to the shear strength of rock joint type I is smaller, which leads to a significant decrease in its peak shear strength. Residual shear strength of also decreases but to a much lesser extent than peak shear strength. The initial stiffness of the rock joint type I decreases but the value of the shear displacement at failure increases (curve 2, *t*/*a*≈10%). Soon, with a further increase in the infill material thickness instead of dilation of the analyzed infilled rock joint type I during the shear process, a contraction is registered. This negatively affects primarily the value of its peak shear strength which is registered at a relatively large shear displacement. The initial stiffness of the rock joint type I is decreasing, as well as its residual shear strength (curve 3, *t*/*a*≈20%).

With a further increase in the infill material thickness, plastic stress-strain behaviour with obvious strain hardening is registered. The maximum value of shear stress is registered at the end of the test, i.e. at the maximum value of the applied shear displacement. Residual shear strength can not be reached (curve 4, quite approximately 30%≤*t*/*a*≤75%).

For the case when the infill material thickness is approximately equal to the height (amplitude) of joint surface asperities *a*, the mechanical behaviour of analysed infilled rock joint type I during shearing corresponds to the mechanical behaviour of the infill material. Due to the influence of joint surface roughness, the peak and residual shear strength of the analyzed rock joint may be slightly higher than the peak and residual shear strength of the infill material. In this general description of the mechanical behaviour of infilled rock joints in the direct shear test under constant normal stress, the mechanical behaviour of the infill material during shearing corresponds to the mechanical behaviour of normally consolidated clays and loose sands during shearing in direct shear test under constant normal stress.

Masoud (2015) performed a very interesting study of the shear strength of natural infilled rock joints. With standard geotechnical field works (drilling and sampling) but very careful, from sandstone rock mass he extracted quality prismatic specimens with orientation dimensions of B/L/H=7.0cm/7.0cm/15cm. All specimens were intersected in the middle by a natural horizontal unfilled joint with smooth and undulated surfece. For all specimens joint roughness coefficient JRC was around 7. After that, in the laboratory, the specimens were separated and then a layer of infill material of a certain thickness over the joint walls was carefully added (Figure 4). Three types of materials for the infill of the rock joints regarding their graining including were used: sand, clay and sandy-clay. Then, all natural specimens were reassembled and tested these specimens in a direct shear test under constant normal stress. Nine different values of ratio *t*/*a* in the range of 0.0 to 1.6 was considered. Four different values of normal stress during shearing of 0.25MPa, 0.50MPa, 0.75MPa and 1.0MPa was considered. Figure 5 shows only part of the results of the laboratory stady carried out by Masoud (2015).

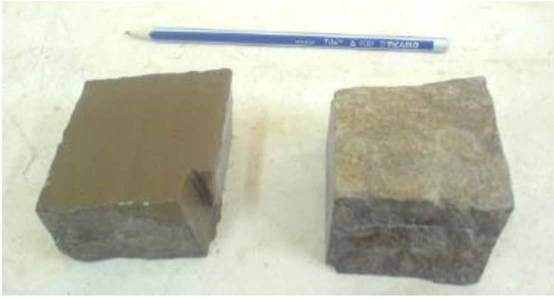

Figure 4. Addition of the clayey infill material over the rock joint (Masoud, 2015)

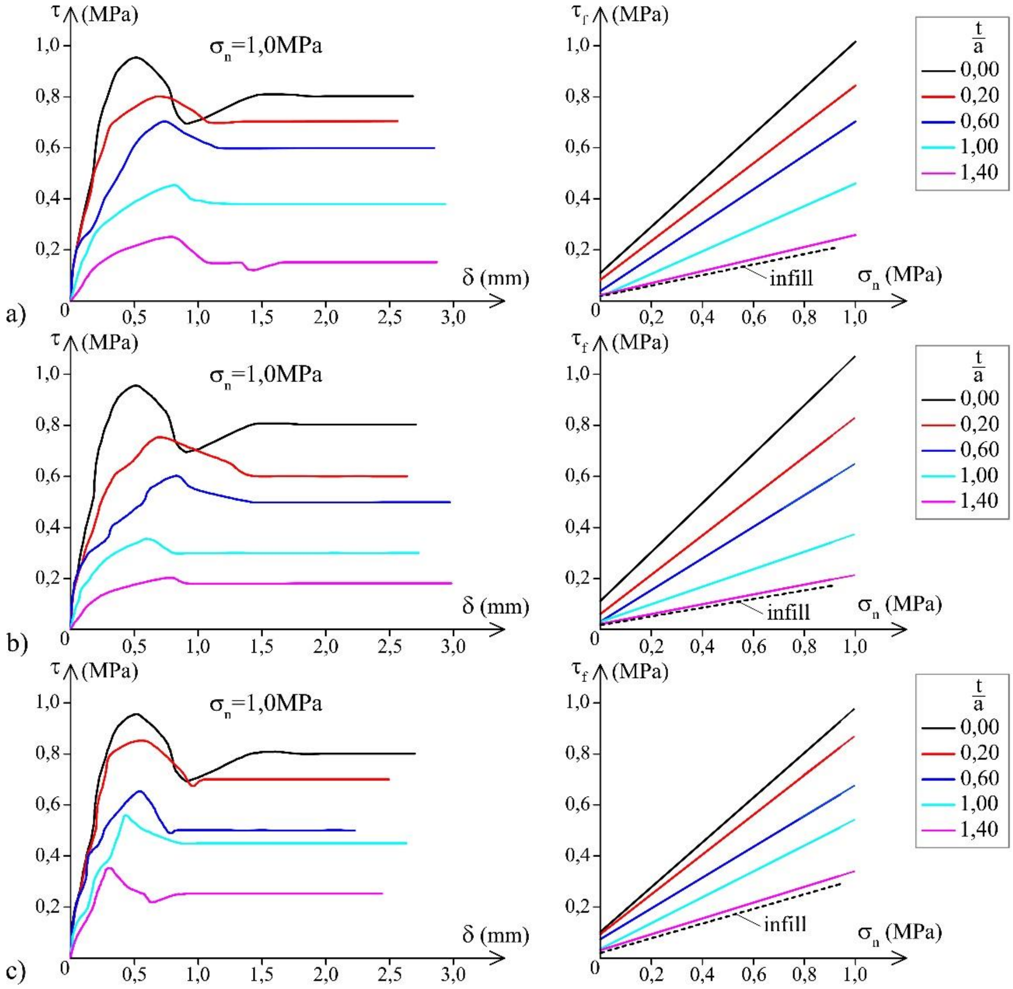

Figure 5. Shear stress τ vs shear displacement δ curves and linear shear strength envelopes for natural infilled rock joints with different values of ratio *t*/*a* in direct shear test along joint under constant normal stress
a) Clayey infill material b) Sandy infill material c) Sandy-clayey infill material (Masoud, 2015)

The presented results confirm the fact that the initial stiffness, peak and residual shear strength of the infilled rock joints decrease with increasing relative infill material thickness i.e. with increasing ratio *t*/*a*. The values of shear displacement at the failure of these rock joints generally increase with increasing infill material thickness. There is a certain deviation regarding the shape of the recored shear stress τ vs shear displacement δ curves in relation to the shape of these curves which was declared as typical by Papaleangas et al. (1993). This deviation primarily refers to the occurrence of strain hardening that Masoud (2015) did not record. The slope of linear shear strength envelope of infilled rock joints decreases with increasing value of the ratio *t*/*a*. This slope actually tends the shear resistance angle of the infill material itself. Also, by increasing the value of the ratio *t*/*a*, the cohesion of the infilled joints i.e. the section of their shear strength envelope on the vertical axis decreases. This is due to a decrease in dilation with increasing infill material thickness, resulting in a smaller curvature of the shear strength envelope in the zone of small normal stresses.

4. CONCLUSION

The presence of a infill material between the walls of a rock joint negatively affects its shear strength. As the infill material thickness increases, the shear strength of the infill rock joint decreases and asymptotically tends to same minimum value. This minimum value of shear strength of infilled rock joint with rough surface is actually equal to the shear strength of the infill material itself. However, for infilled rock joint with planar and smooth surface, the minimum shear strength may be even lower and actually correspond to the shear strength of the contact "planar and smooth joint wall-infill". Of corse, mechanical behaviour of infill rock joints during shearing depends on mechanical characteristic of infill material.

In this paper, the results of displacement controlled direct shear tests along joint in which the rock specimens (joints) were loaded with shear load in one direction to failure are analyzed. However, since we are in the area with high degree of the seismic hazard, it is very important to know the mechanical behavior of rock joints during cyclic shearing. This is one of the tasks of future researches. Also, it is interesting to analyse mechanical behavior of intermittent rock joints.

When studying the shear strength of rock joints, it is always a challenge to conduct direct shear tests along joint on natural rock specimens. One such research is planned to be conducted in the laboratory of the Faculty of Civil Engineering in Podgorica. Marl specimens with natural joints will be used. In addition to standard displacement controlled direct shear tests, it is planned to load rock joints with long-term shear load.

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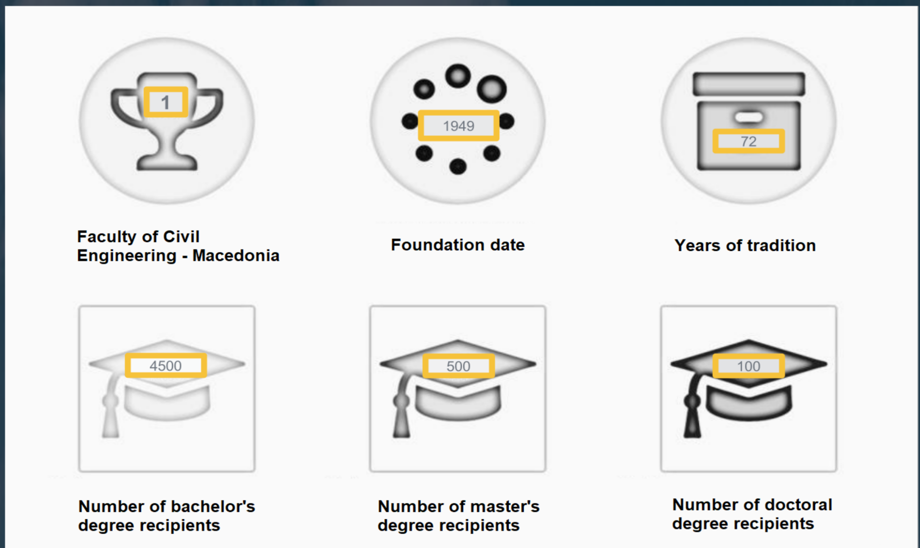

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METHODOLOGY FOR ANALYZING CAPACITY AND LEVEL OF SERVICE USING HCM 2000

The augmentation of motorization level leads us to the need for mobility and demands better infrastructure, in urban and suburban areas. The complexity of this problem is especially notable in urban areas where the space delimitations, functional characteristics and different transportation must be considered.

The intersection between boulevard Krste Petkov Misirkov and boulevard Goce Delcev, in Skopje, has been analysed with the methodology for capacity and level of service, according to Highway Capacity Manual. Both boulevards are with three lanes before the intersection, and two additional lanes for left and right turns in the intersection area, and this is one of the most frequent intersections in Skopje. Number of vehicles is determined by measuring the traffic, and those inputs are used to analyse three solutions: The current solution (signalized intersection), Roundabout and interchange (levelled roundabout).

Calculations are based on custom measurements within a week.

Keywords: intersection, analysis, roundabout, capacity, level of service, Highway Capacity Manual

1. INTRODUCTION

City development affects all the movements and need for transportation. In urban areas, besides the motor traffic, bicycles and pedestrians are important part of city traffic. Part of the motor traffic in cities are buses, city railways, subway and trolleybus. These vehicles must be considered in each analysis, because of their influence [1].

The choice of the type of intersection and thus the applied design elements depend on the category of the road and its function in the network, as well as the ratio of the forecasted intensities and throughput [2]. Traffic conditions at the intersection must be regulated in such a way as to ensure maximum safety of all traffic participants and the required traffic flow. When choosing the type of intersection, one should strive for uniform solutions, which contributes to

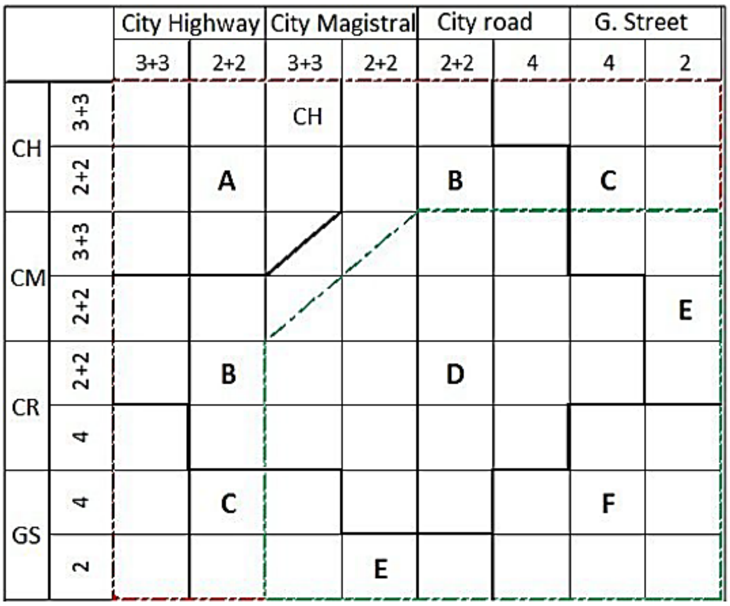

Figure 1. Functional level of intersection

the driver creating a "picture of the expected situation" and recognizability of the road category, which positively affects the driver's behaviour and thus the level of safety.

Intersection type is important and depends on many factors. For instance, if both roads are with similar traffic load, roundabout is recommendable. In case of different traffic load, signalized or unsignalized intersection is better solution. If the roads have more than 4 lanes, classical intersection is the best solution, or intersection with required signalization [3].

There are six types of intersection levels with defined connections and moving regime, according to the roads that are crossing (Fig.1) [4]:

- Functional level A means a interchange where there is no current intersection of the flows.
- Functional level B is applied to CM and CR, the intersection is levelled by cutting the traffic streams of lower rank.
- Levelled intersection with reduced outflow and intrusion is applied at the functional level C (inherent in the intersection of CH with lower rank roads).
- Complete traffic channelling and traffic lights are applied at functional level D. It is applied at city roads.
- Functional level E is applied when connecting gathering streets with streets of higher rank and implies the application of a surface intersections without sewerage of the traffic is applied in the assembly streets - functional level F. They can be signalized or unsignalized.

The Highway capacity manual is used for analyzing capacity and level of service for many various facilities [5]. The analyzed flows are classified as interrupted or uninterrupted flows. Uninterrupted flows are all the flows with no fixed elements (like traffic signals). Traffic flows depends on vehicles interactions and geometric and environmental characteristics.

Interrupted flows, on the other hand, have controlled and uncontrolled access points that interrupt the flow. This includes signals, stop-signs and any type of control that interrupts or slows the traffic. City roads are classified as interrupted because of the signs, signalization and bicycle and pedestrian presence.

The question is how to choose an appropriate traffic solution in the area of intersections and what is the correct choice of solution for intersection?

Such a complex question can be answered only with appropriate traffic analysis in order to check the capacity and level of service for the considered intersection. One way to make such a big decision is by applying the HCM methodology. Depending on how much traffic loads are involved and what spatial constraints occur. There are appropriate methods according to HCM that provide the level of service and capacity for signalized intersection, unsignalized classical or roundabout intersection and interchange.

With these methods, an analysis was made of the intersection of Boulevard Krste Petkov Misirkov and Boulevard Goce Delchev, and the obtained results are demonstrated in the reports.

Calculations have been made for different solutions at the indicated intersection, in order to determine which solution is most favorable.

2. REVIEW OF THE PREVIOUS RELATED STUDIES

In process of planning and design of road intersections the common question is whether to apply a roundabouts or a traditional type of intersection. Numerous studies have been conducted that consider the choice of the type of intersection, mostly for the choice between classical signalized and unsignalized intersection and roundabout [6].

Parameters that are commonly considered in the analysis are: effective intersection capacity, main road capacity, minor road capacity, major road average delay, minor road average delay, major road, 95% queue length, minor road 95% queue length [7].

From the results of the intersection capacity analysis studies based on HCM 2000, it is evident that the application of a roundabout scenario shows higher performance at the intersections than the intersection having a secondary signal [8]. In general, it was found that the two-way stop controlled intersection performed best for relatively low major road one-way volumes, the pretimed signal performed best for relatively high major road one-way volumes, and the roundabout performed best for a mid-range volume between the two.

For the specific case, intersection at Boulevard Krste Petkov Misirkov and Boulevard Goce Delcev, there are no similar studies.

3. CAPACITY ANALYSIS OF INTERSECTION AT BOULEVARD KRSTE PETKOV MISIRKOV AND BOULEVARD GOCE DELCEV

The purpose of the research is the analysis of the capacity and the level of service for the crossroads. In urban areas, there are intermittent flows, either due to the presence of signalization, crossing of pedestrians and cyclists. Such interruptions limit the movement time of the participants in the part of the intersection. The capacity of the roundabout, on the other hand, depends on one side of the surface and on the other side of the time constraints. This paper covers the methodologies for analysis of traffic light and non-traffic light intersection (roundabout), which lists the necessary input data, the procedure for analysis and comparison of the obtained solutions.

For start, it is necessary to know the geometric characteristics of the analyzed intersection (number and width of lanes, longitudinal slopes, etc.) and to provide traffic data. By knowing this data, we can categorize the bands according to the movements they distribute. Further calculations and analyzes are performed for each group of lanes respectively, and the results are summarized at the intersection level.

Taking into account the input data for traffic and geometry, the flow saturation is calculated, through which the capacity of the groups of lanes and the retention is obtained. The level of service is related to the size of delays (the greater delays the lower level of service).

The subject of research is a crossroad between Boulevard Krste Petkov Misirkov and Boulevard Goce Delcev in the city of Skopje.

Boulevard Krste Petkov Misirkov spans in line North - South, while boulevard Goce Delcev spans in line East -West. Both boulevards have three lanes before the intersection and two additional lanes for left and right turns. In the intersection area there are five lanes in total per leg (Figure 2) (Figure 3).

Figure 2. Current solution of intersection - blvd. Goce Delcev (East – West line), Skopje

Figure 3. Current solution of intersection - blvd. Krste Petkov Misirkov (Blvd. K. P. Misirkov - North - Blvd. K. P. Misirkov - South line), Skopje

For the needs of this analysis traffic measuring is made during a week (15.02.2016 - 21.02.2016) in the morning hours, 07:30 - 09:30 from Monday to Friday and 08:30 - 10:30 during the weekend.

The traffic was measured for each approach accordingly, dividing the vehicles according to the type, and then arranging them in the lanes for right movement, left and right turning.

Since measurements were made in a duration of 1 hour, it is necessary to convert this traffic into average daily annual traffic (ADAT).

From these measurements for the traffic, we can calculate average daily traffic (ADT 1) and average daily annual traffic (ADAT 2).

$$ADT = \frac{CO}{FNC} * 100 \quad (1)$$

CO - Load per hour
FNC - n-hour factor (8-10, in this case it's 9)

$$AADT = \frac{ADT}{Ks} \quad (2)$$

Ks - Factor of annual variability (Ks = 1.09 in this case)

The Origin - Destination matrix is presented for 15 minutes traffic for the analyzed intersection (Table 1).

	BGD - E	BGD - W	BKM - N	BKM - S	
BGD - E	0	270	59	81	**410**
BGD - W	173	0	61	23	**257**
BKM - N	167	48	0	91	**306**
BKM - S	173	49	135	0	**357**
	513	**367**	**255**	**195**	**1330**

Table 1. Origin - Destination matrix for the analyzed intersection

It is important to note that these data were obtained on the basis of own measurements over a period of one week, and they can not be used for analysis of the intersection in the future, because there is no data on traffic growth.

For Origin - Destination matrix a cartogram was made, in which the movements in the part of the intersection are presented (Figure 4).

Calculations have been made for different solutions at the indicated intersection, in order to determine which solution is most favorable:

- Four legged signalized intersection
- Roundabout - unsignalized intersection
- Interchange (roundabout) - unsignalized intersection

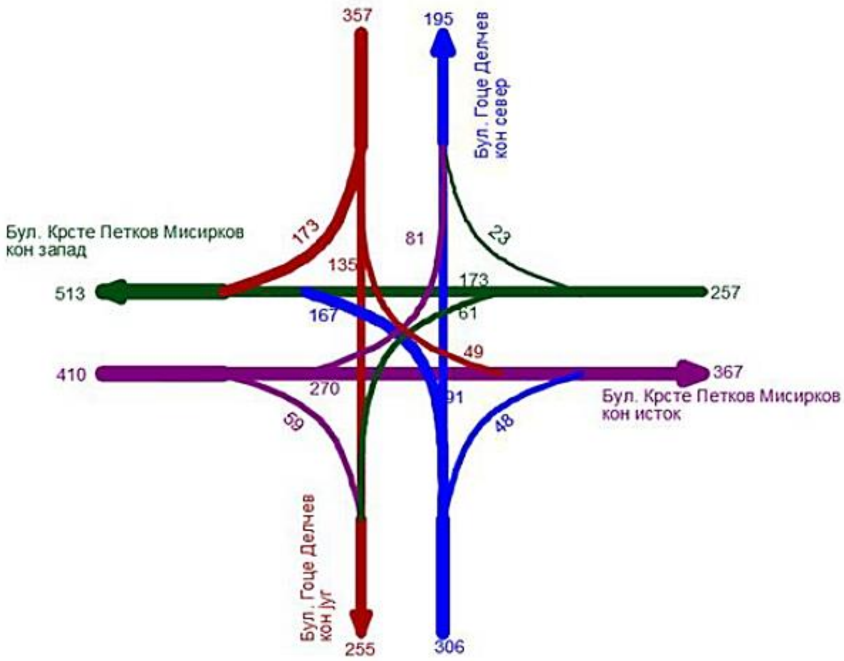

Figure 4. Cartogram of the analyzed intersection (Source: Own research)

3.1. FOUR LEGGED SIGNALIZED INTERSECTION

The intersection between blvd. Krste Petkov Misirkov and blvd. Goce Delcev is four legged signalized intersection. With personal counting of vehicles, the current traffic is obtained. Considering the influence of traffic, geometric and signalization conditions, appropriate correctional factors are used in order to calculate the saturation flow rate.

First step in the calculation is grouping the lanes, so that the capacity and level of service can be calculated for each group [9]. For this research lanes are grouped in 3 groups:

- Left turns and through
- Through
- Right turns

The left turns and through movements are actuated because they depend on the signalization, but the right turns as independent are classified as pretimed.

After grouping the lanes, volume adjustment is made by considering the percentage of heavy vehicles and peak hour factor. Next step is calculation of saturation flow rate, by knowing the number of lanes and appropriate adjustment factor (for lane width, HV, grade, area type, lane utilization...).

Now that both, adjusted flow rate in lane group and adjusted saturation flow are familiar, the capacity analysis can be done. For each group of lanes on each leg, critical lane group or phase is determined by the biggest flow ratio (v/s). Since all of the lane groups have flow ratio smaller than 1, except for the lane for right turns in the leg of Blvd. K. P. Misirkov - South the results for level of service are acceptable. Only the leg of Blvd. K. P. Misirkov - South has level of service F, while the other three have level of service A.

Another indicator of unsatisfying solution for the leg with LOS F is the delay.

In the calculations, the number of buses for some groups of lane is adjusted according to the HCM, it is given as 250 buses (max number given by the manual), even though the number is bigger than this. Also, each lane for right turns is analyzed as two lanes from 2.75m (2x2.75=5.50), because by the manual is not allowed to have lane wider than 4.8m.

With computation of total delay for each lane group, LOS can be determined, for each group lane and for each approach as well (Table 2).

Three of the approaches have LOS "A", while the approach of Blvd. K. P. Misirkov - South has LOS "F", because of the right turns, where the flow is bigger than the capacity.

Lane group capacity, Control delay and LOS determination				
	BGD - E	BGD - W	BKM - N	BKM - W
LOS by approach	A	A	A	F
Approach flow rate $v_a(veh/h)$	3958.06	2476.77	2960.61	3450.37
Intersection delay d_I	5.07	0.27	9.45	129.00

Table 2. Final results for signalized four-legged intersection

3.2. ROUNDABOUT – UNSIGNALIZESD INTERSECTION

Roundabout analysis is divided in two parts, computation of approach flows and computation of circular flow. In order to obtain more realistic results for each lane group, methodology for unsignalized four-legged intersection is used. First step is defining circulating traffic for each entry stream. (For example, for streams 7, 8 and 9 circulating flow is 1, 2 and 10).

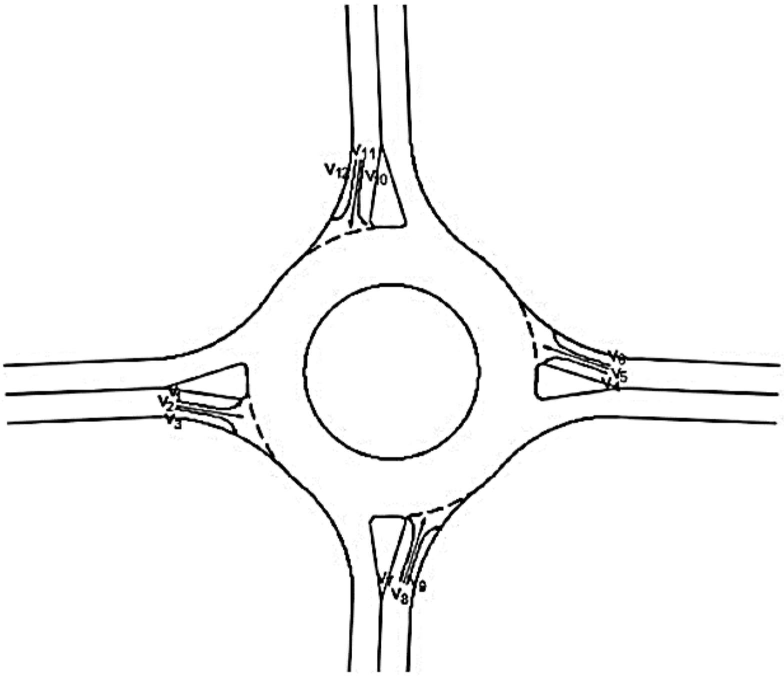

Figure 5. Flow stream definition

Because HCM 2000 only gives solution for roundabout with one circular lane, HCM 2010 methodology for two circular lanes is used in order to compute the capacity and obtain LOS. According to this methodology, the right lane is defined as dominant and the left lane as subdominant lane.

Since v/c ratio is bigger than 1,0 in two approaches (Blvd. G. Delcev - West and Blvd. K. P. Misirkov - North), the delays are bigger and LOS is lower. For these approaches LOS is "F", while for the approach of Blvd. G. Delcev - East LOS is "B" and of Blvd. K. P. Misirkov - South is "C" (Table 3).

Two - lanes roundabout				
	BGD - E	BGD - W	BKM - N	BKM - S
Entry lane capacity (right lane)	425	276	749	1059
Entry lane capacity (left lane)	716	561	500	515
Total capacity	1141	836	1248	1574
v/c	0.69	1.31	1.03	0.82
Control delay	14.89	164.19	52.11	16.68
LOS	B	F	F	C

Table 3. Final results for roundabout with two circular lanes

This results are expected, considering the fact that approaches with 3 lanes before, and 5 lanes in the intersection area are reduced to two-lane approaches and two-lane circular flow. Another anomaly in this concept is the lack of adjustment factors (only factors for heavy vehicles, pedestrians and bicycles are used). Anyway, utilization of roundabout with more than two lanes is insecure solution, considering the number of conflicting points.

3.3. INTERCHANGE (ROUNDABOUT) – UNSIGNALIZED INTERSECTION

Since neither of the previously mentioned solutions is acceptable, another possibility is analysed. By using diamond junction, delays would still remain big, and LOS would be low, so deleveled roundabout is proposed as more acceptable solution. The through movements from the main road are segregated in one level, while all the other movements are lead on another level, with circular flow. The same calculations as in two-lane roundabout is used, just the TH movements from Blvd. Goce Delcev are removed.

From the table is obvious that both approaches of blvd. Krste Petkov Misirkov have acceptable delays and LOS "A", while the approach of Blvd. G. Delcev - East has LOS "B" (which is acceptable) but the approach of Blvd. G. Delcev - West has LOS "F" (Table 4).

This solution is proposed strictly from visual and traffic aspect, with no information for installations, possibility for developing ramps or length of ramps.

Deleveled Two - lanes roundabout				
	BGD - E	BGD - W	BKM - N	BKM - S
Entry lane capacity (right lane)	425	276	1300	1510
Entry lane capacity (left lane)	716	561	903	753
Total capacity	1141	836	2203	2264
v/c	0.69	1.31	0.19	0.32
Control delay	14.89	164.19	52.11	16.68
LOS	B	F	A	A

Table 4. Final results for deleveled roundabout with two circular lanes

4. RESULTS AND DISCUSSIONS

Achieving the required capacity and level of service on any road and intersection as a whole, urban or suburban, is correlated with traffic load and geometric features [10]. In the years to come, with the development of technology and industry, as well as with social changes, traffic planning will become even more complex.

With the help of HCM methods that provide the level of service and capacity for signalized intersection, unsignalized classic or circular intersection and for deleveled roundabout, an analysis was made of the intersection of Boulevard Krste Petkov Misirkov and Boulevard Goce Delchev.

The aim is to achive satisfactory level of service and capacity.

4.1. REVIEW OF THE PROPOSED SOLUTIONS FOR THE INTERSECTION

Each of the proposed solutions has advantages and disadvantages. The solution with signalized intersection has relatively small delay, except for the approach of Blvd. K. P. Misirkov - South. The results are less acceptable for two-lane roundabout, while the deleveled roundabout has similar results as signalized intersection.

If the actual solution is accepted as more favorable, some corrections must be done, so that the problem with low level of service can be solved. One way to solve this problem is by directing the traffic on other existing roads.

Also, special lane for public transport vehicles would also help, because buses have big influence in capacity and level of service.

The bad results for both roundabouts can be because of the reduction of lanes in the intersection area. Anyway, this results are based on personal counting of traffic in short period, without previous information in order to obtain traffic increment, so they should be observed with backup.

4.2. INTERSECTION SAFETY

Studies have shown that roundabouts are safer than traditional stop sign or signal-controlled intersections [11].

Roundabouts reduced injury crashes by 75 percent at intersections where stop signs or signals were previously used for traffic control, according to a study by the Insurance Institute for Highway Safety (IIHS). Studies by the IIHS and Federal Highway Administration have shown that roundabouts typically achieve:

- A 37 percent reduction in overall collisions.
- A 75 percent reduction in injury collisions.
- A 90 percent reduction in fatality collisions.
- A 40 percent reduction in pedestrian collisions [12].

Increased safety of modern roundabouts occurs as a consequence of reducing the number of points of conflict compared to classic intersections, as well as reducing vehicle speed both when entering and while driving through the intersection, which is conditioned by the geometric shape of the intersection.

The reduction of the number of conflict points refers to both conflict points between vehicles and conflict points between vehicles and pedestrians (Figure 6) [13].

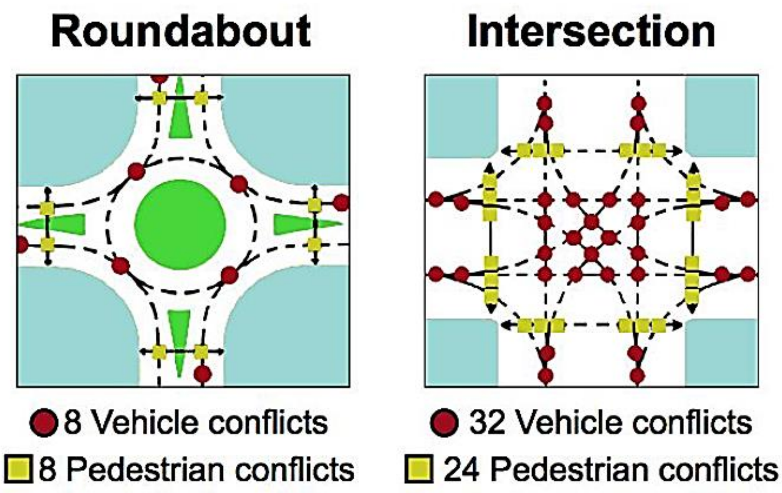

Figure 6. Points of conflict between the traditional intersection and the roundabout

4.3. WAYS TO IMPROVE THE LEVEL OF SERVICES

With the performed analyzes, results can be obtained where the level of service for any of these solutions is not satisfied. In this case, it is necessary to make changes in the existing solution and direction of traffic.

One of the ways to improve the level of service at a given intersection is to redirect part of the traffic on the existing road network, which would relieve this intersection. The possibility to expand the existing road network is not the most favourable solution, because by increasing the number of lanes, the capacity of the leg can be increased, but in the part of the intersection, large delays can occur, which would make it non-functional.

As public transport vehicles play a major role in the functionality of the intersection, it is possible to introduce a special lane for these vehicles, which would not interfere with other road users.

In developed countries, a commonly used way to solve the problem of capacity and throughput in the central urban areas is by introducing a ban on the movement of motor vehicles, restricting the movement in the central areas or charging for the movement of motor vehicles in these areas.

5. CONCLUSION

At the moment when the existing intersection, due to overload or a large number of registered accidents, no longer functions as planned, the question arises whether there is a better solution, another type of intersection that works better. When introducing a new intersection into the traffic network, there is often a dilemma as to which type of intersection to apply. The path to a solution to these problems is not easy. The choice of the most favourable solution when choosing the type of intersection is influenced by aspects such as traffic safety and the quality of traffic flow determined by the capacity, waiting time and the degree of saturation. Other aspects that may influence the choice are the integration of the solution into the environment (surface and aesthetic) and of course the costs.

From the results obtained from the HCM model, due to the heavy traffic load, the most acceptable solution was a interchange (deleveled roundabout) where the movements in the main direction are separated.

The previous results are important because they can determine level of service and capacity for different solutions and improve the traffic performance of them in the future.

Finally, future research should be conducted to extend all aspects of this research using comprehensive field data and traffic measuring. For each major and significant intersection in urban areas it is necessary to make an analysis of capacity and level of service, in order to solve the problem of traffic jams.

It is necessary to make measurements of traffic on a time interval to get a realistic picture of the growth of traffic, which would perform a satisfying capacity in the future.

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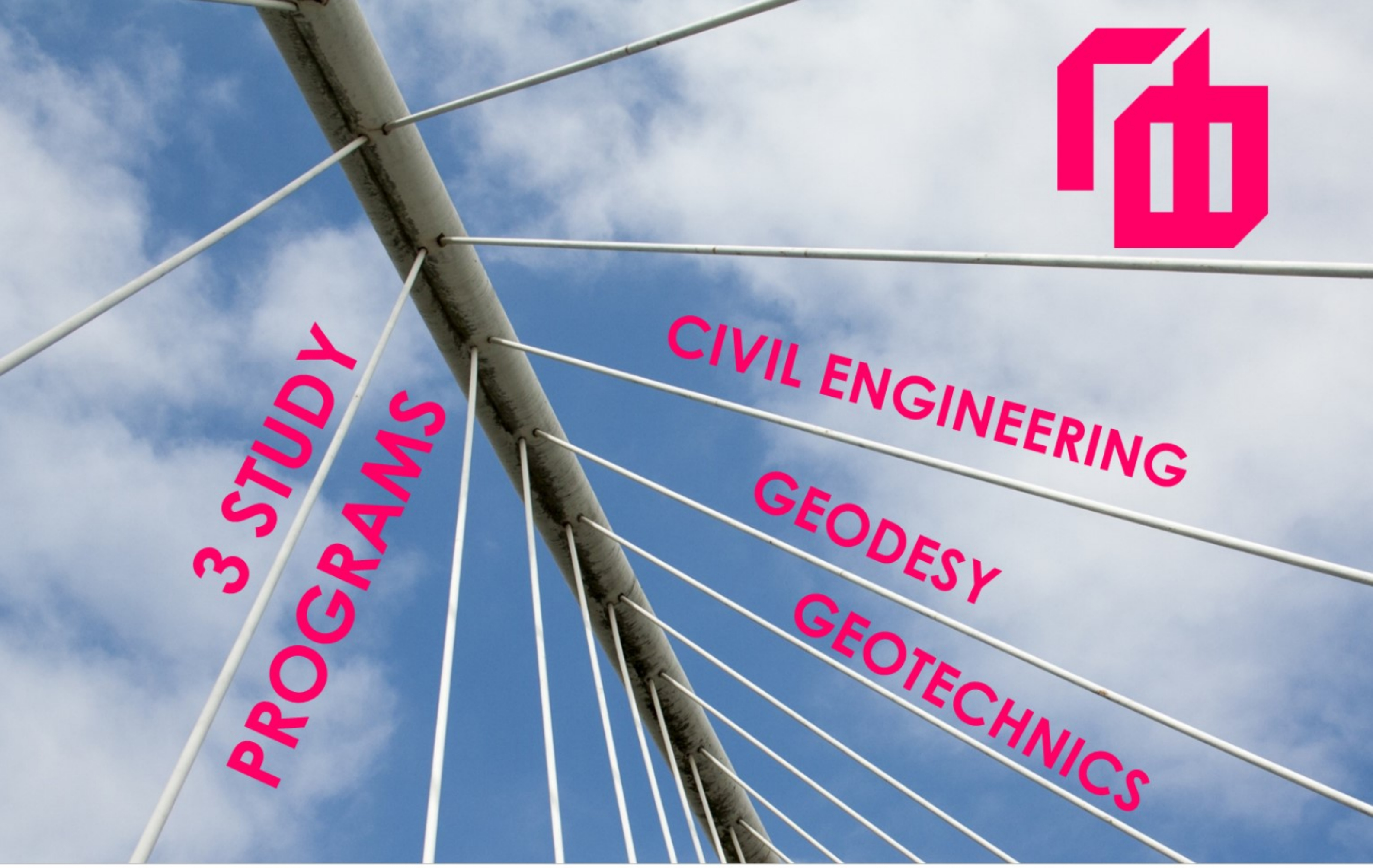

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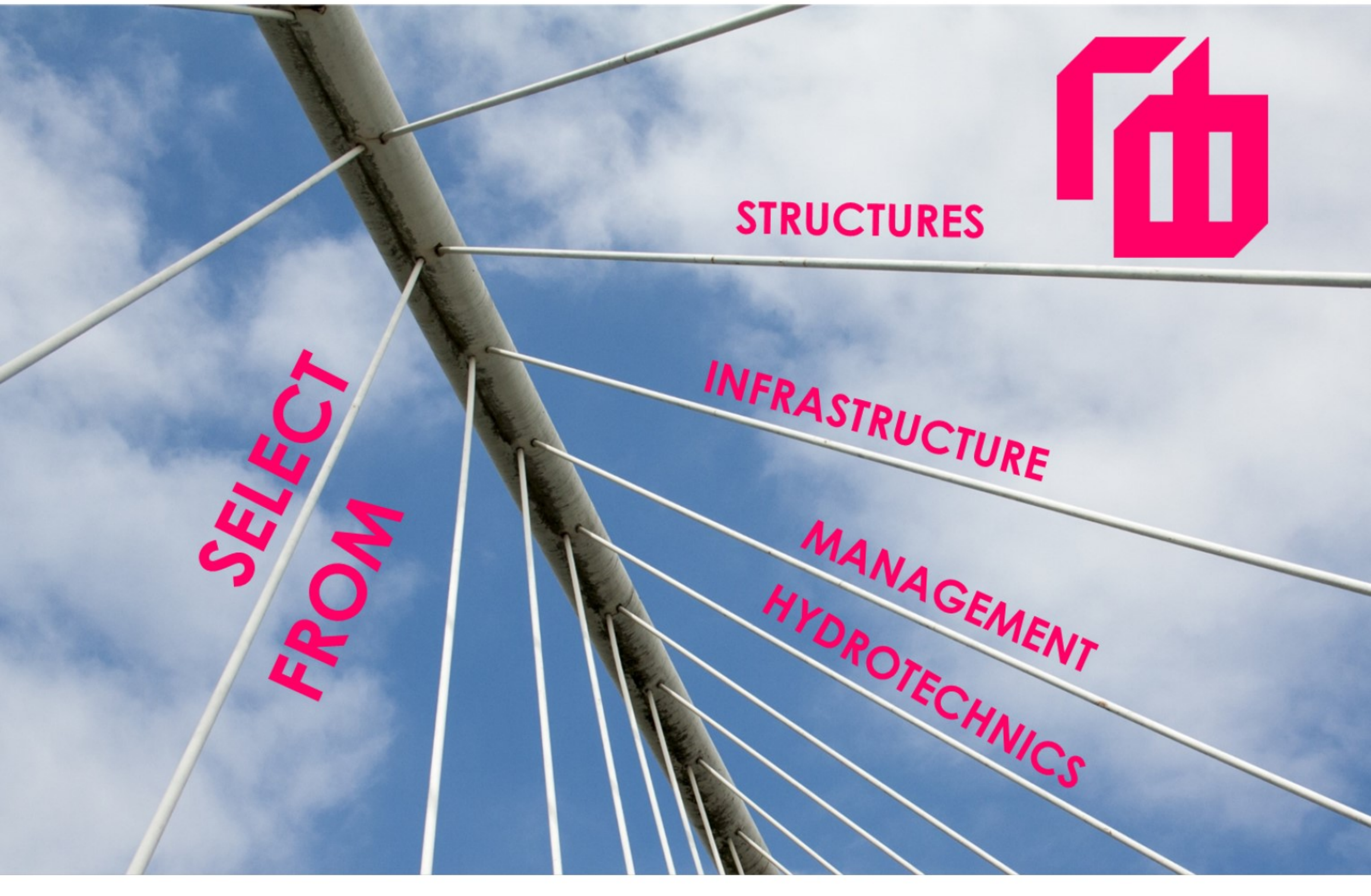

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URBAN PLANNING AS A FACTOR IN DETERMINING AND CHANGING THE VALUE OF REAL ESTATE IN URBAN SETTLEMENTS

The main purpose of this research was to examine the aspects of urban planning that affect the determining of real estate values and their changes, when planning the spatial development of cities and settlements. The basis for the conducted research was review and analysis of the role of urban planning and urban plans, and the significance of real estate and real estate values. Crucial for the case study were the results obtained from examining the relation between urban planning and the value of real estate and determining the aspects that influence urban planning as a factor in determining the value of real estate.

The research was conducted for the city of Skopje, as a capital and also a city based on which the urban policy of the entire country is created. The conducted case studies have been selected based on specific aspects of the urban conditions that directly affect the change of the real estate value.

The presented results showcase the significance of urban planning in the process of development planning, in terms of increasing, decreasing or changing the value of real estate and the findings can assist in making decisions within the process of creating city policies for planning, designing and managing cities and settlements.

Keywords: urban planning, urban plans, real estate, value of real estate

1. URBAN PLANING VS. REAL ESTATES

Urban planning and real estate value are essential for the economic state and the development of cities and settlements. The correlation between them is often emphasized as important in the adoption of policies for future development in many countries, but in Macedonia it is very rarely or not clearly enough taken into consideration. Internationally, there is an extensive scientific literature dealing with this issue and the justification for certain urban interventions and plans is examined by assessing the increase or decrease of the real estate value that would be caused by those plans. In our country, although the methodology for real estate appraisal partially covers aspects of urban planning, it still insufficiently covers and elaborates the relation between urban planning and the real estate value.

1.1. URBAN PLANNING AND ITS ROLE IN CREATING AND MANAGING THE DEVELOPMENT OF URBAN SETTLEMENTS

Urban planning includes the process of planning the structural and aesthetic distribution of land and buildings, in order to ensure a spacious, economic and social efficiency, health and wellbeing of inhabited places. The normative acts for this activity are urban plans that regulate the physical and spatial development of settlements, the purpose of the land and facilities, the requirements for construction of buildings and infrastructure, and thus the rights and obligations of the participants in spatial development.

Urban planning in Macedonia is a continuous process achieved by preparing, adopting and implementing urban plans, in order to provide design, humanisation of space, protection and improvement of the environment and the nature (LUP, 2020). Urban planning in our country is a regulatory system, with a hierarchy of planning documents and spatial planning, and the conditions for future development are specified from the highest planning document to the level of construction, with strict observance of the provided terms and norms.

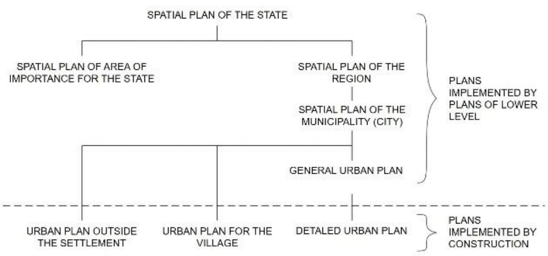

Figure 1. Hierarchical and subordinate scheme of the system of spatial and urban plans

Planning documents that regulate the space from the highest to the lowest level are: PP-Spatial plan of the Republic, GUP-General urban plan, DUP-Detailed urban plan, UPV-Urban plan for the village and UPOS-Urban plan outside the settlement and several other types that regulate specific situations.

The provisions of spatial development are crucial for the future spatial development and construction of cities, and they refer to:
- spatial disposition,
- physical dimensions,
- purpose and manner of building, and
- land and building use.

Planning provisions that regulate the space more precisely are:
- urban boundary;
- boundary of the land-use zones;
- land-use and type of construction;
- front property line;
- boundary of the construction lot;
- build-to line;
- buildable area;
- traffic and levelling solution of primary and secondary network of streets and other traffic infrastructures;
- primary and secondary network of all utility infrastructures;
- maximal height of the buildings;
- general and specific requirements for construction, development and use of land and buildings.

The planning provisions, presented graphically, textually and numerically, define all spatial-physical parameters of the planned supra and infrastructures, as well as all the building requirements and the manners of land and building use, and thus comprise the complex of urban instruments that are practically used for planning and shaping of the space and the settlements (Grchev, 2016).

1.2. REAL ESTATES, THEIR VALUE AND THE METHOD OF ASSESSMENT, IN URBAN SETTLEMENTS

Real estate (English: *immovable, real estate*, German: *Immobilien*), are items that are fixed, i.e. cannot be moved from one place to another without violating their essence, so real estate is considered land and items that are mechanically fixed onto it, such as buildings, but also bridges, dams, roads, etc., and items that are organically bound to the ground, e.g. plantations and fruits until harvested.

In Macedonia, the defining and specifying the term "real estate" is treated in the laws related to real estate cadastre, property rights, property taxes and the like. According to the Law on Real Estate Cadastre, the term real estate refers to land, buildings, specific parts of buildings and other objects, as well as other real estate that is registered in the real estate cadastre (LREC, 2013). “The methodology on evaluating the market value of real estate”, prescribes that: "Real estate means residential houses (buildings for individual housing), residential buildings (buildings for collective housing), office buildings (plants, warehouses, halls and storehouses), business premises (shops), administrative buildings and administrative premises, buildings and apartments for rest and recreation and other facilities (garages, granaries, barns, stables and sheds) as well as construction land, agricultural land, forests and pastures.” (MEMVRE, 2012)

Every real estate has its own value and can be subject to inheritance, sale, lease, etc., but it is also subject to tax payment, taking a loan and mortgage, etc. In order to determine its value, assessment procedures are prescribed and each country determines the basic principles and methods of assessment. The assessment can be made based on several methods, three of which are considered as main and are most often used: sales comparison approach, income approach and cost approach.

The main elements for determining the market value of a real estate are grouped in two classes:

- Basic elements for determining the market value;
- Additional elements for determining the market value.

Table 1. Basic elements for determining the market value of real estate

Basic elements for determining the market value of construction	Type of building
	Floor structure
	Roof structure
	Installation type
	Sub-floor
	Lift
	Sanitary ware
	Facade joinery (windows)
	Isolation and exclusivity
Additional elements for determining the market value of construction	Number of floors in the building
	Microlocation
	Macrolocation
	Attractiveness of the building

The basic elements that refer more to the construction aspects based on the construction type and providing physical quality of the buildings. Contrary to this, the additional elements refer to urban aspects that depend on spatial aspects and the location, the meaning and the relation of the buildings within and to the environment and other buildings and urban contents.

Given the spatial or location aspects, it is important to emphasize the significance of some of the additional elements for determining the market value of real estate. Hence, under *attractiveness* we understand increased interest for buying on a specific location, under *microlocation* we understand proximity to suppliers, health and educational centres, communication, parking space, playgrounds, sport centres etc., and under *macrolocation* is the zone in which the building is located.

When assessing the value of the buildings and the land, we consider the construction value of the buildings, as well as the location factors affecting the price of the building, characteristics depending on the wider location in relation to the city are also taken into consideration (macrolocation conditions) and location conditions of partial importance (microlocation conditions) (MEMVR, 2012). Zones for macrolocation and points for the real estate, i.e. the construction land expressed in m^2, are prepared for the assessment at city level. For example, for determining the market value of real estate, the City of Skopje is divided into zones of microlocations by municipalities, which are 20, and each zone is expressed in value points.

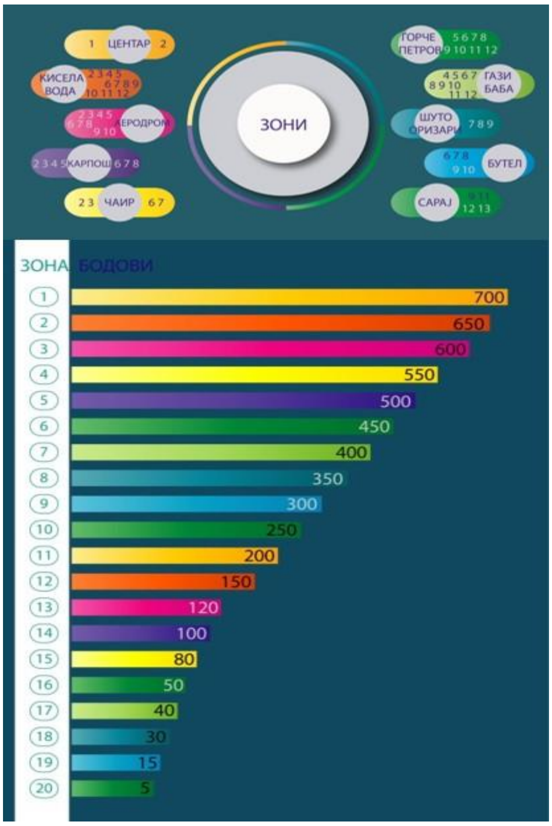

Figure 2. Zones of macrolocation in settlements in Skopje; Zones of macrolocation and points expressed in m^2 in Skopje

The market value of the construction land is determined based on the *Pricelist for determining the market value of construction and agricultural land*, issued by the Councils of the municipalities and the Council of the City of Skopje, pursuant to article 26 paragraph (1) from the Law on Property Tax (LP, 2004).

Table 2. Pricelist for determining the market value of construction and agricultural land issued by the city of Skopje

Construction land within the limits of the GUP of the City of Skopje and in DUP in the corresponding municipality	100 % of the price from the pricelist (factor 1).
Construction land within the limits of the GUP of the City of Skopje, and for the same no DUP was adopted by the corresponding municipality.	80% of the price from the pricelist (factor x 0.8).
Construction land that is not within the limits of the GUP of the City of Skopje, and there is an urbanism documentation (plan) for the same in the corresponding municipality with prescribed use of the building	100 % of the price from the pricelist (factor x 1).
Construction land located in an area of traffic corridor -street	50 % of the price from the pricelist (factor x 0.5).
Construction land located in an area of protected greenery, park, forest, transmission line etc.	30 % of the price from the pricelist (factor x 0.3).
If the subject of assessment is a land under a building registered in a property list, and there is an illegally constructed building on the spot that is not registered in a property list, only the value of the construction land is taken under consideration.	100 % of the price from the pricelist for construction land (factor x 1).

*The market value of agricultural land, forests, pastures, etc., which are outside the urban plan boundary (outside the city with planning documentation), but are in the immediate vicinity of regional roads, major highways and highways, and there are nearby infrastructure facilities that increase the value of the land, then up to 50% of the determined market value of construction and agricultural land is applied.

1.3. URBANING PLANNING ASPECTS THAT APPEAR AS FACTORS IN DETERMENING THE REAL ESTATE VALUE

Urban plans as the main tool in providing conditions for the construction of space, i.e. urban settlements, directly affects the creation of real estate, as well as their value and attractiveness. Real estate and determining the value thereof, in turn, rely on spatial planning aspects, such as micro and macro location, availability of accompanying uses and services etc., for calculations and decisions on transactions and sales. Hence, it is important to determine the aspects of urban planning that appear as factors in creating and determining the value of real estate.

There is an extensive literature in the world that deals with the influence that spatial and urban aspects have on determining the price of real estate. Researchers' discussions go in two directions, one is which spatial (urban) factors influence the creation of real estate and the determining of their value, and the other is how space gains quality and values driven by the creation of new values or investment in improving existing ones. Hence, the following are discussed: the real estate itself; the land and neighbourhood where they are located; distance from the centre; access to and provision of infrastructure and superstructure, utilities and services; quality of the environment, as well as about quality human communities. The research is mainly focused on investigating spatial models for calculation of impact factors and development of market modelling and monitoring tools.

In Poland, for example, the new detailed plans include an accurate calculation of the increase or decrease in value resulting from the change of zones, thus local authorities are aware of the financial consequences for this can only happen in places where public land is at least 50 present of the surface (Rechnio, 2016).

There is a noticeable interest in whether a good urban form can be capitalized on the market. If it is good, the urban form provides a higher quality of life. The monetary value of this effect shall be reflected in the price of the property as well as in other characteristics. A study examining the impacts of urban forms on property values using hedonic price analysis has been conducted (Anselin, 2002).

Hedonic price analyses are widely used to investigate the relation between urban forms and property value. The implicit marginal prices of various urban land use forms were examined by Cao and Cory (1981), taking into account the proximity of health centres, shopping malls, sports and recreation centres, proximity to transportation centres, the possibility of a secure parking space and quality infrastructural network.

In our country, the connection and the influence that urban planning and real estate have on each other is already being investigated. Pioneering research in this area is presented in the paper of Trpkovska (2017), which examines the impact of parks and urban forests on the price of residential real estate in Skopje. It uses the hedonic pricing method of multiple linear regressions that allows a detailed examination of the spatial relation between residential real estate and open green spaces (parks and urban forests). Mitev (2020), on the other hand analyses the influence of sports facilities on the prices of residential real estate in the City of Skopje that is grounded on the postulates of price movements on the real estate market, as inseparable part of one economy.

1.3.1 Determining the relation of urban parameters to the elements determining the real estate value

For the needs of this research, a correlation has been established between urban planning and urban plans and real estate and the value of real estate, comparing the elements for determining the market value of real estate and the provisions of an urban plan and urban parameters.

Table 3. Correlation of the elements for determining the market value of the real estate with the urban parameters

Group of elements	Elements for determining the market value	Correlation with urban parameters
Basic	Type of building	NO
	Floor structure	NO
	Roof structure	NO
	Installation type	NO
	Sub-floor	NO
	Lift	NO
	Sanitary ware	NO
	Facade joinery (windows)	NO
	Doors	NO
	Isolation and exclusivity	NO
Additional	Number of floors in the building	YES
	Microlocation	YES
	Macrolocation	YES
	Attractiveness of the building	YES

It can be concluded that basic elements, which refer to the construction itself, have no relation to urban parameters, however additional elements are those that have a relation and should be considered when planning.

If we inspect the situation in the other direction, i.e. by looking the relation of urban parameters to the elements determining the market value of the real estate, the correlation between them can be determined.

Table 4. To correlation of the urban parameters to the elements determining the market value of the real estate

Provision/ Urban parameters	Relation to the elements determining the market value of the real estate
- Planning boundary - Limit to the planning boundary	**Macrolocation** - within the urban boundary - outside the urban boundary
- Land and building use - Land use zone - Boundary of the land use zone	**Macrolocation/attractiveness** - Zone in which the land or the building is located - Some zones are more attractive the others - Distance and access to other zones with activities from other accompanying content ***Different character!*** - Dependent on retaining or altering of the land use zone
- Construction land - Regulatory lines	***Different character!*** - Dependent on whether it is located on construction land divided on lots for individual construction and use or construction land not divided on lots for general use.
- Front property line	***Different character!*** - Dependent on whether the front property line remains or it shall be changed.
- Construction lot - Boundary of construction lot	**Macrolocation/attractiveness** - Zone in which the land or the building is located - Some zones are more attractive the others - Distance and access to other zones with activities from other accompanying content ***Different character!*** - Dependent on retaining or altering the existing cadastral parcel.
- Build-to line - Buildable area	***Different character!*** - Dependent on retaining or altering the existing building - Dependent on the degree of additional construction
- Maximal height of the buildings	**- Number of floors of the building** - Dependent on the number of permitted levels ***Different character!*** - Dependent on the degree of additional construction
- Building coverage percentage	***Different character!*** - Dependent on the degree of additional construction
- Floor-Area Ratio	***Different character!*** - Dependent on the degree of additional construction
- Primary and secondary network of complete utility infrastructures	**Microlocation** - Zone which is covered or not covered with complete utility infrastructure

Based on the comparison, the key aspects of the impact of urban planning and urban plans on real estate and their value are extracted and the general aspects according to which further examination is conducted are:
- Location (macro and micro, land use zone, connectivity to other needs and attractive contents etc.)
- The scope of changes (existing/new, confirmation/cancellation, decrease/same/ increase, change of land use zone, etc.)
- Equipment (unchanged/increased volume, type and quality, additional purposes and connections in the environment, etc.)
- Attractiveness (retention/change of spatial concept, retained spirit of the place/ improved/disturbed etc.)

2. CASE STUDY - CITY OF SKOPJE

The study of examples, in order to analyse urban planning as a factor in setting and changing the real estate value is conducted for the area of the City of Skopje. The City of Skopje as a capital is a city that is developing with high intensity and where there is most intensive building. The phenomena occurring with regard to its spatial development are the incentive behind creating an urban policy in the whole country and it is reflected in other settlements.

2.1 METHODOLOGY

The research was conducted through:

- Selection of aspects to be considered. Several aspects have emerged from the research, but due to the scope, five key ones have been singled out.
- Collection, review and selection of documents. Used were: General urban plan of the City of Skopje (2012-2022), detailed urban plans for the areas of the selected examples, relevant material referring to the chosen examples etc.
- Analysis of individual examples and field work to check the situation.
- Synthesis of the obtained results and presentation thereof.

Publicly available data, cadastral data, GIS tools and field research were used for the research. The criterion for selection of examples is the diversity and richness of possible arguments.

2.2 ANALYSES OF INDIVIDUAL EXAMPLES

The analysis was conducted based on 5 separate aspects:

1. Expanding the urban construction area and changing the manner of using the surrounding land.
2. Changing the land use within the planning scope.
3. Change of cadastral lot (CL) in urban plans and defining the construction lot.
4. Layout of construction land for public use and their interdependence and mutual influence.
5. Affect of the open green spaces, as a desired quality in urban areas.

2.2.1 Expanding the city construction region and change in the manner of use of the surrounding land

When planning future spatial development of settlements, in the urban plans for settlements (GUP and DUP for Skopje and cities; UPV and UPOS) it is essential to determine the planning scope, i.e. The boundaries of the planning scope. Automatically, everything that enters within the borders becomes a construction area and subsequently the value of the land and the buildings as real estate found in that area change.

We have investigated part of the village area of Vizbegovo, Skopje, where the inclusion of agricultural land within the city directly affects the value of this land. Specific for the location is that for many years this part has not been used due to lack of infrastructure and has not been developed through a detailed urban plan. After the connection to the Bul. Slovenia, this location became attractive and the prescribed uses are M (A+B) - Mixed: A - housing and B - commercial and business use.

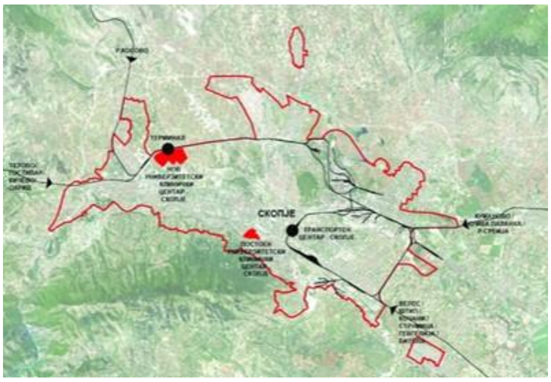

Figure 3. Limits of planning boundary of the City of Skopje - GUP of the City of Skopje 2012-2020

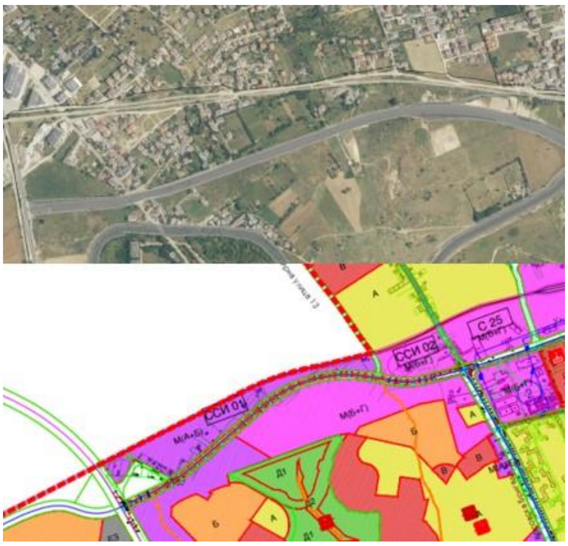

Figure 4. Satellite image of part of the village Vizbegovo and planned expansion of the limits of the planning boundary in the GUP of the City of Skopje Satellite 2012-2020

The implementation of the planning solutions means a change in the value of the land and the large number of illegal constructions here are in the process of or have already been legalized. The data show that the price of land in the village Vizbegovo was around 1000 MKD per m^2, but with the conversion of land use into construction the price rose up to around 5000 MKD per m^2.

Such a transformation has a positive effect if there is interest in using this land for construction and the corresponding planned use, and the owners can have material benefit from the sale or use for their own needs. But, otherwise it can have a negative effect, which is the land being "captured" and devastation, if there are no conditions to use it, it has an inappropriate use or the surrounding use or construction violates its primary conditions.

2.2.2 Change of land use within the planning boundary

The term land use means manner of design, construction and use of construction land and buildings in accordance with the activities that are performed and take place in them. According to the Rulebook on Urban Planning (RUP, 2020), the system of usage classes is composed of six groups: A - Housing; B - Commercial and business uses; C - Public Institutions; D - Production, distribution and services; E - Greenery, sport, recreation and memorial spaces; and E - Infrastructure.

The selected example is located in the city quarter Court Palace, in the Municipality of Centre, covered by the DUP for the city area CS 08 (2011). Located in its proximity are large public buildings Macedonian Radio Television, and in it a gas station and a neglected production facility.

The General Urban Plan of Skopje (2012-2020), prescribes M (A + B) use for this location, i.e. Mixed usage: A - Housing and B - Business and commercial, and the Detailed Urban Plan prescribes for the parcels on which the deserted building is located the land use B4- business premises.

This location became extremely attractive based on to the use prescribed in the plan, but also based on the areas and floors that are allowed for construction. This multiplies the value of the land and future buildings.

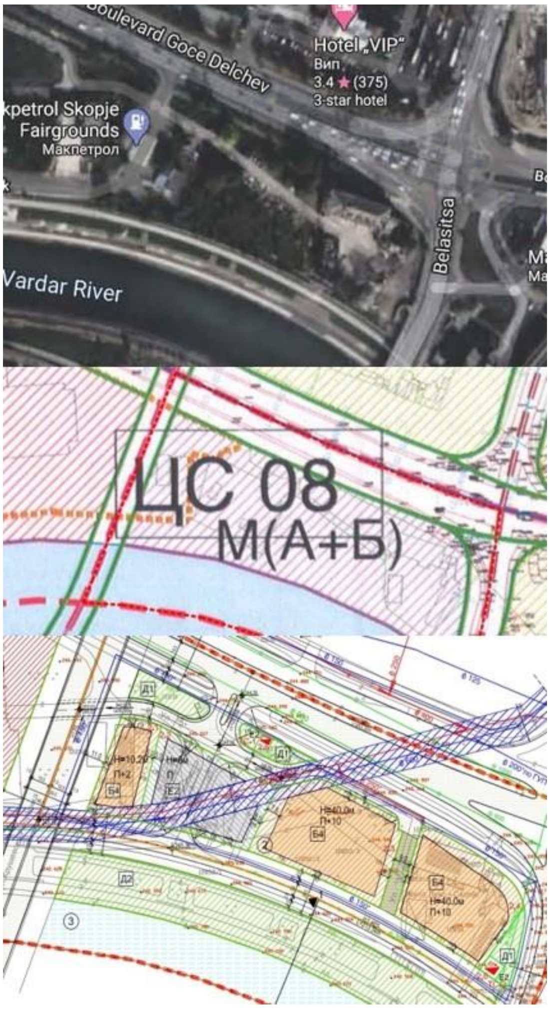

Figure 5. Satellite image of the city quarter Court Palace 2 (CS 08), excerpt from GUP Skopje 2012-2020 and DUP for CQ CS 08-Court Palace 2.

But question is why is there no construction there. The reason behind this is the fact that the location has bad access, but even more so that the investor who is interested in building is looking for other uses, namely B5 hotels. The municipality, on the other hand, considers such use to be inappropriate, given that the famous hotel Continental, which has been unused for many years is located here and that this will reduce its value. Furthermore, the local government considers that this is the right place to accommodate student facilities, due to its proximity to the University campus and the group of natural science faculties.

This disharmonisation of opportunities necessitates and requirements results in this "expensive" city land not being able to exploit its value.

2.2.3 Change of cadastral parcel (CP) in urban plans and defining a construction lot

In urban planning and designing urban plans, two titles or conditions of the parcel are distinguished. One is CP - cadastral parcel, the other is CL - Construction lot. *Cadastral parcel* marks the parcel in its current state and "cadastral parcel" is the main cadastral unit that is part of the land, defined with borders, and is located in one of the cadastral municipalities and it belongs to specific holder/s of the ownership right" (RCPIF; 2013). On the other hand, „*Construction lot* is the smallest unit of construction land that is formed by an urban plan and on which construction of a building is planned and/or a building is already built, where the lot is limited by the front property line and the boundaries of the construction lot and covers the land under the building and the yard or land for regular use of the building ". (LUP, 2020).

In urban planning, the main design of land is construction and its smallest spatial unit is construction lot. The following conditions must be met: to be located next to land for general use (street), and to have dimensions and shape appropriate to the purpose for which it will be used. Hence, the construction lot does not need to correspond with the cadastral parcel. In order to meet the conditions for a land to become a construction lot, cadastral parcels are often subdivided, consolidated or fragmented.

As an example of *consolidation*, we took the city quarter Novo Maalo, in municipality Centre (DUP I 01, 2019). The situation is that there are still small parcels and small houses, and due to their size and having many owners, the previous urban plans have not yet been implemented. DUP for CQ I 01-Novo maalo 1, provides for consolidation of the parcels, which enables the formation of larger construction lots allowing for more height and a large construction area. The gained height is number of floors up to GF+6, contrary to the existing ones which are GF. This is expected to attract investors who would build, and who until now had no interest in building on small parcel that allowed only for small scale of building.

Figure 6. Satellite image of part of the city quarter Novo maalo 2 (I 02), excerpt from the Detailed Urban Plan for CQ I 02- Novo maalo 2.

Such consolidation can bring more value to the land, especially due to the current great interest in building residential buildings, but on the other hand there is a problem in negotiating with many owners. Most often, construction lots are built where there are fewer owners or part of the lot is state owned, which is sold at a very low price.

Figure 7. Satellite image of part of the city quarter Rasadnik (J 14), in the municipality Kisela Voda and excerpt from DUP for CQ J 14 -Rasadnik.

In the second case – *fragmentation*, we took the example of Rasadnik - Kisela Voda. In the current state, this part still has a large area that has not been built. The same location according to DUP for CQ J- Rasadnik, municipality Kisela Voda is planned with large number of parcels and use A2- Housing in residential buildings (DUP J 14, 2019). The surface of the former plant nursery is divided into smaller construction lots and they are planned with number of floors from GF + 6 to GF+12. The land increased its value, especially since it is easy to transform it from agricultural to construction and has a smaller number of owners.

2.2.4 Securing and even distribution of construction land for public use

The basic functions in the city - housing, production and sports and recreation, cannot exist independently. They need the so-called accompanying functions that enable their functioning. Hence, urban planning has a special role in providing space for locating them in the city, as an activity of public interest. Namely, every residential settlement must contain: Buildings for social protection, health, trade, sport and recreation, transport. In addition, it is necessary to provide adequate infrastructure and suprastructure.

The provision of these accompanying functions is crucial for choosing where to live or work, or spending one's free time. Hence the attractiveness of certain parts of the city and thus the greater value of real estate.

As an example we can point out the attractiveness of the city quarters where all the accompanying functions are provided and are easily accessible on foot. For example, the city quarter Bunjakovec has two kindergartens, a primary school, a high school, a polyclinic, a shopping centre, two supermarkets and in the immediate vicinity a green market, a library etc. Around 12.0000 inhabitants live in the city quarter Bunjakovec, and have easy access to all the necessary accompanying functions.

However, according to current detailed urban plans, DUP Bunjakovec 1 (2012) and DUP Bunjakovec 2 (2012), having in mind the gross developed area for construction, the number of inhabitants would increase up to 30.000. Within the territory of Bunjakovec, there is currently an undeveloped area only on the parcel of former factory Treska, and at that location, the urban plan prescribes A2 housing in residential buildings, where there is a possibility to house as many as 4.000 new inhabitants. This jeopardizes the equipment of the quarter and can drastically reduce its quality and attractiveness. The plan does not provide accompanying functions, except for commercial-business, as compatible use within the residential buildings themselves.

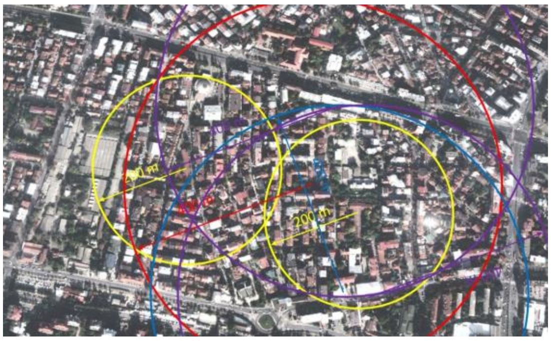

Figure 8. City quarter Bunjakovec. Access to existing functions: yellow - kindergartens 200m; red - schools 400 m; blue - emergency 400 m; violet - stores and markets 400m

From the point of view of landowners, the value of real estate increases many times over and there are conditions for great profit. But, on the other hand, the large offer of housing, and with conditions that will not meet the needs of the new residents, and will endanger the needs of the existing residents, the question is whether the real estate value shall remain high in the future.

Figure 9. DUP Bunjakovec 1 - Location factory Treska. Comparison between the ruling plan as of 2012 and the proposed plan as of 2016.

The downside is that such availability of functions attracts external users and creates congestion. But the biggest disadvantage is the provision of parking spaces. By changing the legislation in the field of urban planning and the non-compliance of urban plans with it and the situation on the ground, unfavourable conditions have been created for the provision of parking lots. Hence, the attractiveness of this area of the city has been reduced.

Furthermore, the problems coming with the additional intensive construction can be very well expected, which in turn is not followed by the provision of additional accompanying functions, but relies on the existing ones. This can affect the decline in attractiveness, and thus the value of real estate. Therefore, the recent DUP proposal provides for reserving of part of the construction area for contents of public character - kindergarten and green spaces.

2.2.5 Impact of the open green spaces on the real estate value

The cities in Macedonia, and especially Skopje, in recent years have become world leaders in pollution, which makes the need for green space a top priority. Citizens faced with the problem of enormously polluted air, see salvation in the use of green city oases and increasingly base their decision on where to live or work on whether there are green public spaces in the immediate vicinity. Providing new green spaces, as well as the rehabilitation of the existing ones, is crucial today in the preparation and adoption of new urban plans.

Figure 10. Satellite image of the city quarter Novo Lisice and DUP for CQ JI 01

The example of Central Park in New York is known worldwide, where the price of real estate is the highest in its immediate vicinity and the price decreases as it moves away from it (Trpkovska, 2017). In Skopje, for example, the settlements of Aerodrom and Novo Lisice are attractive, where the central part provides large green spaces.

This part of the city is further becoming attractive and coveted space for living, due to the equipping of the sport and recreational spaces along the river Vardar, city quarter JI 01 (DUP JI 01, 2014). Although the land is intended for open green spaces, the possibility to design sports fields opens the possibility for profit and real estate can provide greater value. This is an example of how even on land where it is not build or very little is built on, can have an increase of the real estate value.

3. CONCLUSION

The research shows that the role of urban planning should be reconsidered and the repercussions that planning solutions will have on real estate and their value must be taken into consideration when designing urban plans. Namely, the clients and developers, as well as all stakeholders involved in urban planning and the process of adopting urban plans, should take into account the relation between *urban instruments and real estate value*, which is an important economic factor, but also social and environmental one. However, when deciding or determining the methodology for assessing the value of real estate, the categories of micro and macrolocation conditions should also be incorporated or at least expanded. The current methodology does not reflect the essential impacts of urban planning.

It seems that the cooperation and integration of the two stakeholders should be further developed and expanded. The two sectors should be in constant contact and in the future an easily accessible database should be built, of course assisted by GIS technology and similar tools, which would facilitate decision-making in urban planning and development of urban settlements.

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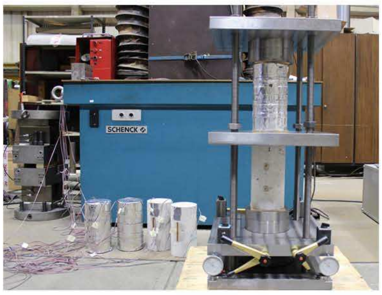

Experimental investigation of concrete creep and creep-recovery phenomenon, Structural testing laboratory KIBKON, RUB-Germany

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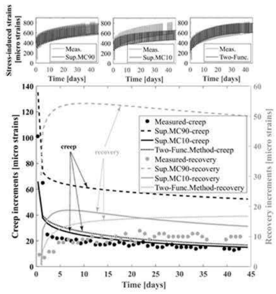

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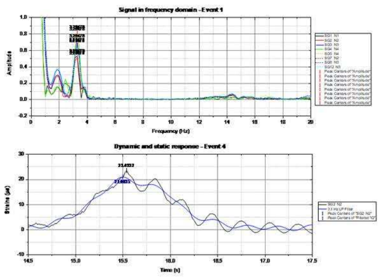
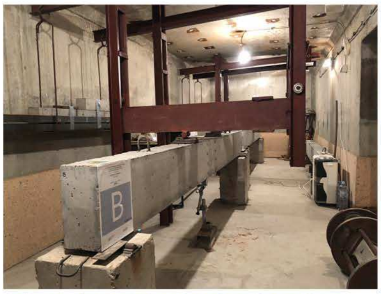


Determination of dynamic amplification factor (DAF) using Fast Fourier Transform and low-pass filtering



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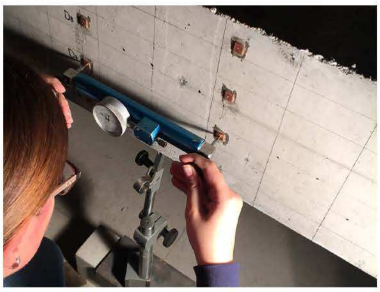

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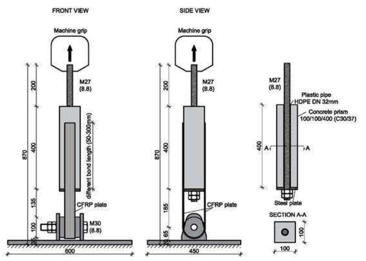

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LONG-TERM BEHAVIOR OF RC BEAMS SUBJECTED TO SUSTAINED LOAD: COMPARISON BETWEEN EXPERIMENTAL AND ANALYTICAL RESULTS ACCORDING TO EUROCODE 2

The influence of concrete creep and shrinkage on the behavior of reinforced concrete elements is most often checked through the serviceability limit states: limitation of stresses, cracking and deflections. Under sustained load, the deformations of the element gradually increase with time and may be many times greater than the initial value. If the temperature remains constant, the gradual development of strain with time is caused by the shrinkage and creep of concrete. In order to determine the influence of different intensity of long–term sustained load on the behavior of reinforced concrete elements during the time, an experimental program has been realized. Eight reinforced concrete beams and an appropriate number of test concrete samples were made and monitored in a laboratory environment with constant ambient temperature and humidity. Seven beams are loaded with different load intensity, whilst six beams are loaded with a sustained load with intensity in which cracks appear in the considered time period. No load was applied to one of the beams and only the shrinkage strains of concrete were observed. Through the experimental analysis was obtained a picture of the long–term behavior of reinforced concrete elements subjected to different intensity of sustained load, as well as its influence on the serviceability limit states. A comparison was also made with the results obtained with modern analytical models in the codes.

Keywords: creep, shrinkage, sustained load, deflections

1. INTRODUCTION

In order to be able to ensure the desired behavior of the structure or part of it in service, it is necessary to control the serviceability limit states: deflection control, crack control, limiting stresses in concrete and reinforcement, vibrations, etc. Their control is proof that in the most unfavorable combination of the service loads, the predicted values will not be exceeded, taking into account the duration of the load.

The long–term influence of the load causes a significant increase in the deflections and the crack width, which leads to a decrease in the tensile stiffness, increased stresses in the reinforcement at the place of the cracks and increased curvature of the section. All this leads to endangering the load-bearing capacity and serviceability of the structure.

During calculating the long-term behavior of structural elements there are three factors we should consider: creep, shrinkage and reduction of tensile strength in the tensile zone of the cracked cross-section due to the formation of cracks during the time and local loss of bond between the concrete and reinforcement [1].

The process of the loading of reinforced concrete elements is accompanied by the process of crack formation. As the load increases, the initially rapid changes in the crack pattern decrease and the pattern gradually stabilizes. Under sustained service load, cracks frequently form with time between the most widely spaced cracks in a cracked tensile region, thereby reducing the average crack spacing with time. In addition, cracks usually form with time in previously uncracked regions thereby increasing the extent of cracking.

Shrinkage and creep as time-dependent deformations have a great influence on the overall behavior of reinforced concrete sections, elements and structures. Many structures have suffered unintended consequences due to inaccurate determination of their influence. In statically indeterminate structural systems that are composed of elements with different deformable properties, there is a redistribution of static influences [2]. Therefore, these impacts must not be neglected and should be properly considered at the design stage. Particularly, the precise assumptions should be made in extremely tall buildings, in segmented bridges, those that have quite large spans as well as pre-stressed structures where cracks may occur in critical sections. Due to all this time spent in the design process on defining the properties of the material should be similar to the time spent on the analysis of the structure.

There is still no precise model for predicting the behavior of concrete during the time, which is primarily due to its highly elastic properties and heterogeneous structure. Most models are based on empirical expressions, so to obtain a model for the practical application it is necessary to improve it with the results of experimental tests.

2. CALCULATION OF DEFLECTIONS ACCORDING TO EUROCODE 2

Deflection control is usually determined using simple rules for limiting the span/depth ratio, which is an adequate approach for common situations.

In addition, another approach is given in Eurocode 2 [3] which involves calculating the curvature of the corresponding cross-section and integrating it along the element to obtain the deflection. Especially in cracked sections, due to the change in curvature that occurs in the area between the cracks and due to the adhesion that still exists between the concrete and the reinforcement, it is necessary to determine the mean curvature. The product of the mean curvature $k_s(t)$, obtained by applying the Simpson rule of numerical integration and the fictitious bending moment M along the element gives the deflection a in the desired cross-section.

$$a = \int_0^l k_s(t)\overline{M}(x)dx \tag{1}$$

During calculating the deflection from long-term loads, three factors should be taken into account: creep, shrinkage and reduction of tensile stress in tensile concrete as a result of increasing the number and width of cracks during the time, as well as local disturbances of the connection between concrete and reinforcement. The mean curvature is determined for the considered time at the moment t, as the sum of the initial curvature of the intersection and its increase due to time-dependent deflections. It is obtained by interpolation between the smallest value for the curve $k^I(t)$ calculated for the cross-section without cracks (state I) and the largest value for the curve $k^{II}(t)$ calculated for the cross-section with cracks (state II) [4].

$$k_s(t) = (1-\varsigma)k^I(t) + \varsigma k^{II}(t) \tag{2}$$

The effect of the tensile concrete between the cracks is included through the coefficient ς.

$$\varsigma = 1 - \beta_1 \beta_2 \left(\frac{M_{cr}}{M}\right)^2 \tag{3}$$

where the coefficient β_1 includes the degree of adhesion between the concrete and the reinforcement (β_1 = 0.5 for smooth reinforcement; β_1 = 1.0 for ribbed reinforcement); the coefficient β_2 includes the influence of the time-dependent properties of the concrete during the time (β_2 = 1.0 for short-term loads; β_2 = 0.5 for long-term and repeated loads); M_{cr} is the cracking moment; M is the considered moment [5].

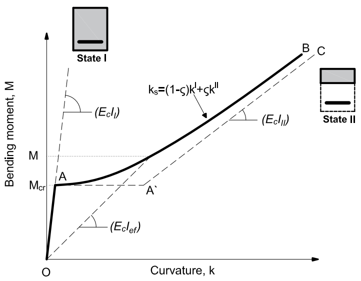


Figure 1. Moment versus curvature relationship [6]

3. EXPERIMENTAL PROGRAM

Having in mind the importance of the long-term effects of concrete structures, in the past years at the Faculty of Civil Engineering in Skopje, experimental investigations have been conducted that they include different types of concrete, as well as different load intensity and duration [7,8,9,10].

In order to determine the influence of long-term sustained loads on the behavior of reinforced concrete elements during the time, an experimental program has been realized in the laboratory of the Faculty of Civil Engineering in Skopje.

Figure 2. Test setup of RC test beams (left) and test setup for creep (right)

The experimental program consists of eight test elements-reinforced concrete beams made of concrete class C35/45, with a 15/28cm rectangular cross-section and 300cm in length. They were monitored in a laboratory environment with constant ambient temperature and humidity. In order to determine the material and time-dependent properties of the concrete, a suitable number of test specimens with different shapes and dimensions were tested. The selected dimensions of the beams allow the use of real concrete and reinforcement. The geometry, required reinforcement and the applied load given in relation to the in-service moment at mid-span (M_e=12.9kNm) are shown in Figure 3.

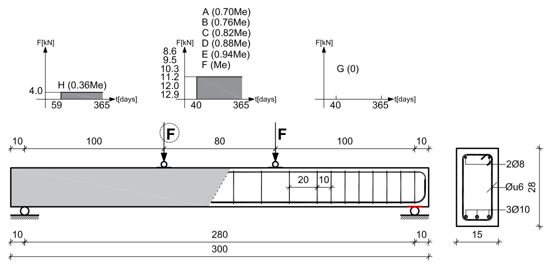


Figure 3. Geometry, load scheme of RC test beams

They are divided into eight groups: A, B, C, D, E, F, G, H. Seven beams are loaded with different load intensity, which are applied to the beam in the form of two concentrated forces. Six beams are loaded with a sustained load with intensity in which cracks appear in the considered period. One beam is not loaded in order to monitor shrinkage strains and any changes in ambient conditions during the same monitoring period.

At the age of 40 days, when the load on the reinforced concrete beams is applied, tests of the specimens were performed to determine the properties of the concrete, as follows: compressive strength, flexural and splitting tensile strength, modulus of elasticity, creep and shrinkage. The test results are shown in Table 1.

Table 1. Experimentally obtained values for the material and time-dependent properties of concrete for t=40 days and t=365 days

C35/45				
Time [days]	$f_{ck,cube}$ [MPa]	f_{ck} [MPa]	$f_{ct,f}$ [MPa]	$f_{ct,sp}$ [MPa]
t=40	48.27	40.82	6.82	3.84
t=365	/	/	/	/
Time [days]	E_c [MPa]	ε_{cs} [‰]	ε_c [‰]	φ
t=40	34100	0.204	/	/
t=365	/	0.460	0.903	1.755

4. RESULTS OF EXPERIMENTAL ANALYSIS

The influence of the long–term sustained load on the behavior and the maximum value of the deflections of the reinforced concrete elements is determined by monitoring the eight test elements over a period of 365 days. The beams are loaded on the 40th day after casting, except for beam H witch is loaded on the 59th day. They are unloaded at 365th day and monitored until the 465th day after casting.

The development of deflection during the time in the middle of the span is shown through the deflection-time diagram in Figure 4, while Figure 5 shows the force-deflection diagram. Table 2 shows the values of the deflection at the moment of loading (a_0) at the age of the concrete t = 40 days, at the moment of unloading (t=365 days) and the final value of the measured deflection (a_t) at the age of the concrete of t = 365 days and t = 465 days. The same table shows the increase in deflection because of the creep (Δa_t).

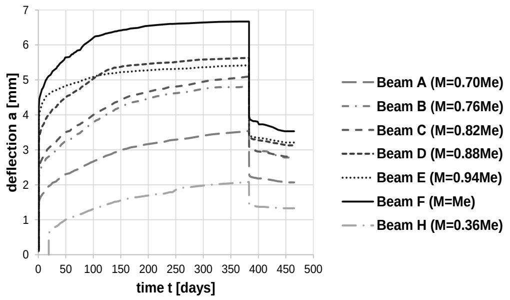


Figure 4. Deflection development during the time for all beams

It can be concluded that the increase of deflections under the influence of long-term loads due to the creep and shrinkage of concrete depends on the level of load, so it is more pronounced for elements that do not have cracks or are in the phase of crack formation, than elements that are in a phase of stabilized picture of cracks.

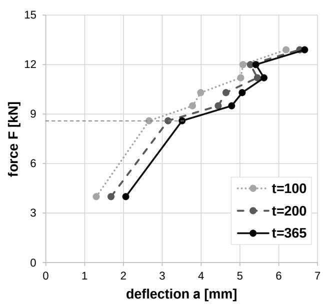


Figure 5. Diagram: force – deflection

Table 2. Measured deflection values at t=40 days, t=365 days and t=465 days

Beam	**loading** / unloading		
	a_0 (**40,59**/365) [mm]	a_t (**365**/465) [mm]	Δa_t (**365**/465) [mm]
A	**1.01**/1.17	**3.51**/2.07	**2.50**/0.90
B	**1.43**/1.62	**4.79**/2.78	**3.36**/1.16
C	**1.76**/1.91	**5.06**/2.81	**3.30**/0.90
D	**2.20**/2.09	**5.62**/3.13	**3.42**/1.04
E	**2.80**/1.86	**5.41**/3.21	**2.61**/1.35
F	**3.14**/2.60	**6.67**/3.53	**3.53**/0.93
H	**0.55**/0.61	**2.06**/1.33	**1.51**/0.72

Figure 6 shows the increase of the instantaneous deflection from the long-term effect of the load in relation to the current during the time.

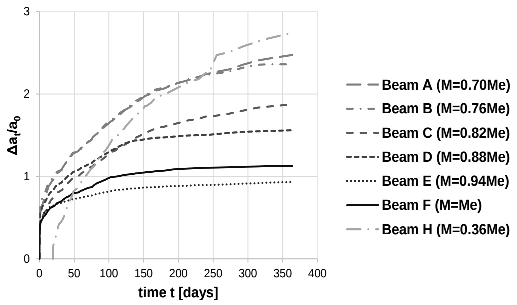


Figure 6. Deflection increase of long-term sustained load / instantaneous deflection during the time

From the comparison shown in Figure 6, it can be seen that in the whole monitoring period there is a more pronounced increase of deflections from the long-term load on the less loaded beams, despite the fact that with increasing load the total deflection is greater.

5. COMPARISON BETWEEN EXPERIMENTAL AND ANALYTICAL RESULTS

Using the method given in Eurocode 2 [3], it was made a comparison between the analytically and experimentally obtained deflections. They were analyzed in the middle of the span of the reinforced beams.

The deflections are calculated with numerical integration, by determining the curvature in multiple cross-sections along the element. Interpolation was performed between the calculated deflection assuming that the whole element is working without cracks and the calculated deflection when the cracks are fully developed, or when there is already a stabilized picture of cracks.

The analytically obtained deflections are calculated by including material and time-dependent properties for concrete class C35/45 (given in Eurocode 2 [3]), and then an improvement is made, which includes the experimentally determined material and time-dependent properties.

The cracking moment, which is used in the analytical calculation of the deflections, is determined theoretically because the total load is applied to the reinforced concrete beams at once. In the calculation where the material and time-dependent properties for concrete class C35/45 are included, it is $M_{cr,1} = 9.14$kNm, while where the experimentally obtained properties are included, it is $M_{cr,2} = 9.32$kNm.

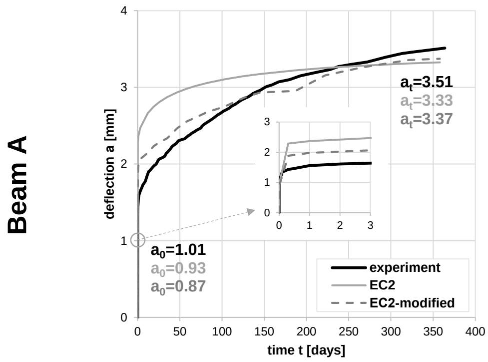
Table 3 shows the initial and final values of the deflections for all beams that are monitored, the deflections in t=40 and t=365 days. The same table shows the percentage maximum and minimum deviations of the experimental from the analytically obtained.

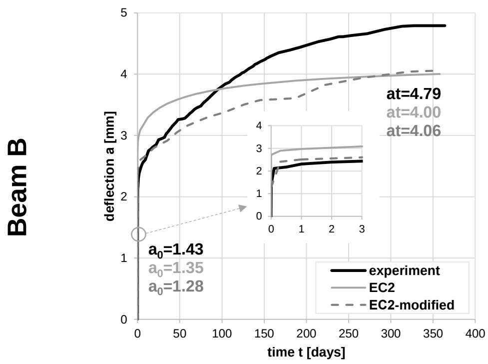
Table 3. Table showing the deflection measured experimentally and calculated analytically according to EC2

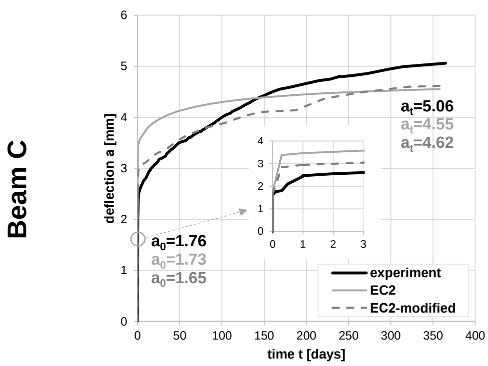
DEFLECTION [mm]						
Beam	t [days]	Exp.	EC2	%	EC2-mod.	%
A	t=40	1.01	0.93	8.6	0.87	16.09
	t=365	3.51	3.33	5.41	3.37	4.15
B	t=40	1.43	1.35	5.93	1.28	11.72
	t=365	4.79	4	19.75	4.06	17.98
C	t=40	1.76	1.73	1.73	1.65	6.67
	t=365	5.06	4.55	11.21	4.62	9.52
D	t=40	2.2	2.15	2.33	2.08	5.77
	t=365	5.62	5.2	8.08	5.16	8.91
E	t=40	2.8	2.54	10.24	2.46	13.82
	t=365	5.41	5.68	4.99	5.71	5.55
F	t=40	3.14	2.97	5.72	2.89	8.65
	t=365	6.67	6.15	8.46	6.24	6.89
H	t=40	0.55	0.4	37.5	0.4	37.5
	t=365	2.06	1.2	71.67	1.24	66.13

The following diagrams show the deflections obtained experimentally (black line), analytically with material and time-dependent properties of EC2 (gray solid line) and analytically - modified (gray dashed line).

From the comparison, it can be noticed that in all beams the values calculated according to EC2 and the values calculated in the same model, but with included material and time-dependent properties from the experiment gave acceptable predictions in relation to the experimentally measured. This is due to the close values of the cracking moment. At beam H we have a complete overlap of the values for the deflections because the cross-section is without cracks, so the cracking moment does not affect the calculation. The values calculated according to EC2 give a closer value for the instantaneous deflection of all beams, which is not the case with deflection during the time.







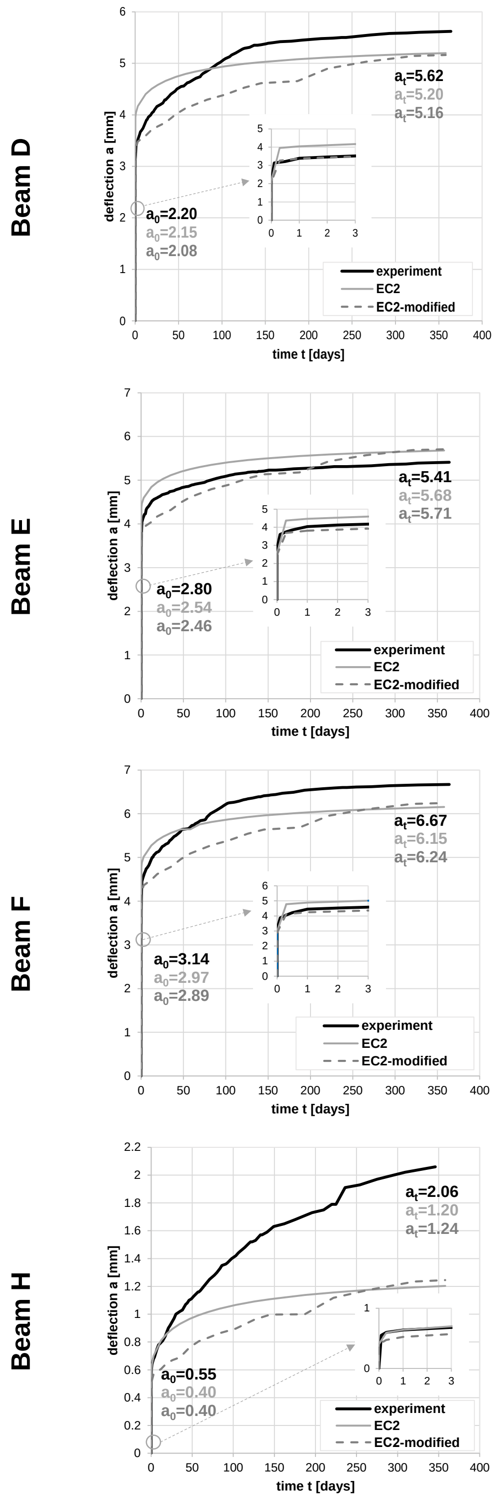


Figure 7. Comparison of analytically obtained deflection with experimentally measured ones

CONCLUSIONS

The following conclusions can be drawn from the experimental investigations of reinforced concrete elements exposed to long–term sustained loads of different intensity and the analysis carried out using analytical models:

- The intensity of sustained load has an influence on the behavior of reinforced concrete elements during the time.
- Higher total deflections are registered in the beams exposed to sustained load with higher intensity.
- A more expressive increase in time of instantaneous deformations is found in beams loaded with a lower load intensity.
- Changes resulting from the long-term effects are more pronounced in the immediate post-load period.
- The considered analytical models proposed in current standards with sufficient accuracy can predict the limit states of reinforced concrete elements exposed to long–term sustained load.

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USAGE OF SHOTCRETE IN NORTH MACEDONIA

Shotcrete is a relatively new technique used in R. N. Macedonia. With the start of the works on the infrastructural projects throughout the territory of North Macedonia, the shotcrete became one of the most important aspect of the projects. Large quantities of sprayed concrete are installed during the construction of tunnels and protection of slopes. Tunnels on the section Demir Kapija-Smokvica and the tunnel on the highway Kicevo-Ohrid are more significant facilities to which this technique was applied. However, we should not forget about the built-in quantities of shotcrete in the landslide protection on the access road to the dam Sveta Petka, as well as when repairing the landslide on the highway Gradsko-Veles. Only in the last building, at the landslide Cucka in front of the entrance of Veles, an area of about 2,000 m² is covered with sprayed concrete.

The technique of sprayed concrete in our country does not stop here. There is a chance to become an even more trendy technique with the announced projects, or projects that are already under construction. One of the many projects that are being carried out is the expressway Prilep - Gradsko that contains the construction of a tunnel and a gallery. Apart from the construction of these elements of the building, sprayed concrete is also used in excavations in rock masses, ie protection of slopes. It is planned to install 500 m³ as the primary support of the tunnel and 20 m³ for slope protection at the gallery. It should be mentioned here that additional quantities have already been applied during the penetration of the route, ≈ 10,000m².

That is a proof that the shotcrete is a trend in our country. The predicted road and railway infrastructure projects pave the way for this technique. However, not only in the infrastructure but also sprayed concrete is expected to be used in other areas, which will further increase the amount of installation. Apart from larger quantities of sprayed concrete, the technique of this process is expected to be improved in the coming years.

Keywords: shotcrete, gallery, tunnel, landslide, infrastructure

1. INTRODUCTION

Sprayed concrete or shotcrete is a technique where the concrete mixture is applied to the surface with the help of high pressure and special equipment. The method of installation can be dry or wet. This technique is flexible and has a fast way of installation, but requires good mechanization and well-trained workers. Can be used alone, but also in combination with anchors, steel mesh and fibers. It is widely used in construction in: tunnel construction; construction of hydro-technical facilities; protection of slopes from landslides; protection of construction pits; mining; reconstruction of buildings; construction of buildings with unusual geometry, etc.

Although sprayed concrete has been used in the world for more than a century in various construction projects, in our country it is a relatively new technique. Mostly, this process is used in tunnel construction and in protection of slopes from landslides. Towards the end of XXcentury, during the construction of the Katlanovo tunnel, the first quantities of sprayed concrete were applied in our country, as the primary support of the tunnel. While larger quantities of shotcrete are installed during the realization of the projects: construction of the dam Sveta Petka; construction of the section Demir Kapija - Smokvica, as well as protection of slopes on the access road to the dam Sv. Petka.

2. TECHNOLOGY OF PRODUCTION AND APPLICATION OF SHOTCRETE

The term technology of sprayed concrete application means including these three basic phases in the process:

- preparation of the mixture for sprayed concrete;

- process of installation of sprayed concrete and

- equipment for installation of sprayed concrete

All three phases are important for performing the shotcrete technique. Preparation of the mixture for sprayed concrete is done according to certain regulations by preparing several combinations of concrete mixture. The difference is in the dosage of certain amounts of each element. Choose the best and most economical, paying attention to the following characteristics: better workability, higher strength, better spraying, minimal rebound and minimal dust. Once the serial preparation of the mixture begins, it should be installed with special equipment.

The basic ingredients for preparing the mixture for sprayed concrete are: aggregate, cement, water and additives. The aggregate used for shotcrete occupies the largest volume in the whole mixture, 75%. When choosing the unit, attention should be paid to the homogeneity of the mixture, workability and mechanical properties.

Portland cement is used to make sprayed concrete without additives, but also other types of cement can be used as an alternative, depending on the conditions. And the third element, ie. water has two roles in this process. The first one, the water together with the cement, starts the hydration, and the second, the wetting of the aggregate. Therefore, the water should be clean, without harmful substances, and should not contain oils, chlorides, sulfates, sugar, salt, etc.

The group of additives used to improve the properties of concrete can include: accelerators, retarders, plasticizers, superplasticizers, aerators, antifreeze and others.

Table 1. Recipe of sprayed concrete installed in several buildings in R. North Macedonia

	Highway D. Kapija-Smokvica	Preseka Tunnel
Cement (kg/m³)	435	524
Aggregate (kg/m³)	1573	1606
Water (kg/m³)	244	220
Accelerators (kg/m³)	No info	3,14 (6%)
Superplasticizers (kg/m³)	6,96	4,19
Total volume weight (kg/m³)	2300	2350

When the sprayed concrete mixture is made, it should be installed, and this is done with special installation equipment using one of the two application procedures. Dry procedure is a method in which the dry mixture of cement and aggregate is installed through a pressure hose, and water is added to the nozzle, just before installation. While wet procedure is a method where the finished concrete mixture is transported and carried to the application rig. This method is newer than the dry one and is being developed with the evolution of NAMT - The New Austrian Tunnel Construction Method.

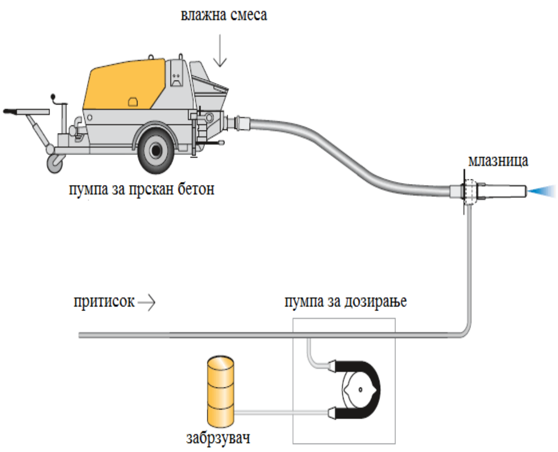

Figure 1. Demonstration of installation of shotcrete - wet procedure

The technical characteristics of the equipment used in the installation of shotcret affect the quality of the sprayed concrete and the success of the whole installation procedure. Therefore, all parts of the equipment to be used should be analyzed in order to make the right choice. The choice of equipment depends on a number of factors, such as: type of installation procedure, project specifications, availability of materials, type of transport of materials, weather conditions, etc.

Sprayed concrete equipment is a technological unit which provides the basic components of the sprayed concrete mixture, their dosage in the required weight ratios, quality mixing of the components, and controlling the mixture in to the outlet nozzle under sufficient pressure.

The basic equipment for making shotcrete are: production pump and nozzle. The pump is used for the production of concrete and through a special hose it is transported to the nozzle. The nozzle, which can be manual or mechanical, is used to apply the concrete mixture. More recently, a robot nozzle has been produced that is very easy to operate and has greater realization.

Figure 2. Robot machine for embedding sprayed concrete

3. CURRENT AND FUTURE USAGE OF SHOTCRETE IN WORLDWIDE

In the world the shotcrete technique began in the United States, with the renovation of the one museum facade. But then it started to be used in the construction of dams, bridges, tunnels, irrigation canals in the field and so on. In one century long time, a lot of money is spent on sprayed concrete, which is a sign that the need for such a technique is increasing every day. The main reason for the growth is the great need for construction of underground facilities, especially in Europe.

The massusage of sprayed concrete around the world can be seen starting from examples in neighboring countries, than in European and around the world. During the construction of the highway G.P. Blace - Pristina, the A1 highway in Serbia, as well as several tunnel solutions through Croatia, Montenegro and Bosnia and Herzegovina. These are proof of the mass useage of this technique. While Turkey has built tunnels with a length of 220 km in just 20 years, and about 300 km are in the design phase. This shows that Turkey will be one of the largest consumers of sprayed concrete.

Table 2. Largest consumers of shotcrete in the world

Europe	North America	Asia and the Pacific
Germany	USA	China
Italy	Canada	Japan
Switzerland	Mexico	Australia

In the world, shotcrete is also used for slopes' protection; protection of construction pits; mining, aviation, industrial floors, etc. However, it is also used for the construction of objects with unusual geometries, such as swimming pools, buildings with a rounded shape, etc.

As proof that it is widely used stand the numbers. According to some research, in 2019 alone, $ 4.88 billion was spent. The largest consumers are the European countries Germany, Great Britain, Russia, Turkey and others. But the other continents should not be forgotten.

Due to the global pandemic, some projects have been put on hold, but this will not have much effect on the amount of embedded concrete. Thus, in 2023, about $ 11 billion is projected to be spent on this technique

4. CURRENT AND FUTURE USAGE OF SHOTCRETE IN NORTH MACEDONIA

After the successful implementation of the above-mentioned projects, the shotcrete received positive reviews from all our experts, as well as greater application in the following facilities.

At the beginning of 2018, a landslide occurred on the regional road R1102, section Veles-Gradsko, and activities were undertaken to repair it last year in February. Landslide rehabilitation at a place called Cucka, was a combination of several methods for landslide remediation and landslide protection (Figure 1). As part of most methods was the protection of slopes with sprayed concrete in combination with several other protective measures: sprayed concrete with anchors and paving net; sprayed concrete with gabions, etc.

The embedded sprayed concrete is prepared as a dry mixture of cement, aggregate and accelerator additive, and water is added before being applied to the surface. The grains of the aggregate are d = 10-20 mm, and the humidity ranges from 3-6%. The water is clean and without harmful amounts of oils, acids, alkalis, sulfates, aggressive CO_2, organic matter, etc. and the temperature should not be less than 25 ° C. Before applying the shotcrete,the surface on which it is applied is previously cleaned and wellmoistened. The thickness of the torque is 10 cm because it is applied in two layers of 5 cm.

The drainage of the shotcreted surfaces will be done by drilling appropriate holes with Ø50 mm and length 0.5 ÷ 1.0 m. The forecasted distance between the boreholes is 6.0 × 6.0 m. The reinforcement is done by placing wire nets MA 500/560, with a wire diameter of 6 mm and a distance of the hooksfrom 10 × 10 to 15 × 15 cm. Anchors that areSN anchors' type with Ø25 mm and lengths L = 3 m and 6 m are placed in a chess pattern.

Figure 3. Slope protection, p.c. Cucka, on the regional road R1102 Veles-Gradsko

During the construction of the tailings pond no. 4 in "Sasa"mine, the technique of shotcrete is applied during the construction of the bypass tunnel; tubes and pipeline for return industrial water. In the tunnel it is used as a primary underpass, while in the construction phase of the tubes and pipeline it is used as protection of the slopes where these infrastructure facilities' routepasses.

Due to the complexity of the terrain where the route of the tubes and the pipeline passes, a large amount of material has been excavated, and as technical solutions for slope protection and erosion protection, sprayed concrete is used in combination with anchors and safety net. The built-in shotcrete is 15 cm thick and it is applied in two layers. The anchoring is with SN-anchors and IBO with Ø32mm, with lengths L = 3, 6 and 9 m, while the installed safety net is reinforced Q-mesh. The drainage of the torquered surfaces will be done by drilling appropriate holes with Ø50 mm and length 2.0 m.

When the bypass tunnelwas built, sprayed concrete was used to support it. Portland cement without additives is used to make this mixture. The grains of the aggregate are not larger than d = 16 mm, and the humidity ranges from 3-6%. The water was determined to be clean after a chemical test. The thickness of sprayed concrete ranges from 10-15 cm depending on the type of tunnel profile. It is applied in several layers, and the surface on which it is applied should be clean. The total amount of sprayed concrete installed during the construction of the tunnel is approximately 1,400m³.

SN anchors Ø20mm (RA 400/500) with a length of 2.5 meters are installed, placed in a symmetrical shape, and the Q196 reinforcement mesh is used.

Table 3.

Name of the project	Installed quantities of sprayed concrete (m³)
Landslide repair and landslide protection on R1102 section Veles-Gradsko	191.55
Tunnel T_1 and T_2, the section of the European corridor 10, Demir Kapija - Smokvica	6,368
Construction of tailings pond No. 4, at the Sasa mine	2,143

Although in recent years there are many more buildings where sprayed concrete is installed on the territory of North Macedonia, we mentioned only 3 at random choice. The examples are just a small step for the future of this technique. The next example of shotcrete usage is the Prilep-Gradsko express highway, on which construction activities are ongoing. When drilling the route of this section, it is necessary to perform excavation in solid rock masses, and as a protective measure the protection of slopes with sprayed concrete in combination with anchors and reinforcing mesh is used. During the construction of this phase, a total of about 1,000 m³ of sprayed concrete was installed.

Figure 4. Protection of slopes, on the expressway Prilep - Gradsko

This building contains other elements where the sprayed concrete will be an integral part: a tunnel with two tunnel pipes L_{right}= 159 m and L_{left} = 139.25 m, as well as the gallery near the tunnel.

For the preparation of this mixture, finely ground Portland cement without additives will be used, which will have to reach MB30. The aggregate grains will be with d_{max} = 16 mm, while the water will have to meet the prescribed requirements according to PBBA. Amounts of accelerator additives will range from 4-6% of the total amount of mixture.

With the announced capital investments in road and railway infrastructure, such as the construction of Corridor 8 and the railway to R. Bulgaria, the future of the shotcrete will be bright. It is planned to install large quantities of sprayed concrete and apply this technique. Apart from the realization of these projects, shotcrete will be used in mining, in hydro-technical facilities, as well as in possible remedial measures for protection of slopes from landslides.

The Banica tunnel, which should be part of the newly planned highway Gostivar - Kicevo, will be constructed according to the new methods for construction of tunnels, according to NAMT which is based on the construction of a flexible primary substructure of sprayed concrete..

According to the prepared technical documentation in the Banica tunnel, it is planned to install a total amount of 3,030 m³ shotcrete with different thickness depending on the classes of the tunnel construction:

- class 1: shotcrete d = 5 cm
- class 2: shotcrete d = 10 cm
- class 3: shotcrete d = 15 cm
- class 4: shotcrete d = 20 cm

As in other buildings, so in the Banica tunnel, the sprayed concrete will be used in combination with anchors, reinforcement mesh, etc. During this operation the anchors are SN-anchors Ø25mm and reinforcement mesh Q131 and Q188.

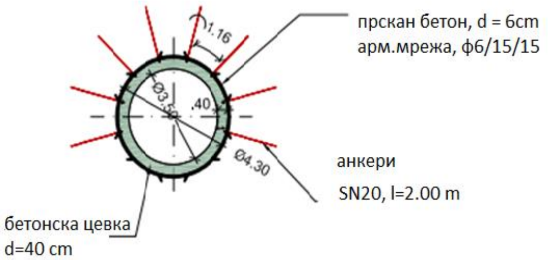

Figure 5. Profile of pipeline, dam HEC Boskov Most

Another facility waiting to be built and play its role is the Boskov Most dam. According to the bill of quantities,in this construction, it's envisaged that shotcrete is used in the construction of the pipeline as a primary substructure with $d_{\text{shotcrete}}$ = 5-6 cm (Figure 5) and other interventions, i.e a total of 6,213 m³.

Table 4.

Name of the project	Installed quantities of shotcrete (m³)
Banica Tunnel, highway Gostivar - Kicevo	3,030
Pletvar Tunnel and Gallery, expressway Prilep - Gradsko	1,020
Construction of dam HEC Boskov Most	6,213

But these are just some of the premises in which we expect to encounter shotcreteusage in the future. And of course, the development of shotcrete will not be limited to the construction of tunnels with NAMT technology, but following the rest of the world to be applied in other facilities: protection of construction pits; construction of industrial floors; construction of airfields; construction of swimming pools; reconstruction of buildings, as well as construction of buildings of irregular shape.

With greater application of shotcrete in our country, we are sure that the process of shotcreting will be improved. We will have larger professional teams for construction, more perfect equipment for installation, etc. Thus, in the future we will not need to "import" companies that will work with this technique.

Also, one of the components in shotcrete, often used in the world, but not yet in R. N. Macedonia, arefibers. The fibers, which can be from steel, synthetic, glass or plastic, replace the reinforcing mesh, which simplifies the process, facilitates the work, reduces the realization time.

The great application of sprayed concrete in various buildings will surely be a challenge for all who deal with this technique, improving the positive properties, as follows:

- greater process automation which reduces human error, time savings, increased productivity, reduced cost;
- improving the conditions when using wet procedure;
- improving the characteristics of the equipment (pump, nozzle, dispensers, etc.);
- educating all participants in the process;
- improvement of the physico-mechanical characteristics of the shotcrete, etc.

5. CONCLUSIONS

Sprayed concrete is a technique where the concrete mixture is applied to the surface with the help of high pressure and special equipment using dry or wet method of installation.

Shotcrete construction technology includes three basic components: preparation of the mixture, installation process and equipment for installation of sprayed concrete. All three elements are important for a quality performance of the shotcrete technique.

The technique of world-class interpretation began in the United States, with the renovation of the museum facade. But then continued with the construction of dams, bridges, tunnels, irrigation canals in the field, etc. If in the last few decades shotcrete has also been used to protect slopes; protection of construction pits;

mining, aviation, industrial fields, etc. However, it is also used for the construction of buildings with unusual geometries, such as swimming pools, buildings with a rounded shape, etc.

The numbers speak as a proof. According to some research, in 2019 alone, $ 4.88 billion was spent. The largest consumers are the European countries Germany, Great Britain, Russia, Turkey and others. But the other continents should not be forgotten, and in 2023 it is predicted that about 11 billion dollars will be spent on this technique.

Although in the world, shotcrete is used for more than a century in various construction projects, in our country it is a relatively new technique and mostly this process is used in tunnel construction and protection of slopes from landslides; construction of the dam St. Petka; construction of the section Demir Kapija - Smokvica, as well as protection of slopes on the access road to the dam St. Petka.

After the successful projects, the shotcrete received positive reviews from all experts, as well as massive application in the following facilities: Landslide rehabilitation and landslide protection on R1102 section Veles - Gradsko; Tunnel T_1 and T_2, the section of the European corridor 10, Demir Kapija - Smokvica and Construction of tailings No. 4, at the mine Sasa with a total amount of sprayed concrete of 8,700 m^3.

With the announced capital investments in road and railway infrastructure, such as the construction of Corridor 8 and the railway to R. Bulgaria, the future of the shotcrete will be bright. It is planned to install large quantities of sprayed concrete and application of this technique in several planned facilities has a planned quantity of over 12000 m^3.

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