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BUILDING MATERIALS AND STRUCTURES

3

Society for Materials and Structures Testing of Serbia
University of Belgrade Faculty of Civil Engineering
Association of Structural Engineers of Serbia

GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

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CONTENTS

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Aims and scope

Building Materials and Structures aims at providing an international forum for communication and dissemination of innovative research and application in the field of building materials and structures. Journal publishes papers on the characterization of building materials properties, their technologies and modeling. In the area of structural engineering Journal publishes papers dealing with new developments in application of structural mechanics principles and digital technologies for the analysis and design of structures, as well as on the application and skillful use of novel building materials and technologies.

The scope of Building Materials and Structures encompasses, but is not restricted to, the following areas: conventional and non-conventional building materials, recycled materials, smart materials such as nanomaterials and bio-inspired materials, infrastructure engineering, earthquake engineering, wind engineering, fire engineering, blast engineering, structural reliability and integrity, life cycle assessment, structural optimization, structural health monitoring, digital design methods, data-driven analysis methods, experimental methods, performance-based design, innovative construction technologies, and value engineering.

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IN MEMORIAM

Проф. др Велимир Дутина, дипл.грађ.инж.
(1956. – 2022.)

Након дуге борбе са тешком болешћу, у Београду је 21. марта у 66. години преминуо наш уважени колега др Велимир Дутина, дипл. грађ. инж., редовни професор у пензији Факултета техничких наука Универзитета у Приштини са привременим седиштем у Косовској Митровици.

Рођен је на Косову и Метохији, у Милошеву, 03. маја 1956. године, од оца Риста и мајке Маре, рођене Кулаш. Као деветогодишњак је остао без оца, прерано постао стуб породице, и до своје смрти је уз велика стручна и научна стремљења неговао породичне и људске вредности. Након завршене основне школе у родном месту и средње техничке школе у Приштини, уписао је Технички факултет - грађевински одсек у Приштини на којем је дипломирао 1981. године. Током школовања, активно се бавио спортом и играо фудбал у тиму ФК Милошево. Спортски дух је био део личности проф. Дутине, и водио га је кроз многе личне и професионалне изазове. Последипломске студије на Грађевинско-архитектонском факултету у Нишу завршио је 1995. године одбраном магистарске тезе под насловом "Оптимално преструктуирање и организовање грађевинских предузећа за производњу и прераду грађевинског материјала". Докторску дисертацију "Моделирање и избор оптималне организационе структуре грађевинских предузећа" одбранио је 2001. године на Грађевинско-архитектонском факултету Универзитета у Приштини са привременим седиштем у Косовској Митровици под менторством проф. др Живојина Прашчевића.

Свој радни век, проф. Дутина је започео 1981. године у грађевинском предузећу ГИК Рамиз Садик које је спадало у грађевинарске гиганте СФРЈ, на пословима планирања и припреме у сектору организације грађења. То га је и определило да се како у стручном, тако и у научном смислу бави организацијом и технологијом грађења и управљањем пројектима.

Своју универзитетску каријеру, започео је 1986. године као асистент приправник на грађевинском одсеку Техничког факултета у Приштини. Од 1995. до 2001. године је радио као асистент на Грађевинско-архитектонском факултету у Приштини. У звање доцента изабран је 2001. године на Грађевинско-архитектонском факултету Универзитета у Приштини, у звање ванредног професора 2006. године, а у звање редовног професора 2012. године на Факултету техничких наука. Дао је кључан допринос формирању Катедре за Менаџмент и технологију грађења коју је водио од оснивања до недавног одласка у пензију. Делио је судбину свог матичног факултета и Универзитета. Након догађаја 1999. године, седиште Увиверзитета је привремено измештено из Приштине у Косовску Митровицу. Држао је наставу и на Државном Универзитету у Новом Пазару, као и на Факултету за грађевински менаџмент у Београду. Проф. Велимир Дутина је пензионисан 2021. године, али је до последњег дана живота, иако борећи се са најтежом болешћу, наставио да ради на реализацији више пројеката.

Један од првих значајних стручних пројеката на којима је учествовао је Пројекат организације и технологије грађења за објекат "Водоводни систем Батлава у Шајковцу", Шајковац-Подујево, 1982. године, из којег се снабдева водом Приштина и шира околина. Професор др Петар Чолић, дугогодишњи директор ГИК Рамиз Садика и редовни професор на предмету Бетонски мостови у пензији Грађевинско-архитектонског факултета, касније Факултета техничких наука Универзитета у Приштини, евоцирао је у нашем недавном разговору успомене и сећања на прве радне дане тада младог инжењера Велимира Дутине и његовом првом послу на овако захтевном инжењерском објекту, који је успешно водио. Као неко ко је пратио и стручни и научни рад, развој и напредовање др Велимира Дутине, од првог приправничког радног дана па до последњег испраћаја на који је отишао као редовни професор, професор Петар Чолић управо овај пројекат издваја као почетно надахнуће које је усмерило стручни и научни пут професора Дутине. На том путу, учио је од најбољих, усавршавао се и напредовао. Професор Дутина је током реформе наставе на ФТН у Косовској Митровици допринео увођењу новог наставног модула Грађевински менаџмент на студијском програму Грађевинско инжењерство. Активно је учествовао у припреми и изради наставних планова и програма за студијски програм Грађевинско инжењерство и Архитектура, као и у њиховој успешној акредитацији, посебно при првој акредитацији факултета 2009. године. Наставу је унапредио увођењем савремених наставних садржаја. Написао је два запажена уџбеника из предмета Менаџмент грађевинских предузећа (2006) и Мерење и вредновање радова у грађевинарству (2012), који су се под овим насловима први пут појавили у нашој научној и стручној јавности. Коаутор је са проф. др Петром Чолићем књиге Развој и достигнућа ГИК "Рамиз Садику" 1951 – 1988 у издању СГИС-а.

Проф. др Велимир Дутина је аутор и коаутор великог броја научних радова публикованих у реномираним иностраним и националним часописима међу којима су Structural Concrete, Изградања, Building Materials and Structures, као и саопштења излаганих на међународним научним конференцијама и домаћим саветовањима, публикованих у зборницима. Учествовао је и руководио реализацијом више научно-истраживачких пројеката финансираних од стране Министарства за науку и технологију Републике Србије и Министарства за заштиту животне средине и просторног планирања. Аутор је великог броја студија и техничких решења из области организације и технологије грађења. Био је рецензент више универзитетских и других уџбеника. Посебан допринос дао је развоју младих кадрова. Био је ментор и члан комисија за одбрану докторских дисертација. Под менторством проф. Дутине велики број студената је урадио и одбранио дипломске и мастер радове. Тематика научних радова проф. Дутине може да се подели у неколико целина. У прву групу радова спадају публикације који се односе на критеријуме и анализу случаја. Друга група радова обухвата истраживања у области критеријума оптимизације и контроле ризика уз унапређивање организације градилишта и управљања изградњом. У трећу групу спадају радови који садрже захтеве организације градње и управљања пројектима са аспекта заштите животне средине. Четврта група радова обухвата примену неуралних мрежа и вештачке интелигенције у решавању проблема предвиђања инвестиционих вредности.

Велики је број значајних стручних пројеката на којима је учесник и руководиц био проф. Дутина. Поред поменутог Пројекта организације и технологије грађења за објекат "Водоводни систем Батлава у Шајковцу", Шајковац-Подујево, 1982. године, издвајају се пројекти организације и технологије грађења за објекте "Фабрика арматуре ФАГАР у Подујеву" (1982), "Пасарела у Приштини" (1984), "Градски фудбалски стадион у Приштини – западна трибина" (1985), "Регулација реке Приштевке", "Стамбено-пословни блок у улици Хајдар Души у Приштини" од 260 станова (1984), "Пословно трговачки центар у Обилићу" (1990), "Основна школа у Штрпцу" (1991) "Студија о концепту будућег модела организовања НДГП Рамиз Садику – Приштина", (1990), "Студија о концепту будућег модела организовања ДИГП Мируша – Клина", (1990), "Пословни објекти у Пећкој Бањи" (1992), "Универзитетско стамбено насеље у Приштини" (1995), "Стамбено пословни блок Бела Чесма у Приштини" (1997), "Студентски дом бр. 3 у К. Митровици" (2004), "Депонија чврстих отпадака Лучка река" Зубин Поток (2007), "Санација и ремедијација депоније Дренско Поље" Лепосавић (2007), "Водоснабдевање Сочанице", Лепосавић (2007), Пројекат организације и технологије грађења за надградњу објекта: "Југословенска банка", Београд (2007), Пројекат организације и технологије грађења и стручни надзор "Студентски дом бр. 5 у К. Митровици" (2007), пројекти организације и стручног надзора на објектима "Студентски дом - Тодор Милићевић у К. Митровици" (2005), "Студентски дом бр. 2 у Косовској Митровици" (2005), "Амфитеатар на Филозофском факултету у Косовској Митровици" (2008), "Фискултурна сала ОШ Стана Бачанин у Лешку" (2008), "Универзална спортска хала у Лепосавићу" (2008), "Адаптација Интерне клинике Здравственог центра Косовска Митровица" (2009), "Реконструкција ОШ Вук Караџић у Прилужју" (2009), стручни надзор над изградњом објеката "Аутобуска станица у Истоку" (1989), "Аутобуска станица у Клини" (1989), "Аутобуска станица у Малишеву" (1989), "Стамбено пословни центар у Штрпцу" (1996), "Трговачко пословни центар у Ораховцу" (1996), "Реконструкција основне школе у Подујеву" (1987), "Дом здравља у Штимљу" (1997), "Балон спортска хала ОШ Вук Караџић у Прилужју" (2010), "Дечије обданиште у Прилужју" (2010), "Дечије обданиште у Осојану" (2010), "5 стамбених објеката у насељу Брђани", Косовска Митровица (2009), "Висока пословна школа у Лепосавићу" (2010), "Стамбени објекти у Прилужју" (2010), "Дом здравља у Д. Гуштерици" (2009), 'Ученички дом у Лешку', стручни надзор над радовима реконструкције великог броја здравствених објеката на Косову и Метохији, започети послови на надзору над изградњом и реконструкцијом објеката Клиничког центра у Косовској Митровици и надзору над изградњом зграде Универзитета у Косовској Митровици (2022) и многи други. Посебно значајни реализовани пројекти су стручни надзор над радовима на изградњи регионалног водовода за водоснабдевање Косовске Митровице, Звечана и Зубиног Потока, пројектовање и стручни надзор "Санација објеката оштећених од земљотреса и изградња нових у Косовско-поморавском округу" 1.000 објеката (стамбени, пословни, школски и црквени објекти), Косовско-поморавски округ (2002-2007) и пројектовање и стручни надзор: "Изградња путних праваца у Косовско-поморавском округу", 100 км насутих путева, Косовско-поморавски округ (2003-2007). Активно је учествовао у очувању манастира, цркава и других знаменитости на Косову и Метохији.

Професор Дутина је био на руководећим функцијама на факултету као Продекан за наставу у два мандата (2005-2009), шеф Катедре за Менаџмент, члан Наставно-научног већа у ужем саставу, као члан и председник више радних тела, стручних већа и Комисија на факултету и универзитету, међу којима се издваја Стручно веће за техничко-технолошке науке на Универзитету. Био је члан Инжењерске коморе Србије као носилац лиценци 310 и 410 и члан СГИС-а.

Професор др Велимир Дутина је кроз читав свој радни век стварао и неговао чврста пријатељства са колегама, како на матичном, тако и на другим факултетима у земљи. Своје велико животно и радно искуство несебично је делио са сарадницима. Био је строг и према себи и према другима, али остаће упамћен пре свега као праведан и добронамеран човек чврстих принципа. Доброчинства је чинио као дубоки лични чин и није се тиме разметао. Тако смо, тек након његове смрти, сазнали да је професор Дутина несебично помагао и примио у своју кућу велики број избеглица. Волео је свој посао, своју породицу, супругу Цецу и ћерке Ану, Невену и Миљану, којима је био посвећен. Волео је своје Милошево и Приштину, у које за живота није успео да се врати. Са пуно љубави је осмислио и уређивао плац и викендицу у Гунцатима код Београда, где је планирао да буде место окупљања и дружења породице и пријатеља. Нажалост, само неколико месеци након пензионисања, смрт га је прекинула. Животни пут проф. др Велимира Дутине је пример како се од нејаког деветогодишњег дечака са села који остаје без оца, радом, упорношћу и јаком вољом постаје цењени редовни професор универзитета који је оставио значајан траг у стручној и научној јавности.

Била је привилегија учити од њега и радити с њим.

Доц. др Лидија Бабић, дипл.грађ.инж.

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Preliminary report

Urban mining potential in Serbia: Case study of residential building material stock

Ana Nadaždi[*1)], Aleksandra Parezanović[1)], Zorana Petojević[1)], Zorana Naunović[2)]

[1)] *University of Belgrade, Faculty of Civil Engineering, Construction Project Management Department, Bulevar kralja Aleksandra 73, 11000 Belgrade*

[2)] *University of Belgrade, Faculty of Civil Engineering, Department of Hydraulic and Environmental Engineering, Bulevar kralja Aleksandra 73, 11000 Belgrade*

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construction and demolition waste management

ABSTRACT

As governments worldwide attempt to develop sustainable waste management strategies, massive amounts of waste have been accumulating. However, developing an effective waste management strategy requires a thorough understanding of waste types and quantites. The existing efforts to identify waste flows in the built environment are unsuitable for countries with non-reliable statistics as they mostly use location-specific parameters such as data on construction, renovation, demolition activity, and generation rates from the literature. The types and quantities of materials embedded are rarely considered. This study aims to fill the identified gap by estimating the quantities of different material types embedded in Serbian residential building stock. It will do so by calculating the volume and weights of building elements and their materials using information from a detailed building stock typology. The results show that the amounts of materials embedded vary significantly from district to district, ranging from 10 in Toplička District to 96.9 million tons in Belgrade. The mineral materials are the highest contributors to the material embedded, implying that future waste management strategies should focus on them. Apart from the formulation of location-specific circular economy and waste management strategies, these results may be useful for planning energy efficiency retrofitting activities, deconstruction and reversible design strategies.

1 Introduction

The construction industry has a considerable impact on national economies, meaning that economic growth is strongly related to the construction sector's growth. In the European Union (EU), this sector alone produced 10.6% of the GDP, 6.2% of all jobs, and achieved investments of 1,402 billion euros in the 2020 [1]. Building construction and operation also accounted for 37% of all energy-related CO_2 emissions and 36% of the world's energy demand [2].Even the waste generated by these activities should not be disregarded because it accounts for more than one-third of all waste [3].

In addition to the economy, the effects on society and the environment also increase as the construction sector expands. And even despite the COVID-19 pandemic, significant investments are predicted in this sector [4]. These investments suggest that primary raw material extraction will increase and that product consumption will rise, increasing the total stock of construction materials. At the same time, the existing stock, especially the materials embedded in the residential building, is either at the end of its service life or is energy inefficient, and the stock requires significant reconstruction or retrofitting. And finally, an increase in investments also implies a higher degree of urban development, which may lead to the demolition of old and vacant buildings, especially in the inner-city areas.

For these reasons, governments have been creating funding programs, policies, and regulations aimed at more prudent energy production and use, efficient natural resource consumption, and more sustainable construction and demolition waste management. For instance, under the Green Deal initiative, the EU has gathered several strategies and action plans to achieve climate and resource neutrality by 2050 [5], such as the Renovation Wave Strategy, which aims to double the energy renovation rate of buildings by 2030 [6], a New Circular Economy Action Plan, which focuses on sustainable consumption of resources and reduction of waste [7] and the forthcoming Sustainable Built Environment Strategy [5].

However, the incorporation of these strategies within the national, regional, or even local housing and waste management policies will require the knowledge of the material stock embedded in the buildings, especially the content and quantity, as well as the predictions of building stock dynamics. To define the material stock database, i.e., the material cadastre, scientific literature distinguishes two approaches: top-down and bottom-up.

[*] Corresponding author:
E-mail address: anikolic@grf.bg.ac.rs

The top-down approach utilizes data from national statistical records to estimate the material flows, which are determined annually as the difference between the inputs and outputs of materials [8]. This approach was used in multiple disciplines in the scientific literature, either to estimate the current material flow or to predict future material flows. One of the first uses of this approach in the built environment was in Japan to estimate the present construction minerals embedded in buildings and roads [9], [10], and in the United States to predict the amount of construction and demolition waste (CDW) until 2052[11] depending on the percentage of waste generated during construction and the service life of materials. Similarly to the United States, a more recent study has included a top-down approach to estimate the amount of CDW from buildings and civil works in India [12]. Although predominantly used for estimating material flows on a national level, this approach mainly focuses on stock additions, thus failing to provide important information such as the spatial distribution, quantities, types, and service life of embedded materials. Without this knowledge, it is impossible to create an effective renovation, demolition, or CDW management strategy that specifically targets buildings of a certain age or structure.

To overcome this, scientists recommend a bottom-up approach or a combination of the two. The idea behind the bottom-up approach is to divide the material stock into different structures, calculate their physical characteristics and use material intensity coefficients or ratios [8] to calculate the amount of the material embedded, i.e. the material intensity. Experts estimate material intensity coefficients [13–15] or calculate them in numerous case studies around the world. When calculated, these coefficients are derived from structures geometries, in most cases, buildings. The physical features of the structures may be estimated in several ways: 1) modeling of different parameters to determine the average size of the floor area of the stock, such as population and housing lifestyle [16] or per capita floor area, local GDP and lifetime of dwellings [17], 2) investigating municipal records and building plans [18], [19], and 3) spatial analysis via Geographic Information System (GIS) [14], [20]–[22]. The material intensity coefficient data are available for buildings in Esch-sur-Alzette, Luxembourg [21], Rio de Janeiro, Brazil [23], Vienna, Austria [20], [24], and Padua, Italy [14] at the city scale, and Germany[18], [25], Sweden [19] and Luxembourg [22] at the national scale.

Except for Kleemann et al. (2016), who combined existing and new building plans and literature, most of these coefficients were calculated from selected case study buildings from different construction periods [18], [19]. Although material intensity coefficients for a particular building offer a high degree of accuracy, they cannot be used as representatives of the entire set of buildings built in that period or at a specific location. For instance, Gontia et al. (2018) investigated 4000 real estate ads and 1000 building plans from 30 municipalities (out of 290) to establish 12 typical single-family and 34 multi-family house buildings.

On the other hand, the studies that calculate material intensity coefficients from statistical records may be used when greater accuracy is required. Still, their application is limited to countries with reliable statistics. In addition, the findings in these studies served to emphasize how sensitive these coefficients are to location-specific factors such as environmental conditions, architectural characteristics, construction techniques and materials applied, and rate of economic growth, highlighting the need for additional case study research on this subject.

To overcome these shortcomings, this study will build on previous knowledge, but instead of using municipal records or expert knowledge, it will use typical buildings (building typologies) at the national scale to calculate intensity coefficients and quantities of different materials embedded in these buildings. The calculation of material quantities may be further used in dynamic residential building stock modeling and in the estimation of more precise amounts of waste that may be generated during the renovation or demolition activity of these buildings. This can, in some cases, affect the decision whether to refurbish or demolish [26]. In addition, the study will focus on the calculation of embedded material quantities at the district level instead of the national level as CDW is often managed regionally. In this way, these regional material intensity coefficients will provide a robust base for further modeling of the residential building stock and CDW flows and different sustainability assessments of CDW treatment options.

2 Methodology

The overall approach of the suggested methodology for the estimation of residential building material stock and material intensity coefficients is based on inventory analysis of typical buildings within a building stock. Typical buildings are representatives of buildings classified into cohorts that were built using construction techniques and materials from the same period and which share similar architectural features. They are usually established on a national scale, as are the ones for residential buildings developed for 21 European countries within the European Project Tabula [27]. If these typologies do not exist within one country, when developed, they should at the very least comprise the layouts and cross-sections of typical buildings and details of building construction and applied materials.

The inventory analysis that is conducted on typical buildings follows the approach and the methodology presented in the thesis by Nadaždi (2022). These include several initial steps performed for each building type to create a unique database: generation of a typical building element list (walls, columns, openings, stairs, etc.), identification of the elements' dimensions (width, height, and length), location within the building (basement, ground floor, 1st floor, attic, etc.), and quantity, identification, and classification of material types. These materials were grouped into four categories: 1) minerals, 2) non-minerals, and 3) metals, to facilitate easier calculation and representation of the material embedded in the residential building stock and for comparison with the existing research. While the mineral category included materials such as concrete, brick, blocks, tiles, plaster, glass, gypsum, etc., non-minerals included plastic, polystyrene, textile, and wood. Although metals can also be considered non-minerals, they were separated because of the substantial differences in how they are treated at the end of their service lives.

The next step of the suggested methodology was to calculate the area, volume, and mass of each individual component of a typical building. Two methods were used to compute the element's area: directly measuring from drawings using a built-in CAD function or multiplying two dimensions and deducting openings where necessary (mostly in cases of walls and slabs). The area and third dimension are multiplied to determine the volume of construction elements, while the volume and material density are multiplied to determine the elements' mass. Densities for most materials are taken from the MASEA online database (Fraunhofer Institute for Building Physics in Holzkirchen et al.

n.d.); those that were not found there were found in textbooks or technical data sheets.

Following this, the total mass of each material category contained in a single typical building, and the accompanying material intensity coefficients were determined. The first was determined by multiplying the number of elements and their masses, then aggregating these values based on material types. Dividing these aggregated masses yielded the material intensity coefficients for a typical building.

While the computation of material category masses for typical buildings was based on building typologies, estimating material categories embedded in buildings per district needed statistical records on the number of buildings per building type and district. However, national statistics frequently do not keep track of the number of buildings constructed in specific districts during a particular construction period but rather the number of dwellings in them. In these instances, it was necessary to follow the assumption that a proportion of buildings per district follows a distribution of dwellings per district, especially for multi-family house buildings. Another assumption that was made was related to the classification of single-family house buildings. While typologies distinguish between two different single-family house building types, free-standing and in a row, statistical data only distinguishes between buildings with one or two dwellings. In these instances, it was presumed that all single-family house buildings are free-standing because the percentage of single-family house structures in a row is insignificant [29].

Additionally, it should be emphasized that this study's focus on material types is limited to key structural and non-structural components and leaves out a number of building components for which it was challenging to gather information from building typologies. These include foundations, shades, window sills, lintels, the mortar between blocks and bricks, chimneys, installation works, and fixtures, etc. This limitation could result in an underestimation of the composition results, especially regarding the quantity of particular constituents like non-minerals and metals.

3 Results and Discussion

In this study, the proposed methodology was used to estimate the quantity and composition of residential building stock materials in Serbia in order to investigate the potential contribution to the circular economy value chain when these materials become waste. With a population of 6.9 million and an area of 88499 km^2, Serbia is divided into 30 districts (five of which belong to the Kosovo and Metohija region) [30]. In its transition to the EU, Serbia has adopted several environmentally-oriented strategies and initiatives, especially towards a circular economy, waste management, and energy efficiency renovation. However, a rate of only 0.3% of GDP investment in environmental protection [31] compared to the EU average rate of 1.8-2.0% of GDP during the previous fifteen years [32] suggests that Serbia needs these carefully planned initiatives.

In addition to this, Serbia´s residential building stock is very old and energy inefficient. Since residential buildings built before 1980 account for around 70% of the whole building stock in Serbia that was built before 2011 [29], it is anticipated that renovation and demolition operations will rise in the future. This indicates that certain residential buildings, especially those constructed after World War II, are rapidly approaching the point at which their demolition is likely. This is further supported in the Strategy on National Housing for the period 2022—2032, which sets the renovation objective of up to 30% of the buildings whose amortization period expires in 2032 [33].

Serbian residential building stock is represented in the National Typology made by [29] within the Tabula project. It is characterized by 26 building types grouped under seven cohorts, i.e., periods of construction (from A to G) and building types (from 1 to 6). When it comes to the period of construction, they included all residential buildings built before 2011, while building types were divided into single-family (free-standing (1) and in a row(2)) and multi-family house buildings (free-standing buildings (3), lamella (4), in a row (5), and high-rise buildings (6)) [29].

Considering the fact that most of the renovation or demolition activities in the coming years will be conducted on residential buildings aged above 40 years, this study will cover only buildings built between 1946 and 1990. To that extent, Table 1—

Table 3 summarizes the total amounts of each material category (mineral, non-mineral and metal) per different types of buildings.

Table 1. Total mass of material categories in one typical single-family house building built from 1946—1990 (in tonnes)

	1946—1960		1961—1970		1971—1980		1981—1990	
	C1	C2	D1	D2	E1	E2	F1	F2
Minerals	200.93	539.40	452.86	209.13	489.55	378.41	321.43	356.20
Non-minerals	193.15	515.31	438.99	196.33	484.72	365.83	316.99	351.09
Metals	5.13	23.67	10.83	10.71	4.83	12.58	4.43	5.11

Table 2. Total mass of material categories in one typical multi-family house building built from 1946—1970 (in tonnes)

	1946—1960				1961—1970			
	C3	C4	C5	C6	D3	D4	D5	D6
Minerals	1,218.40	1,230.36	1,517.94	4,905.32	2,155.31	1,403.31	1,892.91	5,594.62
Non-minerals	26.03	31.40	30.61	373.31	68.24	91.08	85.68	552.63
Metals	28.32	1.53	23.28	158.85	139.45	116.70	10.12	605.46

Table 3. Total mass of material categories in one typical multi-family house building built from 1971—1990 (in tonnes)

	1971—1980				1981—1990			
	E3	E4	E5	E6	F3	F4	F5	F6
Minerals	1,303.71	2,567.78	1,658.12	5,514.47	3,816.81	2,475.57	2,934.92	6,272.39
Non-minerals	476.68	170.45	71.963	1,008.65	89.96	63.81	48.88	444.90
Metals	52.95	73.90	155.43	178.46	361.27	144.88	122.29	674.96

The tables demonstrate that the quantity of all materials in single-family and multi-family house building types increased with time and peaked after the 1970s. The majority of all building types were constructed using mineral materials. When individual building types in all periods are compared, it can be seen that multi-family house buildings consume much more material. For instance, the mass of single-family house buildings ranged from 200.9 to 539.4 tonnes. Free-standing single-family house buildings, which form the majority of residential building stock, have 366.19 tonnes of material on average.

On the other hand, with an average of 6.6 thousand t, high-rise buildings were significant contributors to multi-family house buildings. Except for the final period, other multi-family house building types averaged around 1.8 thousand tonnes of material. However, these values will significantly change for the benefit of single-family house buildings when quantities of material are multiplied by the number of buildings per district.

The other important parameters that were calculated are the material intensity coefficients per building type, which are expressed in tonnes per m^2 of the buildings' gross area and presented in Table 4. In the mineral material category, these coefficients decreased over time, ranging from 2.54 tonnes per m^2 (1946—1960) to 1.38 tones per m^2 (1971—1980) for single-family house buildings, indicating that the population opted for larger houses. Similar patterns can be seen in multi-family house buildings. Depending on the building types, the material intensity coefficients for minerals ranged from 1.81 (1946—1960) to 0.71 (1961—1970) tonnes per m^2.

Table 4. Average material intensity coefficients per building type (expressed in tonnes per m^2)

Building type		Mineral	Non-mineral	Metals
Single-family house buildings				
1946—1960	C1	2.54	0.10	0.001
	C2	2.00	0.09	0.0003
1961—1970	D1	2.08	0.07	0.02
	D2	1.78	0.12	0.00
1971—1980	E1	2.15	0.02	0.02
	E2	1.38	0.05	0.02
1981—1990	F1	1.88	0.03	0.02
	F2	1.48	0.02	0.02
Multi-family house buildings				
1946—1960	C3	1.81	0.04	0.04
	C4	1.28	0.03	0.002
	C5	1.27	0.03	0.02
	C6	1.20	0.09	0.04
1961—1970	D3	1.19	0.04	0.08
	D4	0.81	0.05	0.07
	D5	1.11	0.05	0.01
	D6	1.15	0.11	0.12
1971—1980	E3	1.07	0.39	0.04
	E4	0.71	0.05	0.02
	E5	1.16	0.05	0.11
	E6	0.74	0.14	0.02
1981—1990	F3	1.37	0.03	0.13
	F4	1.31	0.03	0.08
	F5	1.69	0.03	0.07
	F6	0.97	0.07	0.10

Both non-mineral and metal numbers have negligible values in contrast to mineral material categories, but when multiplied by the number of buildings within a district, they may constitute a valuable source of secondary raw materials. Figure 1 shows the number of single and multi-family house buildings per district in Serbia. These numbers were based on the Census 2011 data on dwellings per building type and period of construction, provided by districts [34], and the National Typology data on the number of buildings per building type, provided for Serbia in total [29]. In other words, the National Typology's total number of buildings per building type was multiplied by the share of building types in each district to get the number of building types in each district. As mentioned before, the number of single-family house buildings, both in total values and values for districts, is significantly higher than the number of multi-family house buildings. This implies that all the renovation and/or demolition strategies that local or regional governments may adopt in forthcoming years should be directed to their owners. And in an effort to achieve a more sustainable built environment, their owners may be one of the key game-changers.

In addition, Figure 2 shows the regional distribution of three major material categories embedded in the entire residential building stock constructed between 1946 and 1990. The figure shows that the quantity of materials varies with the degree of economic development in a district. In all three material categories, the Belgrade district, the capital of Serbia and the largest city, has the highest quantities of materials embedded (89.6, 4.1, and 3.2 million tonnes for mineral, non-mineral, and metal material categories, respectively). It is followed by the Južnobačka, Mačvanska, and Nišavska districts, but with far lesser contributions (33 million tonne on average for the mineral material category). These are the districts that, in the process of further urbanisation, may grasp and exploit the full potential of these materials.

On the other hand, the lowest quantities of these materials are found in the Toplička, Pirotska, and Zaječarska districts. The mineral component of the material embedded in their residential building stock varied from 9.6 to 12.2 million tonnes. Combined with a lower degree of economic development and a higher degree of internal migration from these regions, these values imply that the local governments should carefully consider the viability of their waste management and circular economy strategies.

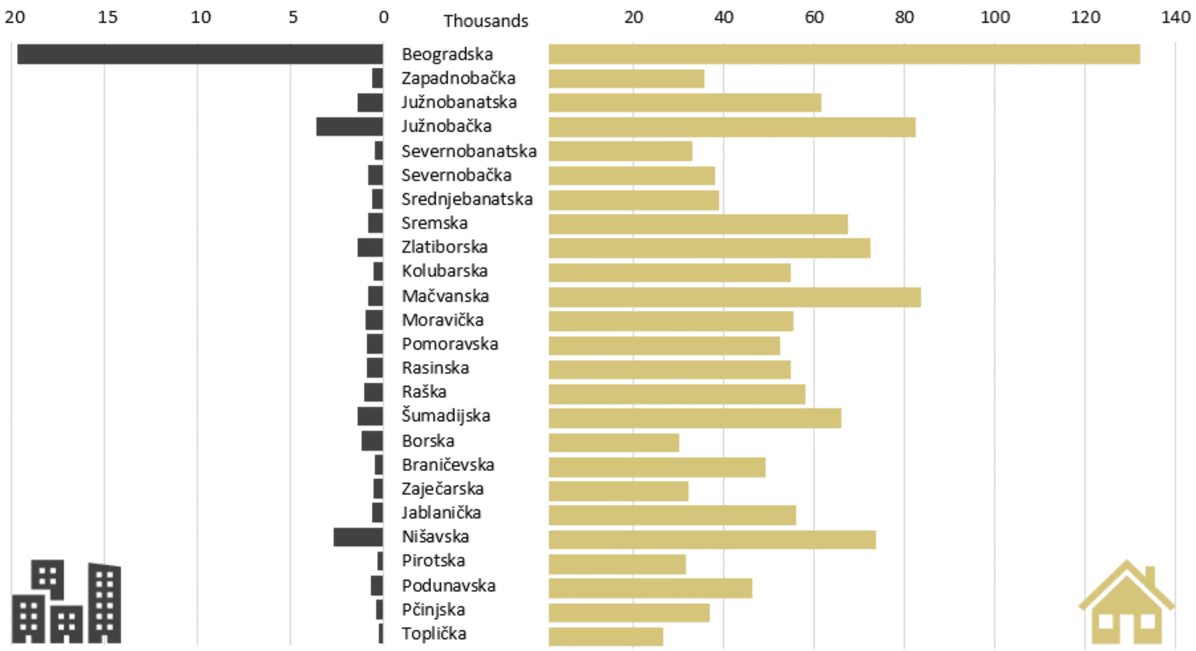

Figure 1. Number of buildings per district: multi-family house buildings (left) and single-family house buildings (right)

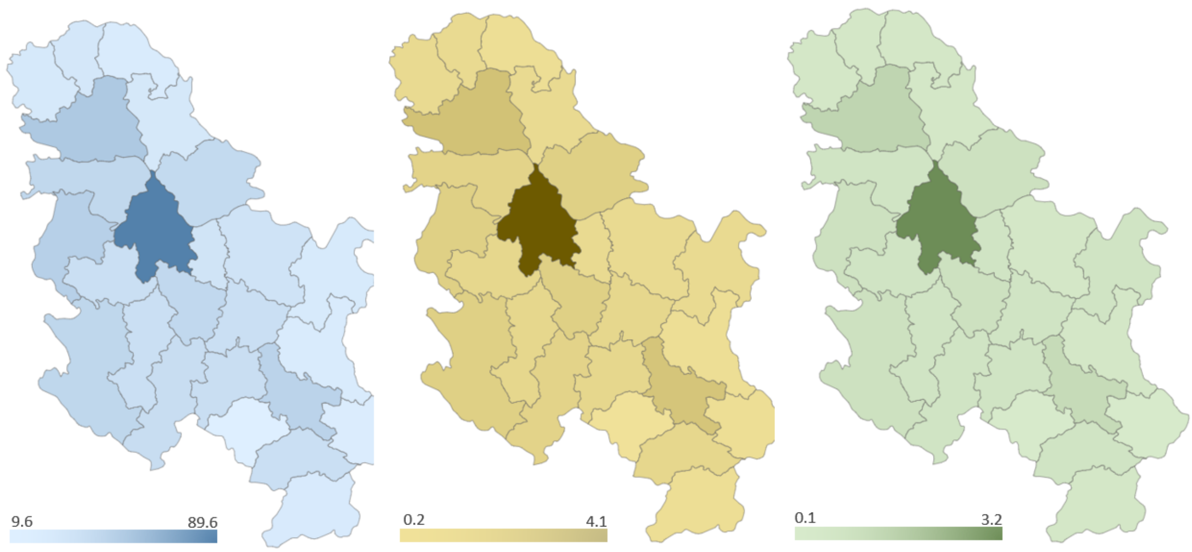

Figure 2. Quantity of material categories per district: minerals (left), non-minerals (centre), metals (right) (in million tonnes)

** Note: Kosovo and Metohija's districts were not depicted on the map since there was no information on the quantity of buildings and building typologies*

4 Conclusion

In the recent decade, academia and practitioners have been trying to develop efficient and viable waste management and circular economy strategies. To that extent, many studies have been conducted to tackle two challenges: to estimate the amount of material within the economy or assess the sustainability of its treatment options before or when these materials become waste. This study focused on the first challenge and the calculation of the construction material embedded in residential buildings. To overcome this challenge, a methodology for the estimation of material content and quantity is proposed. In addition, to facilitate their use the formulation of regional waste management and circular economy strategies, the materials are grouped into three categories: mineral, non-mineral, and metal. The methodology included a bottom-up inventory analysis, which used the information from the National Typology, i.e., the location and geometry of building elements and the material type from which they are made, and calculated their volumes and weights.

These results were then aggregated, multiplied by the quantity of buildings, and regionally distributed for the entire residential building stock in Serbia constructed between 1946 and 1990. The result showed that the largest number of buildings belongs to single-family free-standing house buildings and that the mineral material category is the highest contributor to the material stock. This suggests that any efficient renovation or demolition strategy should target these buildings and materials. Apart from the formulation of waste and energy efficiency-related strategies, the results also provide a robust base for further modeling of or validation of material intensity coefficients obtained in other ways or in different case studies. These results may also be used for building stock modeling and waste estimations from future renovation and demolition activities.

Future research might include a deeper analysis of mineral material composition to estimate the amount and the treatment potential of reusable components to support the greater implementation of circular economy principles in the built environment.

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Preliminary report

Testing bond behaviour of an innovative triangular strand: experimental setup challenges and preliminary results

Žarko Lazić*[1)], Snezana Marinković[2)], Veljko Koković[2)], Gordana Broćeta[1)], Marina Latinović[1)]

[1)] *University of Banja Luka, Faculty of Architecture, Civil Engineering and Geodesy, Vojvode Stepe Stepanovića 77/3, 78000 Banja Luka*
[2)] *University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, 11000 Beograd*

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ABSTRACT

The use of the seven-wire strand in pretension structures so far has shown that such strands have a relatively long development length. An attempt to reduce the development length is reflected through the idea of a different strand geometry that has a more favorable shape. This new shape should have a greater inclination of the outer wires, increasing the resistance that occurs when moving through the concrete. The idea is primarily about the triangular distribution of wires in the cross-section of the strand. This paper reviews tests related to the use of innovative, triangular, and ten-wire strands. The tests exclusively relate to the strands formed by the use of steel wires.

1 Introduction

In the beginnings of prestressed concrete, a smooth prestressed wire made of hardened, high-grade steel was first used. The quality of the material was of the utmost importance due to the possibility of applying greater forces and a lower relaxation loss. Over time, it has been shown that smooth prestressed wire, in the case of force transfer by direct bond, has certain shortcomings in terms of bonding only through adhesion and friction, which is disrupted during the first slips. To improve these shortcomings of the wire, a solution was reached in the form of a strand woven from wires of smaller diameters. The improvement is reflected in the capacity of the bond, where it can be seen that when the strand slips, the bond resistance remains constant or increases. This desired behavior of the strand is attributed to the mechanical locking, i.e. wedging, which does not exist in the case of smooth wires. Thus, today, the strand for prestressing means the conventional 7-wire (7-w) strand.

In the case of pre-tensioned beams (force transfer by direct bond), the most important connection is the bond between the strand and the concrete. Thus, a difference in the bond was observed in the zones of the beam ends and the central zones. End-zones of the girder are called the anchorage zones into the concrete structure (transfer length zones). The development length l_d of the prestressing force is the superposition of the transfer length l_t and the flexural bond length l_{fb}. The bond of the transfer force from the strand to the concrete during release is called the "prestress transfer bond." During the bending of beam elements, when cracks appear, the bond between strands and concrete plays an important role in the subsequent behavior of the elements. The bond that is activated as a result of bending, and which has a significant role in the continuity of the prestressing force transfer, is called the bond due to bending or "flexural bond."

Long-term experimental research [1, 2, 3, 4, 5, 6, 7] has shown that the transfer length and the development length of the prestressing force are influenced by several important parameters. If the diameter of the strand increases, the value of l_t and l_d also increases. The characteristic 28-day strength of concrete has little effect on l_t, while its increase results in a reduction of l_{fb}, and thus a reduction of l_d. However, with higher initial concrete strength at the point of releasing strands, there is a reduction in l_t. An increase in prestressing forces, after losses, results in a higher value of l_t, but also in a lower value of l_{fb}. The technological process of releasing the strands also plays a significant role. Gradual and controlled strand release has up to 30% shorter development length than sudden strand release (partial mechanical or temperature cutting) [8,9]. The surface conditions of the strand have a significant influence on the bond characteristics, such as, for instance, mild surface corrosion, which causes lower l_t values.

The transfer length l_t, according to the observations so far and as a function of the previously mentioned parameters, ranges between approximately 500 and 1200 mm. This means that by applying a 7-w strand, the full input of the prestressing force is moved from the shear span. The motivation for the use of a new geometric shape is to reduce l_t and ensure the contribution of prestressing force to the shear capacity of the girder. Therefore, replacing the standard 7-w strands with a new geometric shape of the strand may lead to an improvement of the bond connection as well as an increase in the shear capacity. Previous assumptions are the main goal of the scientific contribution

* Corresponding author:
E-mail address: zarko.lazic@aggf.unibl.org

within this research and, as such, they must be experimentally approved or disapproved.

For many years, there have been no attempts to improve the bond of the 7-w strand from the view of the geometric change of the cross-section, and thus the wavelength (360° stroke). In the 1960s, the idea of a sharper polygonal cross-section of prestressing strands first emerged. Polygonal shape refers to the regular and linear arrangement of the outer wires arranged around the leading, central wire. The proposal primarily concerned the triangular distribution of wires in the cross-section of the strand. This cross-sectional structure allows a greater inclination of the outer wires, improving the resistance that occurs when moving through a concrete tunnel. To ensure the compactness of the windings when constructing triangular strands with linear contact, which are wound from wires of equal diameter during one technological operation, it is necessary to provide two basic conditions:

- The wires of any concentric winding (twisting) must touch each other and the wires of the adjacent winding;
- The wires must touch the entire length of the strand with the same winding step for all windings (subject to the previous condition).

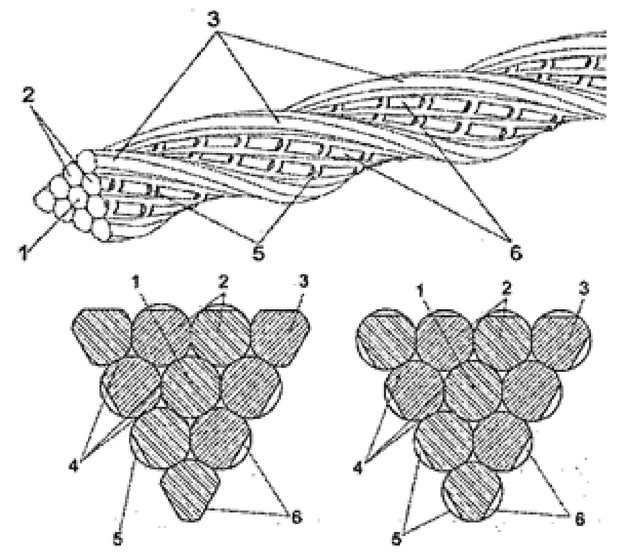

Figure 1. Schematic of innovative, triangular, 10-wire strand [10]

At the time this idea was proposed, it was not technologically possible to produce such a strand with sufficient precision. Only at the beginning of the 21st century, based on the previously proposed solution, patent US2012 / 0240548 A1 was recognized for a geometrically innovative, 10-wire (10-w), and triangular strand (Figure 1). Soon after that, a prototype smooth strand of triangular cross-section, which is the subject of this research, was produced.

It is expected that application of this type of strand will improve the bond behaviour in pre-tensioned structures in which the transfer of force takes place exclusively through the direct bond. The assumption is that the new triangular strand should have a better bond connection due to its geometric shape. The triangular spiral forms a wide periodic profile (360°) continuously. This allows extremely long concrete ribs to be formed, and thus the stress is distributed efficiently over the entire volume. The contact area on those hills has smaller ribs formed in between wires perpendicular to the concrete hills. On the other hand, the surface of the 7-w strand is similar to a circular cross-section, but it has small grooves in which concrete ribs are formed. These ribs have a limited resistance caused by the movement of the strand during higher loads, and the conventional 7-w strand has relatively large end slip.

The first model of the new triangular strand is shown in Figure 2.

Figure 2. Ribbed innovative, triangular, 10-wire strand [10]

Improving the bond between the strand and concrete should reduce the slippage and transfer length, as shown in Figure 3. This is directly related to the higher shear capacity due to the greater contribution of the applied prestressing force. However, the set assumptions need to be experimentally verified, and on the basis of the test results, a model of the bond behavior of a triangular strand should be proposed.

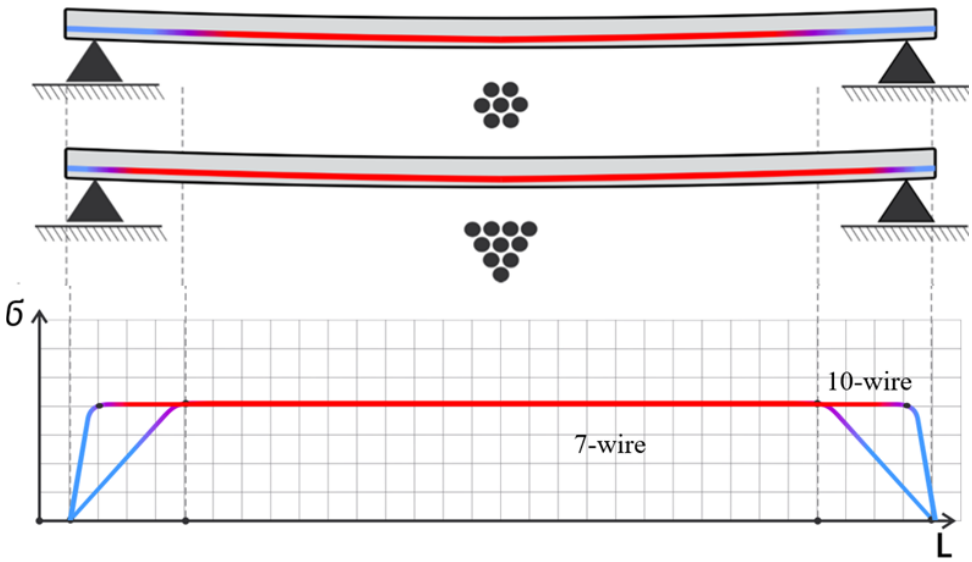

Figure 3. Schematic presentation of the expected contribution of the triangular strand compared to the standard one

2 Experimental program

The testing program is divided into several phases, including pull-out tests of untensioned strands, push-in and pull-out tests of pretensioned strands [11], transfer length tests [12], and shear beam tests [13]. Pull-out tests of untensioned strands are presented in this work.

2.1 Materials

The strands used in the experimental program are shown in Figure 4:

– 7-wire strand, configuration 1 (d = 3.2 mm) + 6 (d = 3.05 mm), with a tensile strength of 1860 MPa;

– 10-wire strand, configuration 1 (d = 2.6 mm) + 6 (d = 2.6 mm) + 3 (d = 2.6 mm) with tensile strength of 1770 MPa;

Figure 4. Geometry of tested strands: (left) standard, seven-wire; (right) ten-wire

Self-compacting concrete (SCC) of class C40/50 was made with the mixture proportion given in Table 1. The same concrete mixture was applied in all tests. Casting of concrete samples was performed in the laboratory of IGM "KABAS" in Teslić, where all other tests will be performed. Based on properly stored material and data on the surface moisture of each of the aggregate fractions, an identical mix for concrete samples was applied in the laboratory in Teslić.

Table 1. Concrete mixture proportion

Class: C40/50

The origin of the aggregate	**Aggregate fractions (mm)**	**Percentage participation (%)**	**Dosing for 1 m^3 of concrete**
Stone pit „Dobrnja" Banja Luka	0-2	38	629 kg
Stone pit „Dobrnja" Banja Luka	0-4	22	367 kg
Stone pit „Dobrnja" Banja Luka	4-8	12	196 kg
Stone pit „Dobrnja" Banja Luka	8-16	28	463 kg
		Total:	1655 kg
Cement	CEM II/B-M(S-LL) 42.5 N		420 kg
Water	City water supply		165 L
W/C	0.393	Residual air	1.70 %
Chemical additive	HRWRA	0.71 %	4.05 L
Mineral additive	Fillers		150 kg
Consistency	SF2		660-750 mm

Testing of concrete component materials and characteristics of fresh concrete SCC C40/50 was performed in the laboratory of the concrete factory "BINIS" in Banja Luka.

The characteristics of fresh concrete that are tested are shown in Table 2. Results of the slump-flow test, „V“ funnel test, and „L“ box test are shown with obtained values and adequate class in the standard (EN 206, reference). After casting, the concrete cube specimens (15x15 cm) were protected with a plastic sheet to prevent the premature loss of moisture. Specimens were stored in the Laboratory for 24h. After that, they were demolded and then placed in the water to be cured by the time of examination.

The measured compressive strengths of SCC C40 / 50 concrete are given in Table 3.

2.2 Pull-out test of untensioned strands

Pull-out tests of untensioned strands were performed on large concrete blocks with a size of 611x450x450 mm (length, width, and height). Each block had 6 strands of which 3 were 7-w and 3 were 10-w strands. Two different surface conditions of strands were tested: AR-as received and W-weathered (Figure 5). Strands were debonded on both sides of the concrete block to a length of approximately 5 cm. Specimens were demolded and mounted on previously ready steel frames where tests will be conducted 24 hours following casting. Pull-out tests were accessed 3 and 7 days from the day of pouring the blocks: the pull-out force was applied on one side of the specimen (active side) and gradually increased over a period of 120-180 seconds until the bond fracture appeared. When the force/slip line reaches the horizontal position parallel to the X-axis, the bond fracture appears, indicating that there is no longer any resistance to the slipping.

The keys to the test were:

a) Measuring points:
  - Values of pull-out force;
  - Slip of the strand on both sides (active and passive);
  - Rotation of the strand on the passive side.

b) Test method:
  - Applying pull-out force on the strand constantly for a period of 120-180 seconds;
  - Equipment used for applying pull-out force was a hydraulic hollow jack with a capacity of 40 tons;
  - Hydraulic jack was placed on top of the concrete blocks.

c) Measuring equipment:
  - For force, oil pressure sensor capacity 500 bar;
  - For displacement, 2 x LVDT sensors on both sides of the concrete block (active and passive ends of the strand);
  - For rotation of the strand rotary encoder was placed on the passive end.

Table 2. Results of SCC C40/50 fresh concrete tests

Test	Class	Values accor. to class	Obtained value
Slump-flow test	SF2	660-750 mm	**710 mm**
„V“ funnel	VF2	9-25 s	**10 s**
„L“ box	PL2	≥ 0,80 with 3 rebars	**0,83**

Table 3. SCC C40/50 compressive strengths

Concrete	Samples	Dimensions (mm)	2 days	7 days	14 days	28 days
			Stress (MPa)	Stress (MPa)	Stress (MPa)	Stress (MPa)
C40/50	1	150x150x150	36.34	52.56	61.77	66.52
	2	150x150x150	35.98	51.80	62.01	65.82
	3	150x150x150	35.58	52.20	61.56	67.14

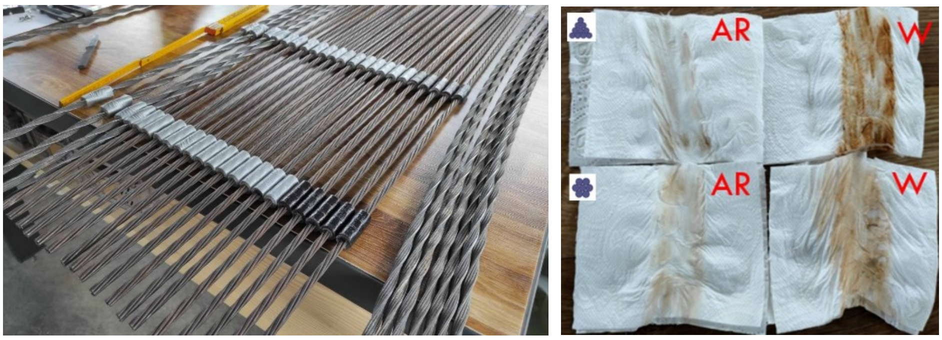

Figure 5. Strand specimens preparation (left) and surface conditions - remains on paper (right)

The schematic presentation of the planned setup with a block specimen that has multiple strand samples is shown in Figure 6. The first used setup of the pull-out test with untensioned strands is called the A setup. The concrete block, with a size of 1240x450x450 mm, was originally planned. Due to the capacity of the mixer, this concrete block was reduced to the size of 611x450x450 mm (Figure 7) to enable the casting of a whole specimen from one mixing.

Figure 8 shows test arrangement and position of the sensors for measuring pull-out force (pressure sensor – S0) and displacement sensors for strand slipping on both sides of the concrete block (1 + 1 LVDT – S1 and S2).

Setup A had imperfections that affected the logic and accuracy of the test results, such as (Figure 9):

– The steel device shown in Figure 9-right, which rested over the anchor chuck taken at one point, had a rotation in both directions when pulling the strand. This was happening in the process of "tuning", so measuring with only one displacement sensor in this way was very unreliable. At least one more displacement sensor must be introduced.

– The height of the steel device itself introduced additional unknowns. The steel device should essentially have a previously tested stiffness, the values of which could be used to compensate for any unwanted measurements in the pull-out test. By removing such a high steel device, a lot of unwanted factors have been eliminated.

– At the passive end, the displacement sensor was set according to Fig.9-left. The way that the sensor made contact from the bottom, perpendicular to the cross-section of the strand, introduced potential problems that can manifest themselves in the form of sensor displacement and sliding during rotation of the strand caused by slipping through concrete.

– Magnetic holders can additionally affect the measurements if they are not strong enough and do not lie on a solid surface. In the new arrangement (setup B), the sensors will be attached directly to the strands according to Figure 10.

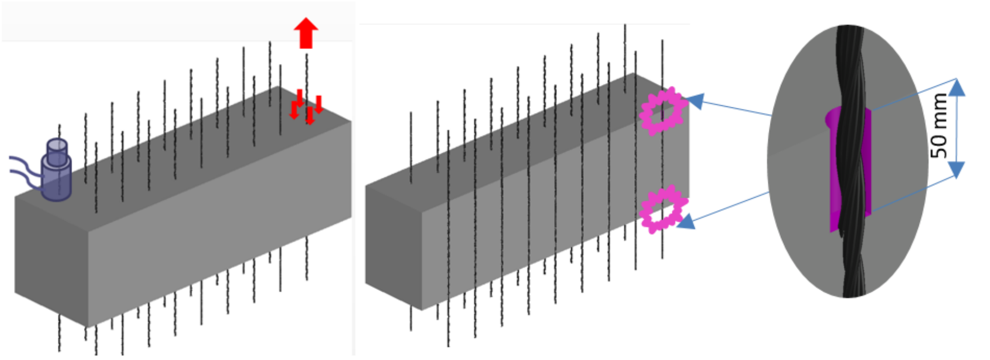

Figure 6. 3D schematic presentation of the block with multiple strand samples

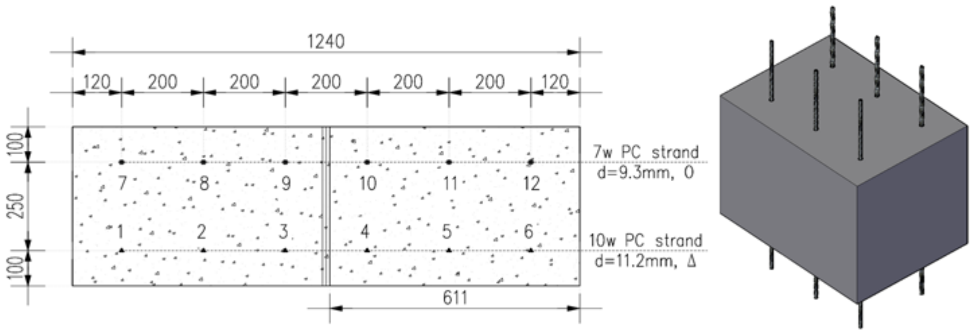

Figure 7. Reduced concrete block specimen with a position of strands – setup A

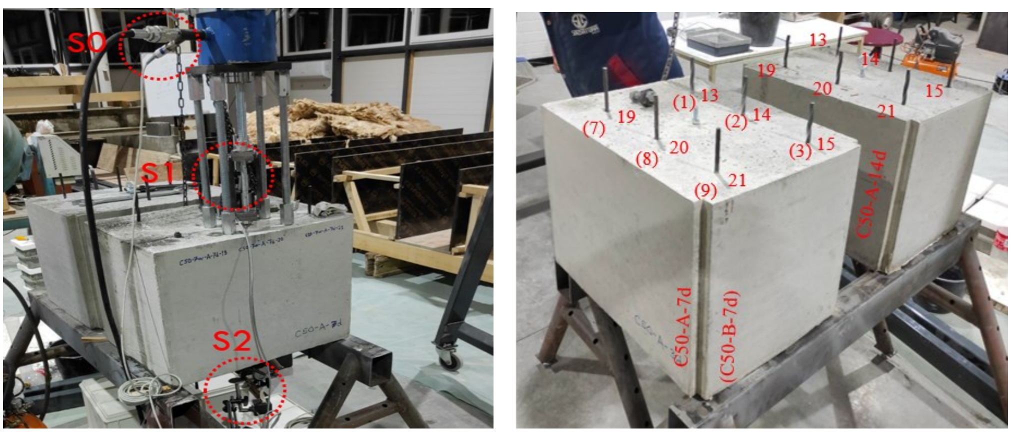

Figure 8. Test arrangement and position of the sensors (left) and concrete specimens with numbering (right)

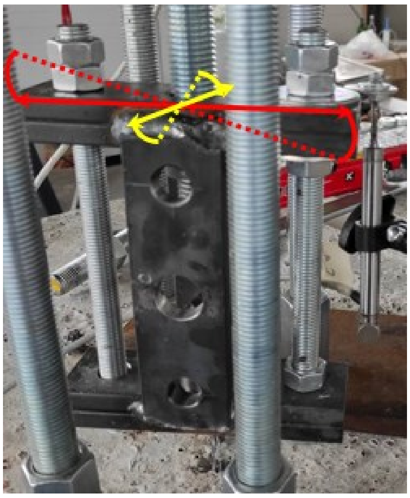

Figure 9. Setup A imperfections on the passive side (left) and active side (right)

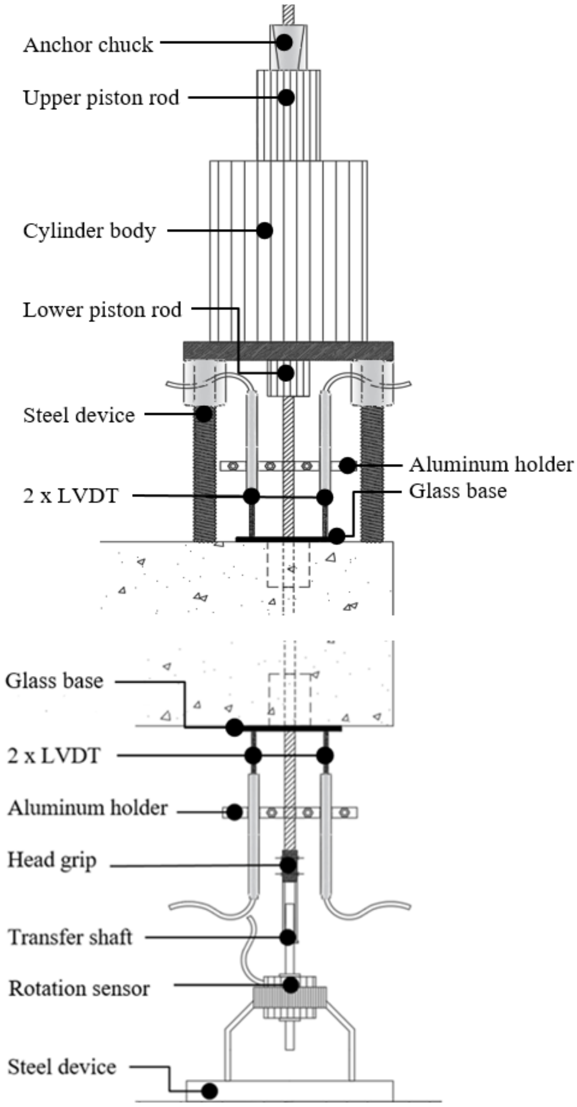

Figure 10. Setup B active side (upper side) and passive side (lower side)

The new setup B was first tested on test specimens, in order to eliminate new unpredicted situations. The specimens were tested according to the previously defined procedure, in order to determine the closest possible moment to full "clamping", i.e. to reduce free movement due to the mutual adjustment of the strand (wedges in the sleeve) and the piston of the cylinder.

The setting of the sensors at the active end is shown in Figure 10-upper side. It can be seen that 2 displacement sensors LVDTs (1 inductive and 1 potentiometer) with a range of 50 mm are placed on each side. Both are mounted directly on the test strand via an aluminum holder (Fig. 10). By pulling the strand out, the strand also rotates. In order for the sensors attached to the strand to rotate freely and together with the strand, it is necessary to reduce the friction in contact with the concrete to a minimum value. This is achieved through glass plates that are placed over the strand and glued to the concrete surface. The contact between the top of the LVDT and the polished glass surface has negligible friction, so it is considered that the LVDT will rotate smoothly with the strand.

On the passive side, the setting of the measuring instruments is shown in Figure 10-lower side. It can be noticed that the setting is completely the same as on the active side (1 inductive and 1 potentiometer LVDT), with the addition of the encoder for measuring the rotation of the strand. The encoder has a range of 2000 impulses. A shaft with a diameter of approximately 8 mm passes through the encoder, which is attached to the headrest. The headrest is attached to the strand. This headrest slides freely over the shaft in the direction of the strand slipping. During the rotation of the strand, the strand can slide upwards without hindrance, and at the same time it does not drag the encoder with it. A glass base was placed over the strand, and on the concrete surface, which enabled uninterrupted rotation of the LVDTs together with the strand (same as the active end).

The force was measured as mentioned via a pressure sensor placed between the cylinder and the oil supply.

The complete final setup of the experiment is shown in Figure 11.

In this approach, the imperfections detected in the setup A were successfully resolved in setup B. Moreover, some fine preparations were performed in setup B which are shown in Figures 12 and 13.

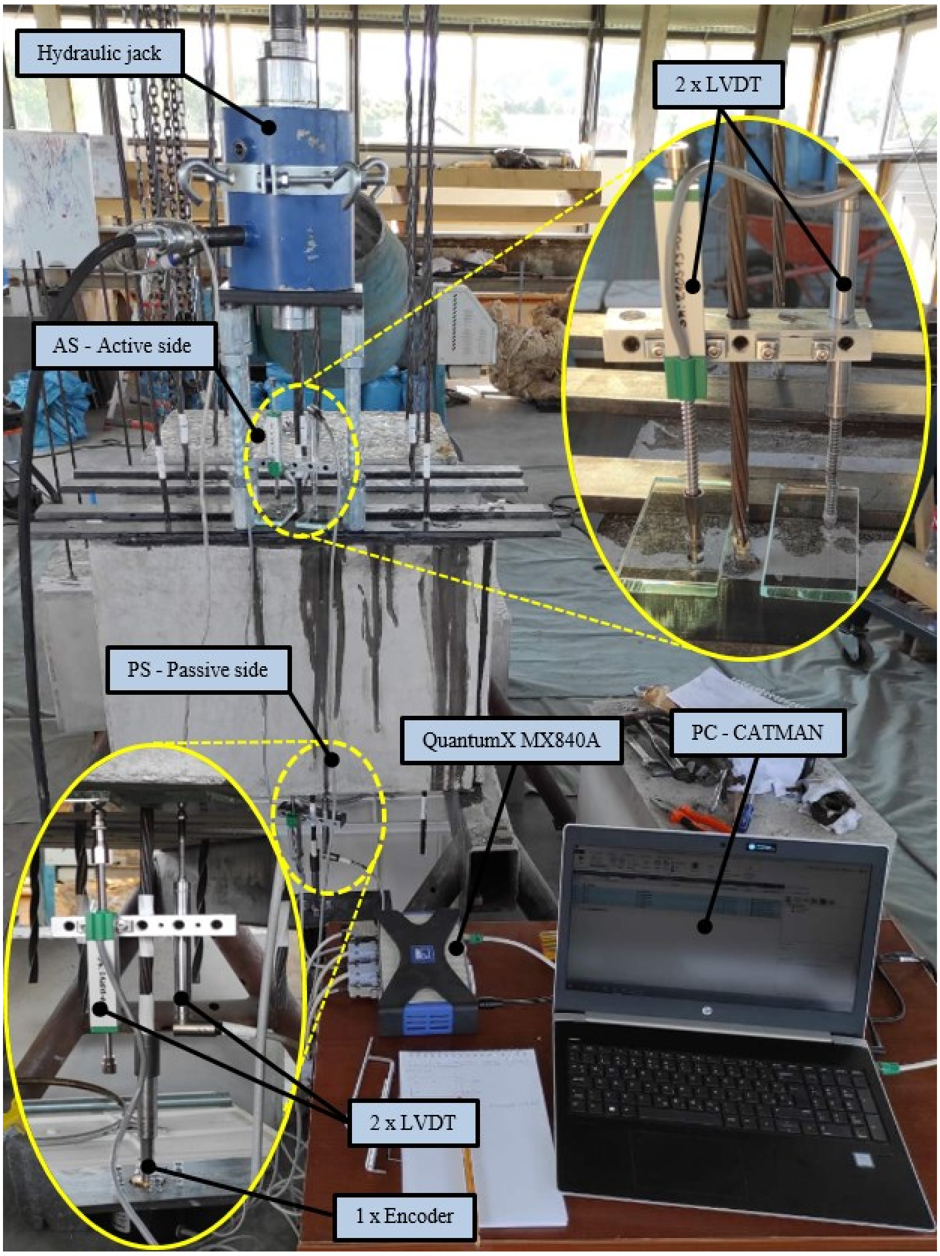

Figure 11. The complete setup B of the pull-out test

Figure.12. Fine preparation of supports on concrete surface

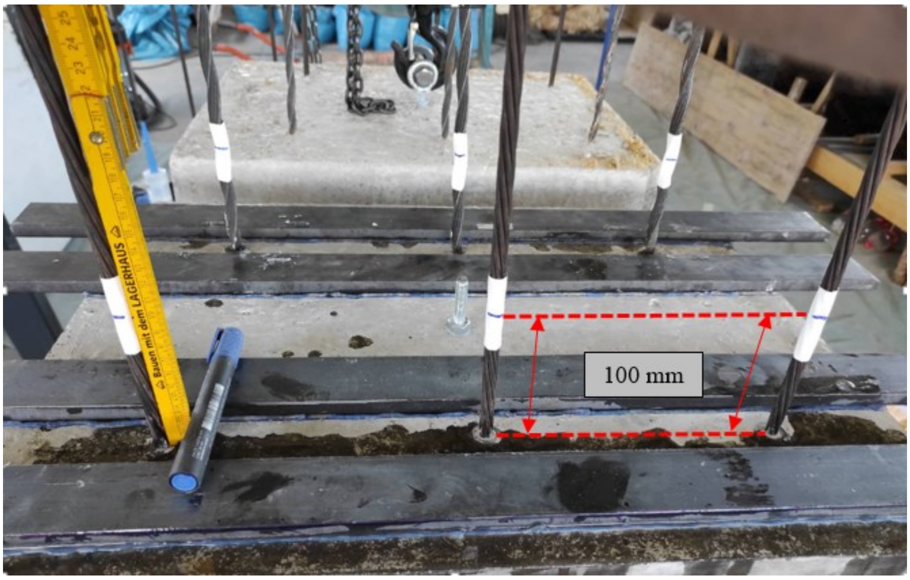

Figure 13. Leveling position of the LVDT's on the active side

3 Experimental results and discussion

The specimen's identification procedure is shown in Table 4 and Figure 14.

Table 4. Summary of the testing program and research variables

Type of strand	Strand condition	Concrete class	Time	Number of large beam samples
7w	A	C40/50	3 days	4
			7 days	
	B	C40/50	3 days	
			7 days	
10w	A	C40/50	3 days	
			7 days	
	B	C40/50	3 days	
			7 days	

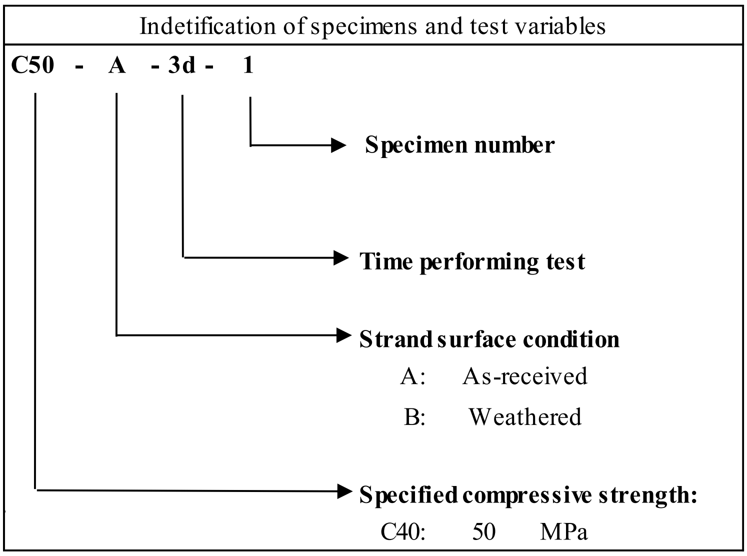

Figure 14. Identification procedure for the block specimens

So far concrete block specimen C50-A-3d was tested with the previously described setup B. The strands were pull-out from the blocks with the procedure described in chapter 2.2. A pull-out force/slip relationship is shown in Figure 15. Dot lines represent the active side, and continuing lines represent passive side results.

As was shown in [14], it was expected that measurements on the active end of the strand (the side on which the force is applied) differ from those obtained on the passive side. This difference is in higher values of slips at lower forces, which is caused by the way the strand is fixed (slip of wedges in the sleeve) and by elastic elongation of the unsupported part of the strand. As a consequence, the active side curve will always have a smaller slope to the horizontal axis. From the moment when the strand completely slips (the strand begins to move through the concrete along its entire length), the curve changes from a linear behavior to a nonlinear behavior. At this point, the connection has been plasticized all the way to the breaking moment by slipping.

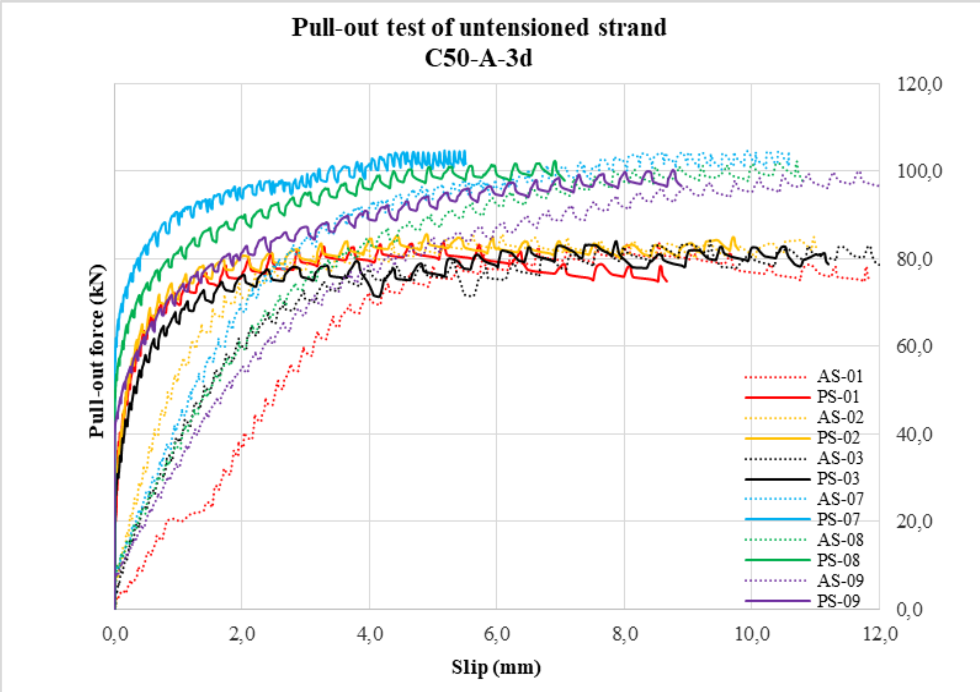

Figure 15. Pull-out force versus Slip response: 1-3 (10-w strands); 7-9 (7-w strand)

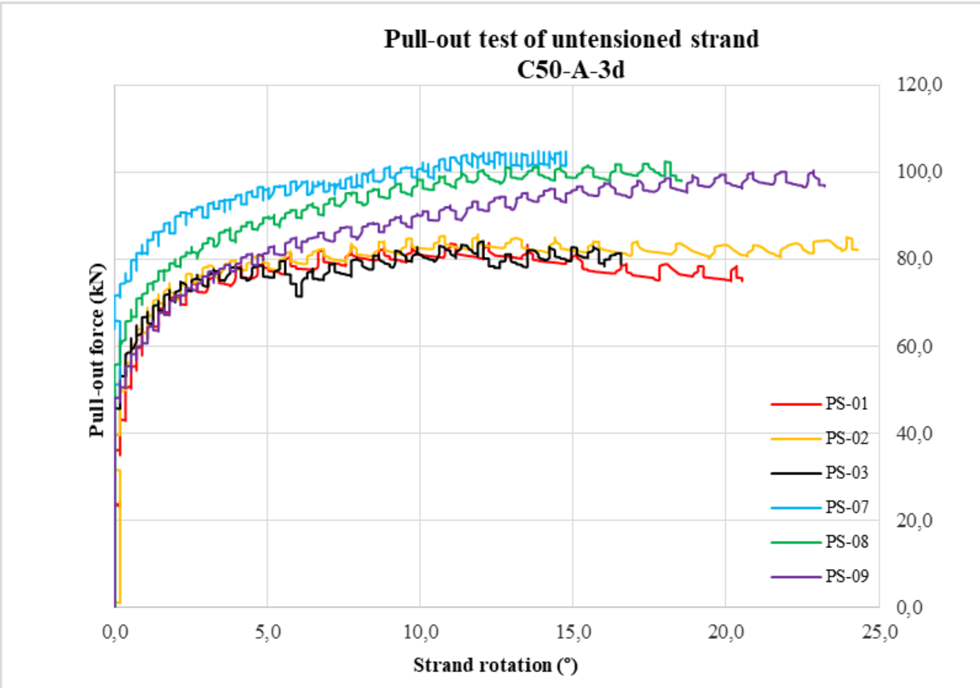

Figure 16. Pull-force versus Strand rotation response: 1-3 (10-w strands); 7-9 (7-w strand)

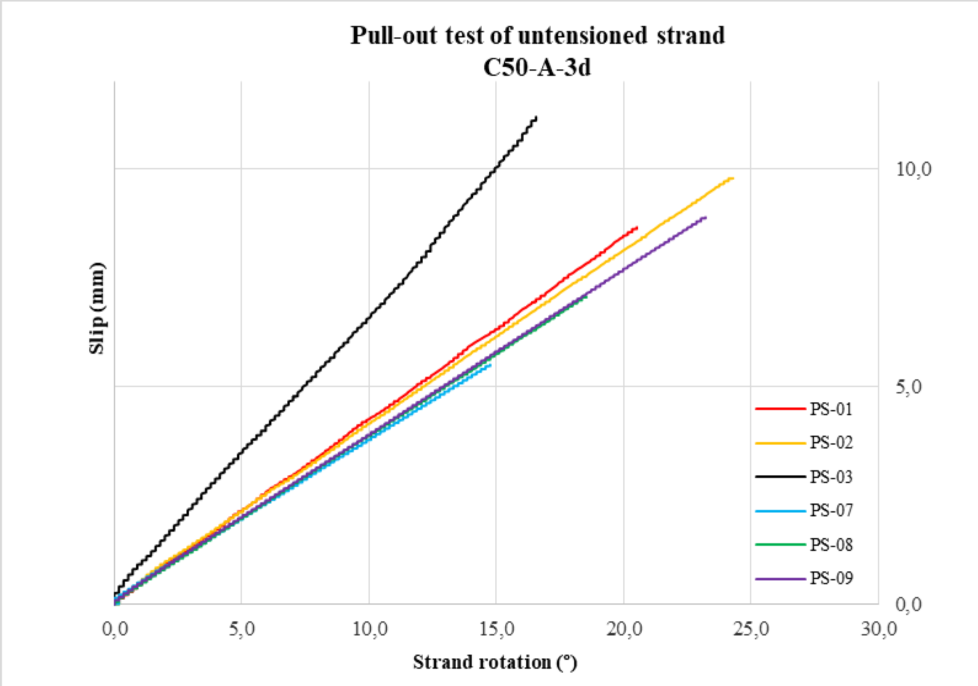

Figure 17. Slip versus Strand rotation response: 1-3 (10-w strands); 7-9 (7-w strand)

From the beginning of plasticization, the slips on the active and passive sides are equalized, i.e., the curve of the active side is parallel and offset by the values of the initial slips. The same behavior was obtained in this test (Figure 15): on the passive side, there is no slip until the moment when the bond plasticization occurs. After that, the strand begins to move completely through the concrete and the slip on the passive side appears.

It can be seen in Figure 15 that 7-w untensioned strands had dominantly higher pick bond strengths than the 10-w strands. Individual results of the 7-w strands are less uniform with respect to 10-w strands. Besides, the AS-01 strand showed measuring imperfections in the range of 20-25 kN. These imperfections are caused by the current fine-tuning of the measuring instrument.

Figure 16 shows the relationship between pull-out forces and strand rotations. It can be noticed that 10-w strand results have very good uniformity in comparison to 7-w strands. Figure 17 presents the results of slip versus strand rotation. The results of the PS-03 strand deviate from other 10-w strand results, which was caused by the dirty shaft of the encoder and its inability to rotate freely. Other results show constant uniformity, which instills confidence in the accuracy of the results.

4 Conclusion

All the tests presented in this paper were non-standard and performed completely innovatively with self-made devices and setups. Initial tests indicate a justified need to investigate the bond characteristics of the innovative triangular 10-w strand.

The pull-out test results obtained on specimen C50-A-3d allowed a comparison of 7-w and 10-w strand bond behavior. Firstly, the results showed logical bond behavior for both strand types. Secondly, 7-w untensioned strands had dominantly higher pick bond strength than 10-w strands, contrary to expectations. Individual results of the 7-w strand were however less uniform in comparison with 10-w strands. Due to the sensitivity and accuracy (3.5%) of the oil sensor pressure used in this test, the results need to be taken with some reserve. In order for the results to be more accurate, it is necessary to have a high degree of repeatability.

Previous observations define the next steps in the pull-out test of the untensioned strand. Instead of the pressure sensor, a calibrated load cell with a capacity of 500 kN will be used. After the pull-out test of untensioned strands is successfully completed, including different specimens' ages and surface conditions, the focus will be moved to the next phase. Further test phases mentioned in the paper, which are part of the Ph.D. thesis of the first author, should give a clear picture of the behavior of the bond connection between the 10-w strand and concrete, as well as its contribution to the shear capacity of the beam elements.

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Technical paper

Analysis of the simultaneous influence of the horizontal seismic load components on buildings

Dušan Stevanović[1)], Ratko Salatić[*2)], Andrej Bubalo[3)]

[1)] *Institute for Testing of Materials Belgrade, Bulevar vojvode Mišića 43, 11000 Belgrade*
[2)] *University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, 11000 Belgrade*
[3)] *Reicon gradnja doo, Pazinska 16, 11000 Belgrade*

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Eurocode 8

ABSTRACT

Earthquake records indicate that earthquake motion is an irregular oscillatory soil movement as a consequence of the heterogeneity of the soil material, as well as due to reflection, refraction, and interference of seismic waves. The trajectories of soil particle movement during an earthquake are proven to be chaotic, so the approximation of seismic effects by a simplified collinear model is very rough from an engineering point of view. The directions of the earthquake during the duration of the earthquake event affects the results of the seismic calculation. In this paper, the simultaneous influence of horizontal seismic load components on buildings has been analyzed. Actual seismic norms deal with this issue and define recommendations that should be applied in the design. This paper discussed how realistic and applicable these recommendations are in standard engineering design. A series of time history analyses of the horizontal stiffness of reinforced concrete regular and irregular structures were performed. Two earthquake events with a markedly changing direction of the ground acceleration vector were taken as the load. Significant differences in the influence values of the adopted representative parameters were determined for the two considered cases of collinear and simultaneous effects. In the conclusion, a critical review of the usual seismic calculation and the provisions of Eurocode 8, related to the effect of the horizontal components of the seismic load, is given. Finally, the paper comments on the introduction of corrective factors in cases where simultaneous action is not considered.

1 Introduction

In seismically active areas, a seismic design is performed to ensure adequate safety and bearing capacity of the building due to seismic load. A seismic load is specific when compared to the other types of loads. Characteristics such as the probability of occurrence, the dynamic characteristics of the load, the intensity of action, and the duration of the earthquake are unknown values at the time when the seismic design is performed. In that sense, the values that determine the seismic load are obtained not through a deterministic but rather a probabilistic approach. This approach requires a greater amount of objective data and a significantly larger number of measured data points related to the effect of earthquakes.

The analysis of these data reveals the characteristics of the seismic load that assert the need to re-examine common engineering practices and procedures. The direction, i.e., the directions of the earthquake action during the earthquake event, have been insufficiently researched. In this paper, the simultaneous effect of horizontal seismic load components on high-rise buildings has been analyzed. Actual seismic norms deal with this issue and define recommendations that should be applied in the design. This paper will discuss how realistic and applicable these recommendations are in standard engineering design. The regularity of the building structure, the stiffness of the structure, and the ratio of frequency characteristics of the forced load to the structure have been adopted as important parameters for analysis. A series of time-history analyses of the horizontal stiffness of reinforced concrete regular and irregular structures were performed. Two earthquake events with a markedly changing direction of the ground acceleration vector were taken as the load.

The results of the performed calculations have been analyzed, and a critical review of the usual seismic design and the provisions of Eurocode 8 [1], related to the effect of horizontal seismic load components is presented in the conclusion. Finally, the paper comments on the introduction of corrective factors in cases wherein the simultaneous impacts are not considered.

* Corresponding author:
E-mail address: ratko.salatic@gmail.com

2 Overview of previous research

The direction of seismic actions has been studied by several authors, applying various methods in the process. Within this subchapter, a brief overview of normative recommendations and previous research will be given.

The proposed orthogonal combination rule for multi-component ground motions was first considered by O'Hara and Cunnif (1963) [2], while Chu et al. (1972) [3] proposed the application of the SRSS (Square Root of the Sum of the Squares) procedure. The type of orthogonal combination procedure, known as the 100% + XX% rule, was introduced by Newmark (1975) [4]. He considered 100% of responses in one direction and an additional 40% in the other direction, in order to conservatively capture a bidirectional load. Rosenblueth and Contreras (1977) [5], based on previous work by A.S. Velestos and Newmark, proposed the 100% + 30% rule, which has been widely used in modern regulations.

Menun and Der Kiureghian (1998) [6] proposed extending the well-known rule of modal combination CQC (Complete Quadratic Combination) to a modal and directional combination rule named CQC3 (Complete Quadratic Combination with Three Components). Hisada et al. (1988) [7] used the ratio of response spectra constructed using both horizontal components of ground motion to those constructed using only one ground motion component as a measure of seismic effects, which was applied in Regulations ASCE/SEI 7-10. MacRae and Mattheis (2000) [8] considered the SRSS rule, the 100% + 30% rule, and the SAV rule (Sum of Absolute Values) for a steel frame building using direct nonlinear dynamic analysis with a varying ground motion angle. The SAV rule uses the sum of the absolute maximum values of the structure response for each direction of the earthquake. The structure displacements were chosen as a comparative parameter, given the fact that in the nonlinear analysis, the forces in elements may not change significantly when the parts of the structure reach yield strength. It was concluded that SRSS, 100% + 30%, and SAV rules depend on the angle at which seismic forces act on the object (rotation of the direction of seismic loading concerning the principal axes of the building) and that all methods produce unconservative results in terms of relative inelastic floor displacements. The conclusions were interpreted in UBC-97 (Uniform Building Code provisions). In their work, Lopez et al. (2001) [9] compared the SRSS rules 100% + 30% and 100% + 40% with the CQC3 rule. Within the CQC3 method, the critical response of the structure was determined, which was compared with the responses of the structure obtained using other methods, and ratios up to 25% difference were obtained. It was found that the critical response increases in cases where the modes with the highest effective mass have close frequency characteristics, so the CQC3 rule should not be applied in that case.

Zaghlool (2001) et al. [10] considered the 100% + XX% rule by applying linear and nonlinear direct dynamic analysis. They analyzed the structure response in the *x*-direction at the time of the maximum response in the *y*-direction, and vice versa. The obtained corresponding component of the response is then divided by the maximum from the same direction, yielding XX% participation. This procedure can be described as the percentage activated of the maximum "strong"-axis response at the time of the maximum "weak"-axis response. As a conclusion of this analysis, the use of 100% + 45% rules was proposed. Sherman and Okazaki (2010) [11] analyzed spatial brace frames by using the nonlinear time integration method. Two criteria were applied for designing corner columns shared by orthogonal frames that resist the effects of earthquakes in both directions. In the first one, corner columns were designed to take 100% of the forces from one direction and 30% from the other direction. In the second one, columns were designed at 100% of the forces from both directions. This approach is analogous, but not completely the same, as the 100% + 30% rule because the column forces were obtained based on the bearing capacity of the whole system and not based on design forces caused by the horizontal effect of the earthquake. They noted that the first approach proved to be unconservative in several cases in terms of the results obtained. For the second approach, they noted that for all calculations, the results were on the side of safety, with more conservative results observed when increasing the height of the building since this change reduces the possibility that all braces will be yielding at the same moment. Bisadi and Head (2011) in their work [12] assessed the 100% + 30%, 100% + 40%, and SRSS rules using nonlinear direct dynamic analysis of bridge structures. They analyzed two cases. In the first case, only the major component of the seismic ground motion record was applied, which is defined as the one with the largest amplitude of the ground motion record (PGA, Peak Ground Acceleration) in the longitudinal and transverse directions of the bridge individually, and then the structure responses were combined using adopted rules. In the second case, the primary and secondary components of the earthquake were applied simultaneously. Next, they combined the responses for both directions using combination rules. After that, they determined the probability of underestimation for each of the combination rules, for each of the two loading cases. It should be noted that the probability of underestimating the results varies depending on whether the force capacity or the displacement capacity of the structure is considered. Cimellaro (2014) et al. [13] proposed the application of a modified nonlinear static analysis that would use the factors of 1.0 and 0.6 for two orthogonal seismic actions. The factor with a value of 60% that differs from the usual 30% was obtained by the calibration of six distinctly irregular models with reinforced concrete spatial frames, using nonlinear direct dynamic analysis. The authors found that the difference between these factors arose as a consequence of observing the nonlinear response, instead of the linear response that the 100% + 30% rule was based on.

3 Seismic load

Earthquakes occur at irregular intervals in space and time. For the assessment of seismic risk, certain laws of earthquake occurrence can be observed, if a sufficient amount of objective data is available. This primarily refers to the basic characteristics of seismic loadings, such as the predominant period, the duration of the earthquake, peak ground acceleration, peak ground velocity, peak ground displacement, and the direction of the earthquake [14]. In this paper, only one parameter will be analyzed in detail, namely the direction of the earthquake in the horizontal plane $x0y$. The effect of the earthquake concerning the z-axis is disregarded in this paper and was not subject to analysis.

An earthquake is a natural phenomenon caused by an abrupt release of energy in the earth's crust, which spreads in the form of seismic waves and manifests itself on the surface as shaking of the ground. Earthquake records indicate that it is an irregular oscillatory movement. It is a consequence of the inhomogeneity of the material through which seismic waves travel. Due to reflection and refraction, on the surface they manifest as three-component oscillatory movements without a stable period and amplitude. Due to earthquakes, ground vibrations occur that have two horizontal components and one vertical component. Vector-wise, ground displacements $x0y$ can be analytically defined by the displacement vector (Figure 1a). It is necessary to adopt a time interval for which the displacement is monitored, since data is collected at discrete moments of time, as a consequence of the impossibility to present the stochastic function as an analytical function. Each moment of time t_i corresponds to the vector r_i. (Figure 1a).

In previous engineering practice, the effect of an earthquake was usually considered a load caused by the collinear displacement of ground particles, since all displacement vectors are on one line. This direction is defined by the angle β of the previously arbitrarily adopted orthogonal global coordinate system (Figure 1b).

As earthquake records are obtained via data (displacement, velocity, or ground acceleration) for two orthogonal directions, the input data for the numerical calculation can be component values of the ground displacement u_x and u_y, that is, ground acceleration a_x and a_y caused by an earthquake (Figure 1c). How the components of this load will be used in calculations, independently or simultaneously, affects the essential result of the calculation. Modern-day software and hardware development has made it possible to consider the simultaneous actions of seismic action components relatively easily today.

Based on the data for a large number of examined earthquakes, the ground particle displacement trajectories can be illustrated by diagrams (Figure 2).

The analysis of the presented diagrams shows that the action of earthquakes in the horizontal plane is usually chaotic, with a significant change in the direction of the displacement vector, so rarely can the action of the earthquake be approximated with the effect in one direction alone. Out of all the trajectory diagrams presented, a single-direction approach can only be acceptable in the case of the diagrams corresponding to the Nicaragua, 1972, and Montana, 1935, earthquakes. The diagram analysis concludes that a rough approximation of the load effect is performed if the earthquake load is considered to be acting only in one direction. One direction cannot realistically describe the majority of the seismic load.

Having that in mind, the regulations have introduced a mandatory consideration of the seismic load in two orthogonal directions, primarily led by the fact that the direction of the future earthquake is not known.

It is recommended that a calculation for each excitation direction be made separately, and then, after assuming the linear behavior of the structure, the final results be determined by the superposition of the results for both orthogonal directions. In cases where we calculate only the maximum values for each direction separately using the response spectrum, the procedure is similar to the problem of determining the resultant effects of various vibration tones.

Since it is quite unlikely that the maximum excitation values in both directions will occur simultaneously, simply adding up the maximum values for individual directions gives overestimated effect values, so it is usually suggested that the square root of the sum of the squares (SRSS) method be applied, which is also used in the modal analysis [14]:

$$u_{imax} = \sqrt{u_{ixmax}^2 + u_{iymax}^2}$$
$$f_{Eimax} = \sqrt{f_{Eixmax}^2 + f_{Eiymax}^2} \tag{1}$$

Expressions (1) represent the displacement and force values, and the indices “x“ and “y“ indicate the direction of excitation. The directions of x and y axes must be orthogonal and are most often adopted as the main axes of the structure. In applying this approach, the results are independent of the chosen excitation directions. The approach is not applicable if the excitation in different directions causes different types of loads in individual elements (e.g. axial forces from vertical load, transverse forces, and moments from horizontal load) [14]. Then a combination of 100% load in one direction and 30% or 40% load in the other direction is recommended.

Applying these problem setups, Eurocode [1] prescribes that horizontal seismic load components act simultaneously. The combination of horizontal components of seismic action can be determined in several ways. The first is that the structure response for each component must be calculated separately, using combination rules of modal responses. The second, the maximum value of each impact in the structure due to the two horizontal seismic components, is determined as the square root of the sum of the squares of the seismic effect for each horizontal component:

$$\sqrt{E_{Edx}^2 + E_{Edy}^2} \tag{2}$$

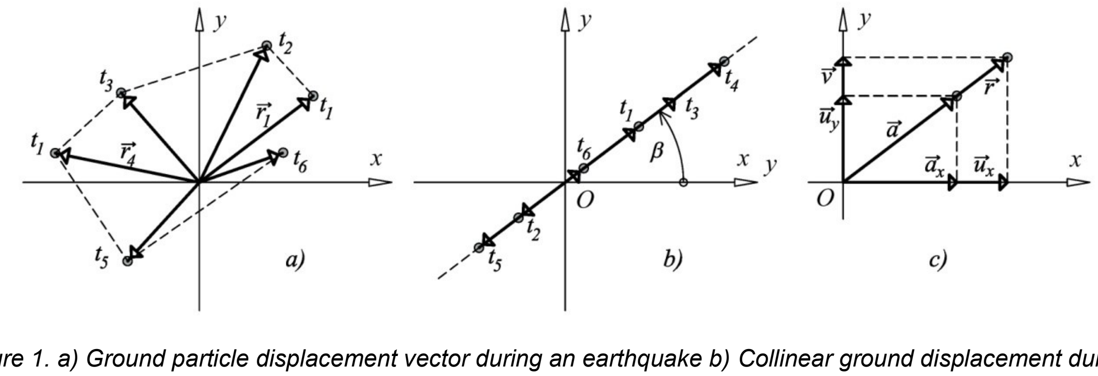

Figure 1. a) Ground particle displacement vector during an earthquake b) Collinear ground displacement during an earthquake c) Components of ground displacement and acceleration due to an earthquake

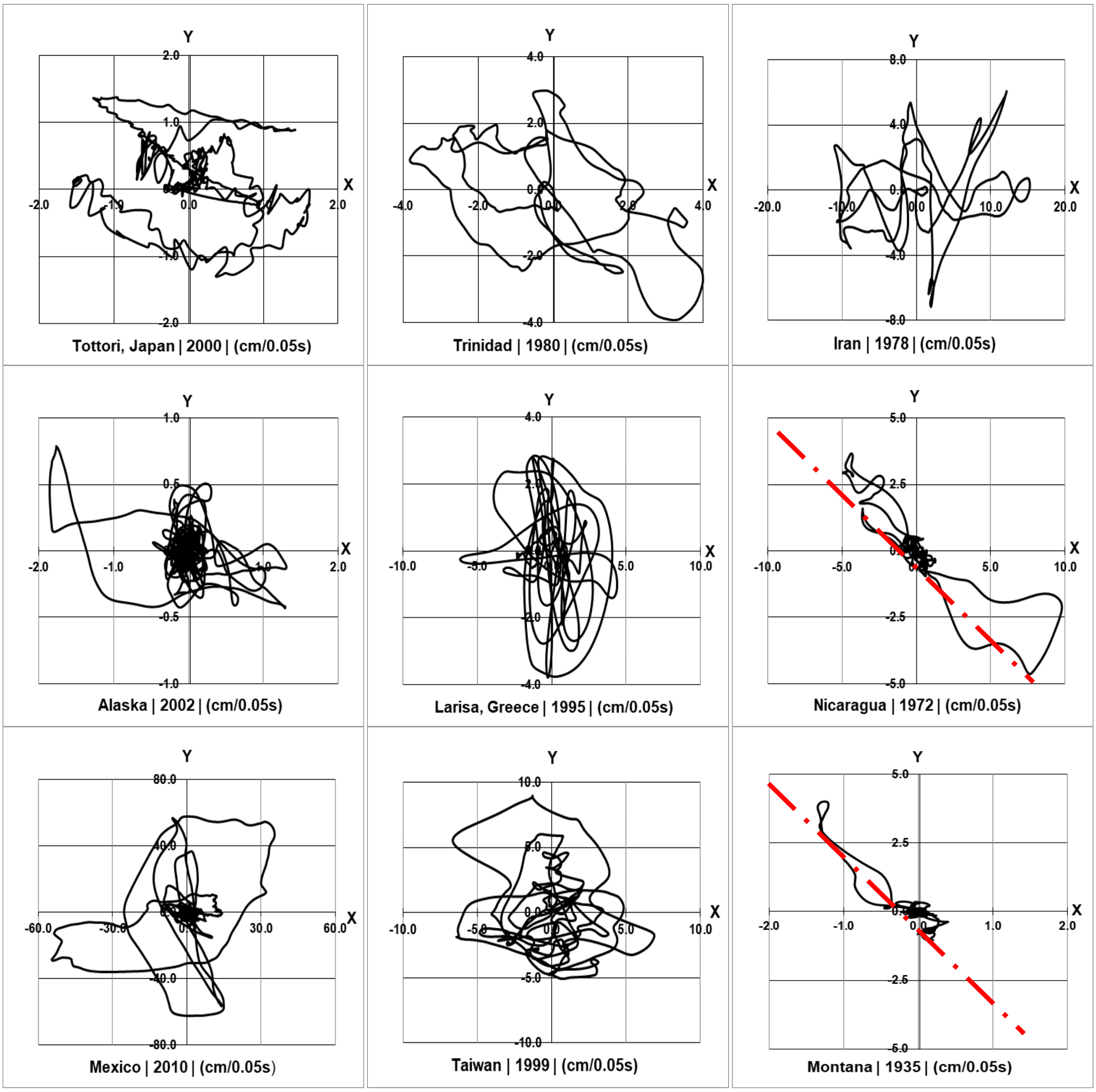

Figure 2. Horizontal action for various earthquake events [15]

The third, less conservative way, assumes governing influences according to the combination of the values of the seismic effects:

$$E_{Edx} \text{"+"} 0.3\, E_{Edy} \qquad E_{Edy} \text{"+"} 0.3\, E_{Edx} \tag{3}$$

where:

E_{Edx} - is the value of effects due to the application of seismic action along the x axis of the structure

E_{Edy} - is the value of effects due to the application of seismic action along the y axis of the structure

In particular, for the nonlinear time history analysis and the spatial model of the structure, the simultaneous effect of the acceleration components in both horizontal directions should be analyzed.

4 NUMERICAL ANALYSIS

4.1 Models for numerical analysis

Numerical analysis was performed for four models of a reinforced concrete multi-story (10-story) structure. The first two models, A and B, represent regular structures with lower and higher horizontal stiffness. Models C and D represent irregular structures with different horizontal stiffness. Different horizontal stiffness was obtained by adding a reinforced concrete core to the frame structure (Figures 3 and 4).

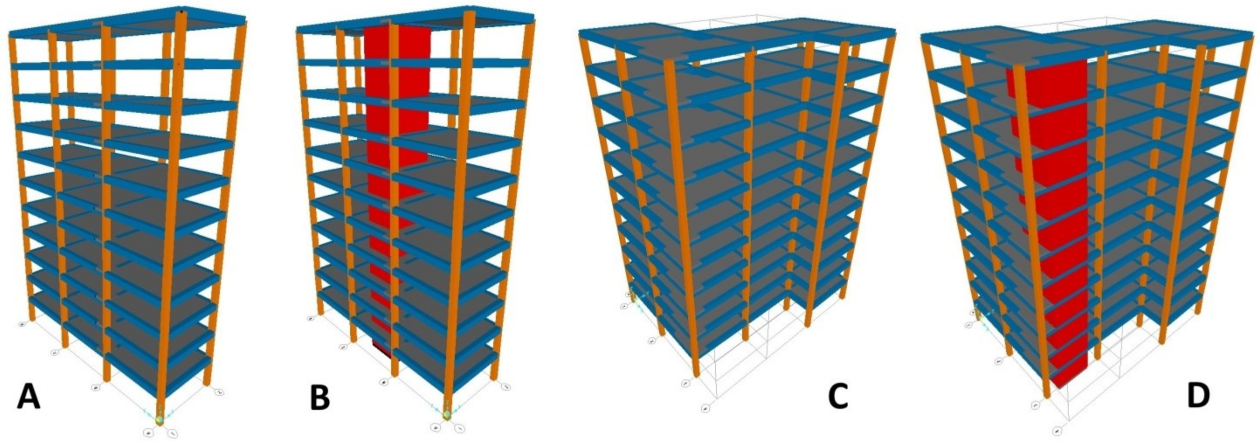

Figure 3. Considered structure models

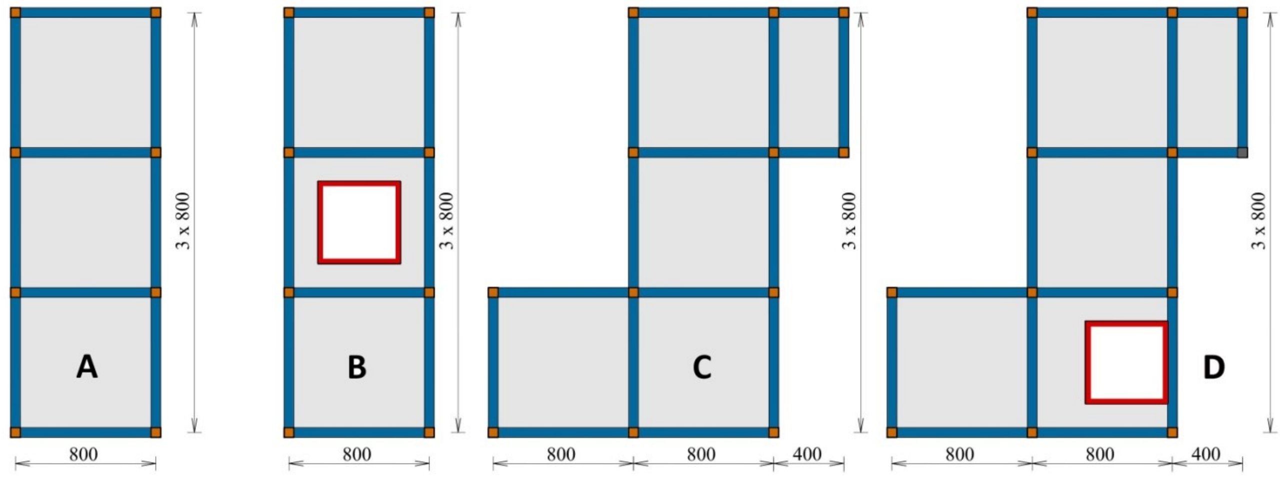

Figure 4. Plan view of the considered models

During the modeling of the structures, given the fact that it is an RC structure, reduced stiffnesses were adopted, which differ depending on the structural element, which is considered to correspond to the behavior of the real structure under the action of seismic loading. For vertical structural elements, columns and walls, the stiffness correction value of 0.70 was adopted, while for horizontal structural elements, beams and ceilings, the stiffness correction value of 0.50 was adopted. The dynamic characteristics for each of the models are given in Table 1.

4.2 Seismic load applied

Analysis was carried out for two earthquake events for which the accelerograms were available for two orthogonal directions, the Gulf of California 2001 and the Lazio-Abruzzo 1984 earthquakes (Figures 5 and 6). The basic data of the record are given in Table 2. The aforementioned earthquakes were chosen for analysis because their ground particle displacement trajectories are markedly chaotic in the horizontal plane (Figure 7).

Table 1. The first three periods of oscillation of the considered models

Structure models	**Oscillation periods** *(s)*		
	T_1	T_2	T_3
A – Regular in plain without RC core	2.35	2.08	1.81
B – Regular in plain with RC core	1.06	0.99	0.78
C – Irregular in plain without RC core	2.17	2.11	1.73
D – Irregular in plain with RC core	1.38	1.13	0.82

Table 2. Earthquake record data

Earthquake record	**Gulf of California**	**Lazio - Abruzzo Italy**
Year	2001	1984
Station	El Centro - Meadows Union School	Athens
Magnitude	5,7	5,8
V_{s30} (m/s)	276,25	585,04
PGA (direction 1)	0,011 g	0,0956 g
PGA (direction 2)	0,0099 g	0,0956 g
Closest distance to rupture – R_{rup} (km)	96,28	18,89
Duration *D5 - 95* (s)	64,7	10,0

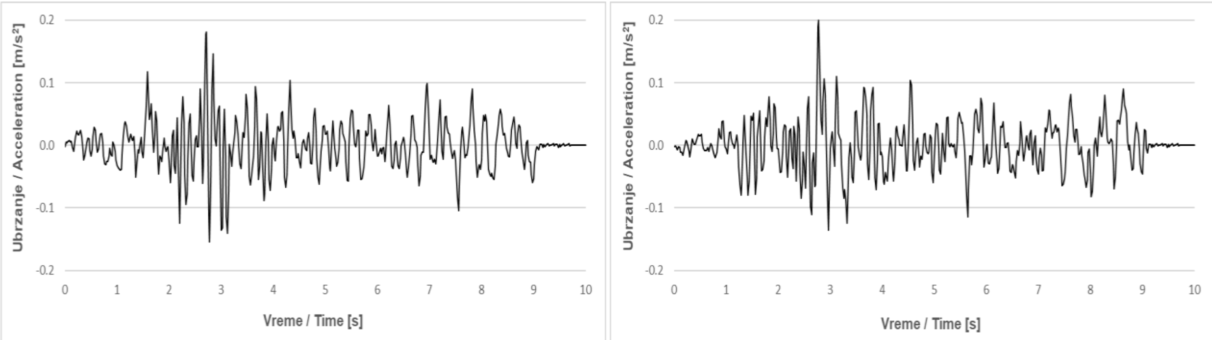

Figure 5. Accelerogram of the Gulf of California earthquake in the direction of both x *and* y *- axes [15]*

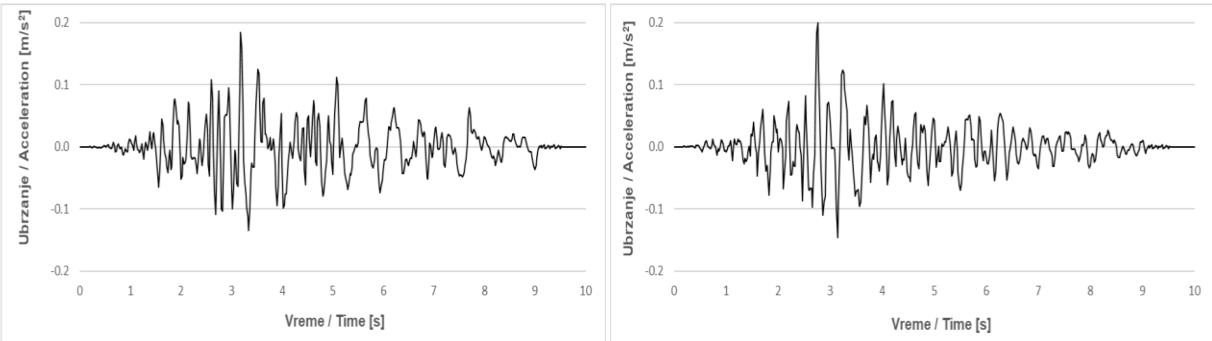

Figure 6. Accelerogram of the Lazio earthquake in the direction of x *and* y *- axes [15]*

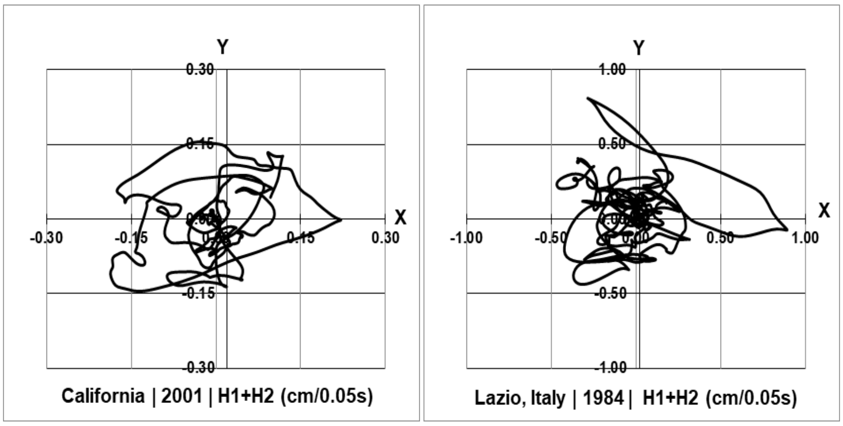

Figure 7. Representation of the ground displacement trajectory in the horizontal plane for the selected earthquakes [15]

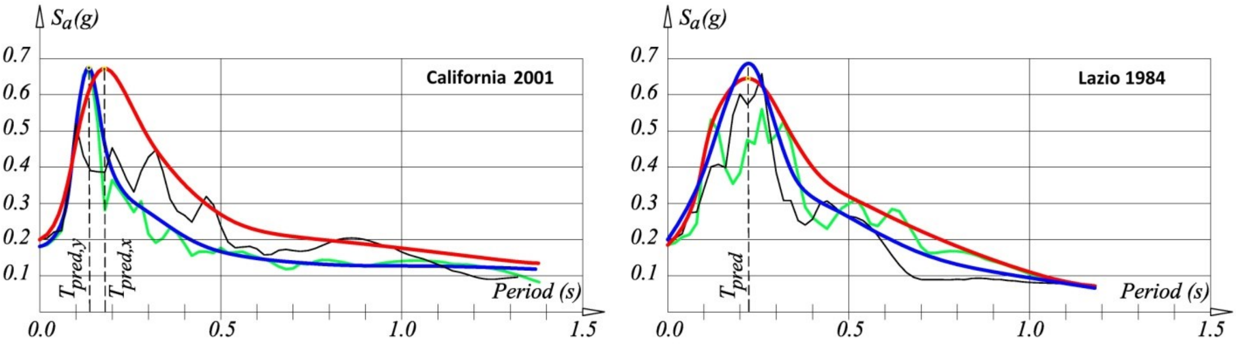

Figure 8. The spectrum of the California and Lazio earthquake response [16]

Ground acceleration records are normed to a value of $0.2g$ for the more intense direction, and then the acceleration values for the other direction were modified with the same multiplier to maintain the same ratio of ground acceleration components both in x and y directions.

In the case of the California earthquake record in the x-direction, the predominant period is 0.17 s, while in the y-direction it is 0.14 seconds. Different values of predominant periods could be due to inadequate earthquake recording or due to a numerical error in the integration procedure. In the case of the Lazio earthquake record, in both directions the predominant period is 0.22 s. Figure 8 presents the original spectrum for two directions (black line - x direction, green line - y direction) and their envelopes (red line - x direction, and blue line - y direction).

4.3 Results of the calculation

The calculation was carried out using the method of linear direct dynamic analysis with the software package SAP2000 ver. 14 [17]. In this study we did not analyze the vertical component of the seismic action and used the assumption that earthquake records, given in orthogonal directions, coincide with the longitudinal and transverse axes of the base of the building. For each of the formed structure models, the seismic load was assigned using three different approaches:

1) according to the EC8 standard, EN 1998-1: 2004 [1],
2) independent collinear effect of seismic loading,
3) simultaneous effect of seismic loading for two seismic load components.

Calculations were performed for each of the assigned loads, to determine the governing deformation and static values. The first calculation is following the provisions of EC8 by applying equations (3). The second calculation, the collinear effect, considered the seismic load in the x-direction and y-direction as acting independently, so the load that causes greater effect was chosen as the governing one. The third calculation refers to the simultaneous action of the acceleration components corresponding to the obtained accelerograms for the two orthogonal directions of the selected earthquakes.

After conducting the analysis using the software package SAP2000 [17], the obtained values of the following components were observed to consider the structure response:

- the resulting displacements in the nodes on the roof slab, (Figure 9)
- columns' bending moments at the building base, (Figure 10)
- total base shear forces.

The resulting displacements in the nodes on the roof slab are determined as the vector sum of the component displacements, which are obtained in the results of the software calculation. These displacements of the roof slab were obtained in all nodes of the slab as a same values, given the fact that the adopted reinforced concrete ceiling was rigid in its plane (Tables 3 and 4).

For the base bending moments, the values that have the largest differences in the calculation values according to Eurocode 8, and the calculation with simultaneous effect are presented (Tables 3 and 4). Differences are given in percentages $\Delta[\%] = ((R_{1,2}-R_3))/R_3$, and R indicates the considered parameter, and the index is the chosen calculation of seismic load.

The total base shear force is equal to the total seismic force. It represents a single global quantity that characterizes the earthquake and is often used to control and analyze the results obtained from a numerical seismic design. Results for total seismic force differences is given in Table 5.

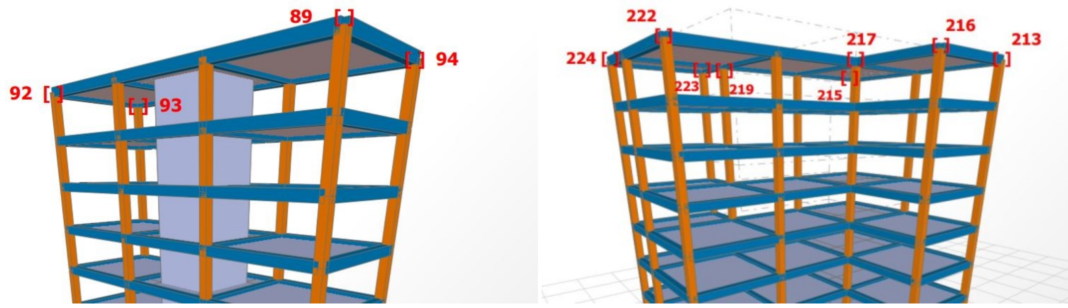

Figure 9. FEA (Finite Element Analysis) model nodes for which the resulting displacements are considered

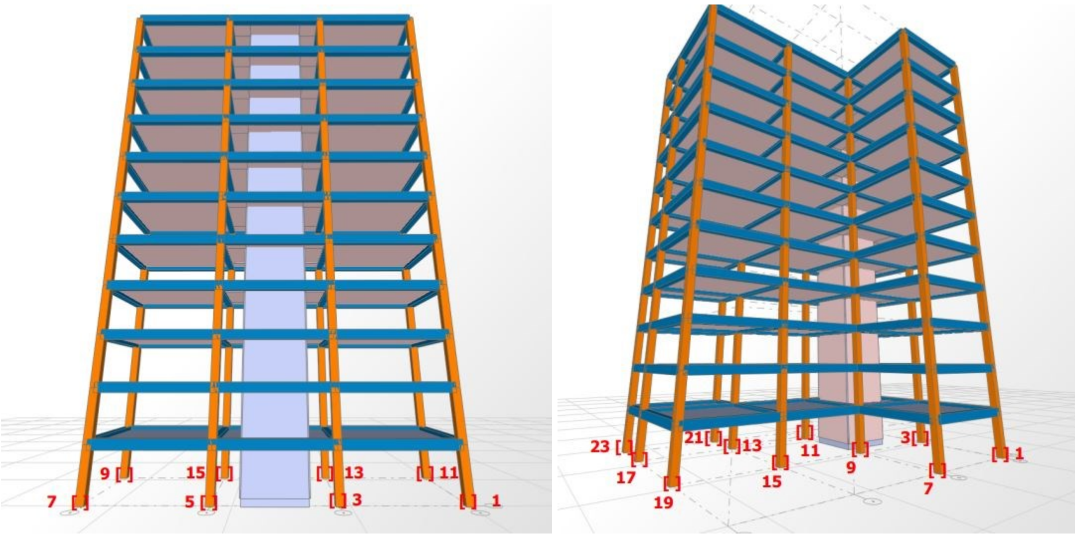

Figure 10. FEA model nodes for which the base bending moment values are considered

Table 3. Results of roof ceiling displacement and bending moments in columns in regular models

		Displacement [cm]				**Bending moment** [kNm]			
	California	**A**	**Δ** [%]	**B**	**Δ** [%]	**A**	**Δ** [%]	**B**	**Δ** [%]
1	EC8	3.40	3.0	3.70	-8.6	144.2	-13.0	66.9	-13.5
2	Collinear	3.27	-0.9	3.56	-12.1	138.3	-16.5	61.2	-20.8
3	Simultaneously	3.30	-	4.05	-	165.7	-	77.3	-
	Lazio	**A**	**Δ** [%]	**B**	**Δ** [%]	**A**	**Δ** [%]	**B**	**Δ** [%]
1	EC8	2.90	0.7	2.73	-14.7	141.6	2.5	38.4	-16.7
2	Collinear	2.80	-2.8	2.61	-18.4	135.9	-1.7	36.9	-20.0
3	Simultaneously	2.88	-	3.20	-	138.2	-	46.1	-

Table 4. Results of roof ceiling displacement and bending moments in columns in irregular models

		Displacement [cm]				**Bending moment** [kNm]			
	California	**C**	**Δ** [%]	**D**	**Δ** [%]	**C**	**Δ** [%]	**D**	**Δ** [%]
1	EC8	3.40	6.6	4.17	-25.4	133.8	-24.9	148.4	-27.7
2	Collinear	3.24	1.6	4.17	-25.4	138.6	-22.2	126.2	-38.5
3	Simultaneously	3.19	-	5.59	-	178.1	-	205.3	-
	Lazio	**C**	**Δ** [%]	**D**	**Δ** [%]	**C**	**Δ** [%]	**D**	**Δ** [%]
1	EC8	2.61	-7.1	2.68	-31.3	155.9	7.2	123.6	41.4
2	Collinear	2.50	-11.0	2.77	-29.0	150.8	3.7	110.4	26.3
3	Simultaneously	2.81	-	3.90	-	145.4	-	87.4	-

Table 5. Results of total seismic force components in irregular models

		Components of total seismic force [kN]							
	California	**C**	Δ_x [%]	**C**	Δ_y [%]	**D**	Δ_x [%]	**D**	Δ_y [%]
		S_x		S_y		S_x		S_y	
1	EC8	675.1	-1.7	656.6	-1.8	1831.0	-12.0	2109.4	-10.2
2	Collinear	675.1	-1.7	656.5	-1.8	1877.0	-9.8	2195.5	-6.6
3	Simultaneously	687.0	-	668.4	-	2080.6	-	2349.4	-
	Lazio	**C**	Δ_x [%]	**C**	Δ_y[%]	**D**	Δ_x [%]	**D**	Δ_y [%]
		S_x		S_y		S_x		S_y	
1	EC8	727.9	2.6	723.8	2.6	1196.7	-8.9	1393.5	-22.4
2	Collinear	725.9	2.3	721.8	2.3	1214.0	-7.6	1469.5	-18.1
3	Simultaneously	709.4	-	705.3	-	1313.6	-	1795.0	-

4.4 Analysis of the numerical analysis results

Analyzing the displacement of the roof slab and column's bending moments in the case of regular structures, there is a difference in the results for the calculation according to Eurocode 8 and the calculation with simultaneous action, for displacement of up to 13.5% while for bending moments it increases to 20.8% (Tables 3, 4). The same parameters for irregular structures have a difference of 25.4% for displacement and 41.4% for bending moments.

The differences in the total seismic force components of regular structures are negligible (2.6%), while in irregular structures they are 22.4% at the most.

When comparing the predominant record periods used in the design of structures, we can conclude that the main oscillation periods of flexible models (Models A and C) are quite distant from the resonant range. The situation is different when analyzing the oscillation periods of stiff models (Models B and D).

We can notice that the oscillation tones of the rigid models are closer to the predominant periods of the considered earthquakes. The periods obtained for the models under consideration are expected. The identical seismic load was assigned to all models, and there is the effect of increasing acceleration in the nodes of rigid models whose oscillation periods are closer to the predominant earthquake period, which indicates the effect of amplification.

If the calculation is performed in accordance with the provisions of the current Eurocode 8, the obtained differences in the results indicate an obvious underestimation of the impact in the seismic calculation. Differences in results from 10% to 20%, even exceptionally up to 41%, indicate that engineering calculations must pay special attention to this fact. This conclusion is in line with previous research on the simultaneous effect of earthquakes, based on both linear and nonlinear analysis. The analysis herein was linear, which may provide a relevant response at the beginning of the earthquake action, before the appearance of nonlinear effects.

Important input data for numerical analysis of the time response of the structure are primarily accelerograms. As the two records define one earthquake event, the dynamic characteristics of the excitation should be the same for both accelerograms in both x and y directions. A certain deviation certainly exists due to the discretization of the input data. A special analysis needs to be conducted in order to determine the sensitivity of the results to the choice of integration time interval, in order to define the criteria for the applicability of the accelerogram record in seismic design.

5 Conclusion

After the analysis of earthquake records, it is obvious that the earthquakes whose action can be approximated by collinear influences have occurred in very small numbers. The trajectories of movement of soil particles during an earthquake are proven to be chaotic, so the approximation of seismic effects by a simplified collinear model is very rough from an engineering point of view. Actual regulations, primarily the Eurocode, prescribe for which cases of seismic calculation it is necessary to carry out an analysis with the simultaneous effect of ground acceleration components. Those normative provisions refer only to more complex and irregular constructions. The problem remains unsolved when calculation with simultaneous action is not mandatory.

Common procedures based on the combination of effects due to collinear effects of earthquakes hide the effects of real earthquake action, because underestimated values of seismic effects are increased by mathematical procedures and not based on the actual response of the structure.

In the paper, the results for the selected types of structure were analyzed in detail, and important differences were determined in the impact values of the adopted representative parameters for the two considered cases of collinear and simultaneous action. Differences were noted in a wide range, from 1% to 40%. The presented difference in results is so significant for some influences that it undoubtedly affects the change in the behavior of the system that absorbs seismic energy with the plastification of the most stressed elements of the system. In the case of irregular building systems, as well as other systems that do not fully meet the requirements of aseismic design, the aforementioned differences may cause unwanted consequences.

The previous conclusion raises the issue of introducing a correction factor for calculations that do not take into account the simultaneous effect of two components of earthquake acceleration. The corrective factor would have the role of increasing the underestimated impacts obtained based on the collinear effect. In this way, for simpler objects, there would be no need to introduce a more complex calculation. Its value should be determined based on the knowledge of the actual impact values compared to underestimated ones, for different categories of objects and types of calculations (regular, irregular, high, low buildings, linear and non-linear design).

In the design which takes into account simultaneous action, special attention should be paid to the analysis of the available records of the ground acceleration components in the x and y directions, i.e. accelerograms. They represent a single earthquake event, and the frequency characteristics of the records for both records should be matched. These errors most often occur due to an inadequate earthquake record or time step in the accelerogram record.

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Technical paper

Two new bridges over the river Vardar in Skopje

Goran Markovski*[1)], Marija Docevska[1)], Viktor Markelj[2)]

[1)] *Faculty of Civil Engineering "Ss. Cyril and Methodius" University, Blvd. Partizanskiodredi 24, 1000 Skopje, N. Macedonia*
[2)] *PONTING Ltd., Strossmayerjeva Street 28, 2000 Maribor, Slovenia*

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ABSTRACT

This paper presents some aspects of the design of two new bridges over the River Vardar in Skopje.

"Mihajlo Apostolski" is an integral bridge, consisting of two separate structures (22.05 + 50.50 + 21.05 m = 93.6 m length and 15.3 m width). The bridge deck is made up of a two-cell prestressed cast-in-situ box girder with a parabolically variable cross section. The abutments and the piers are wall-shaped and supported on single and double row piles, respectively. The side spans, together with the approaching structures, were proposed to be built on traditional scaffolding, while the middle span over the minor riverbed was built with a free cantilever method.

The extradosed bridge at Ljubljanska Street is 27.5 + 56 + 27.5 m = 111 m long and 24.1m wide. The ribbed deck (two post-tensioned main girders, cross girders, and deck slab) is supported from above by 32 parallel stay cables. The 12.6 m high pylons are located at the intermediate supports. In the base, the four piers have a rectangular cross section that has been rotated 45 degrees. The superstructure is supported by pot bearings. A deep foundation on piles was chosen. Cast-in-place construction of the superstructure was foreseen.

The design of the bridges was done according to Eurocodes and respecting the fib and PTI recommendations. Detailed numerical analysis was carried out in the FEM software SOFiSTiK and additional auxiliary software.

1 Introduction

The Vardar River runs along the longer side of Skopje for about 25 kilometers. Such a river orientation necessitates a greater number of bridges to allow for more efficient traffic communication between the city's north and south sides. Following the catastrophic flood of 1962, the river bed was completely regulated to a total width of about 100 m, depending on the location. In the midst of the major bed, a 50-m-wide and 3-m-deep minor bed was formed. On its left and right sides, there are recreational footways that end with 3-m-high quay walls.

Such a cross-section of the river bed (approximately 25 + 50 + 25 m) along with the quite unfavourable hydrological parameters (a small difference between the finished levels of access traffic lines and the flood water level) create real problems in the conceptual design of bridge structures and inevitably lead to certain mutual concessions.

At present, in the area of Skopje City, nine road bridges are functional. All of these are made of concrete (reinforced or prestressed). Most of them have three spans. Several different static systems are present.

Both bridges presented in this paper were designed in 2019 and 2020, respectively. The investor was Skopje, while the project operator was the Geing Ltd. Company from Skopje. The design solution for the bridges was elaborated based on previously prepared projects on the infrastructure, considering the following defined parameters:

- The finished level of the traffic line;
- The position of the structure on the site plan;
- The width of the traffic line, the cycling paths, and the sidewalks;
- Data on flood waters with a return period of 100 years and the necessary hydraulic opening.

The first bridge has already been constructed, while the construction of the second one started in August 2022. The contractor for both structures is Granit Ltd. Company from Skopje.

* Corresponding author:
E-mail address: markovski@gf.ukim.edu.mk

2 Integral bridge "Mihajlo Apostolski"

The "Mihajlo Apostolski" bridge is part of the ASNOM boulevard extension project.

The traffic solution anticipates that the relatively long access ramps to and from the cross-road located on one side (Fig. 1) are a constituent part of the bridge structure. This poses serious limitations on the elaboration of possible variant solutions.

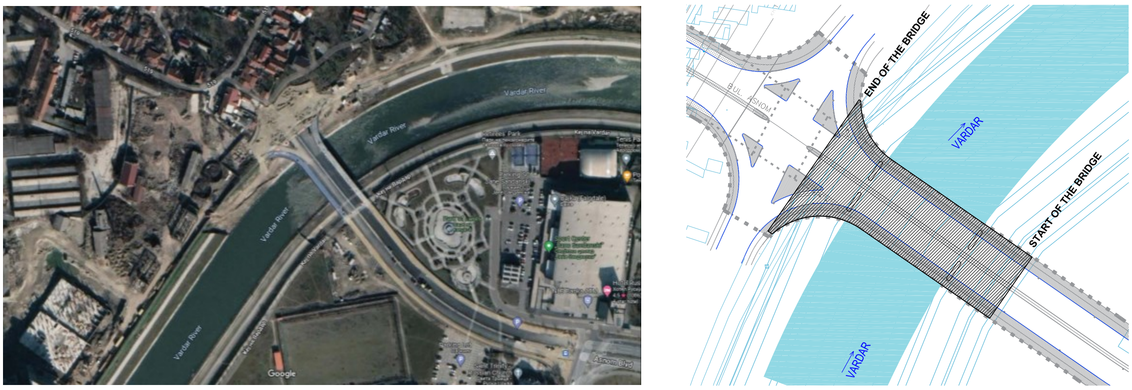


Fig. 1. Position of the bridge on the site plan

A frame with three spans of 22.05 + 50.5 + 21.05 m = 93.6 m (Fig. 2, 3) was selected as the final solution. Such a ratio of spans arose from the condition that the middle piers had to be placed on the slopes of the minor river bed. The bridge consists of two branches that represent separate structural units. The width of each branch is 15.3 m, and it includes: three traffic lanes of 3 m each, 5.2 m for the pedestrian and cycling paths, and two parapet beams for anchorage of the fence systems. The span of the access ramp on the left branch of the bridge is 32.75 m, whereas that of the right one is 25.73 m.

The adopted system is also known as an integral type of bridge. This enables the avoidance of bearings and expansion joints over the abutments that indirectly affect the durability and the cost of the structure. However, on the other hand, the integral solution requires a more detailed analysis of indirect effects as follows: temperature variations, the creep and shrinkage of concrete, and the settlement of supports. Although the standards [1] prescribe this type of bridge for lengths of up to 100 m, lately, such structures have also been constructed to much longer lengths (250-350 m).

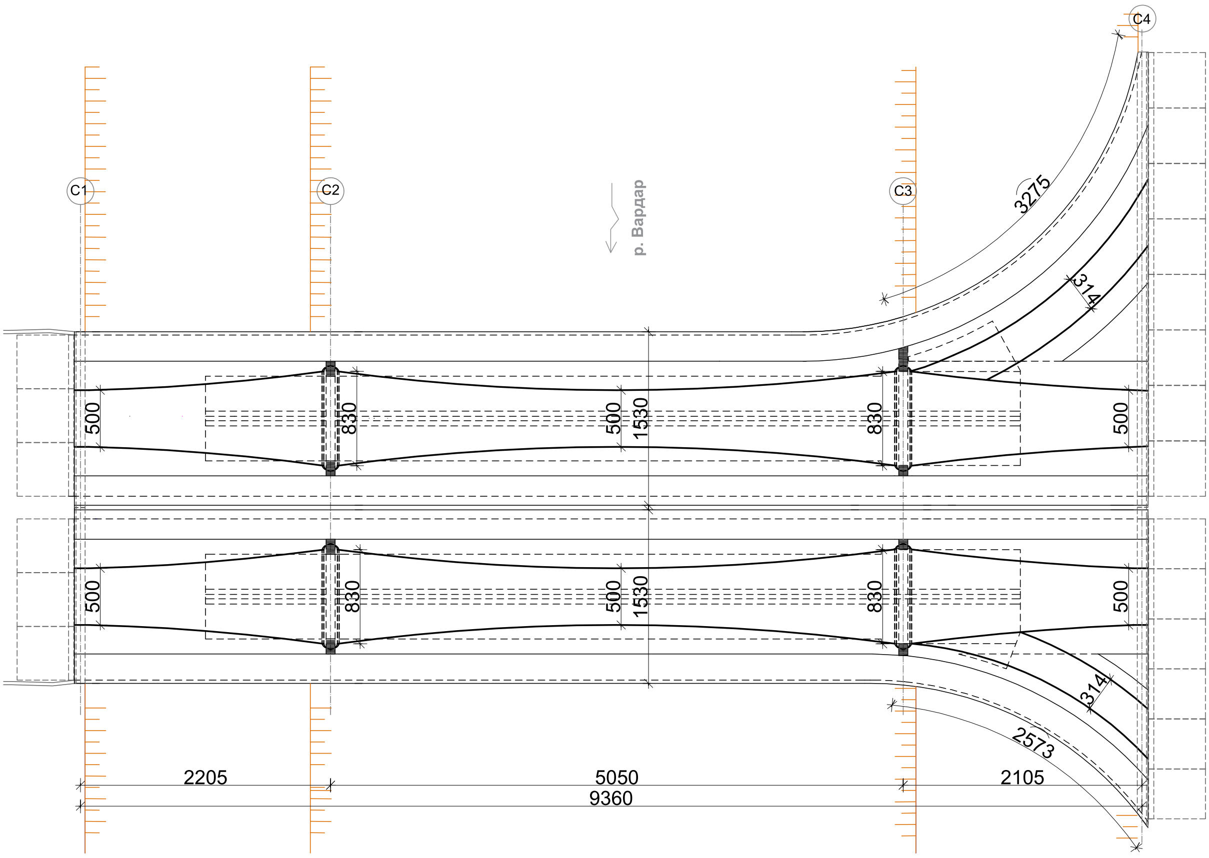


Fig. 2. Bridge plan

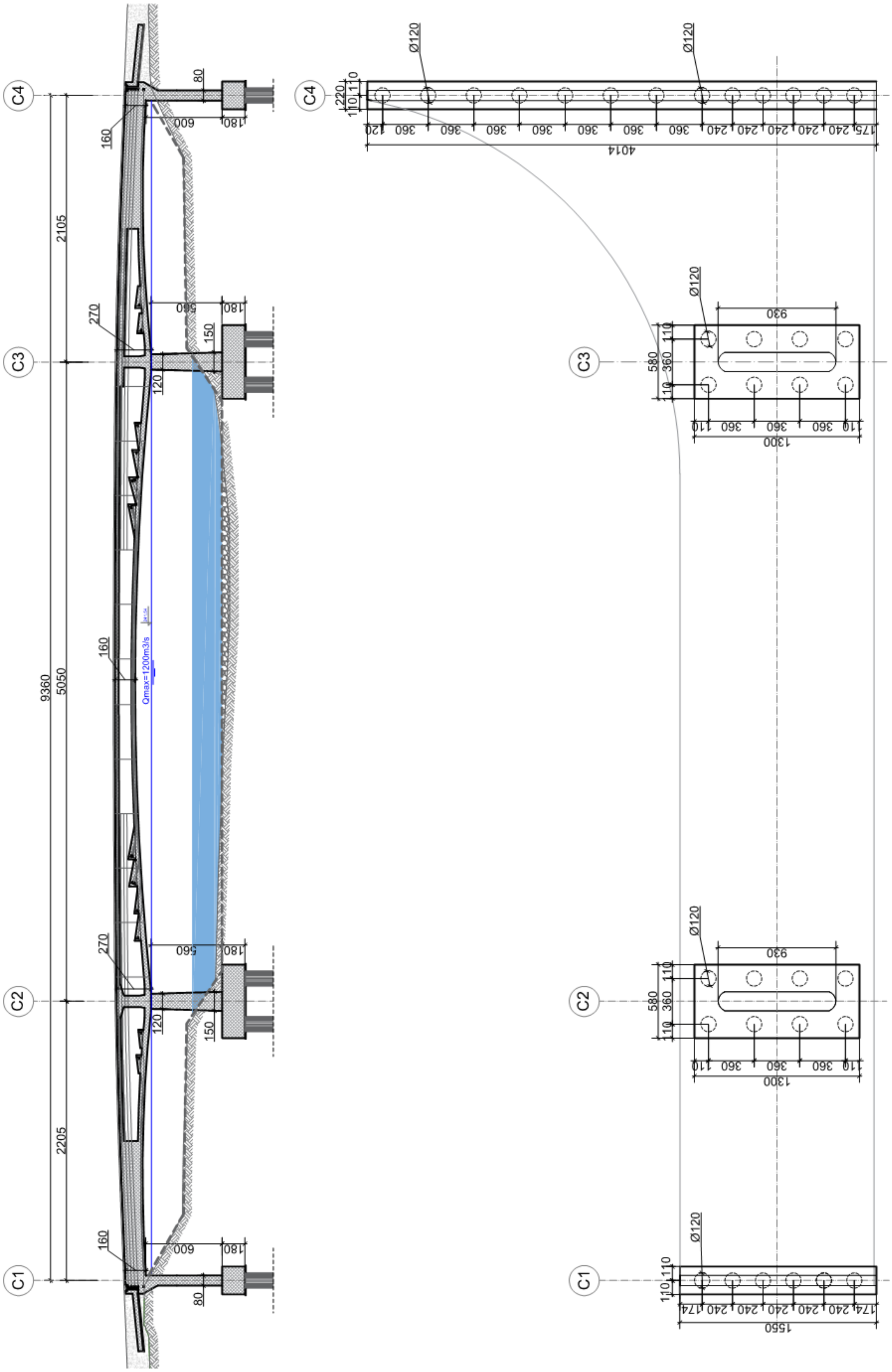


Fig. 3. Longitudinal section (left) and plan of the foundations of the left branch (right)

2.1 Superstructure

The superstructure of the bridge consists of a cast-in-situ, prestressed, double-cell box girder. It has a parabolically variable height and width of the lower deck that also leads to variable inclination of the webs (Fig. 4). The height of the cross-section in the middle of the main span and that over the abutments amounts to 1.60m (L/31.5). Over the piers, it amounts to 2.70 m (L/18.7). The adopted heights are within the recommended limits for this type of structural system. The width of the lower edge of the box girder is also variable, ranging from 5.0 m in the middle of the central span and over the abutments to 8.30 m over the piers.

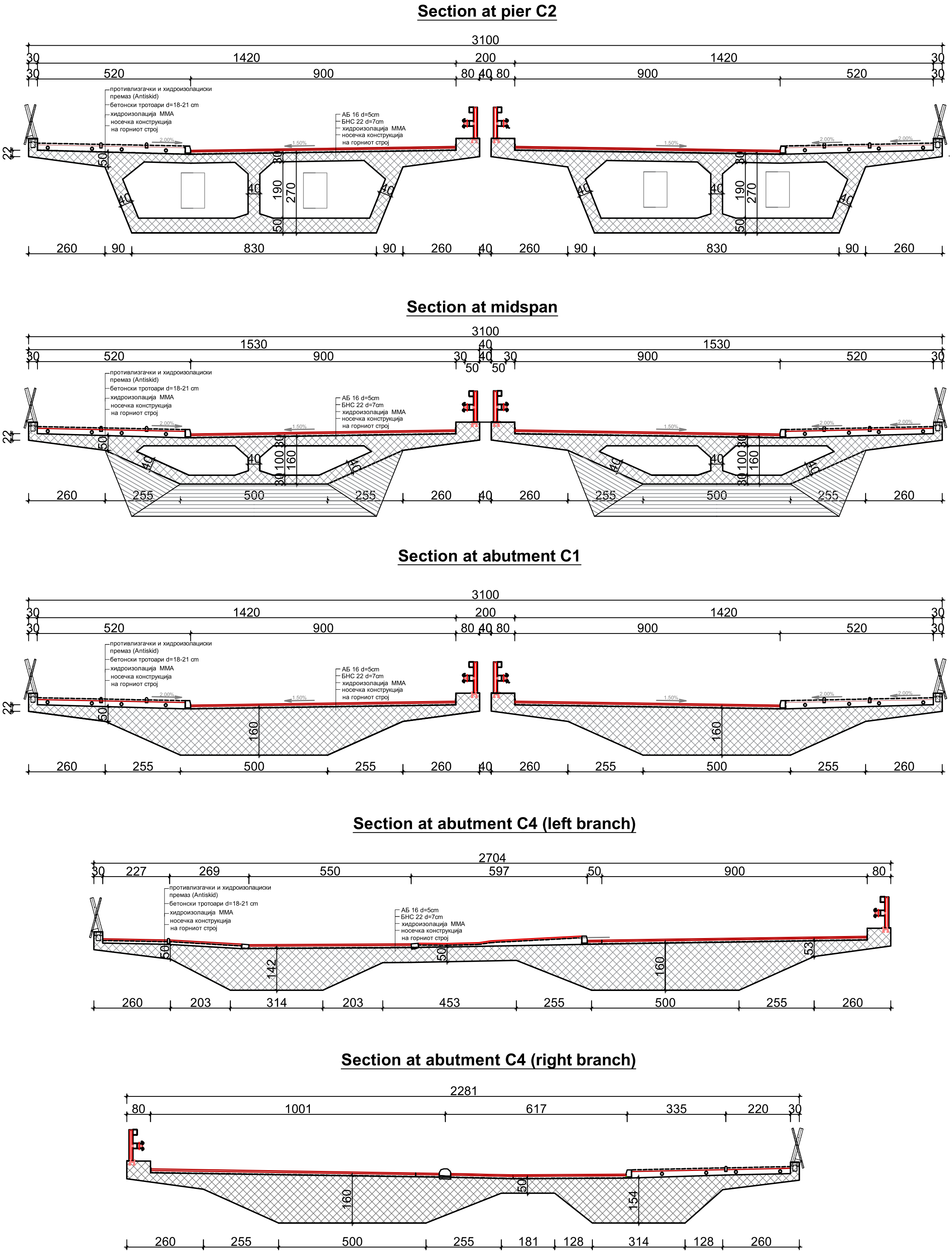


Fig. 4. Characteristic cross-sections of the bridge

The top slab of the box section has a constant thickness of 30 cm along the entire length of the structure, whereas that of the bottom slab starts at 50 cm at the supports and decreases to 30 cm in the midst of the central span. The webs have a constant thickness of 40 cm. The cantilever overhangs are at a span of 2.60 m, whereas their thickness ranges from 22 cm at the parapet beams to 50 cm at the place of fixation into the webs.

At one half of the end spans, the box girder section turns into a solid section to avoid negative reactions at the abutments in all phases of construction and serviceability.

In the part of the access ramps, the girder has a constant width of 3.14 m, whereas its height starts at 2.7 m at the central pier C3 and ends at 1.42 m at the abutment C4 on the left, i.e., 1.54 m on the right bridge branch (Fig.4).

2.2 Substructure

The substructure of each branch consists of two central piers and two abutments. The central piers are in the form of rounded walls with a width of 9.30 m and variable thickness: 1.50 m at the fixation in the pile cap, up to 1.20 m at the top. Their height from the upper edge of the pile cap to the lower edge of the main girder is 5.60 m (Fig. 3).

The abutments are rectangular in plan, with a thickness of 0.80m and a height of 6.00 m (Fig. 3). They are designed without wing walls, which contributes to their greater flexibility.

At the considered location, there are terraced alluvial sediments composed of well-granulated, well-compacted sand and powdered gravel. The foundation below all piers is represented by reinforced concrete piles with a diameter of 1.20 m. The abutments are founded on single-row piles (6 below C1 and 13 below C4) with a length of 12.0 m that are interconnected by a pile beam with a cross-section 2.20/1.80 m. Below the central piers, double-row piles (8 below C2 and C3 each) with a length of 15 m are adopted. The pile cap is 1.80m thick (Fig. 3).

2.3 Equipment

Considering the monolithic connection between the superstructure and the abutments, the adopted expansion joint is placed beyond the bridge, i.e., between the abutment and the transition slab. It enables horizontal displacements of up to +/-40mm and is proportioned for service loads (temperature, braking forces, creep, and shrinkage of concrete). The carriageway expansion joint type is Wd/Wd +80, while the sidewalk expansion joint type is TO 80 (Freyssinet) (Fig. 5).

To make the transition slab remain in a fixed position during horizontal displacement of the structure, a strip-like movable bearing between it and the short element of the abutment has been adopted. This detail has been adopted in accordance with the German Standard [1] and holds for integral bridges with a total length exceeding 50 m (Fig. 5).

For the horizontal bridge surfaces, waterproofing based on methyl methacrylate (MMA) is anticipated. This type of waterproofing consists of a primer layer and a double-layered MMA membrane (Fig. 6). It is applied over the entire bridge surface, where, in the part of the carriageway, there is a double-layered (7 + 5 cm) asphalt, whereas in the part of the sidewalks and cycling paths, monolithic sidewalks (20 cm) are cast, with a finishing anti-skidding, waterproofing layer (Antiskid).

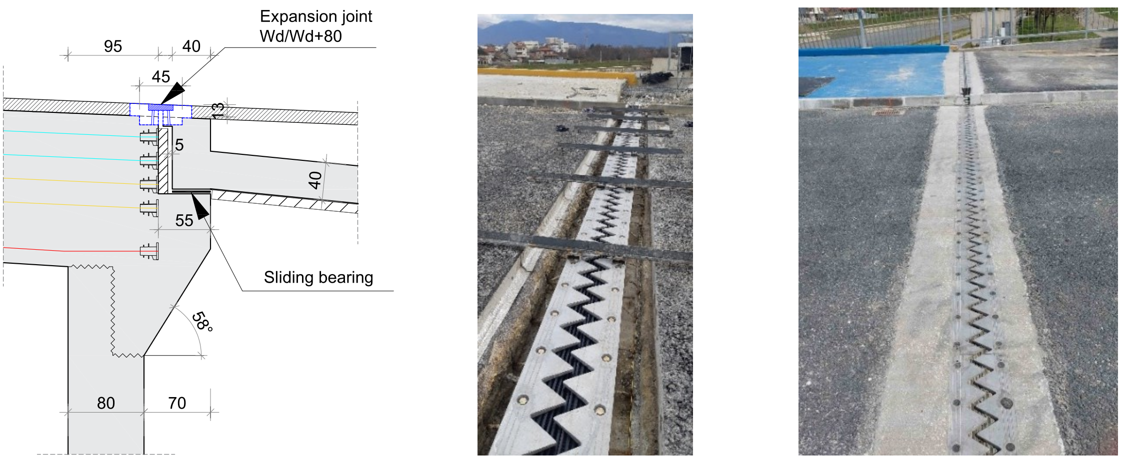


Fig. 5. Detail of an abutment, carriageway and sidewalk expansion joint

Fig. 6. Application of MMA waterfroofing and anti-skidding layer (Antiskid)

On the outside, the box girder is coated with a protective coating called Antikorozin BB. The central piers are coated with granite plates with a thickness of 5 cm.

Anticipated on the side of the footways is a specially designed for that purpose pedestrian steel fence (Prof. Dr.MitkoHadzi-Pulja, grad. architect, Fig. 7), while on the opposite side, an elastic restraint system type H2W4 according to EN 1317 [2] standard is planned.

Anticipated for drainage of atmospheric waters are ACO gullies in compliance with DIN EN 124 [3], pipes of composite material GRP (ISO 25780:2011), and water-purification shafts.

2.4 Methodology of construction

Taking into account the configuration of the river bed, a combined methodology of construction has been selected. The end spans along with the access ramp are proposed to be constructed using the traditional scaffold, whereas free cantilever construction is proposed for the central span that bridges the minor river bed. The central span is constructed in five phases, 4.3 m each from the side of piers C2 and C3. The segment for connection of the left and the right cantilevers has a length of 3.63 m (Fig. 8-10).

From the assumed methodology of construction, the final number and distribution of prestressing cables have emerged. First of all, the cables in the bottom zone of the first and third spans are prestressed (12 cables each), and then the cables in the bottom and top zone of the access ramps (10 in the bottom and 29 in the top zone) are prestressed. For each phase of construction of the central span, 10 cables are anticipated for the top zone, i.e., a total of 50 cables over piers C2 and C3 that are successively prestressed. Following the connection of the two cantilevers, prestressing of the 30 cables in the bottom zone of the central span (the so-called cables for continuation) (Fig. 9) is done.

Fig. 7. View of the constructed bridge

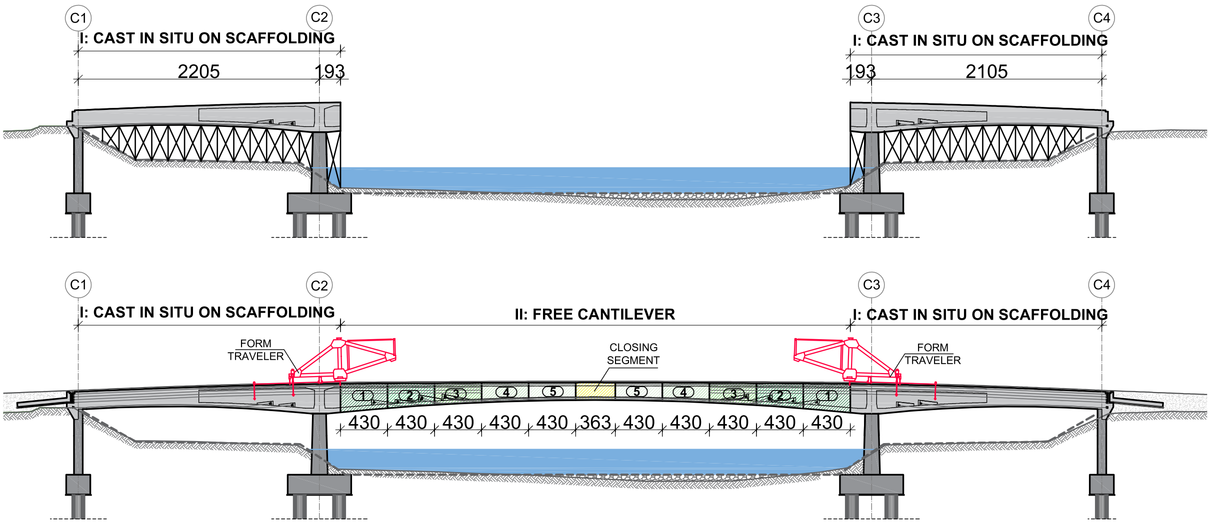


Fig. 8. Proposed methodology of construction

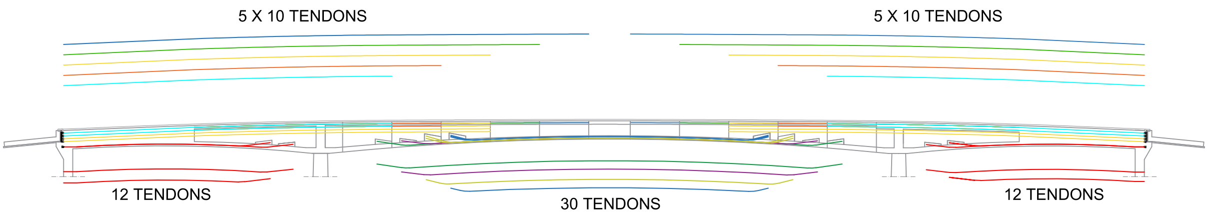


Fig. 9. Prestressing cables in longitudinal section

Fig. 10. Methodology of construction

Anchorage of top cables is done at the end of the constructed segments, while in the case of those for continuation, anchorage blocks are on the bottom slab of the box section.

Detailed numerical analysis has been done for the adopted methodology of construction.

2.5 Numerical model

Complete static and dynamic analysis of the bridge has been done in accordance with the Eurocodes [4-10] by use of the SOFiSTiK software. For designing the structure in the longitudinal direction, a 3D model was built. In this model, all structural elements have been modeled as beam finite elements (Fig. 11). The model also includes the prestressing cables with their real geometry for the purpose of more accurate computation of prestressing losses. The elements have been divided into groups depending on the construction phases in which they are activated. The change in the static system has also been taken into account.

Due to the specificity of the system (an integral bridge), the piles under each pier position have been incorporated into the model. Their deformability has been included through horizontal and vertical springs placed along their perimeter and a spring at the lower base.

Since the access ramp and the main girder of the third span are monolithically connected, their joint behavior has been simulated through mutual connection with rigid elements (“constraints”).

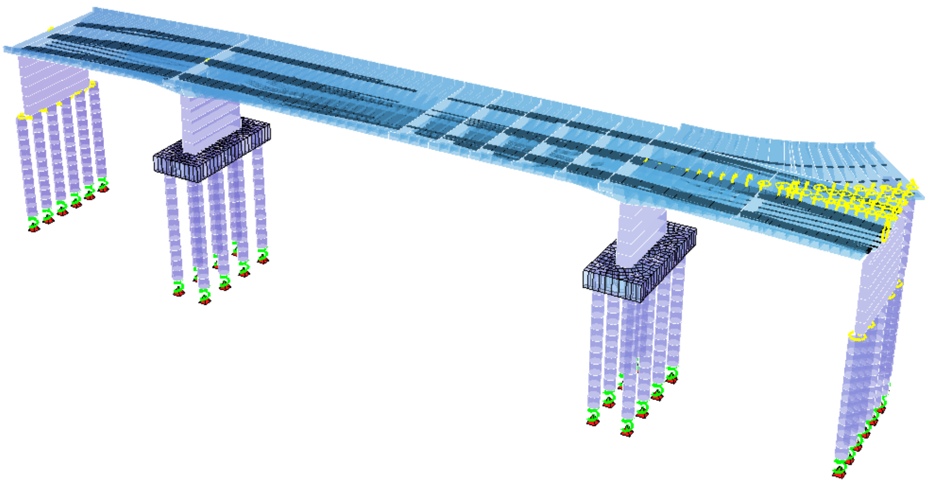


Fig. 11. Numerical model of the bridge in SOFiSTiK

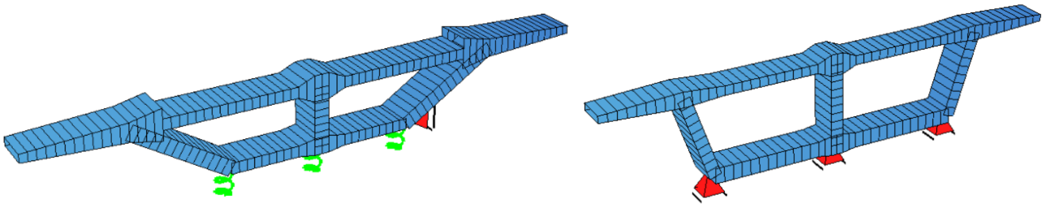


Fig. 12. Numerical model of a single segment in span and above support

Taking into account that this model cannot be used to describe the behavior of the box section in a transverse direction, two additional models of a segment of the box with a unit length have been built. One of these describes the behavior of the box section in span, and it is therefore supported by elastic springs whose stiffness is defined through the vertical deformation at that section. The other model is for the segment above a support, and it rests on rigid supports (Fig. 12). All of the box girder's individual elements are represented by beam-finite elements.

2.6 Analysis

Several different programs were used for the analysis of the structure. The design of the pile foundations was performed in multiple iterative steps between the numerical models built in SOFISTiK and GEO 5. The superstructure was designed separately in longitudinal and transverse directions since the beam model was not sufficient for both. The number, the disposition, and the force in the tendons were defined by the main model and for the most part dictated by the methodology of construction. The segmental construction required analysis of the following construction stages for each segment: casting of the segment, prestressing of top tendons, creep and shrinkage, removing of the traveler formwork, and its placement at the end of the finished segment. The non-prestressed reinforcement in the transverse direction of the box girder was obtained from the analysis carried out on single segments at midspan and at support (Fig. 12). Finally, the load bearing capacity at the cross-sectional level was checked in the auxiliary software CUBUS Fagus 8 using the final dimensions of the box girder and the adopted tendons and non-prestressed reinforcement.

3 Ljubljanska street extradosed bridge

The bridge on Ljubljanska street over the Vardar river in Skopje represents part of the project on the connection of Ilinden Boulevard with Slovenechka Street in Karposh municipality (Fig. 13).

The selected structural solution represents a structure supported from above by parallel stay cables. Considering the configuration of the river bed in the area of the bridge, two variant solutions have been analyzed. The one includes two spans and a higher pylon, whereas the other has three spans and two lower pylons.

Considering that a higher pylon and hence steeper cables limit the line of sight, imposed as the final solution is a structure over three spans (27.5 + 56 + 27.5 m),with two lower pylons and parallel cables. The total length of the bridge is 111 m, while its width is 24.1 m (Figs. 14, 15). It includes 4 traffic lanes of 3 m each, 2 footways and cycling paths of 4 m each, 2 parapet beams for anchorage of the pedestrian railing, and 2 x 1.75 for accommodation of the pylons and the New Jersey restrained system.

The adopted system is known as “extradosed” (a hybrid solution between a traditional girder and a cable-stayed bridge). The main characteristic by which this system is visually distinct from cable-stayed bridges is the small height of the pylons and the mild inclination of the cables. Another specificity of “extradosed” bridges is the higher stiffness of the superstructure compared to that of the cable-stayed bridges and a lower stiffness compared to that of the girder bridges. The higher stiffness contributes to a slight activation of the stay cables under traffic loads that consequently leads to minor changes in the stresses in the cables. Hence, in respect to vertical effects, the cables of “extradosed” bridges mainly sustain the dead loads of the carriageway structure. Therefore, the level of cable stress in these bridges is allowed to be higher compared to that of cable-stayed bridges.

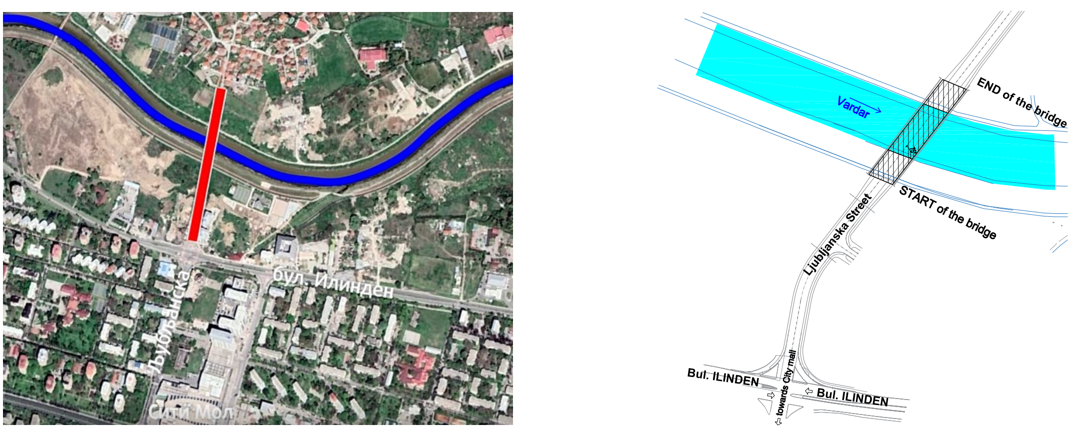


Fig. 13. Position of the bridge on the site plan

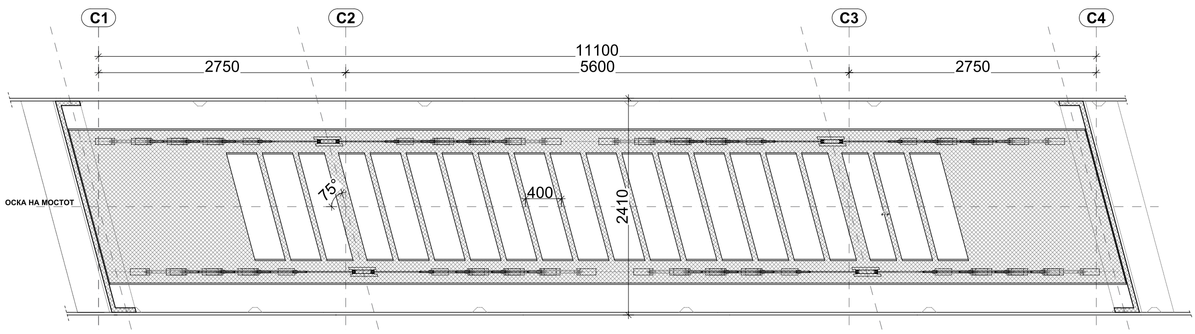


Fig. 14. Bridge plan

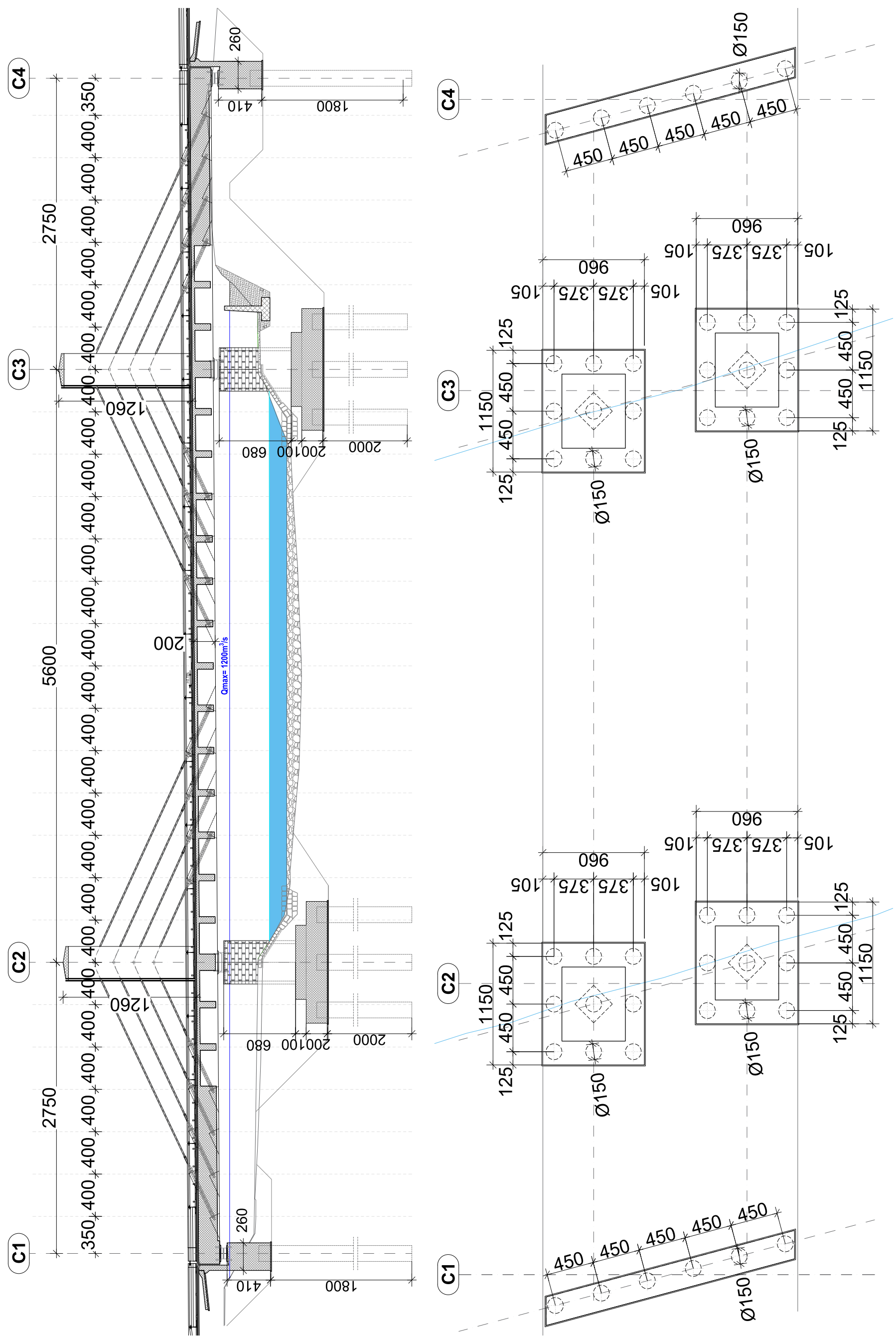


Fig. 15. Longitudinal section of the bridge (left) and foundation plan (right)

Although it is considered that this solution is optimal for bridges with spans of 100 to 200 m, “extradosed” bridges have lately also been constructed for smaller spans, as are the Bergwijk Bridge in Belgium and several overpasses and bridges in Poland [11].

3.1 Structural Elements

The bridge superstructure represents a monolithic ribbed deck “suspended” by parallelly installed 32 stay cables, which is composed of two prestressed main girders, reinforced concrete cross girders, and a bridge deck. The main girders have a constant height of 2 m (L/28). Their cross-section is of a trapezoidal shape with a width of 2.5 m at the lower edge and a width of 2.8 m upwards connection with the bridge deck (Fig. 16). They are placed at an axial distance of 14.5 m.

The middle cross girders have a rectangular cross-section of 0.60/1.80 m and are placed at an interdistance of 4.0 m. They intersect with the main girders at an angle of 75˚ (Fig. 17). The two cross-girders over the central piers have larger dimensions, i.e., 1.5/1.8 m.

The bridge deck has a constant thickness of 30 cm. In the end spans, it increases to 1.8 m (Fig. 17) which enables the avoidance of negative reactions at the abutments in all phases of construction and serviceability. The cantilevers have a span of 3.10 m, while their thickness ranges from 22 cm at the parapet beams to 50 cm at the place of fixation into the main girders. The bridge deck has a constant thickness of 30 cm.

The stay cables are distributed over two parallel planes, where the pylons that are monolithically connected to the main girders (Fig. 16) are positioned over each bearing placed on the central supports. They are designed as reinforced concrete ones with a cross-section of 1.0/3.0 m and a total height of 12.6 m from the carriageway. Due to the limited width of the cross-section, anchorage of the cables is enabled through the so-called saddles (deviators). The height of the pylons to the saddle of the last cable is 9.63 m (L/H=5.8). In the transverse direction, both individual pylons are connected with portal-pretensioned simple supported beams (3.0/0.25-0.5 m).

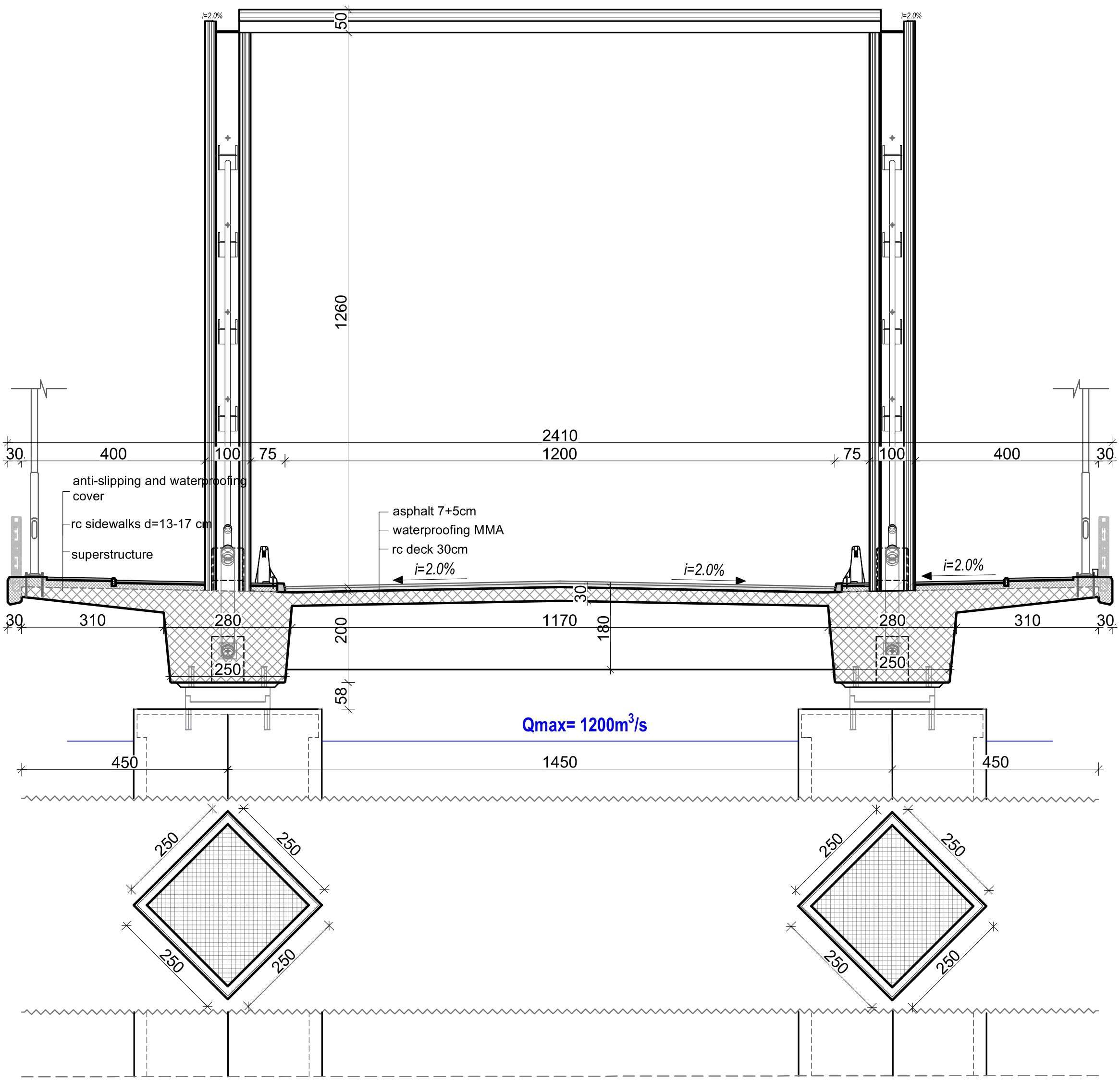


Fig. 16. Characteristic cross-section of the bridge

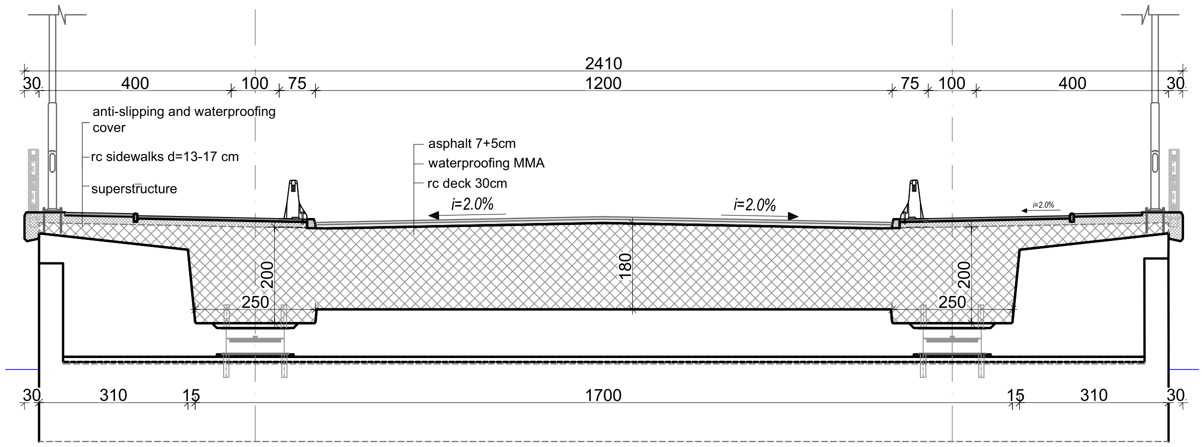


Fig. 17. Characteristic cross-sections in end spans of the bridge

The cable system is composed of a total of 32 mutually parallel stay cables, 8 at each pylon. Passing continuously over the pylon saddles, they are anchored into the superstructure on both sides, at places where main girders are connected with cross girders. The detail of the anchors and anchorage of the cables into the pylon and the main girder is shown in Fig. 18.The adopted type of cables has a three-layer anticorrosion protection, which is of great importance for the durability of such systems. Each strand of a cable is galvanized, and upon it, an external barrier of a polyethylene coat of high density (HDPE) filled with paraffin wax is directly extruded. The cable composed of such protected strands is housed in individual HDPE external protection tubes. Additionally, antivandal tubes are anticipated for the lower part of the cables.

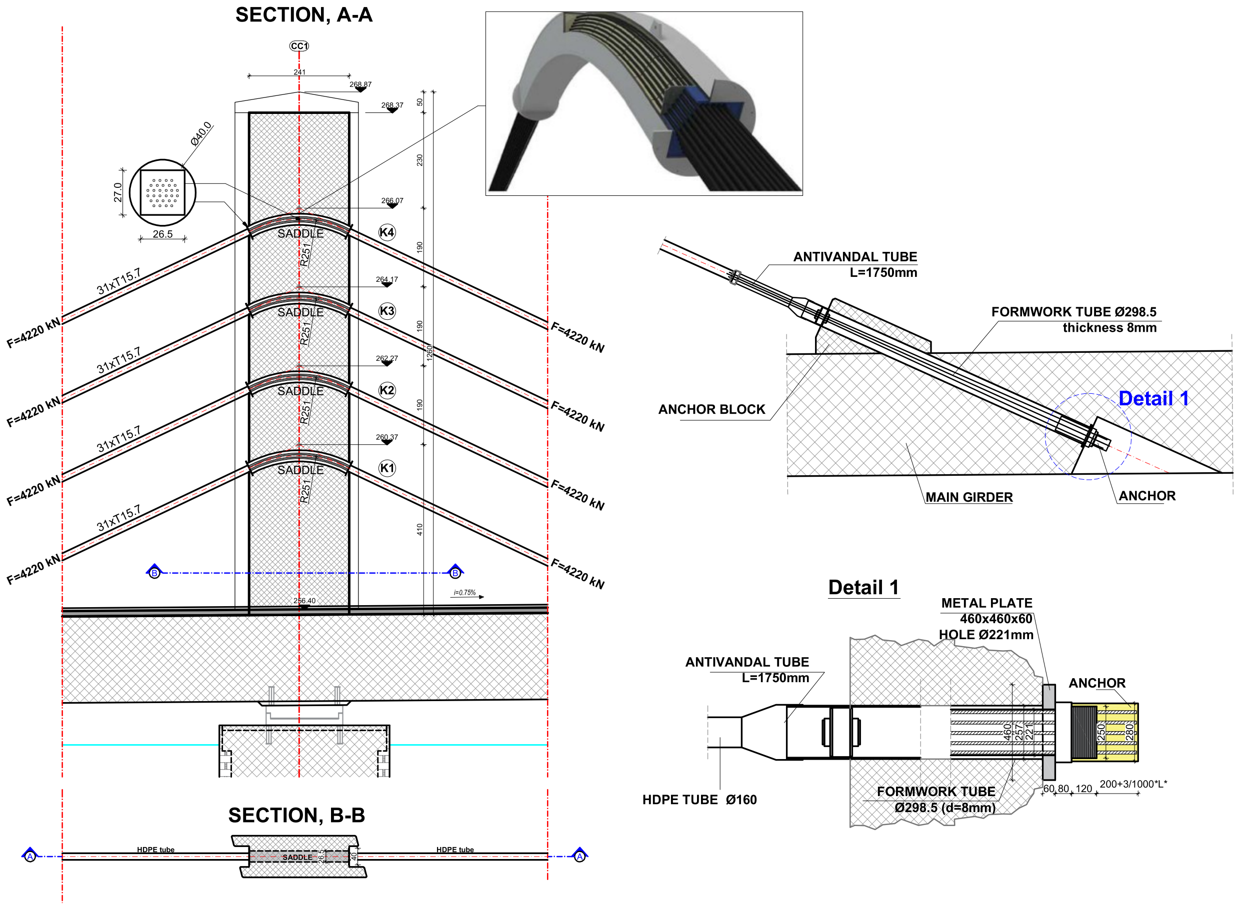


Fig. 18. Detail of a pylon, saddle and anchorage of a cable into the main girder

The hinged connection between the superstructure and substructure is provided through movable and fixed pot bearings. Over the central piers, a total of four fixed bearings are placed. Four longitudinally movable bearings are placed over the abutments. The distribution of movability of the bearings has arisen from the dimensioning of the substructure for the effect of horizontal forces. Their connection to the super- and sub-structure is through steel anchors.

The bridge substructure consists of two abutments and two central piers placed parallel to the Vardar River bed. The abutments that, at the same time, have the role of a pile beam, have a constant thickness of 2.6 m due to the relatively small height (4.1 m). They are designed without wing walls.

The middle piers have a square cross-section that, due to hydraulic and aesthetic reasons, is rotated 45° with respect to the longitudinal axis of the bridge. Each middle pier consists of two individual piers proportioned to 2.5/2.5 m (Fig. 16). Their height from the upper edge of the pile cap to the lower edge of the bearing is 6.80 m. In the upper 50 cm, widening of the cross-section for 15 cm is anticipated on all sides. This is necessary to provide the space necessary for accommodation of the hydraulic jacks during replacement of the bearings.

On the surface of the considered location, there are shallow terraced alluvial sediments composed of gravels, whereas in the deeper layers, there are Miocene sediments classified as marlstone. Deep foundation by application of piles with diameter of 1.5 m has been adopted. Considering this, the deeper layers, specifically the marlstone, play a dominant influence in the behavior of the pile foundation. The abutments are founded on single-row piles (6 below C1 and C4 each) with a length of 18 m. Below each central pier, 3 x 3 piles with a length of 20m are anticipated. They are interconnected by a cascade pile cap, whose thickness amounts to 2.0/3.0 m (Fig. 15).

3.2 Equipment

Based on the computed horizontal displacements of the superstructure due to the total service and seismic loads, two expansion joints over the abutments are anticipated. These enable displacements of up to +/-115 mm. For the carriageway surface, an expansion joint type Wd/Wd + 230 has been adopted, whereas type PL 230 (Freyssinet) has been adopted for the sidewalks. While adopting these expansion joints, the inclination of the bridge at plan has been taken into account.

With the resulting ultimate responses and horizontal displacements of the superstructure, the following types of pot bearings have been adopted (Fig. 19):

- Over the abutments: type GG (movable in longitudinal direction and fixed in transverse direction) with proportioned A/B/H = 1210/1030/550 mm, maximum vertical bearing capacity of V_{ULS} = 20 000 kN, horizontal bearing capacity ofH_{ULS} = 6 000kN and maximum capacity for horizontal displacement of 200 mm.
- Over the central piers: type FX (fixed in all directions) proportioned D/H = 1850/481 mm, with maximum vertical bearing capacity of V_{ULS} = 45 000 kN and horizontal bearing capacity of H_{ULS} = 13 500 kN.

For the vertical elements that are in contact with the external environment, waterproofing in the form of bitumen-based coating is anticipated. For the horizontal bridge surfaces, waterproofing based on methyl methacrylate (MMA) has been adopted. This type of waterproofing consists of a primer and a double-layered MMA membrane. It is applied to the entire bridge surface by way of a cold procedure, where in the carriageway part, there is a double-layered (7+5 cm) asphalt, whereas in the part of the footways and cycling paths, monolithic sidewalks (13-17 cm) are cast, with a finishing anti-skidding and waterproofing layer (Antiskid). The central piers are lined with granite plates with a thickness of 5 cm.

On both sides of the bridge, between the carriageway and the sidewalks, a New Jersey restrained system is planned to be constructed according to the EN 1317 standard [2]. The primary role of this restrained system is the protection of the cables and the pylons against direct impact by vehicles. Additionally, it separates the motor traffic from the pedestrian traffic on the bridge. The shaping of the pedestrian steel fence, the pylons, and the central piers as well as the line illumination of the pylons are works of Prof. Dr. MitkoHadzi-Pulja, grad. arch. (Fig. 20).

ACO gullies in accordance with DIN EN 124 [3], pipes made of composite material GRP (ISO 25780:2011 [12]), and water purification shafts are anticipated for drainage of atmospheric waters

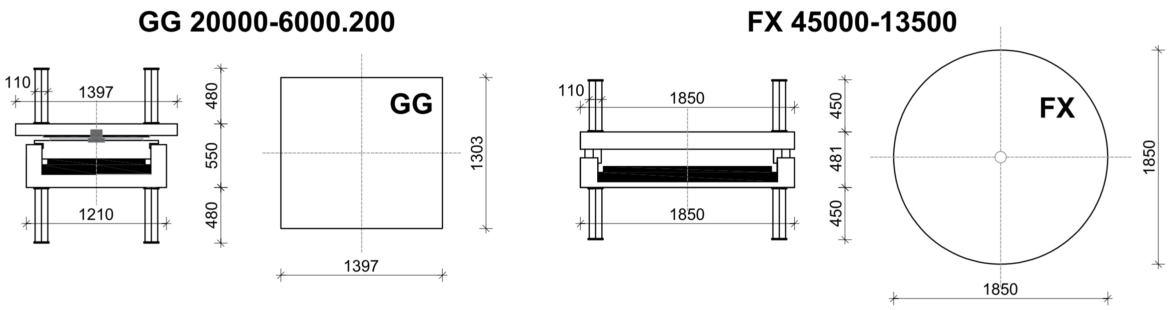


Fig. 19. Detail of the adopted pot bearings

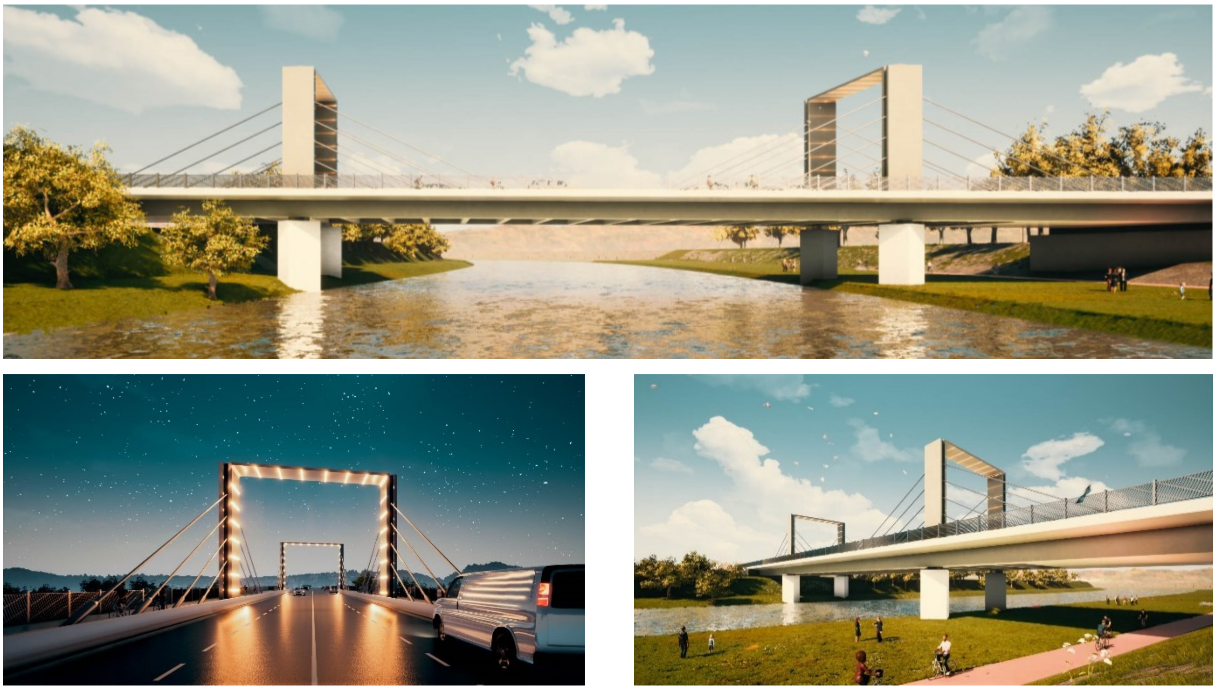


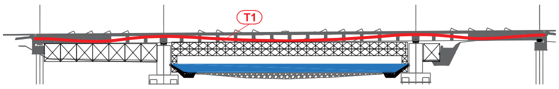
Fig. 20. 3D View of the bridge (Nikola Strezovski, grad. arch.)

3.3 Methodology of construction

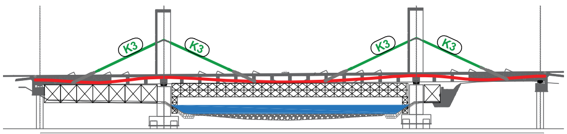
Monolithic construction of the superstructure and its simultaneous concreting are anticipated to be done by use of an integrated scaffold. Anticipated for the central span is the so-called "heavy" scaffold that can entirely bridge the minor river bed, resting on temporary supports placed immediately next to the central piers. After the sub- and superstructures are constructed, the following phases start to be realized (Fig. 21):

- Prestressing of all internal tendons by the full amount of the anticipated prestressing force (14 cables on each main girder) from the side of the abutments;
- Partial tensioning of the second (according to length) external cable K3;

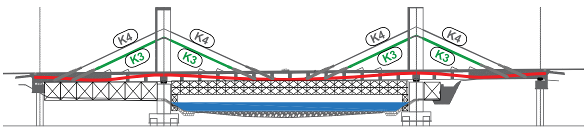
PHASE I: Prestressing of the internal tendons



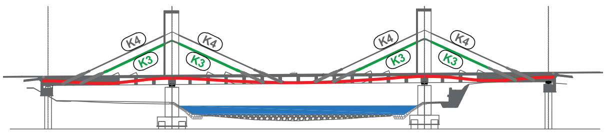
PHASE II: Partial stressing of K3 (1500kN)



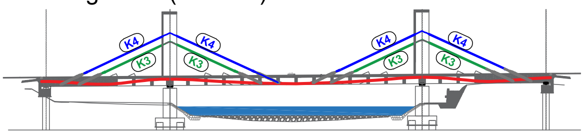
PHASE III: Instalation of K4 (0kN)



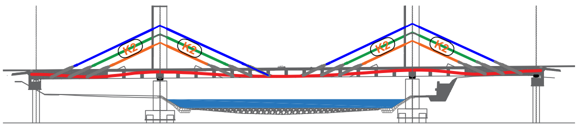
PHASE IV: Dismantling of the scaffold



PHASE V, VI: Restressing of K3 (2350kN) and partial stressing of K4 (3850kN)



PHASE VII: Partial stressing of K2 (3850kN)



PHASE VIII: Application of superimposed dead load

PHASE IX-XIII: Restressing of all cables to the full amount of 4220 kN

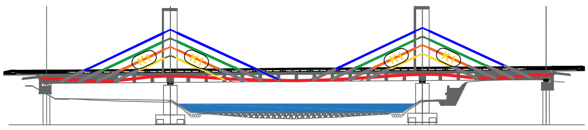


Fig. 21. Proposed methodology of construction

• Placement of the longest external cables K4;
• Dismantling of the scaffold and activation of the entire dead weight of the superstructure;
• Additional external cable tensioning K3 and partial tensioning of K4 and then K2;
• Partial tensioning of K1 after application of the superimposed dead loads (asphalt, sidewalks, railings and alike) and additional tensioning of all cables up to the anticipated force of 4220kN.

The proposed construction methodology is preceded by a detailed numerical analysis that controls the stresses in the bridge superstructure throughout the construction process.

3.4 Numerical model

Using the SOFiSTi software, a complete static and dynamic analysis of the bridge was performed in accordance with Eurocode. For the proportioning of the structure, a 3D model has been formulated. In this model, all structural elements with the exception of the bridge deck have been modeled by linear (beam) finite elements (Fig. 22). The main girders are modeled in T-section and include the effective bridge deck in longitudinal direction. The variation of the effective width along the length of the girders has been taken into account in accordance with EN 1992-1-1 [13]. Since one part of the deck is already included in the flange of the main girders, for these parts, the shell elements by which the bridge deck is modeled are defined without dead weight (γ=0 kN⁄m^3). To prevent doubling of the deck stiffness in longitudinal direction, the shell elements have axial and bending stiffness only in the transverse direction. Beam elements with a rectangular cross-section are used to model the cross-girders. At places where they enter the main girders, they are modeled without dead weight.

For the external cables, linear (cable) elements are used that sustain axial forces only. The pot bearings through which the superstructure rests upon the substructure by way of a hinge, are modeled by spring elements with corresponding stiffness. In the model, the internal prestressing tendons are included through a resultant cable with a sinusoidal run.

The elements are divided into groups depending on the phases of construction in which they are activated. For cable-stayed structures that are constructed monolithically by using the traditional scaffold, it is of particular importance to simulate the separation of the structure from the scaffold and the gradual activation of the dead weight during the phase tensioning of the cables. In the analysis, this has been simulated through nonlinear springs placed below the entire surface of the superstructure that are able to sustain compressive forces only.

To simulate a deformable pier, the model also includes the piles and their interaction with soil. For that purpose, horizontal and vertical springs along the perimeter of the piles as well as a spring on the lower base have been modeled. The stiffness of the springs has been defined by means of the GEO-5 software.

3.5 Analysis

The final type of stay cables (31T15) and 14 internal tendons per girder (19T15) were adopted based on results from both, analysis of construction stages and the analysis of service life of the bridge. The sequences of stay cable stressing, as well as the moment of scaffold dismantling, arose from limiting the stresses in the superstructure during the construction and its service life. Having in mind the sensitivity of the structural system, two additional checks were carried out: replacement of one stay cable and brakeage of the cable due to the vehicle impact. Due to the applied system for anchoring the stay cables in the pilon through the so-called saddle, the maximum allowable differential forces in the cable on two sides of the pilon were controlled. This check was performed in accordance with fib89 [14] and the recommendations of PTI [15] for such structures.

4 In lieu of a conclusion

Bridges are undoubtedly considered among the most impressive works of construction, a synonym for structural engineering. A fruit of primeval creative exaltation, ample engineering knowledge, and spiritual love for the profession. Indicators of the vision of rulers, the intellectual and technological power of nations, the maturity of cities, the importance of states, the development of civilizations. A symbol of human unsubmissiveness to obstacles. A victory of connection over separation.

The design of the “Mihajlo Apostolski” bridges and Ljubljanska street over the Vardar river in Skopje was a challenge to create something new, moving away from destructive iterativeness, encouraging the self-confidence of builders. Each step, no matter how modest, creates faith in the dialectics of the human being, nurtures love toward creation, and initiates hope for new achievements.

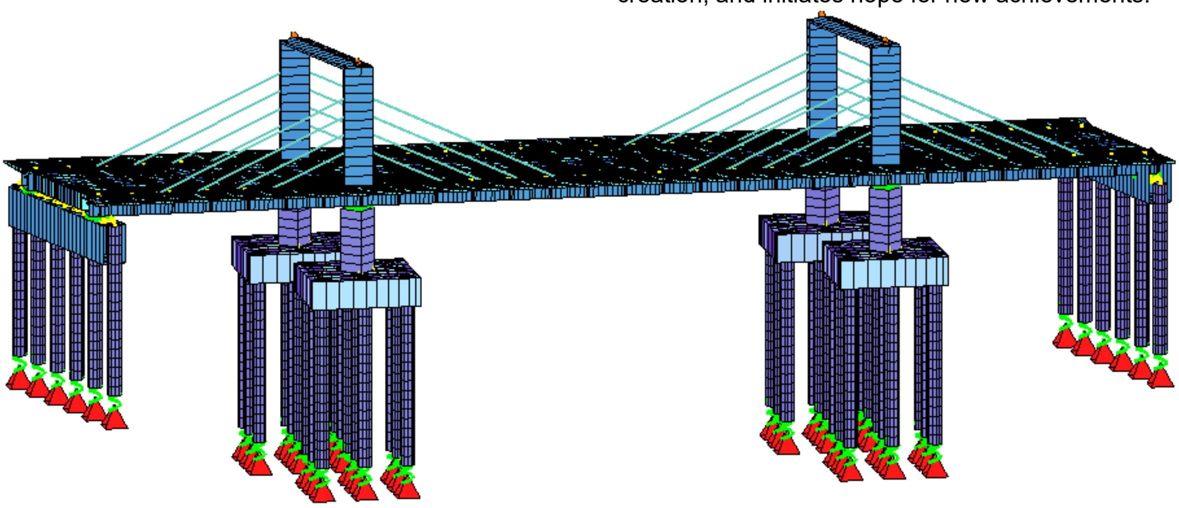


Fig.22. Numerical model of the bridge in SOFiSTiK

Acknowledgement

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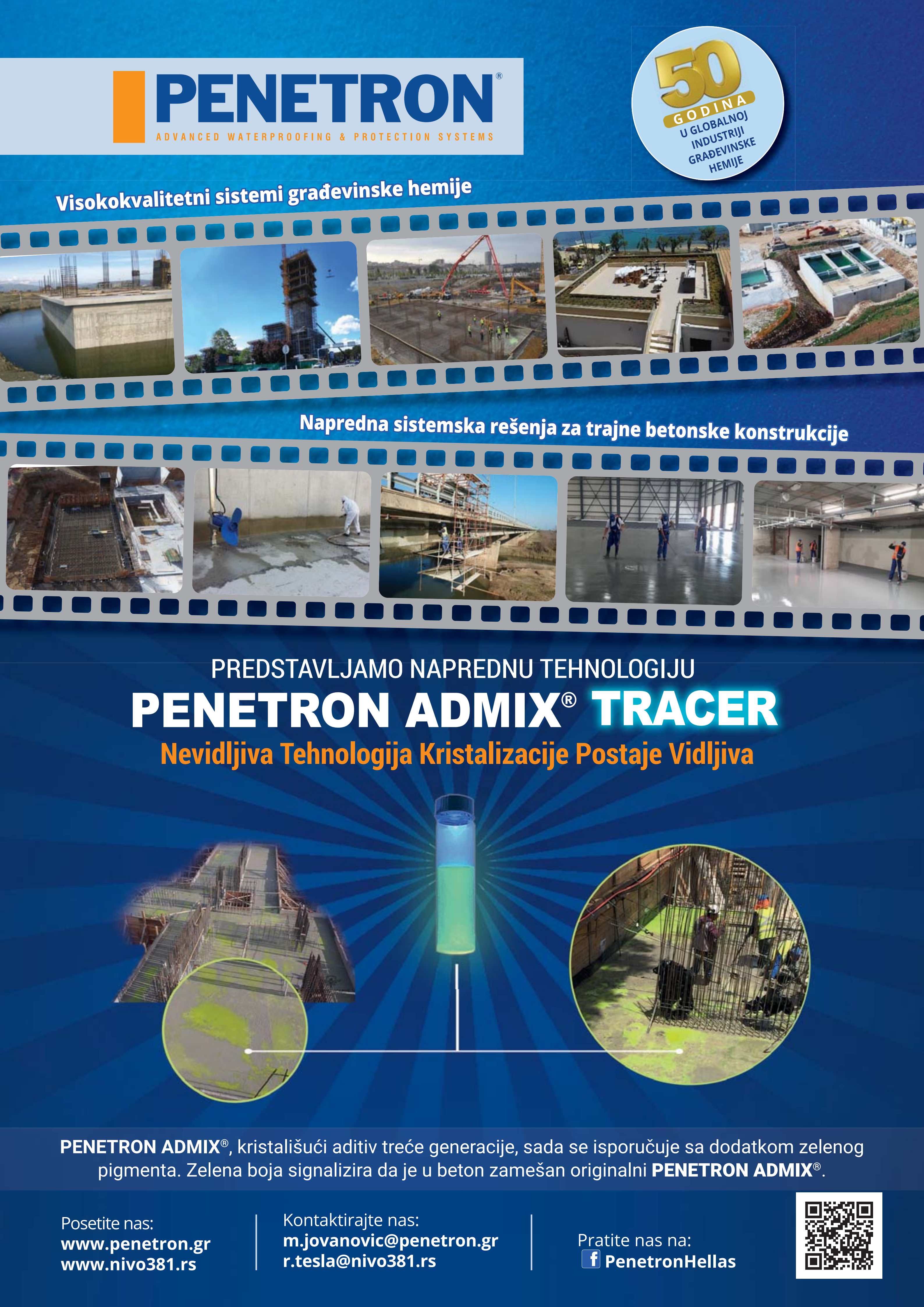

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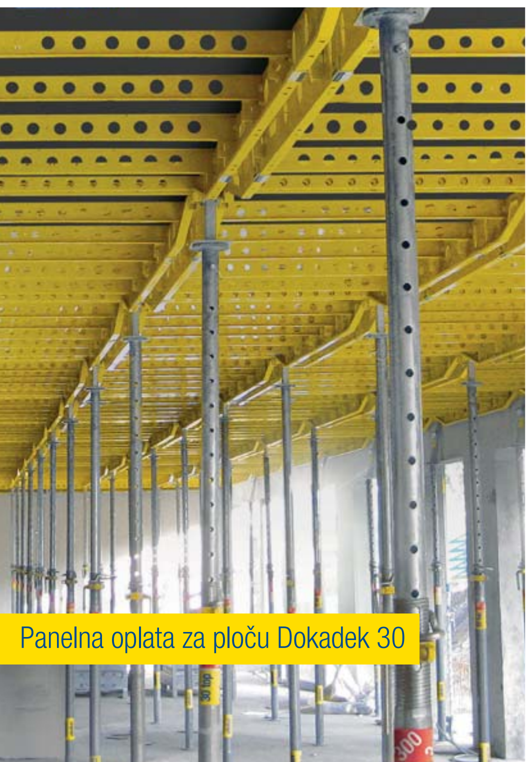

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Put inženjering d.o.o punih 25 godina radi kao specijalizovano preduzeće za izgradnju infrastrukture u niskogradnji i visokogradnji, kao i proizvodnjom kamenog agregata i betona. Preduzeće se bavi i transportom, uslugama građevinske mehanizacije i specijalne opreme.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

NIŠ
Knjaževačka bb, 18000 Niš - Srbija
+381 18 215 355
office@putinzenjering.com

BEOGRAD
Jugoslovenska 2a, 11250 Beograd – Železnik
+381 11 25 81 111
beograd@putinzenjering.com

www.putinzenjering.com

MOBECO