ISSN 2217-8139 (Print)
ISSN 2334-0229 (Online)

UDK: 06.055.2:62-03+620.1+624.001.5(497.1)=861

DIMK

2019.
GODINA
LXII

GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

2

BUILDING MATERIALS AND STRUCTURES

ČASOPIS ZA ISTRAŽIVANJA U OBLASTI MATERIJALA I KONSTRUKCIJA
JOURNAL FOR RESEARCH OF MATERIALS AND STRUCTURES

DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE
SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA

ТЕХНИЧАРСКА ДРУЖИНА
1868 2018

САВЕЗ ИНЖЕЊЕРА И ТЕХНИЧАРА СРБИЈЕ

ПОВЕЉА

Часопис

"Грађевински материјали и конструкције"

издавач Друштво за грађевинске материјале и конструкције Србије

ЗА НАЈБОЉУ ПУБЛИКАЦИЈУ СРБИЈЕ

БЕОГРАД,

02. фебруар 2018.

ПРЕДСЕДНИК

др Игор Марић

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INTERNATIONAL EDITORIAL BOARD

PUBLISHER

Society for Materials and Structures Testing of Serbia, 11000 Belgrade, Kneza Milosa 9
Telephone: 381 11/3242-589; e-mail:dimk@ptt.rs, veb sajt: www.dimk.rs

REVIEWERS: All papers were reviewed

KORICE: Primeri primene EHS preseka: (a) objekat Ziman; (b) aerodrom Barajas; (c) pešački most u Škotskoj prema T.M. Chan i dr.
COVER: Examples of EHS application: (a) Zeeman building; (b) Barajas airport building; (c) pedestrian bridge in Scotland, after T.M.Chan and all.

Štampa/Print: Razvojno istraživački centar grafičkog inženjerstva, Beograd

Publikacija: tromesečno
Edition: quarterly

Financial supports: Ministry of Scientific and Technological Development of the Republic of Serbia

ISSN 2217-8139 (Print)
ISSN 2334-0229 (Online)

GODINA LXII - 2019.

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SADRŽAJ

CONTENTS

CIP - Каталогизација у публикацији
Народна библиотека Србије, Београд

620.1

GRAĐEVINSKI materijali i konstrukcije : časopis za istraživanja u oblasti materijala i konstrukcija = Building materials and structures : journal for research of materials and structures / editor-in-chief Radomir Folić. - God. 54, br. 3 (2011)- . - Belgrade : Društvo za ispitivanje i istraživanje materijala i konstrukcija Srbije = Society for Materials and Structures Testing of Serbia, 2011- (Beograd : Razvojno istraživački centar grafičkog inženjerstva). - 30 cm

Dostupno i na:
http://www.dimk.rs/stg/website/filemanager/files/Casopis_1_2011.pdf. - Tromesečno. - Tekst na srp. i engl. jeziku. -
Je nastavak: Materijali i konstrukcije = ISSN 0543-0798. -
Drugo izdanje na drugom medijumu: Građevinski materijali i konstrukcije (Online) = ISSN 2335-0229
ISSN 2217-8139 = Gradevinski materijali i konstrukcije
COBISS.SR-ID 188695820

PROBABILITY AND DETERMINISM IN BRIDGE MANAGEMENT

VEROVATNOĆA I DETERMINIZAM U UPRAVLJANJU MOSTOVIMA

ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK:624.21 (73)
doi:10.5937/GRMK1902003Y

Bojidar YANEV

1 INTRODUCTION: THE LIMITS OF CERTAINTY

A fundamental notion governing scientific research since antiquity was stated by Pierre-Simon, marquis de Laplace (1749 - 1827) in his Theorie Analytique des Probabilités as follows:

"Given for one instant an intelligence which could comprehend all the forces by which nature is animated and the respective positions of the beings which compose it... nothing would be uncertain..." Probability therefore compensated for a lack of knowledge.

In his Discours sur la Methode, Rene Descartes (1596 - 1650) judged philosophy by the same standard of ultimate determinacy. "I shall say nothing of philosophy, except that it has been cultivated by the most excellent minds that ever lived over the centuries, and that there still remains to be found something that is not disputed, and consequently doubtful." Descartes found people's opinions diverse "not because some are more reasonable than others, but only because their thinking follows different routes and does not consider the same things". A rigorous deductive method would avoid errors and he developed it.

Opinions continued to diverge however. Blaise Pascal (1623 – 1662) found the Cartesian method "largely true", but "useless and uncertain", "mechanical", and "painful". He cautioned that "the heart has reasons that reason knows not". Gottfried Wilhelm Leibnitz (1646 – 1716) estimated that analytic geometry owed more to the genius of Descartes than to his method. Leonard Euler (1707 - 1783) distanced himself from absolute certainty by classifying it into perceptual, demonstrative and moral.

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Whereas the assumptions of causality, determinacy and certainty governed the model of nature, they clearly did not apply to human transactions and games of chance. Beginning with Pascal and Pierre de Fermat (1601? – 1665), the list of illustrious mathematicians investigating problems with uncertain outcomes includes Christiaan Huygens (1629 – 1695), Jacob Bernoulli (1654 – 1705), Abraham de Moivre (1667 – 1754), Laplace, Thomas Bayes (1701 – 1761), Carl Friedrich Gauss (1777 – 1855), Andrey Markov (1856 – 1922), and Andrey Kolmogorov (1903 – 1987), among others. The many diverse and brilliant contributions formed the branch of mathematics loosely known as "probability". In lieu of definition, Bruno de Finetti (1906 –1985) began his comprehensive text on the subject with the following sentence in capitals: "PROBABILITY DOES NOT EXIST."

The numerous definitions and interpretations of the "non-existent" probability converge into two main schools. *Objectivists* or *frequentists* equate the probability of a random occurrence with the relative frequency of outcomes over a large sample. A special case is the "propensity" of occurrence, given a single event. *Subjectivists*, including Bayes and de Finetti, treat probability as a "degree of belief", based on expert knowledge which begins with a prior probability distribution and results in a posterior one. As in the case of determinism, both methods benefit from the accrual of unbiased information. Rather than tending to certainty however, their results approach likelihoods of occurrence. For the purposes of economics, Von Neumann and Morgenstern developed a Theory of Games [1].

While mathematics was refining its treatment of uncertainty, thermodynamics and quantum mechanics observed phenomena with random and indeterminate outcomes untreatable by deterministic models. In nature, as in society, certainty and determinism had revealed their limit.

2 UNCERTAINTIES IN ENGINEERING

The International Organisation of Standardization (ISO) has identified the following three types of uncertainty in engineering products and processes:

2.1 Randomness

Natural phenomena are random. Extreme events, such as atmospheric, seismic and hydraulic disturbances are inherently unstable. Their characteristics are indeterminate, as are live loads on structures. Material properties, such as yield, ultimate strength, fracture toughness, chemical resistance are not constants.

Randomness lends itself to stochastic analysis. *The random causes for bridge deterioration and accidents can be modelled statistically, given ample data* (Ph. 1). Since that is never the case, stochastic analysis must take into account the deterministic assessments based on phenomenological models.

2.2 Ignorance

All information is incomplete. Many of the quantitative parameters describing the “as built” and extant states of a structure cannot be fully known. Measurements of structural and material behaviour are often absent, inaccurate or irrelevant. Data are insufficient for an adequate stochastic analysis of, for example, vulnerabilities, potential hazards, material properties, amount and effect of repair and maintenance work, among others.

Deterministic analysis counteracts ignorance by two means. On one hand, there is perpetual incentive to quantify and formalize all assessments, eliminating “guesswork”. On the other, qualitative expert judgments and opinions contain all the admittedly vague wisdom accumulated over the professional experience. Since determinism is inevitable, so is the formalized treatment of its vagueness. Thus, randomness entails ignorance, which implies vagueness.

2.3 Vagueness

By definition, vagueness defies definition. Bertrand Russell stated that “everything in this world is vague to a degree we don’t realise until we try to make it precise”. The most commonly used qualitative assessments are particularly susceptible. They include condition ratings, load ratings, remaining useful life, redundancy, safety, reliability, vulnerability, potential hazards, socio - economic constraints. Vaguely defined bridge and element conditions have been modelled by fuzzy sets, genetic algorithms and neural networks, producing ranges of possibilities with perceived likelihood.

McNeill and Freiberger [2] list various combinations of vagueness with other uncertainties resulting in fuzziness, including the following:

Nonspecificity: Ambiguity or lack of informativeness. A one-to-many relation between statement and possible meaning. Can be addressed by crisp set theory. This definition appears similar to ignorance.

Dissonance: Pure conflict. Treated as Bayesian probability of one statement being correct as opposed to another.

Confusion: Pure and potential conflict. There is conflict and the meaning of the data is unclear. Treated by 'possibility' theory.

Fuzziness: Vagueness, e.g. to what degree does a term apply. Treated by fuzzy set theory, which considers Bayesian probability (e.g. randomness) as a subset.

For the purposes of Bayesian probability, Thoft-Christensen and Baker [3] distinguish physical, statistical and model uncertainties. Melchers [4], recognizing that predecessor, organizes uncertainties into phenomenological, decision, modelling, prediction, physical, statistical and human (error and intervention). Any one of the latter seven groups can contain the former three in numerous combinations of somewhat fuzzy distinction.

Ang and De Leon [5] identify the following two types of uncertainty, which seem related to randomness and ignorance, respectively:

Aleatory: a non-deterministic property of natural randomness, modelled by random variables.

Epistemic: an inability to correctly represent a possibly deterministic reality, particularly significant in risk-informed decisions.

Photo 1-BY. Brooklyn and Manhattan Bridges across East River

3 UNCERTAINTIES IN BRIDGE MANAGEMENT

The engineering practice encounters all of the uncertainties described so far in combinations which do not lend themselves to easy separation. Thus it is critically important to model uncertainties according to their source and to recognize the limitations of the adopted methods. Several examples from the field of bridge management illustrate this point.

3.1 Present worth (PW) and life-cycle cost analysis (LCA)

The mathematically formalized modelling of uncertainty has been fundamental to economics at least since the publication of [1], as it has been to gambling since Pascal. Recent upheavals in the global financial markets have demonstrated that purely "ferquentist" models are unsuitable for discontinuities, instabilities, the ignorance associated with the phenomena, and the vagueness typical of human behaviour. In contrast, the highly uncertain discounting of future costs and benefits to an estimated present worth is modelled by the purely deterministic present worth analysis (PWA). Contrary to inflation rates, discount rates "*i*" are subjective and apply to specific investments, rather than to the overall economy. Hudson et al. [6] wrote:

"The discount rate selected by most agencies is a policy decision, but usually it is the difference between the interest rate for borrowing money and the inflation rate."

Hawk [7] defined three components of discount rates as follows:

$$i = (1 + cc)\,(1 + fr)\,(1 + pi) - 1 \qquad (1)$$

where:

cc = "real" opportunity-cost of capital

fr = required premium for financial risk associated with the considered investments

pi = anticipated rate of price inflation

Neglecting the higher order terms is justified by the relatively small values involved and reduces Eq. (1) to the following:

$$i = cc + fr + pi \qquad (2)$$

The present worth of an amount "a" occurring " years into the future is reduced by the factor $1/(1+ i)^N$. If amounts "*a*" occur annually, their cumulative present worth over $N - 1$ years from the present is obtained by Eq. (3).

$$a \sum_{n=0}^{N-1} 1/(1+ i)^n = a\,(1 + 1/i)\,[1 - 1/(1 + i)^N] \qquad (3)$$

$$\lim_{N \to \infty} a \sum_{n=0}^{N-1} 1/(1+ i)^n = a\,(1 + 1/i) \qquad (4)$$

If "*N*" tends to infinity, the non-convergent infinite series (*a, a, a, a*,.......) converges to a finite sum determined by "*i*", as in Eq. (4). Fig. 1 [8] shows the curves defined by Eq. (4) within the realistic range of values of "*i*". Their purpose is to illustrate the limited attention span of PWA. That span is quantified by the ratio of the finite and the infinite sums obtained in Eqs. (3) and (4), as shown in Eq. (5).

$$a\,(1 + 1/i)\,[1 - 1/(1 + i)^N] \,/\, a\,(1 + 1/i) = [1 - 1 / (1 + i)^N] \qquad (5)$$

For a given discount rate "*i*", the period "*N*" can be selected such that the error "*ε*" due to ignoring the present worth of increments "a" beyond "*N*" becomes negligible, as shown in Eq. (6):

$$N = - \ln \varepsilon / \ln (1 + i) \qquad (6)$$

Eq. (6) implies that, for a given ε, the relationship between N and the cumulative present worth is linear. Higher values of "*i*" correspond to a shorter financial "attention span". At i = 3%, that span declines fast after 70 years and all but vanishes after 100. The American Association of State Highway and Transportation Officials (AASHTO) recommends a 75-year useful life for new bridges, however 40 years is not uncommon. Leeming [9] observed that "any costs beyond 30 or 40 years have a negligible influence on the outcome". He concluded:

"If maintenance of our bridge stock is to remain a fixed percentage of the total governmental expenditure on construction, then there is an argument for a zero discount rate in calculating the net present value of maintenance."

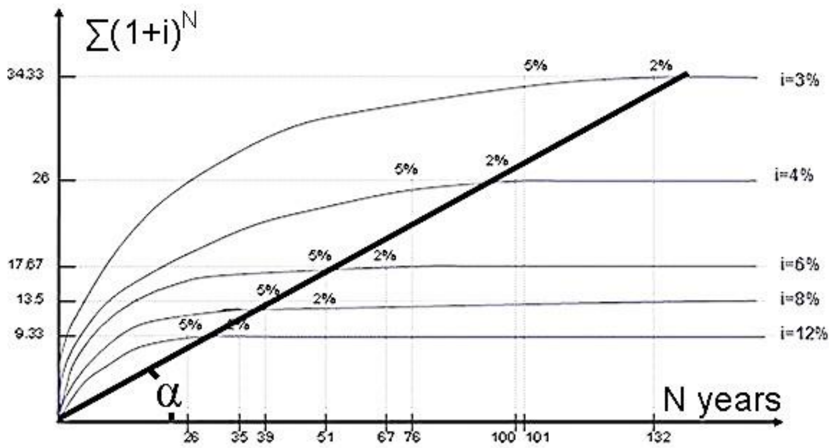

Figure 1. Cumulative present worth of infinite uniform series at different discount rates

In order to minimize the effect of discounting without completely eliminating it from life-cycle cost analysis, a low discount rate i = 2% is typically assumed. *For infrastructure facilities requiring periodic maintenance and replacements* (Ph. 2), De Gramo et al. [10] recommend "perpetuity", e.g. a uniform series of indefinitely running payments. In order to provide for annualized payments X, a principal P must be set aside at annual interest in % (in ≠ i), such that P in = X. If the payments are not annual but arise at k periods, the relationship becomes:

$$X = P\,[(1 + in)^{k} - 1] \tag{7}$$

Where: P is the capitalized value of X.

On the feasibility of capitalized annual maintenance expenditures, Leeming [9] commented:

"Governments do not usually put aside sums of money for future expenditure, but maintain out of income from the taxes we pay... It would be necessary to invest at 8% compound [interest] in order to keep pace with the increase in the road construction price index [UK]."

Photo 2-BY. The Queensboro Bridge

3.2 Bridge design

The design of vehicular bridges in the United States has been regulated by AASHTO (originally AASHO) since 1928. In 1998 AASHTO superseded the earlier codes with the Load Resistance and Factor Design (LRFD) specifications [11]. The reasoning behind this innovation is clearly detailed in [3] and [12]. Traditional design, based on the deterministic allowable stress and ultimate strength methods, requires the supply of structural resistance "R" to exceed the demand of all loads "Q" by a "safety" factor or by an array of "load" factors. The central innovation of LRFD is to treat Q and R as normally distributed statistically independent random variables. The margin of safety is represented by the limit state function defined as $g\,(Q, R) = R - Q$. Failure occurs when $g\,(R,Q) < 0$. The overlap between the two distributions shown in Fig. 2 is the "reliability boundary".

In this notation, the probability p_f of failure can be expressed as follows:

$$p_f = 1 - F_u\,(\beta) \tag{8}$$

F_u is the standard normal cumulative distribution function and β is the reliability index defined as:

$$\beta = (\bar{R} - \bar{Q}) \,/\, (\sigma_R^{2} + \sigma_Q^{2})^{1/2} \tag{9}$$

R and Q are mean values, σ_R and σ_Q are standard deviations.

AASHTO LRFD (1998) was calibrated to a target reliability index β = 3.5, implying a probability of failure of 0.0233% for new structures. For older structures β is estimated closer to 2.5, corresponding to a failure probability of 0.621%. Both values are based on assumptions about the probability of failure of individual structural components in non-redundant systems. The reliability index β has been calibrated for existing satisfactory design loads. The result is a recommendation for strength and load factors. Attempts are made to optimize

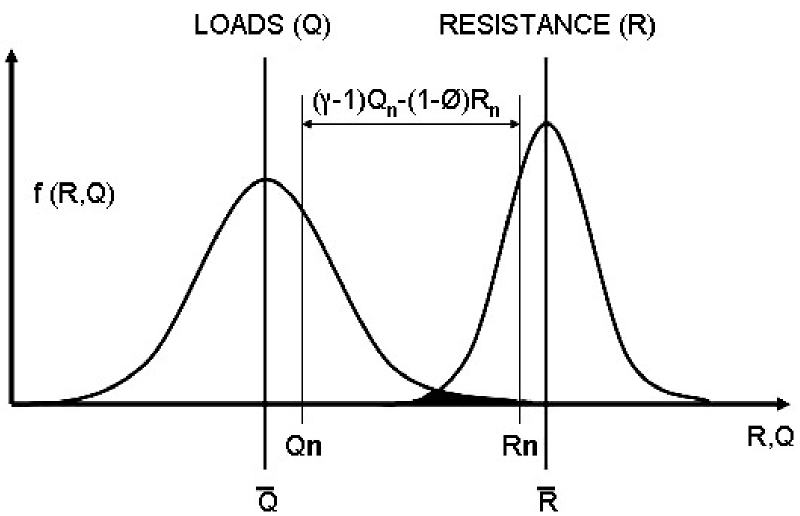

Figure 2. Normally distributed supply of structural resistance R and demand of loads Q

β with respect to cost, however life-cycle cost estimates are highly speculative. A number of NCHRP publications report on the calibration of β for structural redundancy and extreme events.

3.3 Bridge condition

Bridge condition is subjected to all the uncertainties identified in Section 2. Since the collapse of the Silver Bridge over the Ohio River at Point Pleasant in 1967, a federal law mandates the biennial inspections of all vehicular bridges in the United States. The National Bridge Inventory (NBI) contains and annually updates condition evaluations of roughly 630,000 bridges. Rather than seek the perfect model of these evaluations and then attempt to gather the data needed for their reliable determination, bridge managers rely on a variety of more or less independent assessments. The set shown in Fig. 3 supports bridge management decisions at New York State Department of Transportation (NYS DOT) and is similar to those employed by other bridge owners and FHWA. A brief description of each follows.

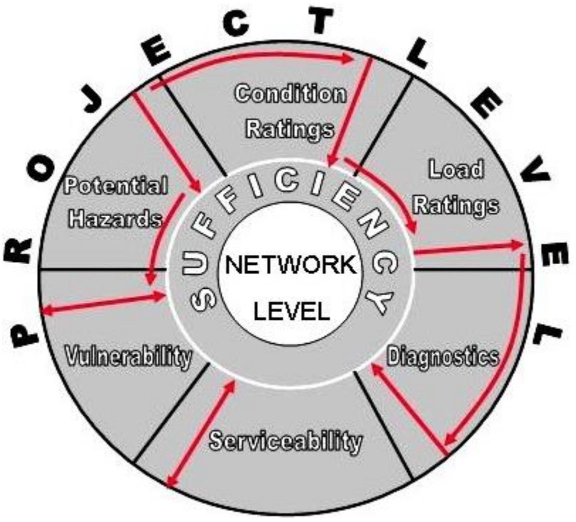

Figure 3. Various concurrent systems of bridge condition assessment

3.3.1 Structural condition rating

Structural condition is subject to randomness and causality. Condition ratings are vague. They can be descriptive or prescriptive. The policy adopted by the Federal Highway Administration (FHWA) is to compare inspection findings to the presumed "as built" condition. Hence a descriptive condition rating says more about the state of maintenance than about the load carrying capacity. NYS DOT rates the condition of bridges and their elements on a scale of 7 (new) to 1 (failed). Inspections are visual.

The NYS database is component- and span-specific. A weighted average formula combines the worst ratings of 13 key structural components throughout a bridge to obtain an overall condition rating as shown in Eq. (10). Included are primary member, secondary member, deck, piers, seats, bearings, backwalls, wingwalls, abutments, curbs, wearing surface, sidewalks, joints. Yanev [8] recommended the inclusion of paint for steel structures.

$$R = \sum_{i=1}^{13} W_i R_i / \sum_{i=1}^{13} W_i \tag{10}$$

R is the overall bridge condition rating; R_i is the minimum condition rating of element "i" observed on the bridge during the inspection (not necessarily in the same span), W_i are the element weights of the NYS DOT bridge condition formula, ranging from 10 for primary member to 1 for curb.

The alternative is the prescriptive approach, favored for example by the American Railroad Engineering and Maintenance Association (AREMA). Conditions are described by the amount and urgency of the remedial work, recommended by the inspecting engineer (Ph. 3). It is assumed that all recommended actions will be executed, hence the method is suited for a network in superior condition. The rating-descriptive method must be supplemented by a prescriptive evaluation, such as "potential hazards" for prompt corrective actions.

A 4- and 5- level rating system for commonly recognized (CoRe) structural elements was developed for the use of the FHWA Bridge Management System (BMS) PONTIS. In 2014 AASHTO proposed and FHWA introduced the Bridge Element rating protocol. Elements are quantified in 4 "condition states". The vagueness of

Photo 3-BY. Suspenders in Manhattan Bridge the original 10-level qualitative NBI ratings may be reduced, however the quantities rated in the 4 "condition states" are susceptible to randomness and ignorance. The number of bridges with comparable quantities would increase, requiring more detailed evaluations in order to prioritize their needs qualitatively.

3.3.2 Load rating

Load rating is obtained through calculations based on the design of the structure described in Section 3.2. Significant departures from the as-built condition require new ratings. NBI recognizes inventory and operating ratings, the former reflecting the regularly presumed structural capacity, the latter – its extreme capacity. In a well-functioning system, the qualitative condition ratings should inform about visible deterioration before the quantitative load ratings determine that the structure is functionally deficient.

3.3.3 Potential hazards

NYS DOT designates conditions perceived as potential hazards as "flags". Flags can be structural or safety (where the former always implies the latter, but not vise versa). Their urgency can vary from requiring prompt interim action (PIA) within 24 hours to low priority (allowing for monitoring until the next regular inspection). Yanev [8] reported a correlation between flag incidence and condition ratings of the most frequently flagged bridge elements, such as decks, primary members, railings, expansion joints and so on. Hazards related to traffic accidents and climatic changes occur at a relatively steady rate, whereas those caused by structural conditions increase predictably with deterioration.

3.3.4 Vulnerability (NYS DOT)

This rating anticipates hazards, rather than react to them. NYS DOT has developed procedures for addressing vulnerabilities related to the following causes: hydraulic, seismic, collision, overload, steel details, concrete details, sabotage.

Vulnerability is determined first through a review of the inventory, then confirmed by field inspections. The rating prioritizes the pre- and post event needs of the potentially vulnerable structures. Procedures for mitigating the conditions (for example by capital rehabilitation) and for responding to them in emergency mode are established.

3.3.5 Serviceability (NBI)

Serviceability is said to be appraised, rather than evaluated, however the federal rating is once again from 9 to 0. The quality of service is influenced by structural conditions, but depends also on factors, such as importance, obsolescence, and poor geometric alignment.

3.3.6 Sufficiency (NBI)

Sufficiency is an overall rating combining structural (55%) and serviceability (30%) factors, weighted by importance (15%). Albeit vague, this rating helpfully illustrated the state of the national vehicular bridge network. The 4 Element Level "condition states" cannot generate it and bridge managers are hard-pressed to find a substitute.

3.3.7 Diagnostics

Diagnostics is a rapidly developing field of condition assessment. It utilizes the non-destructive testing (NDT) and evaluation (NDE) techniques which are becoming commercially available for the first time. The developments follow three partly independent paths. Scientific research focuses on measurable events, commercial production develops marketable technologies, bridge owners must manage the life-cycle of their assets optimally. The resulting 3-dimensional space shown in Fig. 4 [8] defines the domain of structural health monitoring.

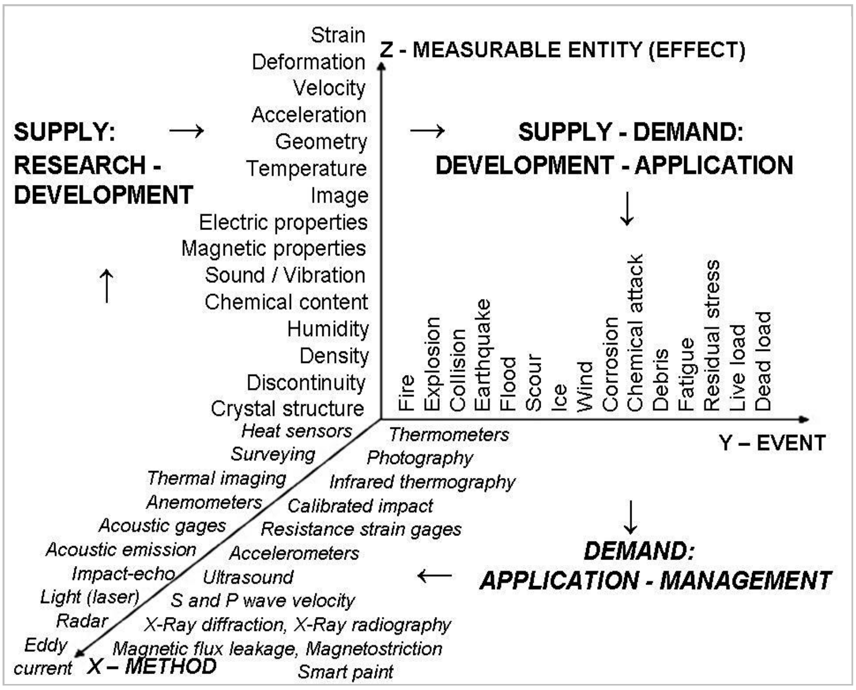

Figure 4. Bridge health monitoring by NDT and NDE

4 DETERMINISM AND PROBABILITY OF THE CONDITION ASSESSMENTS

The described assessments are grouped in Tab. 1 according to their contribution to decision support on the project and network levels in the short and long term. Rigorous overall optimization is not possible, however deterministic and probabilistic methods are combined towards an actionable outcome, either by limited optimization, vaguely or deterministically.

According to Tab. 1, short term and project level decisions are usually based on deterministic assessments, whereas long term and network level ones tend to employ probabilistic techniques. The relationship between deterministic and probabilistic methods in the most important assessments is briefly reviewed.

4.1 Bridge condition (rating) deterioration

The condition ratings described in the preceding Section should enable the modeling of bridge condition deterioration, and hence, provide estimates of infrastructure life-cycle needs. While the needs must address actual conditions however, the estimates are often based on condition ratings.

Condition models are phenomenological. They take into account material properties, design, construction and maintenance practices, service demands, climate, and so on. Consequently, they may vary from one project to another. If a reliable phenomenological model of condition deterioration existed, condition ratings would not be needed every two years for all bridges. A current proposal to relax the mandate for biennial inspections makes that argument, however the primary motive is economy.

Condition rating models are indispensable on the network level. The abundant data lends itself to stochastic methods of evaluation. The FHWA bridge management program PONTIS relied on the Markov chain model, but is replacing it with a Weibull distribution one, in part because of the former inability to model past history. Accumulation of new data may contribute to the decision.

There has been considerable debate whether the likeliest deterioration path is convex, concave or "*S*"-shaped (and if so, type Fig. 5-a or -b). Yanev [8] recommended a distinction between the project and network levels, as well as between the phenomenological models of condition and the stochastic models of condition rating. For condition ratings on a network level, the shape of the curve depends on the distribution of the data points along the two axes, as Fig. 5 demonstrates. For a large bridge network, a near-normal distribution is likely along both axes (*X*, representing time and *Y*, representing condition rating). Hence, the straight line of Fig. 5-c may be sufficiently accurate given other uncertainties.

For a reality check, the New York City bridge condition and sufficiency ratings for a typical year are plotted with respect to the structural age in Fig. 6-a and -b. The Bridge Element "condition states" cannot generate a bridge sufficiency rating. The need for it becomes immediately obvious. A stochastic model of the general

Table 1. Deterministic and probabilistic assessments on the project and network levels

Level	Deterministic by expert opinion	Probabilistic by various methods
Project	Condition rating	Condition (phenomenological)
	Load rating for AASHTO design loads	Load rating using probabilistically calibrated factors
	Potential hazards prioritized by urgency	
	Vulnerability prioritized by deterministic decision tree	
	Diagnostics: field measurement	
	Serviceability	
	Sufficiency	
	Needs: short term	
Network	Condition rating Worst case, Linear regression Other	Condition rating deterioration Markov chains, Normal, Weibull distribution, Monte Carlo simulation, Other
	Load rating based on the worst condition on a traffic corridor	Load rating based on design criteria
	Potential hazards Correlation with condition rating	Potential hazards Risk = likelihood x penalty
	Vulnerability	Vulnerability using probabilistic forecasts of extreme events
		Diagnostics: stochastic analysis
	Serviceability	Serviceability using economic and deterioration forecasts
		Sufficiency using economic and deterioration forecasts
	Needs: short term, emergency, Present worth	Needs: life-cycle structural conditions

deterioration pattern would reduce the data scatter of Fig. 6 to a near - straight line, as in Fig. 5-c, obtaining a useful life of approximately 85 years. That would correspond to roughly US $300 million in annual rehabilitation expenditures. The worst conditions in both graphs however, jointly indicate a useful life of 30 years. During the last two decades annual expenditures have exceeded US $500 million, corroborating that result. Many factors contribute to the data scatter. Rehabilitations are not always reflected in the database by changing the age of the bridge. Hundred-year old bridges can be rated as new after a billion-dollar rehabilitation (Ph. 4). Bridges in average condition receive numerous undocumented repairs. Thus, stochastically obtained forecasts invariably underestimate the needs. The worst ratings are both least compromised statistically and most urgent deterministically.

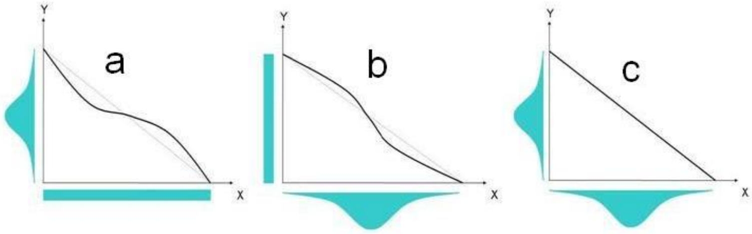

Figure 5. Cumulative distribution curves corresponding to normal distribution along one or both axes

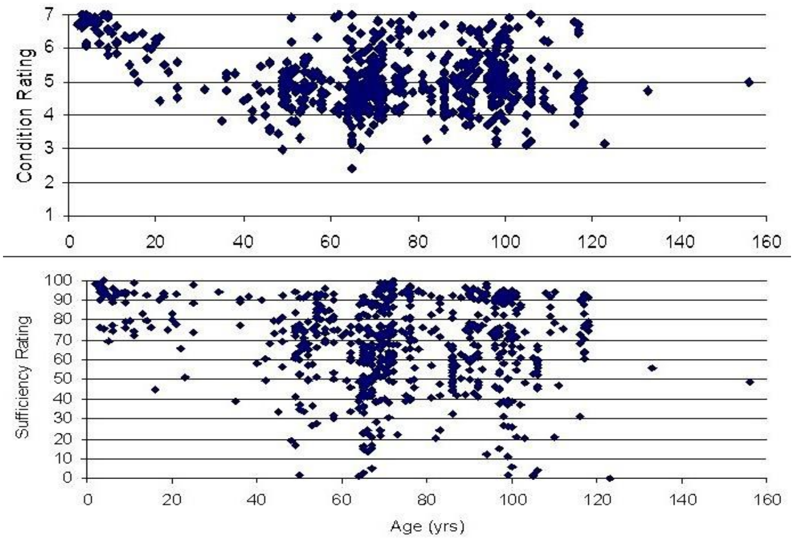

Figure 6. Condition (a) and sufficiency (b) ratings for New York City bridges over time

Photo 4-BY. Old and new suspenders

4.2 Load rating

The LRFD specifications assume a normal or log-normal probability function resulting in a linear failure function. When the failure function is not linear, the reliability index β can be estimated by Monte-Carlo Simulation (MCS) or by the iterative linearization procedures known as FORM (First Order Reliability Method) or SORM (Second Order Reliability Method), described by Ang and Tang [13]. The LRFD load rating model is appropriately conservative, but not always realistic. AASHTO recognized this and has recently approved load testing as an additional method of establishing the capacity and integrity of vehicular bridges. On railroad bridges, where loads are more predictable, visual assessments and load ratings can be more conclusive.

4.3 Potential hazards

Since network level need estimates are based on condition ratings, rather than on conditions, the costs of emergencies and hazard mitigation are not directly included. Hazard is associated with risk, which is defined as the product of likelihood of occurrence and the penalty. Some sources multiply by an additional factor for the likelihood of timely discovery. Since the penalty is a perception, it is assigned deterministically. The likelihood of occurrence is treated as a random variable, but it too can be assigned deterministically, pending the accrual of sufficient data.

Although potential hazards are phenomenologically correlated with conditions, the correlation with condition ratings, reported in [8] is not the same. Dominant is the correlation with actual events. After a fatal bridge-related accident in New York City in 1989, reports of potential hazards escalated from 800 to 2,000 / year.

4.4 Diagnostics

The 3-dimensional space of Fig. 4 might be interpreted as an indication that academic research, commercial production and bridge ownership have divergent objectives. More constructive would be the Cartesian view that they operate under a different balance between determinism and uncertainty. Bridge owners operate in a deterministic domain where actions must be taken and implemented, available monies must be committed and accounted for and, ultimately, safe traffic must be maximized. Their ideal bridge that does not need diagnostics at all. Their preferred methods are deterministic. Manufacturers are supplying a product and depend on the demand. In their view every bridge should be instrumented. They are governed by economic forecasts. Depending on their subject, researchers develop phenomenological or random models.

4.5 Serviceability

This is ultimately, a deterministic rating, based on qualitative assessments. In a safe system, the network level assessments must be the more conservative ones. In general, and in New York City, bridge life-cycles are governed by serviceability deficiencies, rather than by structural failures, as Fig. 6 shows. Although safe service is a rigid constraint, its maintenance cannot be rigorously optimized because the priorities and their quantification are heterogeneous, as illustrated in Fig. 7. User costs due to traffic closures are a product of time delays, average time cost and number of users. The losses due to accidents are subjective. Life-cycle maintenance costs and benefits are not directly quantifiable. In contrast, the clarity of the process and the deliverable product of construction make it the preferred option for bridge improvement in the short term. FHWA has advanced the alternative of “Bridge Preservation” on grounds of serviceability and life-cycle benefits.

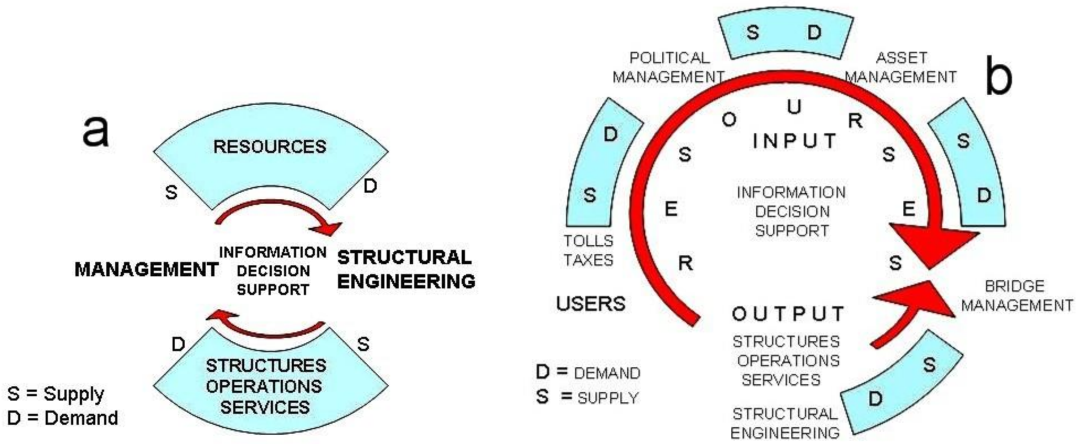

Figure 7. Bilateral and multi-layered models of infrastructure management

5 CONCLUSION

As all of society, engineering, and management in particular increasingly modify deterministic methods by allowing for uncertainty. Throughout most of history, the infrastructure was governed by supply and managed deterministically, as in the bilateral model of Fig. 7 – a. In contrast, democracies are governed by demand in a multi-layered interaction of leadership, management and the electorate, as in Fig. 7 – b. The major interested groups operate under diverse priorities.

The different explicit or implicit models borrow selectively from the method of Descartes and the view of Pascal. Users seek to maximize traffic while minimizing the taxation for its maintenance. The private sector is constrained by supply / demand and maximizes short-term return on investments. The political community is constrained by 4-year election terms. Bridge managers (tolled facilities excepted) rely on public funds which fall short of the estimated needs for maintaining the assets in their highest "ratings" or best "condition states". As the probabilistic treatments of uncertainty gain wider application, it is increasingly necessary to clarify the range of their validity in these heterogeneous overlapping domains. The following observations emerge:

- Modelling random processes by normal and Weibull distributions is deterministic in its own way. Not all of the variables are independent, as their models imply. For example, bridge conditions, condition ratings and potential hazards are vaguely, but not randomly correlated.
- Future costs / benefits are discounted deterministically, imposing a limited "attention span" on life-cycle planning. In the meanwhile, the uncertainty of the market is taken as proof that the physical infrastructure cannot be managed deterministically. The "reverse" influence of the infrastructure in the market is taken into account deterministically, only when its condition approaches failure. In 2019 AASHTO has rated the overall condition of the United State infrastructure as D+ (from A to D). This purely deterministic qualitative assessment has become a main argument in a Congress debate on funding allocation.
- Quantitative statistical methods are better suited for modelling future expectations and strategic guidelines, than for supporting immediate and tactical decisions. Poor reasoning cannot be replaced by a mathematical model but by correct reasoning.
- Determinism and uncertainty are not mutually exclusive alternatives, but complementary at every step of engineering in general, and the bridge management process in particular. Appropriately, the "reliability based" AASHTO LRFD bridge design specifications ultimately make deterministic choices. The transition of NBI from 10-level qualitative ratings to 4-level "condition states" illustrates the same point. The always inevitable uncertainty, consisting of randomness, ignorance, and vagueness must be treated with a well-balanced mix of probability and determinism.

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ABSTRACT

PROBABILITY AND DETERMINISM IN BRIDGE MANAGEMENT

Bojidar YANEV

Probabilistic models and methods are replacing deterministic ones in many areas of theoretical and applied sciences. To keep up with this development, the engineering profession is revising design specifications and updating forecasting techniques, whereas economics is adopting increasingly complex statistical models. At the intersection between the two fields, the management of the infrastructure must use effectively all new tools without abandoning established and reliable existing practices. The gathering and evaluation of meaningful data are critical to decisions with profound consequences in terms of monetary cost and quality of life. The article presents the author's observations drawn from his bridge management experience in New York City over 30 years.

Keywords*:* bridge, deterioration determinism, management, probability, uncertainty
Research area*:* infrastructure management

APSTRAKT

VEROVATNOĆA I DETERMINIZAM U UPRAVLJANJU MOSTOVIMA

Bojidar YANEV

U brojnim oblastima teorije i primenjene nauke, probablistički modeli i metode predstavljaju zamenu za determinističke. Da bi održala korak sa tom činjenicom, inženjerska struka trenutno revidira projektantske uslove/specifikacije i ažurira tehnike predviđanja, dok ekonomska struka istovremeno usvaja sve složenije statističke modele. Između ove dve sučeljene oblasti, od menadžmenta (upravljača) infrastrukture se očekuje da efikasno primenjuje sve novije alatke, a da se istovremeno ne odrekne uhodanih i pouzdanih metoda koje danas postoje. Prikupljanje i procenjivanje najvažnijih podataka predstavlja ključni faktor u odlučivanju, sa dalekosežnim posledicama u smislu novčanih troškova i kvaliteta života. U ovom radu predstavljeni su autorovi zaključci do kojih je došao na osnovu iskustava u oblasti upravljanja mostovima u gradu Njujorku.

Ključne reči: upravljanje infrastrukturom, most, određivanje dotrajalosti, menadžment, verovatnoća, neizvesnost

FLEKSIONO IZVIJANJE NAKNADNO TERMIČKI OBRAĐENIH I HLADNOOBLIKOVANIH STUBOVA ELIPSASTOG POPREČNOG PRESEKA: NUMERIČKA UPOREDNA ANALIZA

FLEXURAL BUCKLING OF HOT-FINISHED AND COLD-FORMED ELLIPTICAL HOLLOW SECTION COLUMNS: NUMERICAL COMPARATIVE ANALYSIS

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ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK: 624.014.2.075.4.072.7
doi:10.5937/GRMK1902015J

1 UVOD

Šuplji profili kvadratnog, kružnog i pravougaonog poprečnog preseka imaju već tradicionalnu primenu u građevinskoj praksi. S druge strane, šuplji profili elipsastog poprečnog preseka (EHS) postali su dostupni na tržištu poslednjih godina [19]. U poređenju sa šupljim kružnim poprečnim presecima (CHS), EHS preseci iste površine imaju veću nosivost na svijanje oko jače ose inercije. Prednost EHS preseka ogleda se u atraktivnoj arhitektonskoj formi, zbog čega imaju primenu u konstrukcijama koje ostaju vidne u prostoru. Poslednjih godina, elipsasti poprečni preseci korišćeni su kao stubovi Ziman objekta Univerziteta u Varviku (2003), aerodroma Barajas u Madridu (2004) i aerodroma u Korku u Irskoj (2006), zatim kao nosači staklene fasade aerodroma Hitrou u Londonu (2007) i kao lučni nosači pešačkog mosta *Society bridge* u Škotskoj (2005) [6]. Neki od navedenih primera prikazani su na slici 1.

U važećem Evrokodu za proračun čeličnih konstrukcija EN 1993-1-1:2005 [10], nisu definisani kriterijumi za proračun elemenata EHS poprečnih preseka. Međutim, u revidiranoj verziji Evrokoda EN 1993-1-1:2015 [11], koja još uvek nije publikovana i zvanično

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1 INTRODUCTION

Square, circular and rectangular hollow sections have been widely used in construction for decades. However, elliptical hollow sections (EHS) have been recently introduced to the construction market [19]. Comparing to circular hollow sections (CHS), EHS of the same area have greater bending capacity around the major axis of inertia. The EHS are used in exposed steelwork due to their advantage of attractive aesthetic appearance. In the past years, the elliptical hollow sections were implemented as columns in the Zeeman building at the University of Warwick (2003), in the Barajas airport building in Madrid (2004) and the Cork airport in Ireland (2006), then as supporting members for a glass façade in the Heathrow airport building in London (2007) and as arches of the pedestrian Society bridge in Scotland (2005) [6]. Some of mentioned examples are presented in Figure 1.

In the current Eurocode for the design of steel structures EN 1993-1-1:2005 [10], there are no design criteria defined for EHS structural elements. However, in the draft version of Eurocode EN 1993-1-1:2015 [11] which is not published and enacted yet, there are added

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usvojena, dodate su odredbe koje se odnose na EHS profile. Navedena pravila za proračun uglavnom se zasnivaju na istraživanjima nosivosti šupljih preseka pod dejstvom aksijalnog pritiska [3], savijanja [4] i kombinovanih uticaja [14]. Takođe, naučnici su predložili procedure za klasifikaciju elipsastih poprečnih preseka [13] i sproveli su ispitivanja stubova od EHS profila na fleksiono izvijanje [5]. Treba naglasiti i to da su se navedena istraživanja bavila isključivo naknadno termički obrađenim elementima.

regulations about EHS profiles. Those design rules are mostly based on the studies that had investigated behaviour of hollow sections under axial compression [3], bending [4] and combined effects [14]. In addition, researchers proposed the procedure for elliptical cross-section classification [13] and performed the study of flexural buckling of EHS columns [5]. It should be emphasized that mentioned studies were exclusively focused on hot-finished members.

Slika 1. Primeri primene EHS preseka [6]: (a) objekat Ziman; (b) aerodrom Barajas; (c) pešački most u Škotskoj
Figure 1. Examples of EHS application [6]: (a) Zeeman building; (b) Barajas airport building; (c) pedestrian bridge in Scotland

Druga uobičajena metoda proizvodnje elemenata šupljih poprečnih preseka jeste hladno oblikovanje. Analize nosivosti hladnooblikovanih EHS poprečnih preseka nisu dovoljno pokrivene eksperimentalnim i numeričkim ispitivanjima i postoji svega nekoliko radova koji se bave ovom tematikom [7,17]. Imajući u vidu različite procese proizvodnje u slučaju hladnog oblikovanja i naknadnog termičkog obrađivanja, te – kao posledicu toga – različite osobine materijala, očekuje se i drugačije ponašanje konstruktivnih elemenata proizvedenih na ova dva načina. Stoga, neophodna su dalja istraživanja u ovoj oblasti.

Proces hladnog oblikovanja sprovodi se na sobnoj temperaturi, tako što se čelični limovi savijaju u željeni oblik prolazeći kroz set valjaka. Kako bi se proizveo šuplji poprečni presek, savijeni čelični limovi se potom zavaruju po dužini dodirne izvodnice. Uobičajeno je da se elipsasti oblik preseka formira od prethodno proizvedenog kružnog profila koji se izlaže dodatnoj hladnoj deformaciji. Procesu naknadne termičke obrade predstoji isti set radnji kao u slučaju hladnog oblikovanja, nakon čega se proizvod podvrgava dodatnom tretmanu u peći na povišenim temperaturama. Treba naglasiti da se navedeni proces naknadne termičke obrade razlikuje od procesa vrućeg valjanja. Vruće valjanje ne uključuje

Cold-forming is another common production process of tubular structural elements. Experimental and numerical investigations on cold-formed EHS have not been broadly analysed and there are very few papers focused on this topic [7,17]. Considering different fabrication processes in the case of cold-forming than in the case of hot-finishing method, and as an effect, different material properties, dissimilar behaviour of hot-finished and cold-formed structural members is expected. For that reason, further researches in this field are necessary.

The process of cold-forming is conducted at ambient temperature by compressing and squeezing steel sheets through set of rollers. In order to produce a tube section, steel sheets are afterwards welded alongside the edges. Commonly, elliptical shape is formed from previously developed circular section that is additionally exposed to cold deformation. Before hot-finishing process, the same set of procedures as in the case of cold-forming should be performed, subsequently followed by additional heat treatment in a furnace. It should be noted that mentioned hot-finishing process differs from hot-forming process. The hot-forming process excludes any rolling in ambient conditions, but only at temperatures above the material re-crystallization temperature, which means that the obradu na sobnoj temperaturi, već isključivo na temperaturama iznad temperature rekristalizacije, što znači da se kompletan proces proizvodnje sprovodi na povišenim temperaturama. Iako se naknadno termički obrađeni i vrućevaljani elementi pri proračunu obično tretiraju na isti način, nedavno istraživanje pokazuje da su njihove mehaničke karakteristike drugačije [20].

Ovaj rad bavi se uporednom numeričkom analizom ponašanja zglobno oslonjenih naknadno termički obrađenih i hladnooblikovanih EHS stubova izloženih čistom aksijalnom pritisku i njihovom nosivošću na fleksiono izvijanje oko slabije ose inercije. Analiza pokriva set stubova relativne vitkosti do 2.5. Osnovu ove studije predstavlja istraživanje čije su rezultate autori prethodno publikovali [15], a koje je u ovom radu prošireno na veći set podataka, uz prikazivanje dodatnih poređenja i donošenje daljih zaključaka.

Rezultati numeričke analize upoređeni su s proračunskim kriterijumima definisanim u revidiranoj verziji Evrokoda EN 1993-1-1:2015 [11], koji se baziraju na metodi ekvivalentnog prečnika, kao i s proračunskim odredbama iz Severnoameričke specifikacije za projektovanje hladnooblikovanih čeličnih elemenata AISI-S100 [2], koje se zasnivaju na metodi direktne čvrstoće (DSM).

Istraživanjem su obuhvaćeni preseci klase 4, čije vitkosti zadovoljavaju granične uslove za klasifikaciju efektivnih preseka prema EN 1993-1-1:2015 [11]. Kako bi se stekao uvid u nosivost ovih poprečnih preseka, čije je određivanje neophodno za pomenuta poređenja s proračunskim odredbama, bilo je potrebno sprovesti i numeričku analizu lokalnog izbočavanja pritisnutih poprečnih preseka. To je učinjeno simuliranjem ponašanja kratkih stubova.

Numerička analiza određivanja nosivosti stubova na izvijanje sprovedena je u softverskom paketu *Abaqus* [1], koji se bazira na primeni metode konačnih elemenata (MKE). Geometrijske karakteristike poprečnih preseka elemenata usvojene su u skladu sa EHS proizvodima koji se danas mogu naći na tržištu [19]. Poprečni preseci odgovarajuće geometrije uključeni su u analizu lokalne stabilnosti, zadavanjem odgovarajuće dužine elementa i graničnih uslova radi simulacije ponašanja kratkog stuba. Nelinearno ponašanje materijala u oba slučaja modelirano je putem eksperimentalnih rezultata testova na zatezanje, koji su objavljeni u prethodnim istraživanjima [5,17,21].

2 NUMERIČKA ANALIZA

Istraživanje fleksionog izvijanja prikazano u ovom radu obuhvata geometriju elipsastih poprečnih preseka, koja odgovara naknadno termički obrađenim presecima koje proizvodi *Tata Steel* [19]. Analizirana su tri različita poprečna preseka, dimenzija 150x75 mm i varirane debljine zida 3, 4 i 5 mm.

Napravljena su četiri različita numerička modela za svaki poprečni presek, s dužinama elementa 700, 1500, 2300 i 3100 mm, kao što je učinjeno u eksperimentalnom ispitivanju fleksionog izvijanja naknadno termički obrađenih EHS stubova, koje su sproveli Čen i Gardner [5]. Na taj način, uključen je opseg relativnih vitkosti stubova, značajnih za opisivanje krivih izvijanja. Stubovi su zglobno oslonjeni, a s obzirom na to što je simulirano samo izvijanje oko slabije ose inercije, dodatni bočni

whole manufacturing process is conducted at elevated temperatures. Although hot-finished and hot-formed sections are often treated equally in design, recent research shows that their mechanical properties are different [20].

The paper is focused on a comparative numerical analysis of pin-ended hot-finished and cold-formed EHS columns under pure axial compression and addresses their flexural buckling capacity about minor principal axis. The analysis covers a whole range of column overall slenderness ratio up to 2.5. A baseline for this study is the research which results were previously published by authors [15], and that has been extended in this paper to wider set of data and further comparisons. In addition, conclusions have been made as well.

The results of numerical analysis are compared to the design criteria defined in Eurocode draft standard EN 1993-1-1:2015 [11], based on the equivalent diameter method, and to the design regulations defined in North American specification for the design of cold-formed steel members AISI-S100 [2], based on the Direct Strength Method (DSM).

According to the limiting slenderness for cross-section classification defined in EN 1993-1-1:2015 [11], sections of the class 4 are included in the study. In order to determine the resistance of such cross-sections which is required for mentioned comparisons with the design criteria, it was necessary to incorporate a numerical analysis of local buckling of compressed members into the research. This was performed through simulations of stub column tests.

Numerical analysis of columns´ compressive capacity is performed by using the software package Abaqus [1], based on the finite element analysis (FEA). Geometrical properties of the column cross-sections are adopted according to EHS products that could be found on the market nowadays [19]. Cross-sections of the corresponding geometry are included in a local buckling analysis, setting an appropriate member length and boundary conditions for stub column simulations. Material non-linear behaviour in both cases is included through experimental tensile test results published in previous researches [5,17,21].

2 NUMERICAL ANALYSIS

The flexural buckling study presented in this paper includes geometry of elliptical hollow sections corresponding to hot-finished sections manufactured by Tata Steel [19]. Three different cross-sections are adopted in analysis, all of the overall dimensions 150x75 mm and with the varied wall thickness of 3, 4 and 5 mm.

Four different numerical models for each cross-section were developed, with member lengths of 700, 1500, 2300 and 3100 mm, as it was performed in the experimental research of flexural buckling of hot-finished EHS columns done by Chan and Gardner [5]. Therefore, a range of non-dimensional column slenderness of the importance for describing buckling curves is included. Columns are pin-ended and there were no lateral supports added along the column height as only buckling oslonci duž visine stuba nisu uključeni u analizu.

Uzimajući u obzir znatan broj numeričkih rezultata koji se baziraju na analizama nosivosti vitkih EHS preseka klase 4 [11], takođe su sprovedene numeričke simulacije ponašanja kratkih stubova kako bi se ocenila redukcija nosivosti poprečnog preseka usled elastičnog izbočavanja. Zadata je visina kratkog stuba od 450 mm i aplicirani su granični uslovi uklještenja na krajevima.

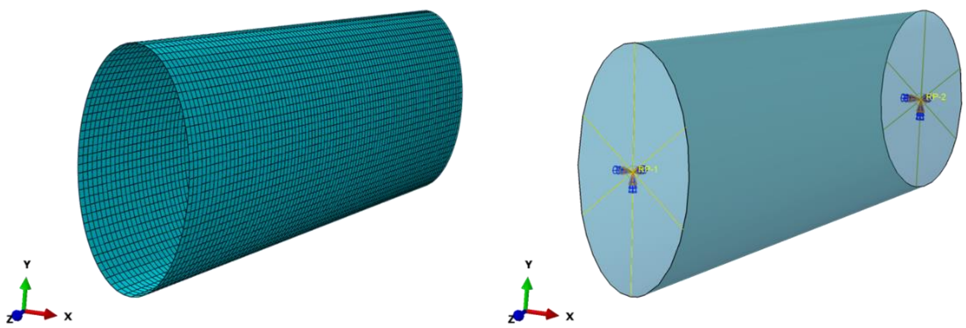
Numeričke simulacije sprovedene su u programskom paketu *Abaqus* [1], koji se bazira na primeni metode konačnih elemenata. Korišćeni konačni elementi su površinski S4R elementi s redukovanom integracijom i sa šest stepeni slobode u svakom čvoru. Izabrana je uniformna gustina mreže, s veličinom elementa od 5 mm (slika 2.a), koja je jednaka maksimalnoj zadatoj debljini zida profila i koja je usvojena na osnovu analize konvergencije rezultata, kako bi se pri numeričkom proračunu istovremeno postigla efikasnost i preciznost. Svi čvorovi na krajevima stuba povezani su putem komande *coupling* s referentnom tačkom u težištu preseka na kraju stuba, u kojoj su aplicirani granični uslovi (slika 2.b). U slučaju modela kratkih stubova, svi stepeni slobode kretanja su sprečeni, sem podužnog pomeranja na mestu unosa opterećenja, dok su kod modela vitkih stubova rotacije na krajevima dozvoljene, a pomeranja sprečena (sem u podužnom pravcu na mestu unosa opterećenja). Za sve modele, aksijalni pritisak je zadat kao koncentrisano opterećenje u referentnoj tački.

about minor principal axis was simulated.

Taking into account that significant number of numerical data is based on slender EHS of the class 4 [11], numerical simulations of stub column tests are additionally performed to quantify reductions of cross-section resistances due to elastic local buckling. The member length is set to 450 mm and fixed-ended boundary conditions are applied.

Numerical simulations are performed in the finite-element software package Abaqus [1]. Finite elements implemented in numerical models are shell elements S4R with reduced integration and six degrees of freedom per node. Uniform mesh density is chosen with finite element size of 5 mm (Figure 2.a), equal to the maximum applied wall thickness and adopted according to the mesh convergence study, in order to achieve computation efficiency and accurate results at the same time. All edge nodes are connected by coupling to the reference point at the cross-section centroid at each member's end, to which boundary conditions are applied (Figure 2.b). For stub column models all degrees of freedom are restrained except in the longitudinal direction at the loading point, while for slender column models rotations are allowed and displacements are restrained (except in the longitudinal direction at the loading point). For all models, axial compression is introduced by applying a point load at the reference point.

(a) (b)



Slika 2. Numerički model kratkog stuba: (a) mreža konačnih elemenata; (b) granični uslovi
Figure 2. Numerical model for stub column simulation: (a) FE mesh; (b) boundary conditions

U prvoj fazi numeričke simulacije, sprovedena je analiza sopstvenih vrednosti kako bi se odredili kritični oblici izvijanja elementa. Zatim je sprovedena statička analiza modifikovanom Riksovom metodom radi simuliranja izvijanja stubova, uzimajući u obzir materijalnu i geometrijsku nelinearnost.

2.1 Početne geometrijske imperfekcije

Globalne i lokalne geometrijske imperfekcije uključene su u numeričku analizu. Relevantne kritične forme izvijanja i izbočavanja, određene putem analize sopstvenih oblika izvijanja, zadate su kao početne deformisane forme elementa. Sama veličina globalne i lokalne deformacije definisana je zadavanjem odgovarajućih amplituda imperfekcija.

Prema preporukama, datim u EN 1993-1-5: 2006

In the first step of numerical simulation, Eigenvalue analysis is conducted in order to obtain buckling modes. Secondly, the static analysis with modified Riks method is used for simulating the column buckling behaviour, accounting to the material and geometrical non-linearities.

2.1 Initial geometric imperfections

Overall and local geometric imperfections are incorporated in the numerical model. Relevant critical buckling modes of global and local buckling obtained in Eigenvalue analysis are set as initial deformed shapes of the element. The value of overall and local deformation is applied through setting adequate imperfection amplitudes.

According to the recommendations given in EN

[12], preporučena vrednost amplitude geometrijske imperfekcije u numeričkim simulacijama je 80% fabričke tolerancije. Za odstupanje od pravca štapa šupljeg kružnog poprečnog preseka, fabrička tolerancija jeste 1/500 dužine elementa [8,9]. Međutim, u numeričkoj analizi – sprovedenoj u ovom radu – imperfekcija je zadata kao polusinusna funkcija sa amplitudom 1/1000 dužine štapa, kao što su u numeričkoj parametarskoj analizi uradili Čen i Gardner i što je pokazalo najbolje slaganje sa eksperimentalnim rezultatima [5].

Fabrička tolerancija za debljinu zida CHS preseka data je u zavisnosti od prečnika *D* i debljine zida *t*: za *D* ≤ 406.4 mm i *t* ≤ 5 mm, maksimalna imperfekcija ne bi trebalo da bude preko 10% debljine zida [8,9]. Ispitivanja nosivosti na pritisak EHS preseka koji su naknadno termički obrađeni [3] i hladnooblikovani [7], pokazala su da je najbolje slaganje između eksperimentalnih i numeričkih rezultata postignuto kada su magnitude lokalnih imperfekcija zadate kao 1/100 i 1/50 debljine zida, respektivno. U skladu s tim, navedene početne lokalne imperfekcije uključene su u numeričke modele. Međutim, tokom analize uočeno je da je uticaj zadatih imperfekcija na globalno izvijanje zanemarljiv, čak i u slučaju najkraćih elemenata. U slučaju kratkih stubova, rezultati su osetljiviji na zadate magnitude lokalnih imperfekcija, ali i dalje je razlika u vrednostima graničnih nosivosti dobijenih za amplitude lokalnih imperfekcija 1/10 i 1/100 debljine zida preseka, bila manja od 1%.

2.2 Karakteristike materijala

Materijalne karakteristike hladnooblikovanih i naknadno termički obrađenih čeličnih preseka proračunom su obuhvaćene rezultatima testova na zatezanje, koji su sprovedeni pri eksperimentalnim ispitivanjima elipsastih i ovalnih šupljih preseka. Za razliku od EHS profila kod kojih je zid preseka zakrivljen duž celog obima elipse, ovalni preseci sastoje se iz dva paralelna zida koji su međusobno povezani sa dva polukružna prstena [21].

Kvak i Jang [17] odredili su krive napon–dilatacija ispitivanjem epruveta izvađenih iz različitih delova duž obima – kako hladnooblikovanih, tako i naknadno termički obrađenih EHS preseka. U slučaju naknadno termički obrađenih profila, uočene su neznatne razlike u ponašanju epruveta izvađenih iz najravnijih delova i ugaonih delova. Nasuprot tome, osobine materijala kod hladnooblikovanih preseka nisu konstantne duž obima elipse – ugaone epruvete imaju veću granicu razvlačenja i čvrstoću na zatezanje, ali nižu duktilnost. Ovo je rezultat procesa proizvodnje pri kojem su zakrivljeni delovi preseka podvrgnuti hladnom savijanju i izloženi plastičnim deformacijama. U istraživanju naknadno termički obrađenih EHS profila koje su sproveli Čen i Gardner [5], ispitivane su samo epruvete iz najravnijeg dela elipsastog preseka. Slično tome, kada su Zu i Jang [21] ispitivali ovalne hladnooblikovane preseke, materijalne karakteristike određene su samo za epruvete izvađene iz ravnog dela ovalnog profila.

Sumarno, krive napon–dilatacija korišćene u numeričkim modelima u ovom istraživanju date su na slici 3 i navedene u sledećoj listi:

− epruvete iz naknadno termički obrađenih preseka

1993-1-5:2006 [12], the advised value of geometric imperfection amplitude for numerical simulation is 80% of fabrication tolerance. The fabrication tolerance is 1/500 of member length for out of straightness of circular hollow section column [8,9]. However, for numerical analysis performed in this research, the imperfection is applied through half-sine wave function with the amplitude of 1/1000 of member length, as it was done by Chan and Gardner in numerical parametric study and that showed the best agreement with experimental results [5].

A fabrication tolerance for wall thickness of CHS is given depending on a section diameter *D* and a wall thickness *t*: for *D* ≤ 406.4 mm and *t* ≤ 5 mm, the maximum imperfection should not exceed 10% of the thickness [8,9]. Researches of EHS compressive resistance of hot-finished [3] and cold-formed sections [7] showed that the best agreement between experimental and numerical results is achieved when the magnitude of local imperfection is applied as 1/100 and 1/50 of the section wall thickness, respectively. Accordingly, mentioned initial local imperfections are included in numerical models. However, during the analysis it is observed that the influence of the applied local imperfection on the global buckling was negligible, even in the case of the shortest column members. In the case of stub column models, results were more sensitive to the local imperfection magnitude, but still the difference in a value of the ultimate load obtained with local imperfection amplitudes of 1/10 and 1/100 of the section wall thickness was less than 1%.

2.2 Material properties

Material properties of cold-formed and hot-finished steel sections are taken into account through the results of tensile coupon tests that are conducted in experimental investigations of elliptical and oval hollow sections. Unlike EHS where cross-sections' wall is curved all along the perimeter, oval hollow sections consist of two parallel flat walls that are interconnected by two semi-circle rings [21].

Quach and Young [17] obtained stress–strain curves for material coupons taken from different parts along a section perimeter, for both cold-formed and hot-finished EHS. In the case of hot-finished sections, insignificant differences in material behaviour of the flattest coupons and corner coupons are observed. Contrary, material properties of cold-formed sections are not uniform throughout elliptical perimeter – corner coupons have greater yield stress and tensile strength, but lower ductility. This is due to the manufacturing process, as curved parts of section were subjected to local cold bending and undergo plastic deformations. In the research of hot-finished EHS done by Chan and Gardner [5], only coupons from the flattest portion of section were tested. Similarly, when cold-formed oval hollow sections were investigated by Zhu and Young [21], only mechanical properties of flat coupons are obtained.

The summation of all stress–strain curves implemented in numerical models in this research is presented in Figure 3 and in the following list:

− hot-finished coupon (HF) tested by Chan and Gardner [5];

(HF) koje su testirali Čen i Gardner [5];

– epruvete iz naknadno termički obrađenih preseka (HF) koje su testirali Kvak i Jang [17];

– epruvete iz najzakrivljenijeg dela hladnooblikovanih preseka (CF – corner) koje su ispitivali Kvak i Jang [17];

– epruvete iz najravnijeg dela hladnooblikovanih preseka (CF – flat) koje su ispitivali Kvak i Jang [17];

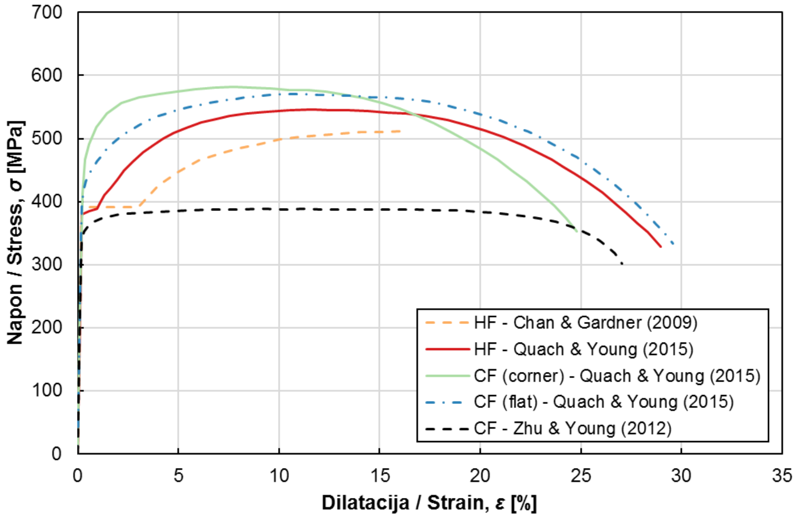
– hladnooblikovane epruvete (CF) iz ravnog dela ovalnog preseka koje su testirali Zu i Jang [21].

– hot-finished coupon (HF) tested by Quach and Young [17];

– cold-formed coupon from the most curved part of section (CF – corner) tested by Quach and Young [17];

– cold-formed coupon from the flattest part of section (CF – flat) tested by Quach and Young [17];

– cold-formed coupon (CF) from the flat part of oval hollow section tested by Zhu and Young [21].



Slika 3. Krive napon–dilatacija materijala naknadno termički obrađenih i hladnooblikovanih čeličnih preseka
Figure 3. Stress–strain curves for hot-finished and cold-formed steel sections' material

Posmatrajući sliku 3, uočava se da je oblik materijalnih krivih različit za hladnooblikovane i naknadno termički obrađene preseke. Hladnooblikovani materijal odlikuje se postepenom plastifikacijom i ojačanjem, dok naknadno termički obrađen materijal ima oštro izražen početak plastifikacije. Odlika drugog materijala jeste plato s jasno uočljivim naponom na granici razvlačenja f_y, dok u slučaju hladnooblikovanih materijala, vrednost granice razvlačenja f_y nije jasno uočljiva i neophodno je odrediti konvencionalnu granicu razvlačenja $f_{0.2}$, koja odgovara plastičnoj deformaciji od 0.2%. Ponašanje naknadno termički obrađenih čelika približno odgovara ponašanju osnovnog čeličnog materijala. Suprotno tome, proces hladnog oblikovanja izaziva promene u dijagramu napon–dilatacija izvornog materijala. Međutim, kod materijala dva prezentovana naknadno termički obrađena EHS preseka, dužina platoa razvlačenja se razlikuje. To je posledica razlike u nastavku procesa proizvodnje nakon hladnog valjanja, s obzirom na to što su naknadno termički obrađeni preseci, koje su testirali Čen i Gardner, bili izloženi temperaturi od 900 ^{0}C, dok je za EHS elemente koje su ispitivali Kvak i Jang termički tretman sproveden na temperaturi od približno 750 ^{0}C.

Deo krive napon–dilatacija koji je značajan za simuliranje fleksionog izvijanja jeste deo pre dostizanja čvrstoće na zatezanje f_u. Iz tog razloga, materijalne krive koje su dali Čen i Gardner predstavljene su na slici 3 na isti način kao što je dato u njihovom istraživanju, bez dela nakon dostizanja maksimalnog napona.

S ciljem uključivanja materijalnih karakteristika u numerički model, sem definisanja Jangovog modula

It can be observed from Figure 3 that the shape of the material stress–strain curve is different for cold-formed and hot-finished sections. Cold-formed sections exhibit gradual yielding behaviour with enhanced material properties, whereas hot-finished sections have sharp yielding stress–strain curves. The feature of the latter one is a yield plateau with noticeable yield stress f_y, while for cold-formed sections, the value of yield stress f_y is not obvious and 0.2% proof stress $f_{0.2}$ needs to be determined. Behaviour of hot-finished steel fits better the behaviour of basic steel material. Contrary, a cold-forming process causes a change in the stress–strain relationship of the basic material. However, even for the presented two hot-finished EHS, the length of the yield plateau differs. This is due to the difference in manufacturing process after cold-rolling, as hot-finished sections tested by Chan and Gardner were exposed to the temperature of 900 ^{0}C, while for EHS tested by Quach and Young, heat treatment was performed at the temperature of approximately 750 ^{0}C.

The part of a stress–strain curve which shows the importance for simulation of flexural buckling is the part before reaching the material strength f_u. Therefore, the material curve obtained by Chan and Gardner is presented in Figure 3 in the same way as it is given in their research, without post ultimate material behaviour.

In order to incorporate material properties into the numerical model, except defining Young's modulus and Poisson's ratio in the elastic domain, it was necessary to define true stress and true plastic strain for describing plastic behaviour of material. It is done according to the elastičnosti i Poasonovog koeficijenta u elastičnom domenu ponašanja, bilo je neophodno definisati stvarni napon i stvarnu plastičnu deformaciju za opisivanje plastičnog ponašanja materijala. To je učinjeno u skladu sa sledećim izrazima datim u korisničkom uputstvu za *Abaqus* [1]: $\sigma_{true} = \sigma(1+\varepsilon)$ i $\varepsilon_{true,pl} = \ln(1+\varepsilon) - \sigma_{true}/E$, gde je E modul elastičnosti.

2.3 Zaostali naponi

Zaostali naponi uglavnom se javljaju kao posledica hlađenja nakon termičke obrade, usled procesa zavarivanja ili kao rezultat sprečenih povratnih deformacija izazvanih procesom proizvodnje. Tokom fabrikacije šupljih profila, hladnooblikovani elementi izloženi su plastičnim deformacijama, pa se nakon elastičnog rasterećenja, javljaju zaostali naponi. U toku termičke obrade kod naknadno termički obrađenih elemenata, znatan deo zaostalih napona se oslobađa, s obzirom na to što pri povišenim temperaturama granica razvlačenja i modul elastičnosti opadaju [20]. Merenja zaostalih napona kod elipsastih poprečnih preseka sproveli su Lo i Gardner za naknadno termički obrađene elemente [16], Čen i Jang za hladnooblikovane elemente [7] i Kvak i Jang za naknadno termički obrađene i hladnooblikovane EHS profile [17]. Zaostali napon koji utiče na ponašanje pritisnutih elemenata je zaostali napon u podužnom pravcu. Eksperimentalni rezultati pokazali su da naknadno termički obrađeni šuplji preseci imaju znatno manje zaostale napone od hladnooblikovanih preseka i da oni iznose 10–15% vrednosti granice razvlačenja. Prema tome, ne očekuje se značajan uticaj ovih napona na ponašanje konstruktivnih elemenata i zbog toga oni nisu uključeni u numeričku analizu fleksionog izvijanja koju su sproveli Čen i Gardner [5]. Kod hladnooblikovanih preseka, maksimalni zaostali napon savijanja može dostići približno 75% konvencionalne granice razvlačenja $f_{0.2}$, dok maksimalni membranski napon iznosi približno 25% napona $f_{0.2}$. Iako su zaostali naponi savijanja velikih magnituda, numerička ispitivanja pokazala su da ovi naponi ne utiču na ponašanje kratkih stubova [7]. Stoga, u ovom radu zaostali naponi nisu eksplicitno uključeni u numeričku analizu ni naknadno termički obrađanih, niti hladnooblikovanih elemenata.

3 REZULTATI I DISKUSIJA

Rezultati numeričkih simulacija ponašanja pritisnutih poprečnih preseka kratkih stubova i ponašanja stubova pri fleksionom izvijanju prikazani su u delovima 3.3 i 3.4, respektivno. Napravljeno je poređenje ponašanja naknadno termički obrađenih i hladnooblikovanih EHS profila, kao i poređenje proračunskih predikcija prema revidiranoj verziji Evrokoda za proračun čeličnih konstrukcija EN 1993-1-1:2015 [11] i Severnoameričkoj specifikaciji za projektovanje hladnooblikovanih čeličnih elemenata AISI-S100 [2]. Radi boljeg razumevanja prezentacije rezultata, prvo su u delovima 3.1 i 3.2 ukratko opisane procedure koje daju pomenuti standardi.

following relations as specified in Abaqus user's manual [1]: $\sigma_{true} = \sigma(1+\varepsilon)$ and $\varepsilon_{true,pl} = \ln(1+\varepsilon) - \sigma_{true}/E$, where E is Young's modulus.

2.3 Residual stresses

Residual stresses usually arise due to cooling effects after hot-finishing, employed welding processes or by the prevention of springback introduced during manufacturing operations. During a production process of hollow sections, cold-formed members are exposed to plastic deformation, so after elastic unloading, residual stress is induced. During heat treatment, a considerable amount of residual stress in hot-finished sections is released, as at elevated temperatures yield stress and Young's modulus decrease [20]. Measurement of residual stresses in elliptical hollow sections is done by Law and Gardner for hot-finished members [16], by Chen and Young for cold-formed members [7] and by Quach and Young for both hot-finished and cold-formed EHS [17]. The residual stress in a longitudinal direction is important for compressed member behaviour. Experimental results show that longitudinal residual stresses in hot-finished hollow sections are significantly smaller than those in cold-formed hollow sections, rating 10–15% of the material yield strength. Therefore, it is expected that they have a insignificant influence on the structural behaviour and thus excluded from the numerical analysis of flexural buckling developed by Chan and Gardner [5]. In the case of cold-formed sections, the maximum bending residual stress can reach approximately 75% of the material 0.2% proof stress, while the maximum membrane residual stress is about 25% of the 0.2% proof stress. Although bending residual stress has the high amplitude, numerical investigations showed that it does not affect the structural response of stub columns [7]. For that reason, the residual stresses are not explicitly included in the numerical analysis of this research, neither of hot-finished, nor of cold-formed members.

3 RESULTS AND DISCUSSION

The results of numerical simulations of cross-sectional behaviour and flexural buckling behaviour are shown in sections 3.3 and 3.4, respectively. A comparison between hot-finished and cold-formed EHS is given, as well as a comparison with design predictions according to Eurocode draft standard for the design of steel structures EN 1993-1-1:2015 [11] and North American specification for the design of cold-formed steel members AISI-S100 [2]. Design procedures according to the mentioned standards are briefly presented in sections 3.1 and 3.2 for better understanding of the result representation.

3.1 Proračunske procedure prema EN 1993-1-1

Prema EN 1993-1-1:2005 [10], vrednost nosivosti na izvijanje N_b određuje se množenjem nosivosti poprečnog preseka N_c s bezdimenzionalnim redukcionim koeficijentom izvijanja χ, definisanim kao:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} \leq 1 \qquad (1)$$

gde je:

$\Phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$;

α – koeficijent imperfekcije;

λ – relativna vitkost elementa.

Procedura proračuna zasnovana je na Peri-Robertsovim formulama i na linearnom izrazu za parametar imperfekcije $\alpha(\lambda - 0.2)$, kojim se u proračun fleksionog izvijanja uključuju početne geometrijske imperfekcije, zaostali naponi i ekscentričnost opterećenja. Vrednost koeficijenta imperfekcije zavisi od krive izvijanja. Prema EN 1993-1-1:2015 [11], elipsasti šuplji profili tretiraju se na isti način kao kvadratni, pravougaoni i kružni šuplji profili, za koje izbor krive izvijanja zavisi od granice razvlačenja materijala i procesa proizvodnje. Za naknadno termički obrađene poprečne preseke kvaliteta čelika od S235 do S450, definisana je kriva izvijanja *a*, dok je za hladnooblikovane preseke – to kriva *c*.

Za poprečne preseke klase 1, 2 i 3, relativna vitkost elementa data je u izrazu:

$$\lambda = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \frac{L_{cr}}{i \cdot \pi}\sqrt{\frac{f_y}{E}} \qquad (2)$$

gde je:

L_{cr} – dužina izvijanja za posmatranu ravan izvijanja;

i – poluprečnik inercije za odgovarajuću osu;

A – površina poprečnog preseka;

N_{cr} – elastična kritična sila fleksionog izvijanja.

Za poprečne preseke klase 4, relativnu vitkost elementa treba sračunati sa efektivnom površinom preseka A_{eff}:

$$\lambda = \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}} \qquad (3)$$

Revidirana verzija Evrokoda EN 1993-1-1:2015 [11] definiše granične vitkosti za klasifikaciju i daje izraze za određivanje efektivne površine EHS preseka. Prvo, treba sračunati ekvivalentni prečnik D_e za pritisnute EHS preseke kao:

$$D_e = h\left(1 + \left(1 - 2.3\left(\frac{t}{h}\right)^{0.6}\right)\left(\frac{h}{b} - 1\right)\right) \qquad (4)$$

ili konzervativno kao

$$D_e = \frac{h^2}{b} \qquad (5)$$

3.1 Design procedures according to EN 1993-1-1

According to EN 1993-1-1:2005 [10], the value of the member buckling resistance N_b is obtained by multiplying the cross-section resistance N_c with the buckling reduction factor χ, defined as:

where:

$\Phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$;

α is an imperfection factor;

λ is a non-dimensional slenderness.

The procedure is based on Perry-Robertson equations and the linear expression for the imperfection parameter $\alpha(\lambda - 0.2)$ that accounts for initial geometric imperfections, residual stresses and load eccentricity on the predicted flexural buckling resistance. The value of an imperfection factor depends on the flexural buckling curve. According to EN 1993-1-1:2015 [11], elliptical hollow sections should be treated equally as square, rectangular and circular hollow sections, for which buckling curve selection depends on a material yield strength and a fabrication process. For hot-finished hollow sections of a steel grade from S235 to S450, the buckling curve is defined as *a*, while for cold-formed sections, it is *c*.

For cross-sections of class 1, 2 or 3, a non-dimensional slenderness is given by:

where:

L_{cr} is a buckling length for the considered buckling plane;

i is a radius of gyration for the corresponding axis;

A is a cross-sectional area;

N_{cr} is an elastic critical buckling load.

For the cross-sections of the class 4, a non-dimensional slenderness should be calculated with an effective area of a cross-section A_{eff}:

Draft version of Eurocode EN 1993-1-1:2015 [11] states limiting proportions for EHS classification and defines an effective area of EHS. Firstly, an equivalent diameter D_e for EHS in compression should be calculated as:

or conservatively as

gde su:

t – debljina zida preseka;

h i b – dimenzije poprečnog preseka ($h > b$).

Vitkost poprečnog preseka koja se poredi s graničnim vitkostima radi određivanja klase preseka, definisana je kao odnos D_e/t. Za pritisnute preseke graničnu vitkost za klasu 3 treba uzeti kao $90\varepsilon^2$, $\varepsilon^2 = 235/f_y$.

Za pritisnute EHS preseke klase 4, efektivna površina određuje se kao:

$$A_{eff} = A\sqrt{\frac{90\varepsilon^2}{D_e/t}} \tag{6}$$

za $D_e/t \leq 240\varepsilon^2$.

3.2 Proračunske procedure prema AISI-S100

Severnoamerička specifikacija za projektovanje hladnooblikovanih čeličnih elemenata AISI-S100 [2] eksplicitno ne definiše procedure za proračun EHS preseka. Međutim, metoda direktne čvrstoće (DSM), novi proračunski koncept uveden u AISI-S100, deo E3.2 [2], ne zahteva klasifikaciju poprečnih preseka, niti određivanje efektivne površine preseka. Prema ovoj metodi, izvijanje pritisnutog elemenata definisano je samo jednom krivom izvijanja. Nominalna nosivost pritisnutog elementa P_{ne}, koja istovremeno uzima u obzir uticaj plastifikacije preseka i globalnog izvijanja, definisana je kao proizvod bruto površine preseka A i napona pritiska f_n, koji treba sračunati na sledeći način:

– za $\lambda_c \leq 1.5$,

$$f_n = \left(0.658^{\lambda_c^2}\right) f_y \tag{7}$$

– za $\lambda_c > 1.5$,

$$f_n = \left(\frac{0.877}{\lambda_c^2}\right) f_y \tag{8}$$

gde je:

$$\lambda_c = \sqrt{\frac{f_y}{f_{cre}}}$$

f_{cre} – elastičan kritičan napon izvijanja (najmanji napon od kritičnog napona za fleksiono, torziono i torziono-fleksiono izvijanje).

Nominalna nosivost preseka, usled aksijalnog pritiska, P_{nl} sračunava se na sledeći način:

– za $\lambda_l \leq 0.776$,

$$P_{nl} = P_{ne} \tag{9}$$

– za $\lambda_l > 0.776$,

$$P_{nl} = \left(1 - 0.15\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4}\right)\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4} P_{ne} \tag{10}$$

where:

t is a section wall thickness;

h and b are overall dimensions of a cross-section ($h > b$).

A cross-section slenderness that is compared with limiting slenderness for obtaining a cross-section class, is defined as the ratio D_e/t. The limiting slenderness for the class 3 should be taken as $90\varepsilon^2$, $\varepsilon^2 = 235/f_y$, for sections in compression.

For EHS of the class 4 in compression, an effective area is to be determined as:

for $D_e/t \leq 240\varepsilon^2$.

3.2 Design procedures according to AISI-S100

North American specification for the design of cold-formed steel members AISI-S100 [2] fails to explicitly define regulations for EHS design. However, Direct Strength Method (DSM), the new design approach introduced in AISI-S100, Section E3.2 [2], does not require cross-section classification, neither determination of an effective cross-sectional area. According to this method, flexural buckling is defined merely by one buckling curve. Nominal axial strength P_{ne} for yielding and global buckling is obtained as a product of a gross sectional area A and a compressive stress f_n, that should be calculated as follows:

– for $\lambda_c \leq 1.5$,

– for $\lambda_c > 1.5$,

where:

$$\lambda_c = \sqrt{\frac{f_y}{f_{cre}}}$$

f_{cre} is an elastic global buckling stress (the minimum of critical stresses for flexural, torsional and flexural-torsional buckling).

A nominal axial strength for local buckling P_{nl} is calculated in the following way:

– for $\lambda_l \leq 0.776$,

– for $\lambda_l > 0.776$,

gde je:

$$\lambda_l = \sqrt{\frac{P_{ne}}{P_{crl}}}$$

P_{ne} – nosivost pritisnutog elementa koja podrazumeva uticaj plastifikacije preseka i globalnog izvijanja, i određuje se kao što je prethodno opisano;

P_{crl} – elastična kritična sila izbočavanja preseka, koja može biti određena različitim numeričkim metodama, uključujući metodu konačnih traka.

Metoda konačnih traka originalno je razvijena za otvorene hladnooblikovane profile s ravnim delovima poprečnog preseka. U praksi, ova analiza može se sprovesti primenom programa *CUFSM* [18], kao što je urađeno u ovoj studiji. Primena metode konačnih traka na elipsaste poprečne preseke delimično je testirana i upoređena sa eksperimentalnim i numeričkim rezultatima koje su dali Čen i Jang [7].

3.3 Ponašanje kratkih stubova

Simulacije ponašanja kratkih stubova daju uvid u nosivost poprečnog preseka na pritisak, izuzimajući efekte globalnog izvijanja. Na slici 4, prikazani su oblici loma kratkih stubova poprečnog preseka EHS 150x75x3 mm, dobijeni za zadate različite materijalne krive napon–dilatacija. Kao što se može primetiti, javlja se samo izbočavanje. Slični oblici loma uočeni su i za druge analizirane poprečne preseke.

where:

$$\lambda_l = \sqrt{\frac{P_{ne}}{P_{crl}}}$$

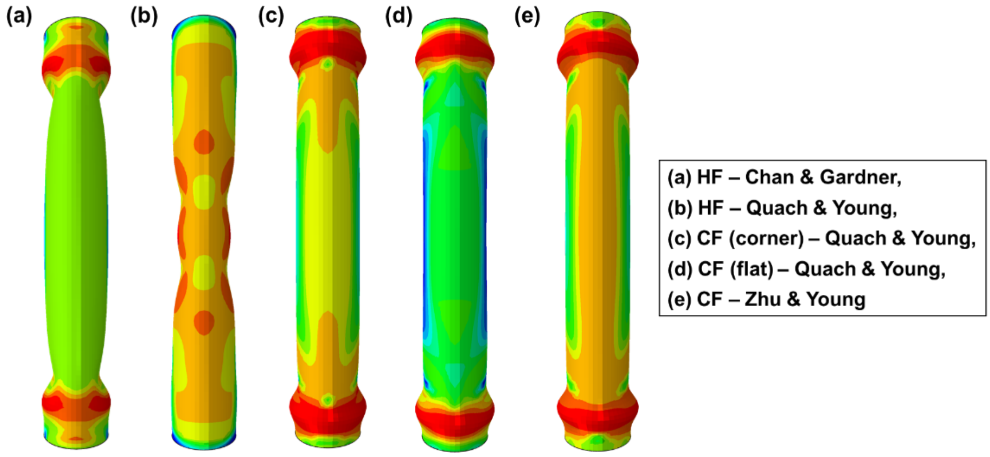
P_{ne} is a nominal axial strength for yielding and global buckling, determined as previously described;

P_{crl} is a critical elastic local column buckling load, that could be determined by different numerical methods, including finite strip analysis.

The finite strip method is originally developed for cold-formed open sections with plate elements. For practical use, this analysis could be performed in the software CUFSM [18], as it is done in this study. Applicability of the finite strip method on elliptical hollow sections is partially tested and compared to the experimental and numerical results developed by Chen and Young [7].

3.3 Stub column behaviour

Simulations of stub column tests give insight in the cross-section compressive resistance excluding overall buckling effects. In Figure 4, there are presented failure modes of stub columns of the cross-section EHS 150x75x3 mm obtained for different material stress–strain curves that were applied. As it could be noticed, only local buckling appears. Similar failure modes were observed for the other analysed cross-sections.

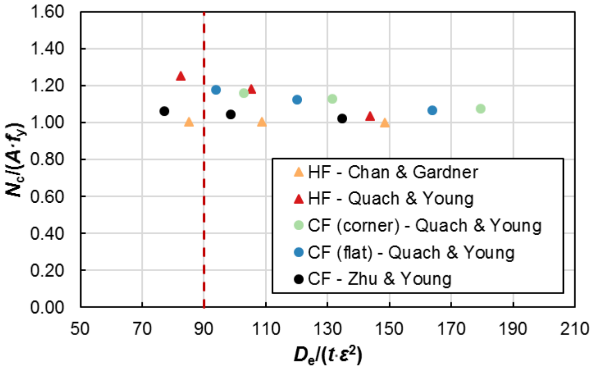


Slika 4. Oblici loma kratkih stubova
Figure 4. Failure modes of stub columns

Na slici 5, date su numerički dobijene vrednosti graničnih nosivosti analiziranih poprečnih preseka N_c, normalizovane proizvodom bruto površine preseka i napona na granici razvlačenja $A \cdot f_y$, koji odgovara koeficijentu redukcije za izbočavanje, kao što je definisano u [12]. Rezultati su predstavljeni u funkciji od $D_e/(t \cdot \varepsilon^2)$, kako bi se dobio uvid u vezu između numeričkih rezultata i granične vitkosti za klasu 3 koja je definisana kao $90\varepsilon^2$ prema EN 1993-1-1:2015 [11] (isprekidana linija na slici 5). Svi odnosi $N_c/(A \cdot f_y)$ jesu veći od 1.0, što dovodi u pitanje pouzdanost definisane granične vitkosti za potpuno pritisnute EHS preseke. Moglo bi se razmotriti pomeranje granične vitkosti sa $90\varepsilon^2$ na neku

In Figure 5, there are given numerically obtained values of ultimate resistances for analysed cross-sections N_c, normalised with a product of a gross sectional area and a yield stress $A \cdot f_y$, corresponding to the reduction factor for plate buckling, as defined in [12]. The results are plotted against $D_e/(t \cdot \varepsilon^2)$, to give a better overview of the correlation between numerical results and the defined limiting slenderness for class 3 of $90\varepsilon^2$ according to EN 1993-1-1:2015 [11] (dashed line in Figure 5). All ratios $N_c/(A \cdot f_y)$ are larger than 1.0, questioning the reliability of the cross-section limiting slenderness for totally compressed EHS. Shifting limiting slenderness from $90\varepsilon^2$ to some larger value might be veću vrednost, uz dodatna istraživanja na većem setu podataka za različite geometrije poprečnih preseka.

discussed, followed by additional investigations of the greater range of cross-section geometries.



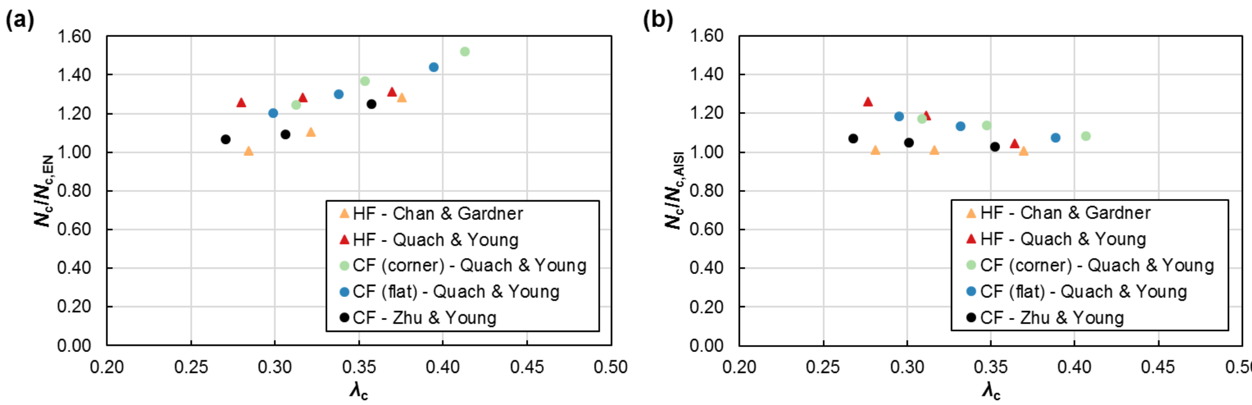
Slika 5. Poređenje numerički dobijenih nosivosti kratkih stubova sa $A \cdot f_y$
Figure 5. Comparison of numerically obtained ultimate loads of stub columns with $A \cdot f_y$

Poređenje prediktivnih nosivosti aksijalno pritisnutih EHS preseka, koje definišu EN 1993-1-1:2015 [11] i AISI-S100 [2], dato je na slici 6, gde su odnosi $N_c/N_{c,EN}$, odnosno $N_c/N_{c,AISI}$, predstavljeni u zavisnosti od relativne vitkosti preseka λ_c (definisane kao λ_l u AISI-S100). Za određivanje kritične sile izbočavanja prema AISI-S100, sprovedena je numerička analiza u softveru *CUFSM* [18], dok je za proračun prema EN 1993-1-1:2015, kritičan napon izbočavanja sračunat kao što je dato u [3].

Za analizirane poprečne preseke, primećeno je da numerički rezultati više odgovaraju predikcijama koje daje AISI-S100, nego onim koje daje EN 1993-1-1:2015. Sve sračunate nosivosti veće su od nosivosti poprečnih preseka dobijenih numeričkim putem. Najveći odnos $N_c/N_{c,EN}$ jeste 1.52, dok najveći odnos $N_c/N_{c,AISI}$ iznosi 1.26. Generalno, nisu primećeni različiti trendovi u ponašanju hladnooblikovanih i naknadno termički obrađenih kratkih stubova. Iako je AISI-S100 standard definisan za hladnooblikovane elemente, predikcije ne potcenjuju numerički dobijene vrednosti nosivosti za analizirane naknadno termički obrađene poprečne preseke.

Comparison between EN 1993-1-1:2015 [11] and AISI-S100 [2] design predictions for EHS resistance under pure axial compression is presented in Figure 6, where the ratio $N_c/N_{c,EN}$, i.e. $N_c/N_{c,AISI}$, is plotted against a non-dimensional section slenderness λ_c (defined as λ_l in AISI-S100). For obtaining a critical elastic load of local buckling according to AISI-S100, numerical analysis in the software CUFSM [18] is performed, while for calculation according to EN 1993-1-1:2015, a critical elastic buckling stress is taken as presented in [3].

For analysed sections, it is observed that numerical results fit AISI-S100 predictions better than EN 1993-1-1:2015 predictions. All calculated design resistances are larger than numerically obtained cross-section strengths. The largest ratio $N_c/N_{c,EN}$ is 1.52, while the largest ratio $N_c/N_{c,AISI}$ is 1.26. In general, different trends in behaviour of cold-formed and hot-finished stub columns are not noticed. Although AISI-S100 procedure is defined for cold-formed members, it does not underestimate numerically obtained values of the section strength for analysed hot-finished sections.



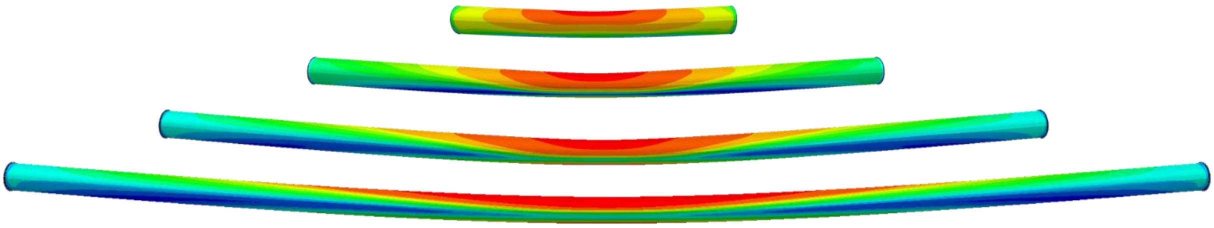
Slika 6. Poređenje numerički dobijenih nosivosti kratkih stubova s nosivošću poprečnog preseka prema: (a) EN 1993-1-1; (b) AISI-S100
Figure 6. Comparison of numerically obtained ultimate loads of stub columns with cross-section design strengths according to: (a) EN 1993-1-1; (b) AISI-S100

3.4 Fleksiono izvijanje

Tipični modeli loma stubova, dobijeni numeričkom analizom putem MKE, predstavljeni su na slici 7, na primeru elemenata, preseka EHS 150x75x3 mm i različitih dužina od 700 mm do 3100 mm, sa zadatom krivom napon–dilatacija, koju su dali Kvak i Jang za naknadno termički obrađene profile [17]. Oblik loma određen je izvijanjem oko slabije ose inercije, bez pojave efekata izbočavanja.

3.4 Flexural buckling behaviour

Typical column failure modes according to the finite element analysis are presented in Figure 7, on the example of columns EHS 150x75x3 mm of various lengths from 700 mm to 3100 mm, with the applied material stress–strain curve given by Quach and Young for hot-finished elliptical sections [17]. Column failure occurred due to flexural buckling about minor principal axis, without local buckling effects.



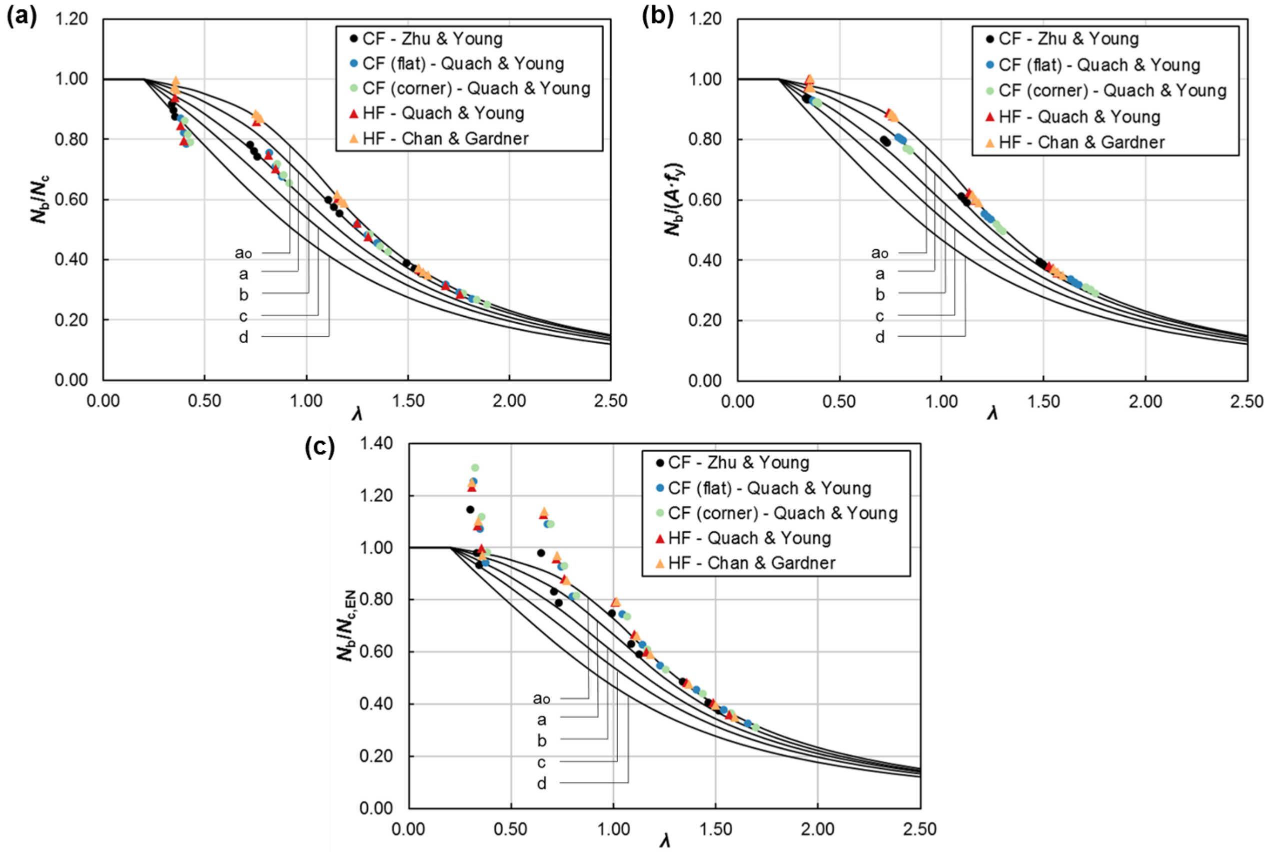
Slika 7. Tipični modeli loma stubova
Figure 7. Typical columns failure modes

Rezultati analize fleksionog izvijanja najčešće se predstavljaju putem zavisnosti bezdimenzionalnog koeficijenta izvijanja, koji je jednak normalizovanoj nosivosti stuba, usled dejstva aksijalnog pritiska N_b/N_c, i relativne vitkosti elementa λ. Međutim, slika 8 pokazuje da takva prezentacija rezultata može biti zbunjujuća i neprimenljiva u pojedinim slučajevima. Na slici 8 predstavljena su granična opterećenja dostignuta tokom aksijalnog pritiska elementa, dobijena numeričkom analizom, normalizovana sa: (1) nosivošću poprečnog preseka na pritisak, dobijenom iz simulacija ponašanja kratkog stuba N_c (slika 8.a); (2) proizvodom bruto površine preseka i granice razvlačenja, $A \cdot f_y$ (slika 8.b); i (3) nosivošću sračunatom prema EN 1993-1-1:2015, uzimajući u obzir efektivnu površinu u slučaju preseka klase 4, $N_{c,EN}$ (slika 8.c). Kako su ove tri vrednosti različite, tako se i predstavljeni rezultati međusobno razlikuju. Na primer, posmatrajući sliku 8.c, moglo bi se zaključiti da krive izvijanja definisane u EN 1993-1-1:2015 u opštem slučaju dovode do rezultata koji su na strani sigurnosti. Međutim, pre izvođenja takvog zaključka, treba biti svestan relativne greške koja se može desiti pri određivanju klase poprečnog preseka i efektivne površine, kao što je opisano u prethodnom delu 3.3. Navedene konzervativnosti dovode do dodatnih nepreciznosti pri određivanju bezdimenzionalnog koeficijenta izvijanja. U prilog navedenom, uočava se viši stepen međusobnog poklapanja podataka na slikama 8.a i 8.b, dok raspodela rezultata prikazanih na slici 8.c odstupa u odnosu na ekvivalentne vrednosti na slikama 8.a i 8.b.

Sa slike 8 zapaža se da svi rezultati leže između krivih *a* i a_0 za stubove s relativnom vitkošću većom od 1.5. Kako vitkost elementa opada, tako je varijacija u dobijenim rezultatima veća. Stoga, za stubove s relativnom vitkošću manjom od 0.5, rezultati za hladnooblikovane EHS profile leže iznad krive *c* (slika 8.b). Nasuprot tome, rezultati za naknadno termički obrađene uzorke leže iznad krive *a*. S povećanjem vitkosti, izvijanje hladnooblikovanih elemenata približnije odgovara višim krivama izvijanja. Međutim, rezultati ispitivanih naknadno termički obrađenih EHS preseka u celom opsegu vitkosti odgovaraju krivama izvijanja *a* i a_0.

The results of flexural buckling analysis are usually presented through relationship between a buckling reduction factor, equal to a normalised column resistance due to axial compression N_b/N_c, and a non-dimensional slenderness λ. However, Figure 8 shows that such presentation of the results might be confusing and inapplicable in some cases. In Figure 8, there are presented ultimate loads reached during axial compression of members obtained from numerical analysis, normalised with: (1) a compressive cross-section resistance as it is obtained from stub column simulations N_c (Figure 8.a); (2) a product of a gross cross-sectional area and a yield strength, $A \cdot f_y$ (Figure 8.b); and (3) a design strength as it is calculated according to EN 1993-1-1:2015, taking into account an effective area for cross-sections of the class 4, $N_{c,EN}$ (Figure 8.c). As those three values are not equal, given results differ as well. For example, by looking at Figure 8.c, some might conclude that buckling curves defined in EN 1993-1-1:2015 in general lead to safe-sided results. However, before making such a conclusion, one should be aware of the inaccuracy that could be made while determining cross-section class and an effective sectional area, as explained in the previous section 3.3. Mentioned conservativeness lead to additional inaccuracies at the point of buckling reduction factor determination. In addition to the stated, it could be observed that Figures 8.a and 8.b fit each other rather, while results presented in Figure 8.c vary comparing to equivalent values in Figures 8.a and 8.b.

It could be observed from Figure 8 that all results lie in-between curves *a* and a_0 for columns of a non-dimensional slenderness greater than 1.5. As the slenderness tends lower, the variation of the results is greater. Therefore, for columns of the non-dimensional slenderness lower that 0.5, the results for cold-formed EHS lie above the curve *c* (Figure 8.b). Contrary, the results for hot-finished specimens lie above the curve *a*. By increasing slenderness, buckling of cold-formed members fits higher curves. However, the results for studied hot-finished EHS for all range of slenderness correspond to *a* and a_0 buckling curves.



Slika 8. Krive izvijanja i numerički rezultati normalizovani sa:
(a) nosivošću poprečnog preseka, dobijenom iz numeričke analize ponašanja kratkih stubova; (b) $A \cdot f_y$;
(c) nosivošću poprečnog preseka prema EN 1993-1-1
Figure 8. Column buckling curves and FEA results normalised with:
(a) cross-section strengths from FEA of stub columns; (b) $A \cdot f_y$;
(c) cross-section design strengths according to EN 1993-1-1

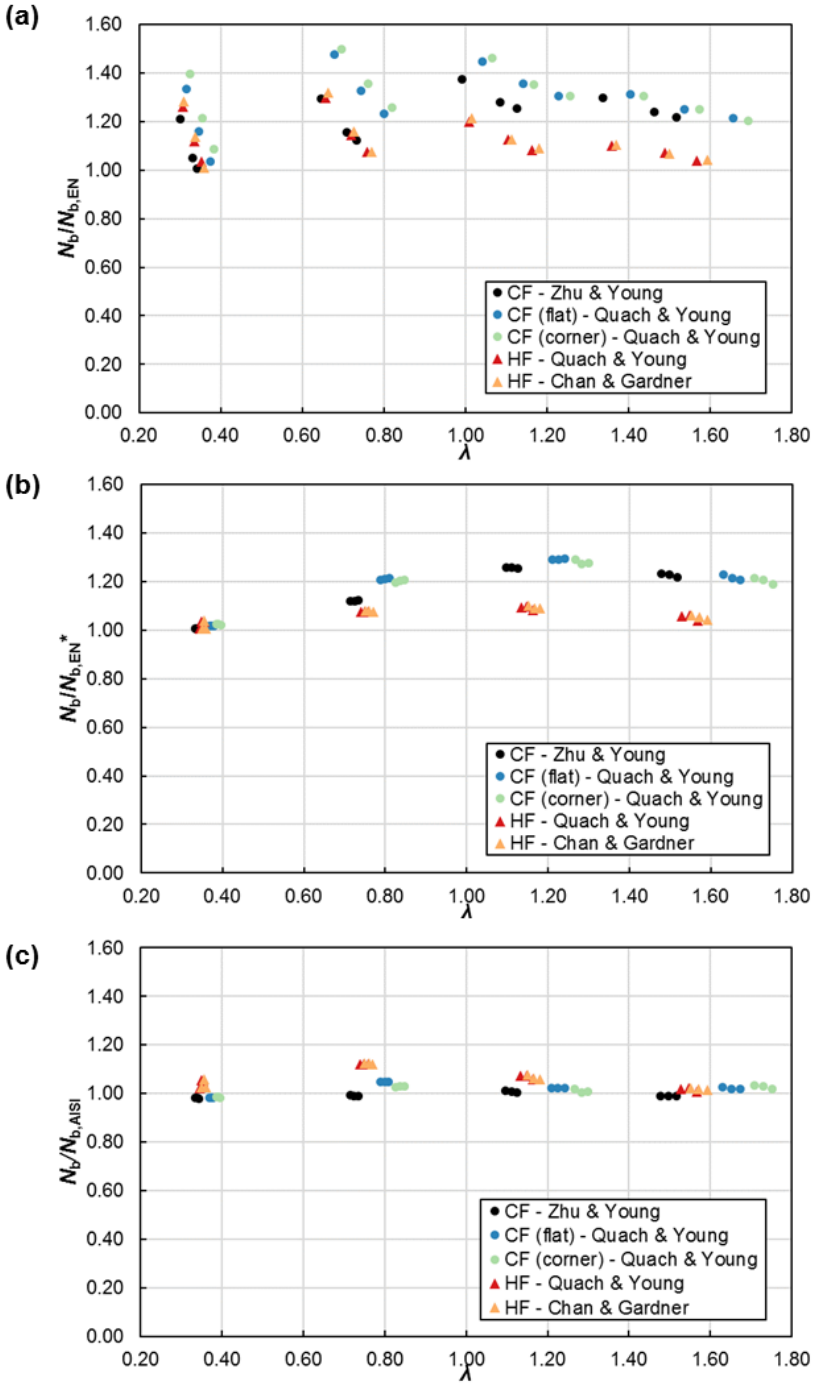
Poređenje predikcija koje daju EN 1993-1-1:2015 i AISI-S100 predstavljeno je na slici 9, gde je odnos graničnog opterećenja i definisane nosivosti stuba na izvijanje $N_b/N_{b,EN}$, odnosno $N_b/N_{b,AISI}$, prikazan u funkciji relativne vitkosti stuba λ (definisane kao λ_c u AISI-S100). Primećuje se da Severnoamerička specifikacija (slika 9.c) daje predviđanja bliža numeričkim rezultatima nego što daje EN 1993-1-1:2015 (slika 9.a). Podudaranje rezultata posebno je dobro za hladnooblikovane elemente, dok je rasipanje rezultata veće u slučaju naknadno termički obrađenih EHS stubova. Ovakav ishod nije iznenađujući, imajući u vidu to što je AISI-S100 standard za projektovanje hladnooblikovanih čeličnih elemenata. Svega nekoliko rezultata nije na strani sigurnosti, međutim, u tom slučaju razlika u numerički dobijenoj i prediktivnoj nosivosti stuba ni u jednom slučaju nije veća od 1%.

S druge strane, procedura koju definiše EN 1993-1-1:2015 dovodi do konzervativnih predikcija, u jednom slučaju čak za 50% većih u poređenju s rezultatom dobijenim numerički. Kao što je prethodno napomenuto, razlika u rezultatima jeste posledica konzervativnog usvajanja efektivne površine poprečnog preseka. Kako bi se ovakva greška umanjila i kako bi se stekao bolji uvid u određene proračunske procedure definisane za fleksiono izvijanje, na slici 9.b dat je odnos $N_b/N_{b,EN}$*,

Comparison of EN 1993-1-1:2015 and AISI-S100 predictions is presented in Figure 9, where the ratio of an ultimate column load and a design column strength $N_b/N_{b,EN}$, i.e. $N_b/N_{b,AISI}$, is plotted against a non-dimensional column slenderness λ (defined as λ_c in AISI-S100). It can be observed that North American specification (Figure 9.c) gives predictions closer to numerically obtained results than EN 1993-1-1:2015 (Figure 9.a). The result match is especially good for cold-formed members, while dissipation is greater in the case of hot-finished EHS. Such outcome is not surprising as AISI-S100 is the standard for the design of cold-formed steel members. Only few results are unsafe-sided, however in that case the difference in numerically obtained buckling resistance and a design prediction is never larger than 1%.

On the other side, the procedure that follows EN 1993-1-1:2015 leads to conservative predictions, in one case even up to 50% greater than the numerical result. As previously underlined, the difference in the results is a consequence of the conservative adoption of the cross-sectional effective area. In order to reduce such an error and to give a better overview of the specific design procedure defined for flexural buckling, in Figure 9.b the ratio $N_b/N_{b,EN}$* is obtained disregarding an effective cross-sectional area and calculating with a gross zanemarena je efektivna površina preseka i umesto nje, računajući s bruto površinom, takav postupak označen je kao modifikovani EN 1993-1-1. Međutim, i dalje je uočljivo precenjivanje definisanih nosivosti veće od 10% u slučaju analiziranih hladnooblikovanih elemenata relativne vitkosti veće od 0.80.

sectional area instead, marking that procedure as the modified EN 1993-1-1. However, the overestimation of a design strength greater than 10% is still noticeable for all analysed cold-formed members of a non-dimensional slenderness larger than 0.80.



Slika 9. Poređenje numeričkih rezultata s nosivošću stuba na izvijanje prema: (a) EN 1993-1-1; (b) modifikovanom EN 1993-1-1; (c) AISI-S100

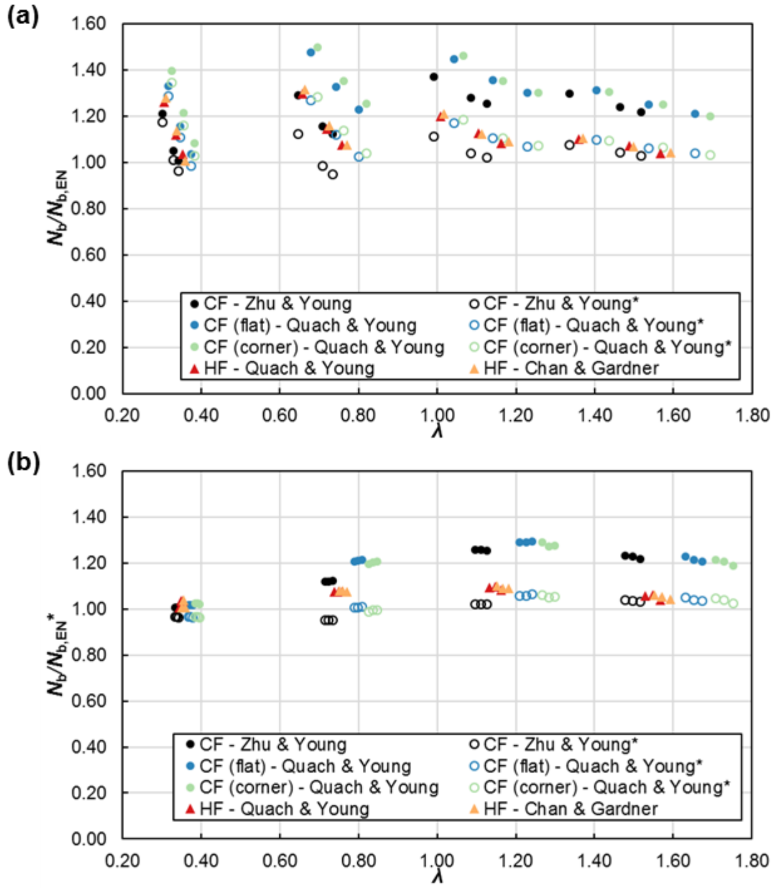
Figure 9. Comparison of FEA results with column design strengths according to: (a) EN 1993-1-1; (b) modified EN 1993-1-1; (c) AISI-S100

U sledećem koraku, iscrtani su grafici na slici 10, koji obuhvataju prikaz rezultata sa slike 9 i dodatno poređenje između numeričkih rezultata i proračunskih predikcija prema Evrokodu, korišćenjem krive izvijanja *a* takođe za hladnooblikovane elemente (prstenasti simboli na slici 10). Kao što je očekivano, u opsegu veće vitkosti elementa, nosivosti stubova sračunate s krivom *a* približnije su onim dobijenim numeričkom analizom. Nasuprot tome, usvajanje krive *a* u domenu manje vitkosti elementa ne vodi do rezultata na strani sigurnosti.

Primećeno je da se rezultati dobijeni korišćenjem materijalnih krivih koje su dali Kvak i Jang [17] za najravnije i najzakrivljenije epruvete ne razlikuju znatno. Međutim, razlike u fleksionom izvijanju hladnooblikovanih elemenata s materijalnim karakteristikama, koje su dali Kvak i Jang [17] i Zu i Jang [21], jesu uočljivije. To se može objasniti različitim oblicima materijalnih krivih, što se vidi iz različitih odnosa konvencionalne granice razvlačenja i čvrstoće na zatezanje. Ipak, prethodno je naglašeno da epruvete koje su testirali Zu i Jang nisu izvađene iz elipsastih, već iz ovalnih profila i zbog toga moraju biti uzete s rezervom. Naknadno termički obrađeni EHS elementi s materijalnim krivama, koje su publikovali Čen i Gardner [5] i Kvak i Jang, prate isti trend ponašanja.

In the next step, there are plotted graphs on Figure 10 including the results shown on Figure 9 with an addition of a comparison between numerical results and design predictions according to the Eurocode by applying curve *a* to cold-formed members as well (unfilled circles in Figure 10). As expected, design column strengths calculated with the curve *a*, for larger slenderness region predict results closer to the results of the finite element analysis. In the opposite, adoption of the curve *a* in low slenderness domain does not lead to safe-sided predictions.

It is noticed that the results conducted by using material curves given by Quach and Young [17] for flat and corner coupons differ insignificantly. However, the differences in flexural behaviour of members with applied cold-formed material properties of Quach and Young [17] and of Zhu and Young [21] are more noticeable. It could be explained by the different shape of material curves, which is observed by comparing a ratio between 0.2% proof stress and a tensile strength. Although, it is outlined that material coupons tested by Zhu and Young are not taken from elliptical, but from oval sections and for that reason should be taken with caution. At the point of hot-finished EHS, members with applied material curves given by Chan and Gardner [5] and Quach and Young follow the same trend.



*Slika 10. Poređenje numeričkih rezultata s nosivošću stuba, imajući u vidu krive izvijanja a (naknadno termički obrađeni EHS i hladnooblikovani EHS – označeni sa *) i c (hladnooblikovani EHS) prema: (a) EN 1993-1-1; (b) modifikovanom EN 1993-1-1*

*Figure 10. Comparison of FEA results with column design strengths accounting buckling curves a (hot-finished EHS and cold-formed EHS - marked with *) and c (cold-formed EHS) according to: (a) EN 1993-1-1; (b) modified EN 1993-1-1*

4 ZAKLJUČCI

U ovom radu predstavljena je numerička evaluacija fleksionog izvijanja elemenata elipsastih poprečnih preseka. Formirani su modeli naknadno termički obrađenih i hladnooblikovanih stubova i odgovarajući modeli kratkih stubova za svaki analizirani poprečni presek i zadate materijalne karakteristike. Simulirana je globalna imperfekcija sa amplitudom 1/1000 dužine stuba, dok je amplituda lokalne imperfekcije varirana od 1/100 do 1/10 debljine lima, ali nije pokazala značajan uticaj na rezultate. Materijalne krive napon–dilatacija zadate su putem rezultata testova na zatezanje, koji su publikovani u drugim istraživanjima EHS preseka. Analizirano je samo izvijanje oko slabije ose inercije.

Dobijeni numerički rezultati za naknadno termički obrađene i hladnooblikovane elemente upoređeni su međusobno i s proračunskim kriterijumima definisanim u revidiranoj verziji Evrokoda za proračun čeličnih konstrukcija EN 1993-1-1:2015 [11] i Severnoameričkoj specifikaciji za projektovanje hladnooblikovanih čeličnih elemenata AISI-S100 [2]. Uočeno je i to da je Severnoamerički standard precizniji nego Evrokod u predviđanju i nosivosti kratkih stubova i nosivosti stubova na izvijanje, za sve analizirane naknadno termički obrađene i hladnooblikovane elemente. Samo za nekoliko stubova, AISI-S100 predikcije nisu na strani sigurnosti, ali ni u tim slučajevima – ne za više od 1%. Ispostavilo se da je granična vitkost za klasu 3 aksijalno pritisnutih preseka prema EN 1993-1-1:2015 potcenjena. Navedena konzervativnost dovodi do rezultata na strani sigurnosti (do 50%) za definisane nosivosti poprečnog preseka i nosivosti stuba na izvijanje. Stoga, preporučuju se modifikacije.

U oblasti veće vitkosti elemenata, kada se fleksiono izvijanje događa u oblasti elastičnih napona pre dostizanja granice razvlačenja, uočeno je slično ponašanje kod stubova proizvedenih hladnim oblikovanjem i naknadnom termičkom obradom. Suprotno tome, za manje vitkosti elemenata, hladnooblikovani stubovi teže nižim krivama izvijanja u odnosu na naknadno termički obrađene. Zaključuje se da kapaciteti stubova u pogledu nosivosti na pritisak veoma zavise od osobina materijala, posebno u oblasti nelinearnog ponašanja materijala.

Poređenjem krivih izvijanja, datih u EN 1993-1-1:2015 [11] s numeričkim rezultatima, uočeno je da u oblasti manje vitkosti elemenata, rezultati odgovaraju krivama *a* i *c*, koje su respektivno definisane za naknadno termički obrađene i hladnooblikovane elipsaste poprečne preseke analiziranog kvaliteta materijala. Međutim, u oblasti veće vitkosti, izvijanje svih modela odgovara krivoj *a*. Stoga, treba biti svestan da standard daje konzervativne preporuke za analizirano izvijanje oko slabije ose inercije pri primeni na sve hladnooblikovane elemente.

Ova komparativna analiza daje uvid u različito ponašanje hladnooblikovanih i naknadno termički obrađenih stubova EHS poprečnih preseka izloženih fleksionom izvijanju, koji su za sada samo delimično pokriveni standardima za projektovanje i koji nisu dovoljno eksperimentalno i numerički istraživani. Za donošenje opštih zaključaka, u budućim istraživanjima trebalo bi uključiti i druge odnose dimenzija EHS profila, kao i veći opseg vitkosti poprečnih preseka. Iako je sličan trend primećen u fleksionom izvijanju oko slabije i jače ose inercije

4 CONCLUSIONS

This paper presents numerical evaluation of flexural buckling of elliptical hollow section members. Column models of hot-finished and cold-formed sections are made, and corresponding stub column models for each cross-section and the applied material properties are developed as well. An overall imperfection is included in simulations with an amplitude of 1/1000 of a member length, while a local imperfection amplitude is varied from 1/100 to 1/10 of a plate thickness, but shows insignificant effect on the results. Material stress–strain curves are applied through the results of tensile coupon tests published in other researches of EHS. Only buckling about minor principal axis is analysed.

The obtained numerical results for hot-finished and cold-formed members are compared among themselves and with the design criteria defined in the draft version of the Eurocode for the design of steel structures EN 1993-1-1:2015 [11] and North American specification for the design of cold-formed structural members AISI-S100 [2]. It is observed that North American standard provides better predictions of both a stub column strength and a buckling column resistance than Eurocode, for all analysed hot-finished and cold-formed members. Only for a few columns AISI-S100 design predictions are unsafe-sided, but even in that case not more than 1%. The limiting slenderness for pure compression for the class 3 according to EN 1993-1-1:2015 turned out to be underestimated. Mentioned conservativeness leads to safe-sided results (up to 50%) for design predictions of both cross-section compression strength and column buckling strength. Therefore, modifications are suggested.

In the range of higher column slenderness, when flexural buckling happens in the elastic stress domain before reaching the yielding stress, similar behaviour is observed for cross-sections produced both by cold-forming and hot-finishing. Contrary, for a lower slenderness, cold-formed members tend to lower buckling curves in comparison with the hot-finished ones. It is concluded that compressive capacities of columns are highly dependent on their material properties, especially in non-linear stress–strain domain.

Comparing buckling curves given in EN 1993-1-1:2015 [11] to the numerical results, it is noticed that in the lower slenderness area, the results fit curves *a* and *c*, which are defined for hot-finished and cold-formed elliptical hollow sections of the analysed steel grades, respectively. However, in the higher slenderness region, buckling of all models correspond to the curve *a*. Therefore, one must be aware that design predictions for the analysed buckling about minor axis of inertia are conservative if adopted for all cold-formed members.

This comparative study provides an outlook on the different behaviour of cold-formed and hot-finished EHS members exposed to flexural buckling, that are still only partly covered by design standards and insufficiently experimentally and numerically investigated. For making further general conclusions, the other aspect ratios of EHS should be included in future researches, as well as the larger range of a cross-section slenderness. Although there is a similar trend observed in flexural buckling response both about minor and major axis of naknadno termički obrađenih EHS stubova koje su ispitivali Čen i Gardner [5], u budućim istraživanjima moglo bi se ispitati da li isto važi za hladnooblikovane stubove.

inertia in the research of hot-finished EHS columns done by Chan and Gardner [5], in some future work it might be tested if the same applies on cold-formed columns.

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REZIME

FLEKSIONO IZVIJANJE NAKNADNO TERMIČKI OBRAĐENIH I HLADNOOBLIKOVANIH STUBOVA ELIPSASTOG POPREČNOG PRESEKA: NUMERIČKA UPOREDNA ANALIZA

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Iako se poslednjih godina šuplji profili elipsastog poprečnog preseka (EHS) mogu naći na tržištu, pravila za njihovo dimenzionisanje su tek delimično uvedena u odgovarajuće standarde za projektovanje. Cilj ovog rada jeste da prikaže komparativnu numeričku analizu ponašanja centrično pritisnutih hladnooblikovanih i naknadno termički obrađenih EHS stubova, usled fleksionog izvijanja. Analizirani su zglobno oslonjeni stubovi različitih vitkosti bez bočnog pridržanja duž elementa, uzimajući u obzir početne geometrijske imperfekcije. Nelinearno ponašanje materijala modelirano je na osnovu publikovanih eksperimentalnih rezultata testova pri zatezanju, sprovedenih u prethodnim ispitivanjima. Takođe, izvršene su numeričke simulacije ponašanja kratkih stubova kako bi se utvrdila nosivost poprečnog preseka na pritisak. Rezultati numeričke analize sprovedene metodom konačnih elemenata upoređeni su s računskim vrednostima graničnih nosivosti u skladu s revidiranom verzijom Evrokoda EN 1993-1-1:2015 i Severnoameričkom specifikacijom za projektovanje hladnooblikovanih čeličnih elemenata AISI-S100. Uočeno je da Severnoamerički standard daje preciznije predikcije – i za nosivosti poprečnog preseka i za nosivosti stuba na fleksiono izvijanje. Metoda ekvivalentnog prečnika, opisana u Evrokodu 3, daje konzervativnije rezultate. Prema dobijenim numeričkim rezultatima, slično ponašanje pri fleksionom izvijanju uočeno je kod naknadno termički obrađenih i hladnooblikovanih stubova veće vitkosti, dok u oblasti manje vitkosti, hladnooblikovani pritisnuti elementi teže nepovoljnijim krivama izvijanja. Primećeno je da Evrokod potcenjuje nosivosti na izvijanje hladnooblikovanih stubova veće vitkosti.

Ključne reči: elipsasti poprečni preseci, fleksiono izvijanje, krive izvijanja, efektivna površina poprečnog preseka, metoda direktne čvrstoće, kratki stub, numerička analiza

SUMMARY

FLEXURAL BUCKLING OF HOT-FINISHED AND COLD-FORMED ELLIPTICAL HOLLOW SECTION COLUMNS: NUMERICAL COMPARATIVE ANALYSIS

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Even though structural elements of the elliptical hollow section (EHS) have been introduced to the construction market in the past years, the rules for their design were only partially introduced into the suitable codified procedures. The aim of this paper is to present a comparative numerical analysis of behaviour of cold-formed and hot-finished EHS columns exposed to flexural buckling under pure axial compression. Pin-ended columns of a various slenderness without lateral restrains and with incorporated geometrical imperfections are analysed. Material nonlinear behaviour is included through published experimental tensile test results, conducted in previous researches. In addition, numerical simulations of stub column behaviour are performed in order to obtain a cross-section compressive resistance. Results of the finite element analysis are compared to the design criteria defined in the draft version of Eurocode EN 1993-1-1:2015 and in North American specification for cold-formed member design AISI-S100. It is observed that North American standard provides more accurate predictions of both a cross-section resistance and a buckling column resistance. Equivalent diameter method described in Eurocode 3 led to more conservative results. According to the obtained numerical results, similar buckling behaviour is observed for both hot-finished and cold-formed columns of higher slenderness, while in the lower slenderness region, cold-formed compressed members tend to lower buckling curves. Underestimation of a column buckling resistance according to Eurocode regulations is noticed for cold-formed members of a higher slenderness.

Key words: elliptical hollow sections, flexural buckling, buckling curves, effective cross-sectional area, direct strength method, stub columns, numerical analysis

INTERACTIVE ALGORITHM FOR GEOMETRIC MODELLING DOUBLE-CURVATURE ARCH DAMS

INTERAKTIVNI ALGORITAM ZA GEOMETRIJSKO MODELIRANJE DVOJNO ZAKRIVLJENIH BRANA

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ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK: 627.82.036.3:004.925.8
doi:10.5937/GRMK1902033M

1 INTRODUCTION

In suitable terrain and geological conditions, arch dams consistently outperform other concrete dam types in terms of load capacity, safety and economic efficiency. Benefiting from the concrete compressive strength and the advantageous geometry, arch dams effectively transmit static and dynamic loads to the abutments and foundation. To further decrease the amount of construction material and push the financial savings to higher levels, arch dams structural design and construction techniques focus on variable geometry minimizing the thickness. The dam design process starts with the arch dam initial layout. The next phase consists of gradual adjustments of the dam complex geometry and comparison of results of structural analyses. The analyses in the successive optimization steps consist of structural evaluation and sensitivity calculations of critical parameters, such as displacement and stresses developed in the dam body, foundation and abutments. In response to the results of the complex analyses, the engineer gradually updates the arch dam geometry model in the Euclidean three-dimensional space. The double-curved shape of an arch dam requires judicious evaluation and optimization of the design parameters yet, the most widespread arch dam design techniques are still dependent on engineering judgment and experience and involve recommendations given in the traditional technical documents.

With the rapid increase of computer capacity, the computer-aided engineering design became the standard technique for design and optimization modelling of arch dams. The optimization process of the dam geometry and structural parameters is carried out until the dam optimal design satisfies the allowable design criteria [1-4]. The software accuracy and capacity to carry out complex structural and sensitivity analyses and fine-tuning of the dam geometry contribute to the effectiveness of the final dam shape [3-7]. The existing numerical methods can be categorized into three broad groups: geometric and feature modelling [12-17], solid modelling [18-19] and surface modelling [20-22]. Wassermann (1983) [5], used eight-node isoparametric solid finite elements to model a double-curvature arch dam and to solve the optimization problem by the sequential linear programming method [1]. To achieve general 3D surface design optimization (Mortenson 1985) [4], the Bezier surface for shape design parameterization was applied [23-24]. Seyedpoor et al.

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[8] focused on soft computing techniques for optimal design of arch dams. The authors first simulated the dam-water–foundation rock system using the finite element method, and then conducted the optimization process. To further accelerate the optimization process, the generally time-consuming dynamic finite element analyses were replaced with the neuro-fuzzy inference system combined with the particle swarm optimization [8]. On the other hand, to create a solid model of an arch dam, the computer-aided design platform CATIA V5 [25-26] develops arch dam geometric characteristics applying geometric and feature modelling methods [17].

Although equipped to carry out sophisticated analyses, current computer-aided design models have a range of shortcomings. As indicated by Kirk Martini [27], the developed algorithms for geometric modelling are often not in full compliance with requirements for practical and applicable engineering drawings, i.e., while traditional engineering drawings apply a concise description of the dam geometry that can easily be modified and decomposed in a top-down manner, [17,27] the current computer models generally use a bottom-up approach [27]. The latter approach appears not well suited for design purposes, in particular for cases where an update of the dam geometry can be a demanding task. To overcome the above issues, considerable research was conducted on the effectiveness of computer-based geometric modelling techniques to capture practical design problems. A few algorithms exist in the literature for automatic generation of dam geometry based on mathematical description of the dam structural geometry considering top-down approach. For example, Goulas [23] developed a procedure for the preliminary design of double-curvature arch dams, compatible with the well-known DIANA software [24]. D. J.Vicente [28] presented an innovative software for preliminary geometric design of double-curvature arch dams FEM models based on the US standards [28]. Likewise, to optimize the dam geometry for static and dynamic loads, Jalal Akbari [5] suggested a practical computer algorithm applying Hermit cubic splines. It can be concluded from the above discussion that improvement and sophistication of the computer-based geometric modelling and visualization techniques are needed to capture the necessary practical design issues.

In this paper, an original algorithm for a user-friendly design of arch dams is presented that effectively combines the advantages of the traditional geometric-design methods with innovative graphical and computational capabilities [12,13]. The main objective is to further facilitate the decision-making in the dam design process. The generated design algorithm was implemented in and runs parallel with the ADAD-IZIIS software for structural analyses of concrete arch and gravity dams [29]. The implemented design process includes the following successive steps: (i) define topography and select potential dam micro location(s); (ii) design dam initial layout; (iii) generate finite element mesh of a single and double-curvature arch dam together with a portion of the foundation rock to account for dam-foundation interaction, (iv) generate boundary element mesh representing the boundaries of the fluid domain to account for the fluid-dam interaction, (v) introduce construction joints within the dam body and peripheral joints at the dam-foundation interface, (vi) analyze the stress-strain state under static and dynamic loads, (vii) compare the analysis results, and (viii) return to step (ii) with the updated dam geometry (if necessary). To demonstrate the effectiveness of the proposed method for geometric design, an example is given of a 130 m high double-curvature arch dam located in a V-shape canyon.

2 TOPOGRAPHY AND OPTIMAL LOCATION

The proposed dam geometric design process starts with generation of a mathematical model of the terrain topography. The digital elevation model of the potential construction site(s) is created from elevation points from geodetic surveys (e.g., light detection and ranging LIDAR data) and/or existing numerical topographic maps. The coordinates of the point raster are defined in a fixed global Cartesian system (X_G-Z_G-Z_G), where the Z_G axis points vertically upward, Fig. 1a. Equally spaced contour lines are then mathematically presented with short straight-line segments or, in case of insufficient data density, with polynomials of a second order passing through three neighbouring points. In the vertical direction, further densification of the spacing between contour lines can be done where necessary.

The topography is defined in a global coordinate system, which remains unchanged throughout the design procedure. The geometry of the arch dam components, however, is determined in both the global coordinate system and in a changeable local Cartesian coordinate system (X_L-Y_L-Z_L-α). The local coordinate system facilitates generation and visualization of the crown cantilever and the horizontal arches and updating of their geometry. The origin of the local coordinate system, Xc-Yc-Zc, and the deflection angle α with respect to the global coordinate system, can change during the location selection process. In the presented example, the arch dam local coordinate system was rotated by 87º in the counter clockwise direction, Fig. 1.

The local coordinates of any point of the arch dam are transformed into global coordinates with the following equation,

$$\begin{Bmatrix} X_G \\ Y_G \\ Z_G \end{Bmatrix} = \begin{bmatrix} \cos\alpha & -\sin\alpha & 0 \\ \sin\alpha & \cos\alpha & 0 \\ 0 & 0 & 1 \end{bmatrix} \begin{Bmatrix} X_L \\ Y_L \\ Z_L \end{Bmatrix} + \begin{Bmatrix} X_C \\ Y_C \\ Z_C \end{Bmatrix} \tag{1}$$

The horizontal coordinates Y_C and X_C are different from zero, while the vertical coordinate Z_C could equal zero in case when the absolute global Z_G coordinates are used for definition of the vertical 2D model of the crown cantilever in the reference plane, Fig. 1b.

The algorithm allows the possibility to select among the multiple potential locations the one that offers at this stage the most cost-effective solution. The selection process is based on a comparative study, where the user defines a number of vertical cross sections in the studied narrow portion of the canyon with successively increasing dam heights. In general, canyons with complex topography and geometrical singularities require higher number of potential locations and dam elevations. In Fig. 2 is illustrated such an example of selection of the dam optimal location and height.

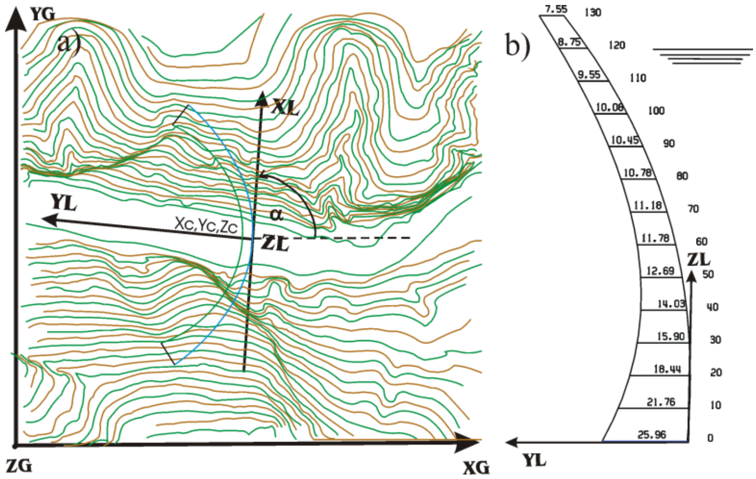


Figure 1. Global (G) and in local (L) coordinate systems for: a) Location of the crown arch of the dam, and b) Cross section of the crown cantilever representing the reference plane in the local coordinate system. The origin of the local coordinate system is indicated with Xc-Yc-Zc.

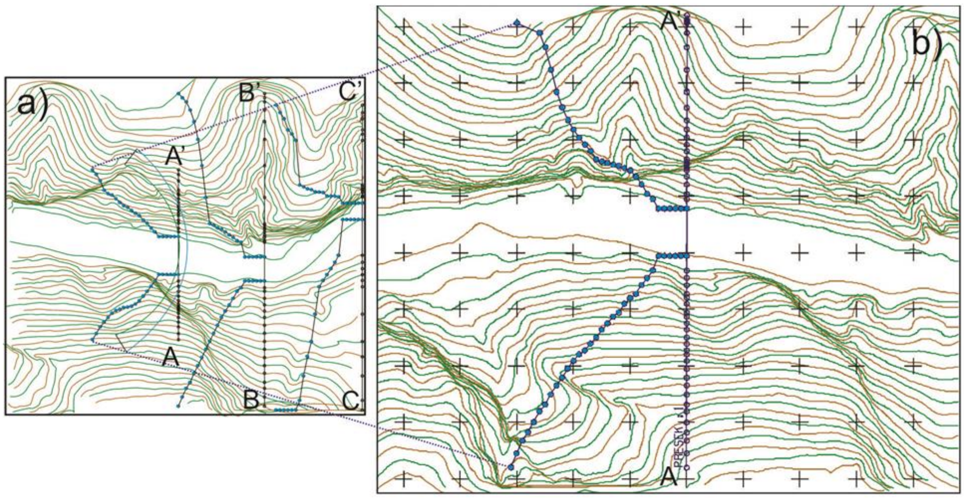


Figure 2. Selection of optimal dam location: a) potential locations (straight lines) and respective cross section profiles (overturned projections) of the dam foundation, and b) final location of the dam (cross section A-A').

Three locations with different dam heights are explored in Fig. 2a. In the selection process the dam height was gradually increased with successive height increments of 10m. The optimal location, cross section A-A' in this case, was selected in the narrowest portion of the canyon up to the extent of the steepest portion of the sidewalls and with respective shortest crest length. Any increase of the dam height beyond the final 130 m, could have resulted in unattractive solution from an economic viewpoint due to the progressively increased length of the dam crest.

3 GEOMETRY OF CROWN CANTILEVER AND ELEVATION ARCHES

Following the selection of the dam location, the arch dam geometrical model is built in a form of circular segments assigned to each incremental height. The curved shape results from the combination of both the crown cantilever geometry and the curvature of the composite arches. As the geometry of a double curvature dam curves horizontally towards the abutments and vertically towards the crest, the design of the arches is conducted in parallel to the definition of the crown cantilever.

3.1 Crown cantilever geometry

The initial dimensions of the crown cantilever depend on the shape of the selected topographic profile, mainly characterized by the vertical distance from the dam crest to the foundation rock, referred to as structural height (H), and straight line distances between the abutments at the dam crest and at 0.15H including assumed excavation to the solid rock. The crown cantilever is with vertical curvature to counter the development of tensile stresses in the structure and has two curved surfaces on the upstream and downstream face of the dam.

The crown cantilever is defined in the Y_L-Z_L reference plane with the following local coordinates, Fig 1b:

• Y_{LEn}, Z_{LEn} - coordinates of the extrados points with respect to the local reference system X_L-Y_L-Z_L, where: $X_{LE}=0$ and $Y_{LEn}\neq0$, $Z_{LEn}\neq0$ for n=1,K , where K represents the number of selected elevations for the dam design;

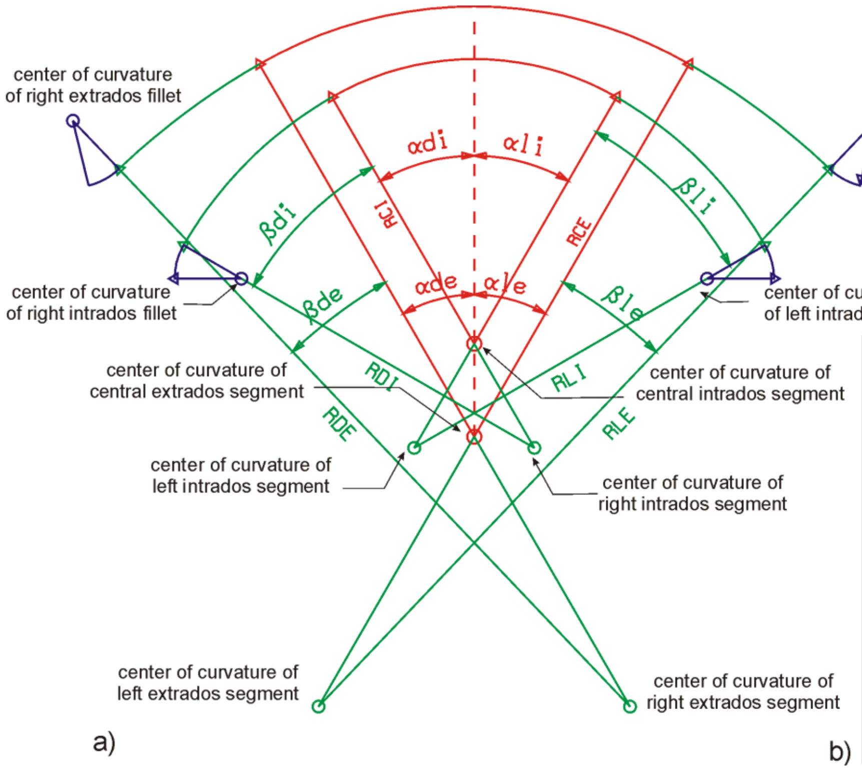
• Y_{LIn}, Z_{LIn} - coordinates of the intrados points with respect to the local reference system X_L-Y_L-Z_L, where: $X_{LI}=0$ and $Y_{LIn}\neq0$, $Z_{LIn}\neq0$ for n=1,K.

The reference plane Y_L-Z_L and $X_L=0$ is the imaginary upstream to downstream vertical cross section through the crown cantilever and the line-of-centres related to the central segment of the dam, Fig 1b. In the given example, the crown cantilever was divided in a number of superimposed substructures corresponding to each 10m incremental dam height. In this way, the thickness of the crown cantilever becomes a design variable as function of the geometrical features of the selected topographic profile, Fig. 1b. To mathematically determine the shape of the crown cantilever, curve fitting was applied through the coordinates of nodes representing its upstream and downstream faces. Among the tested functions, a third order polynomial was found to reasonably simulate the curved shape of the crown cantilever. The adopted shape of the crown cantilever indicates a thin arch dam as the base thickness to height ratio is 26m/130m=0.2.

3.2 Shape of individual arches

In the horizontal direction, the arches consist of one central and two peripheral segments. Three pairs of lines-of-centres are therefore needed to generate the circular segments. One pair of lines-of-centres defines the central segment, whereas the other two pairs delineate symmetrical or asymmetrical peripheral segments. Together they approximate an elliptical shape of a composite arch that reproduces the line of thrust for U or V-shaped canyons.

The number of lines-of-centres relative to the extrados and intrados arcs determines whether the arches are of uniform or of variable thickness in the horizontal direction. Uniform thickness of the segment corresponds to a single line-of-centre and variable thickness is obtained for a pair of spatially different lines-of-centres. On the other hand, variable centres of curvature along the dam height contribute to the vertical curvature of the dam. The dam curvature is highest in the upper central part and decreases towards the abutments and the base of the dam accompanied with a gradual increase of the thickness. An output of the algorithm for a horizontal cross section together with relevant design parameters is given in Fig. 3. They describe the arch geometry with increasing thickness from the reference plane toward the abutments.



b)

LEVEL 60.00	EXTRADOS	INTRADOS
RC	96.65	64.00
αl	30.00	30.00
αd	30.00	30.00
RL	166.65	91.00
βl	13.57	30.00
RD	166.65	91.00
βd	13.57	30.00
RML	-15.00	15.00
δl	30.00	30.00
RMD	-15.00	15.00
δd	30.00	30.00

Figure 3. Output of arch parameters at structural height of 60m (elevation 395m): a) arch designed with one central (red colour) and two peripheral segments (green colour) and b) quantification of the arch parameters. The dam fillets are indicated with dark blue colour.

In the horizontal direction, the arches are determined with the central and peripheral radii of extrados and intrados and by the corresponding central and peripheral angles, Fig. 3a. They are determined with respect to the local coordinate system XL-YL-ZL, as follows:

- R_{CI} - radius of the central segment of intrados arc and R_{CE} - radius of the central segment of extrados arc;
- α_{LI}- left angle of the central segment of intrados arc and α_{LE} - left angle of the central segment of extrados arc;
- R_{LI} - radius of the left peripheral segment of the intrados arc and R_{LE} - radius of the left peripheral segment of the extrados arc;
- β_{LI} - angle of the left peripheral segment of the intrados arc and β_{LE} - angle of the left peripheral segment of the extrados arc;
- R_{DI} - radius of the right peripheral segment of the intrados arc and R_{DE} - radius of the right peripheral segment of the extrados arc
- β_{DI} - angle of the right peripheral segment of the intrados arc and β_{DE} - angle of the right peripheral segment of the extrados arc;

To reduce the accumulation of stresses in the horizontal direction, the arch dam structure may require increase of the thickness at the abutments. Hence, for a smoother distribution of lateral loads to the foundation rock, the adopted procedure pays particular attention to the design of the fillets along the abutments, Fig. 3a. The parameters needed for the design of the fillets are:

- RML_E and δL_E - radius and angle at the end of the extrados at the left dam side;
- RML_I and δL_I - radius and angle at the end of the intrados at the left dam side;
- RMD_E and δD_E - radius and angle at the end of the extrados at the right dam side;
- RMD_I and δD_I - radius and angle at the end of the intrados at the right dam side;

The local coordinates of the centres of curvatures for the central segment correspond to the sum of the extrados and intrados radii and the already defined local coordinates of the extrados and intrados of the cantilever. Likewise, the local coordinates of the centres of curvature for the peripheral segments are calculated by adding-up the length of the extrados and intrados radii to the coordinates of the points at compound curvature of the respective central and peripheral segments. In this respect, the radii of right and left extrados fillets are negative. The values are presented on the screen in a table format and can be easily modified by the user, Fig. 3b.

4 GRAPHICAL OPTIONS

Throughout the design process, the algorithm allows for visual assessment of the dam horizontal and vertical cross sections and their position with respect to the canyon topography. In addition, the relevant design parameters for the geometry of extrados and intrados curvatures are shown on the screen in a table format. This extremely useful option gives not only a detailed insight into the dam geometry and dimensions, but also facilitates eventual corrections in the optimization process of the desired design parameter(s). In average, following the initial dam layout, an update of the dam geometry takes only several minutes.

4.1 Arches with one central and two peripheral segments

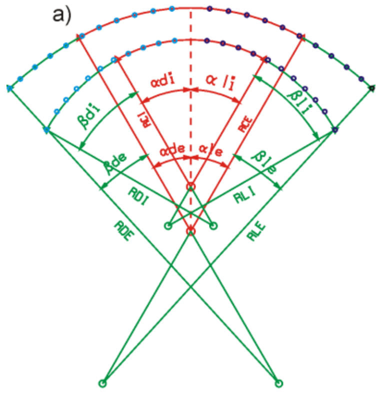
With the exception of the bottom arch, all other arches are designed as composite arches with one central and two peripheral segments. In Figs. 4 are given composite arches at elevation of 385m (structural height of 50m) and 465m (130m high crown arch), respectively. These figures show that the central and peripheral segments have different centres of curvature of the extrados and intrados arcs, indicating arch segments of variable thicknesses. As mentioned earlier, the arch thickness is the lowest at the reference plane and increases toward the abutments, whereas the cantilever thickness decreases with dam height and is lowest at the dam crest.

4.2 Full-radial arch abutments

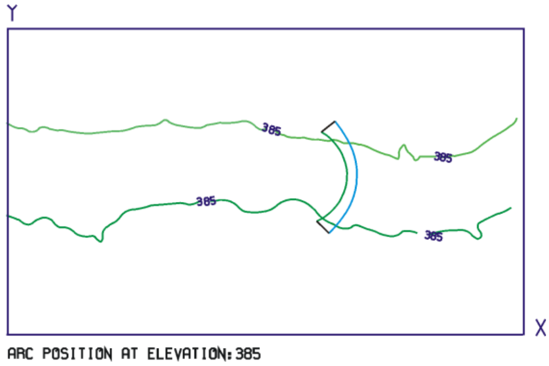
Accurate design of the full-radial arch abutments perpendicular to the central axis of the arc provides solid resting and stress distribution in the foundation rock. The algorithm gives the possibility to observe to which extent the arch abutments satisfy the full-radial design, and enables effective correction of the arch resting when necessary. For a better stability, the abutments should extend radially from the extrados centre of the peripheral segment. To satisfy this requirement, the initial shape of the abutments shown in Fig 5a was corrected for the one depicted in Fig 5b. Figs. 5 also shows a central segment of uniform thickness obtained assigning the same centre of curvature for the extrados and the intrados arcs.

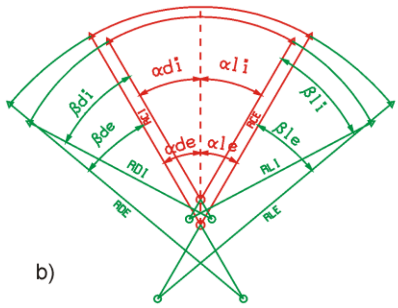
4.3 Bottom arch consisting of one central segment

The function of the bottom arch is to provide embedment of the dam foundation into solid rock. Fig. 6a depicts the initial layout of the bottom arch at elevation of 335m consisting of one central segment. As can be seen in plan this design fails to enable full contact with the foundation rock. Following a slight shift of the bottom arch in downstream direction, a rapid correction was made by broadening the left and right angles of the intrados from the initial 4º to 65º, and left and right angles of the extrados from the initial 5º to 50º degrees. As well, the curvature of the intrados was subsequently slightly decreased by enlarging the radius of intrados by 3 meters, Fig. 6b. These interactive corrections enabled a solid contact between the arch and foundation rock at both banks.



LEVEL 50.00	EXTRADOS	INTRADOS
RC	88.88	58.50
αl	30.00	30.00
αd	30.00	30.00
RL	158.88	76.50
βl	12.29	30.00
RD	158.88	76.50
βd	12.29	30.00
RML	0.00	0.00
δl	0.00	0.00
RMD	0.00	0.00
δd	0.00	0.00





LEVEL 130.00	EXTRADOS	INTRADOS
RC	165.30	138.50
αl	30.00	30.00
αd	30.00	30.00
RL	230.00	155.60
βl	20.00	32.00
RD	230.00	155.60
βd	20.00	32.00
RML	0.00	0.00
δl	0.00	0.00
RMD	0.00	0.00
δd	0.00	0.00

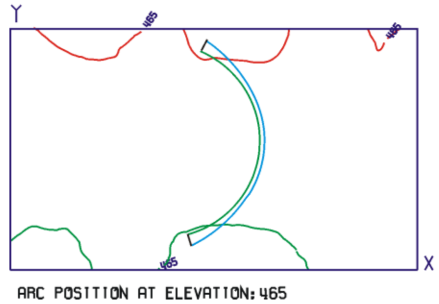
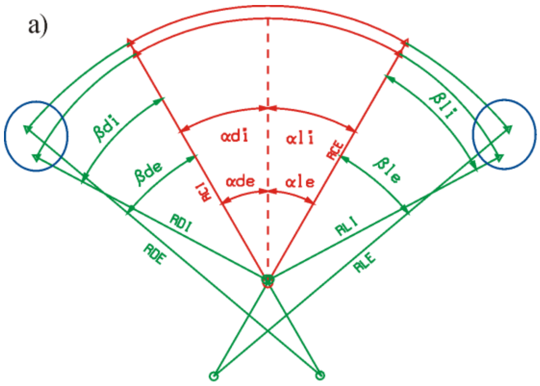


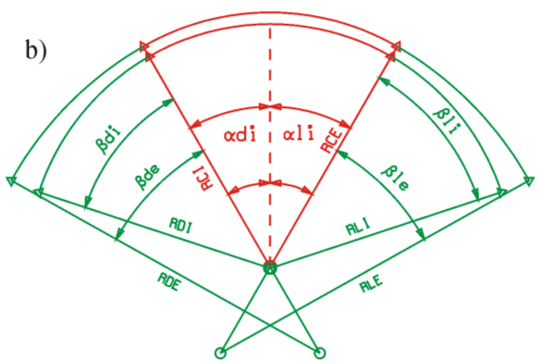
Figure 4. Arch geometry designed with one central and two peripheral segments supported at the abutments: a) dam height of 50m (elevation 385m) and b) dam crest with height of 130m (elevation 465m). The blue nodes on the arcs indicate the location of the nodes describing the substructures.



LEVEL 130.00	EXTRADOS	INTRADOS
RC	165.30	157.75
αl	30.00	30.00
αd	30.00	30.00
RL	230.00	159.00
βl	20.00	32.00
RD	230.00	159.00
βd	20.00	32.00
RML	0.00	0.00
δl	0.00	0.00
RMD	0.00	0.00
δ l	0 00	0 00

41.80

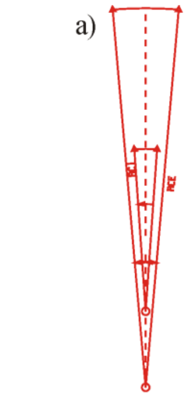
30.80



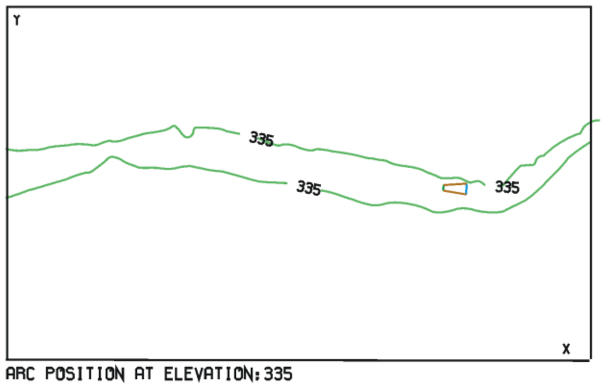
LEVEL 130.00	EXTRADOS	INTRADOS
RC	165.30	157.75
αl	30.00	30.00
αd	30.00	30.00
RL	230.00	159.00
βl	30.80	41.80
RD	230.00	159.00
βd	30.80	41.80
RML	0.00	0.00
δl	0.00	0.00
RMD	0.00	0.00

Figure 5. Geometry of the crown arch (elevation 465m) designed with one central segment with constant thickness and two peripheral segments with variable thickness: a) initial embedment of the arch, and b) corrected embedment providing better resting and stress distribution.

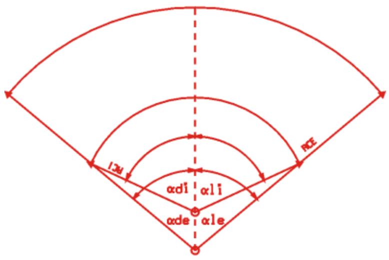
a)



LEVEL 0.00	EXTRADOS	INTRADOS
RC	70.00	30.00
αl	5.00	4.00
αd	5.00	4.00
RL	0.00	0.00
βl	0.00	0.00
RD	0.00	0.00
βd	0.00	0.00
RML	0.00	0.00
δl	0.00	0.00
RMD	0.00	0.00
δd	0.00	0.00



b)



LEVEL 0.00	EXTRADOS	INTRADOS
RC	70.00	33.00
αl	50.00	65.00
αd	50.00	65.00
RL	0.00	0.00
βl	0.00	0.00
RD	0.00	0.00
βd	0.00	0.00
RML	0.00	0.00
δl	0.00	0.00
RMD	0.00	0.00
δd	0.00	0.00

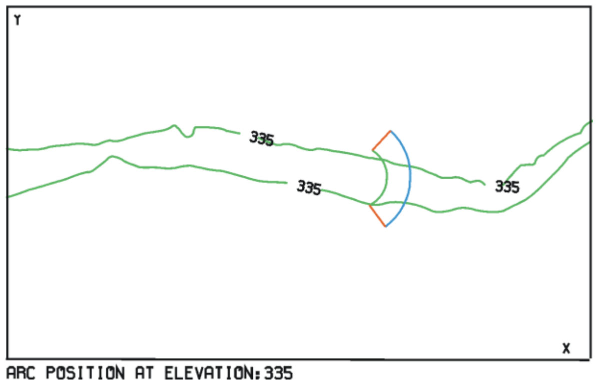


Figure 6. Arch geometry at the dam bottom (elevation of 335m) designed with one central segment for: a) improper partial support at the abutments, and b) final appropriate support at the abutments obtained by broadening after displacing the segment in downstream direction.

4.4 Dam continuity in the vertical direction

The geometry of double curvature arch dams is characterized with downstream overhanging near the crest and with upstream undercutting of the heel. Overhanging of the upper arches occurs typically at the crown cantilever and gradually diminishes towards the abutments. In the arches near the dam crest, this condition contributes to the development of low tensile stresses on the upstream face, and counterbalances the development of tension on the downstream face. Severe overhanging, however, can develop unacceptable tension at the upper upstream face causing closure of contraction joints and preventing appropriate grouting. On the other hand, the contact between the concrete and foundation rock undercuts the upper concrete portions at the upstream dam face. The moment generated by the weight of the upper dam portions compresses the concrete and outweighs the occurrence of tension from the reservoir pressure. In extreme cases, undercutting may cause instability during construction with potential overturning in the upstream direction.

During the geometrical design and structural analyses of arch dams, a special attention must be given to achieve smooth transition of dam faces taking into account the overhanging and undercutting effects. In this respect, the algorithm allows for an efficient observation and assessment of the continuity of the system of independent arches in plan. This helps to detect eventual shape deviation among neighbouring arches in horizontal cutting planes. The user can also select the option to display the overhanging and undercutting zones along the cantilevers in vertical cross sections. Figs. 7 give the mutual position of two 10m high adjacent arches with a contact at elevation of 375m and along the structural heights between 30-40m and 40-50m. Gradual changes of the dam shape can be observed with a slight overhanging effect of the upper arch.

At this stage of the design process, the user can intervene to adjust the position of individual arches to improve the continuity of the dam faces and/or the embedment of the abutments. This is achieved by a slight shift or rotation of individual arches or even of the entire dam with respect to the canyon axis. The coordinates of the updated dam shape and location are automatically modified and stored in conformity to the amended local coordinate system.

4.5 Lines-of-centres

The lines-of-centres represent the loci of centres of circular arc segments applied to determine the extrados and intrados faces of the dam. To avoid presence of excessive tensile stresses in the lower part of the dam, they are positioned closer to the crown cantilever resulting in shorter radii, higher curvature and stiffer arches as shown in, Fig. 8.

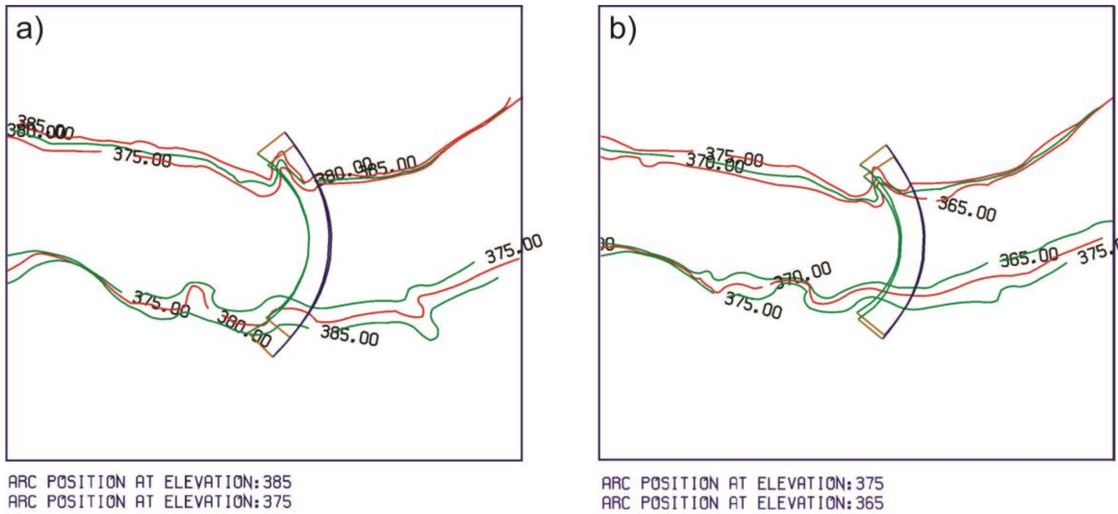


Figure 7. Plan view of the mutual positions of two 10m high arches with contact at 375m: a) horizontal cutting plane at elevation of 385m, and b) horizontal cutting plane at elevation of 365m

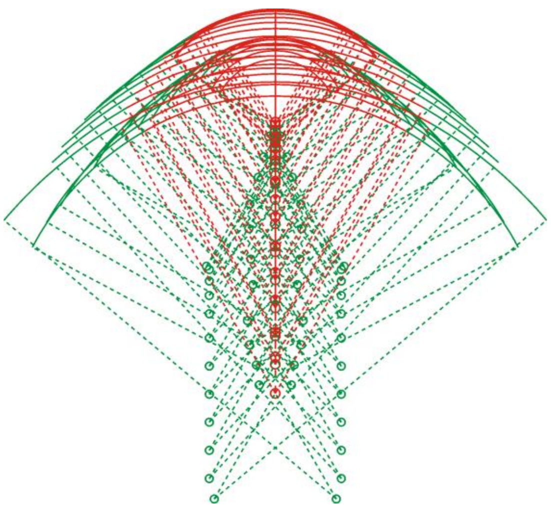


Figure 8. Line of centres, radii and angles of three segment arches with variable-thickness.

Arch segments with uniform thickness are determined by a single line-of-centres, whereas arches with variable-thickness require additional lines-of-centres.

Fig. 9 depicts the lines-of-centres for a three centred arch dam. The central segment of the dam is defined by the coupled lines-of-centres LOC-1/LOC-2 and LOC-3/LOC-4, where the former designs the crown arch with constant thickness (Fig. 9b and Fig. 5b) and the latter with variable thickness (Fig. 9c and Fig. 4b). These loci of the centres are coplanar with the crown cantilever and defined in the reference plane. Likewise, the coupled line-of-centres LOC-5/LOC-6 and LOC-7/LOC-8 define the shape of the arches at the left and right outer segments, respectively. The central and the peripheral segments of the dam are coplanar at an angle of compound curvature as measured from the reference plane.

Since the line-of-centres LOC-5/LOC-6 and LOC-7/LOC-8 are more distant from the base of the crown cantilever than the line-of-centres LOC-1/LOC-2 and LOC-3/LOC-4, the radii of arches of the peripheral segments are larger than the radii of arches of the central segment, resulting to a smaller curvature of the outer segments comparing to the central segment of the dam (Figs. 8 and 9). As can be observed in Fig. 9, the centres of the curvatures and the respective radii are positioned in a way that provides smooth transitions, hence avoiding abrupt changes in the geometry and stress concentrations.

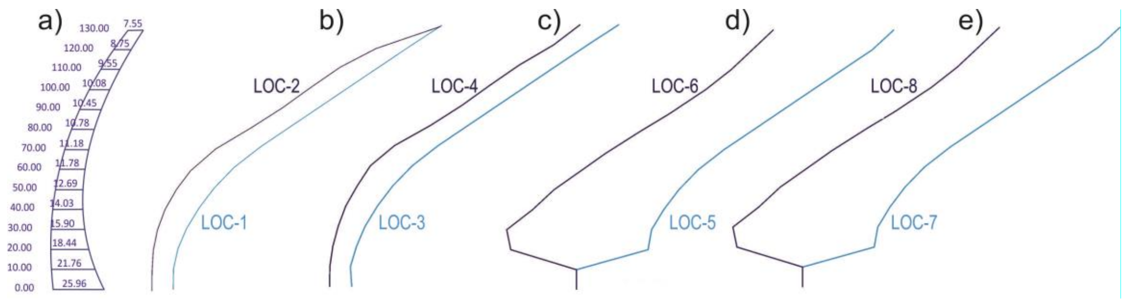


Figure 9. Lines-of-centres for a three-centred arch dam with a) crown cantilever, b) constant thickness of the central segment LOC-1/LOC-2, c) variable thickness of the central segment LOC-3/LOC-4, d) left outer segment LOC-5/LOC-6, e) right outer segment LOC-7/LOC-8.

4.6 Foundation Rock and Abutments

Dam abutments extend into the solid rock. As it was the case with the dam gradually changing geometry, the excavation for the abutments has also to provide smooth transition zones with depth. To meet this requirement, the user can verify in plan and in cross section the lateral extent of the arches, their position with respect to the topography and the embedment depth in the canyon walls. In Fig. 10 is given the interface between the intrados and extrados arcs and the foundation rock at elevation of 385m that describes this option. The layout of the selected foundation profile in Fig. 10 considers removing the weak soil and altered rock up to the solid rock. In the given example it was assumed that the foundation rock is approximately 7-10m below the ground surface, where the top 3-5m are related to the surficial soils and regolith and as much is excavated into the underlying solid rock. On the other hand, Fig. 11 illustrates the computed contacts of the dam intrados and extrados abutments with the ground surface and the excavation depth into the solid rock layer. This information helps to obtain accurate estimate of the volume of excavation needed for proper embedment of the dam abutments.

Figure 10. Interface between the intrados and extrados arcs and the foundation rock at elevation of 385m: a) right abutment, and b) left abutment. Points of intersection of the arcs with the terrain contour lines are indicated with red hollow circles.

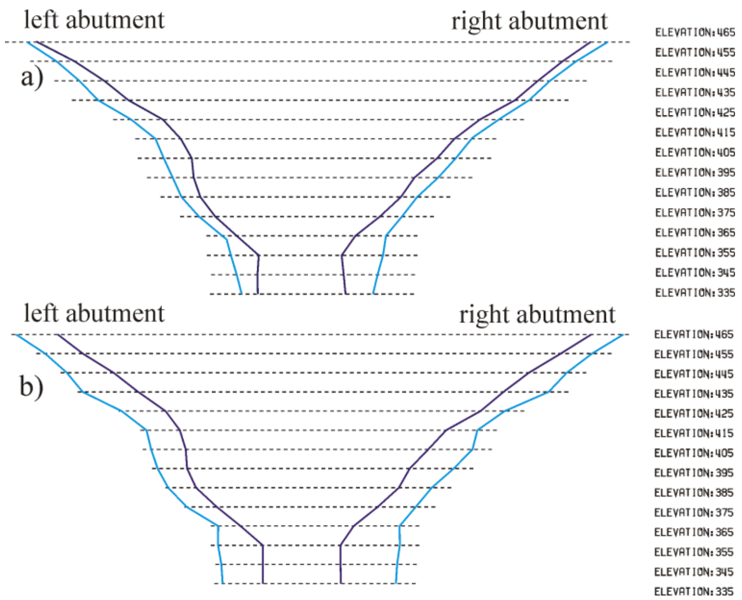


Figure 11. Longitudinal section of the planned contacts with ground surface (dark blue line) and with solid rock (light blue line): a) intrados face, and b) extrados face.

5 GENERATION OF FE MODEL OF THE DAM AND BE MODEL OF THE FLUID DOMAIN

Static and dynamic structural analyses are conducted in parallel to the design of the geometric layout of the dam. To this end, a FEM-BEM numerical model of the example double-curvature arch dam was built in a format compatible with the ADAD-IZIIS software, which runs the analyses. The dam model consists of substructures with 15 and 20-nodes, further divided in a number of 15 and 20-node finite elements of Serendipity type. Each substructure represents part of a monolithic block, situated between two adjacent elevations spaced evenly at 10m and between two construction joints distanced 15m horizontally along the arches. Construction joints are placed at the lateral contact of two adjacent monolithic blocks. Within each substructure, two different types of nodes describing the substructures and the finite elements are required to generate the mathematical FE model. To fully determine the substructures, respective coordinates are automatically generated for i) basic nodes describing the construction joints between the monoliths and ii) nodes interpolated between the basic nodes along the arcs. The position of the basic nodes along the arcs and the nodes interpolated in-between are presented in Figs. 4a and 12, respectively. In order to define all nodes used for generation of 15 and 20 nodded substructures, additional interpolation of nodes is required between two superimposed arcs. Fig. 12 depicts interpolated nodes along the arcs at elevations of 395 and 405 together with vertically interpolated nodes.

The FE model of the example double-curvature arch dam together with the construction joints is given in Fig. 13. It was generated automatically using the predefined shapes of the arcs and the coordinates for the basic nodes at all design elevations. In this case, the final FE model contains 11,170 external nodes connecting 199 substructures discretized into 6,294 finite

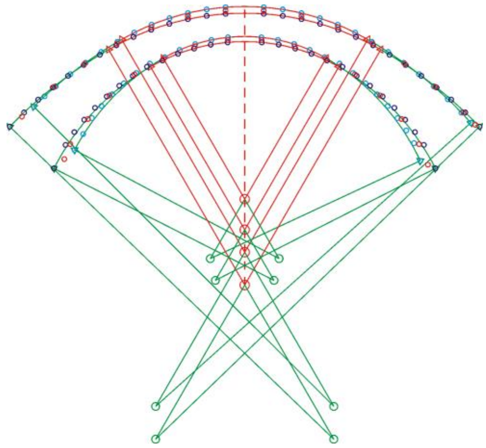


Figure 12. Interpolated nodes: horizontally along the arcs at elevations 405 (dark blue circles) and 395 (light blue circles) and vertically between these two elevations (red circles)

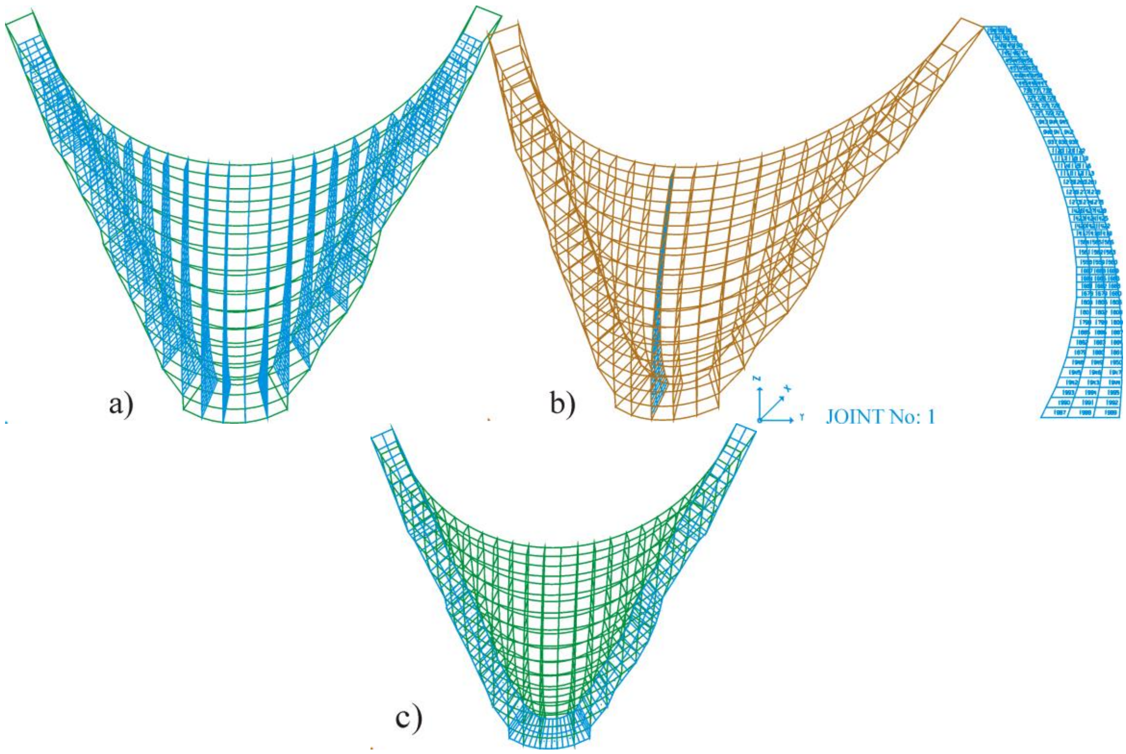


Figure 13. Perspective view of the mathematical model of the arch dam: a) construction joints at the vertical contacts between the monolith blocks, and b) construction joints discretized by contact elements. c) contact elements between the dam abutments and the foundation rock

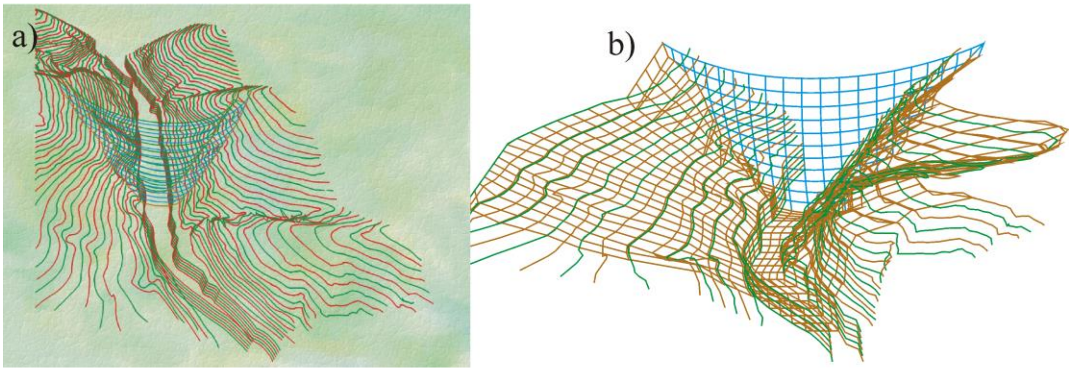


Figure 14. 3D perspective view: a) canyon topography and dam geometry, and b) generated BE mesh of the dam-fluid-foundation system bounded by the dam-fluid interface, canyon walls, river bed, (boundary elements at the water mirror and the upstream truncation surface are not presented).

elements. The band of the system of matrices is 3,111. The model also contains 2,013 contact elements at the interface between monolithic structural blocks, which simulate the effect of the construction joints, Fig. 13b. If needed, automatic refinement of the contact element mesh can be further conducted upon user provided mesh parameters. Fig. 13c. presents the contact elements generated between the dam abutments and the foundation rock. For cases when the solid rock mass is not included in the analysis, these contact elements act as spring elements with such mechanical properties that simulate a non-rigid support at the foundation base.

The next step consists of discretization of a BE model of the dam-fluid-foundation system taking into account the complex topographical conditions of the canyon [30-31]. The mesh generation process of the fluid domain is simple requiring that the user specifies the number of vertical planes intersecting the terrain together with respective distances from the top of the crown cantilever. Evidently, complex highly irregular terrain conditions will require increased number of vertical planes and a more refined mesh [30-31]. In this respect, the proposed algorithm offers a 3D perspective view of the canyon and the dam, Fig. 14a. It is also possible to extract the boundary elements at the extrados directly from the upstream face of the FE model, Fig 14b. This option allows the user to confirm that adequate distribution of the hydrodynamic pressure is attained along the dam-fluid interface.

6 CONCLUSION

An original algorithm for geometric modelling and effective design of double-curvature arch dams is presented. It was developed and implemented to run in parallel to the ADAD-IZIIS program for structural analyses of arch dams. Combing FEM-BEM modelling, the algorithm enables accurate FE modelling for the dam and BE modelling of the reservoir domain adjusted to the terrain configuration. The proposed interactive design starts with the generation of a mathematical model of the terrain topography. The next step consists in selection of the optimal dam location based on comparisons of the dam heights and extents.

The arch dam geometrical model is built in a form of circular segments assigned to each incremental height. A variety of different graphical presentations of the dam structural geometry and terrain topography are displayed on the screen including cross-sections, plan views, contacts at the dam abutments, and 3D views of the dam mathematical model. Correction of the parameters is possible at each step of the design. The practical engineering drawings and descriptions of the dam geometry are automatically modified following the introduced changes. Thus, they provide competent insight in the design and the design possess itself with abundant range of details and clarity.

The presented example of geometric design of a 130m high double curved arch dam in a V-shape canyon demonstrated the above capabilities. Compared to the existing commercial software, the developed algorithm showed a few significant advantages: the dam-reservoir design is directly dependant on the configuration of the site, the number of graphical options available at the push of a button offer an intuitive and user-friendly environment.

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ABSTRACT

INTERACTIVE ALGORITHM FOR GEOMETRIC MODELLING DOUBLE-CURVATURE ARCH DAMS

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Alen HARAPIN
Ana NANEVSKA

A rapid and efficient algorithm for interactive geometric modelling of arch dams is presented. It combines the advantages of the traditional geometric-design with innovative computational capabilities offering simple procedures for otherwise complex process of laying out double-curvature arch dam-reservoir coupled systems. The key parameters taken into account are: terrain topography, shape and thickness of crown cantilever, reference cylinder, thickness and curvature of individual arches, excavation depth, concrete volume, vertical and peripheral construction joints and automatic generation of finite element and boundary element models. The proposed algorithm was implemented in and runs parallel to the ADAD-IZIIS FEM-BEM, a finite element-boundary element software for structural analyses of concrete arch dams. To demonstrate the performances of the proposed algorithm, an example of a 130m high double-curvature arch dam was considered in a narrow V-shape canyon. The number of graphical options available at the push of a button, such as vertical and horizontal cross sections and 3D perspectives, allows the user to rapidly conduct the dam design within the optimization process.

Keywords: arch dam; geometric modelling; FEM-BEM numerical modelling; terrain topology; ADAD-IZIIS software

REZIME

INTERAKTIVNI ALGORITAM ZA GEOMETRIJSKO MODELIRANJE DVOJNO ZAKRIVLJENIH BRANA

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Ovaj članak se odnosi na efikasno formulisan algoritam za interaktivno geometrijsko modeliranje lučnih brana. Algoritam predstavlja kombinaciju tradicionalne metode za dimenzionisanje kombinovan sa inovativnim kompjuterskim veštinama koje pružaju mogućnost jednostavnog procesuiranja kompleksnog projektovanja fleksibilno spregnutog sistema dvojno zakrivljene lučne brane – rezervoara. Ključni parametri koji se uvode u predmetni algoritam su topologija terena, oblik i debljina glavne centralne konzole, referentni cilindar, debljina i zakrivljenost pojedinih lukova, dubina iskopa za temelje u terenu, vertikalne konstruktivne razdelnice (fuge), perimetralna (obimna) fuga i automatsko generisanje numeričkih modela od konačnih i graničnih elemenata. Algoritam je ugrađen u kompjuterski program ADAD:IZIIS namenjen statičkoj i dinamičkoj analizi lučnih brana. Program je orijentisan na primenu metoda konačnih i graničnih elemenata. Da bi se demonstrirala primena algoritma odabran je primer lučne dvojno zakrivljene brane visoke 130m koja je postavljena u tesni kanjon oblika V-tipa. Prikazan je veći broj grafičkih priloga koji se dobija jednostavno selekcijom tražene opcije. Jasna prezentacija značajnih inženjerskih crteža na ekranu omogućuje brzo optimalno dimenzionisanje i formiranje matematičkih modela lučnih brana.

Ključne reči: lučna brana, geometrijsko modeliranje, MKE-MGE numeričko modeliranje, topologija terena, ADAD-IZIIS softver

IN MEMORIAM

Profesor dr **Sekula Živković**, dipl.inž.građ.
Professor **Sekula Zivkovic**, Ph.D.
(1944-2019)

Prof. dr **Sekula Živković,** diplomirani građevinski inženjer, redovni profesor Građevinskog fakulteta u penziji, rođen je 4. maja 1944. godine, u selu Makce (opština Veliko Gradište). Osnovnu školu završio je 1959. godine u Rabrovu (opština Kučevo), a Građevinsku srednju tehničku školu 1963. godine u Beogradu.

Građevinski fakultet Univerziteta u Beogradu upisao je školske 1963/64. godine, a diplomirao je u aprilu 1968. godine, na Odseku za konstrukcije s prosečnom ocenom 8,00 i ocenom 10 na diplomskom radu iz predmeta Betonske konstrukcije.

Odmah po diplomiranju, zaposlio se u GK „Komgrap" – Fabrika montažnih elemenata „Standard-beton" u Beogradu, gde je – u okviru Konstrukciono-tehnološkog biroa – radio do 1972. godine, na mestu inženjera za tehnologiju betona i prefabrikaciju betonskih elemenata. U ovom periodu, upisao je i poslediplomske studije na Građevinskom fakultetu u Beogradu, školske 1970/71. godine, koje je uspešno završio, s prosečnom ocenom 8,90. Kao inženjer za tehnologiju betona, imao je kraći studijski boravak u Istočnoj Nemačkoj, gde je u Istočnom Berlinu i Drezdenu posetio nekoliko velikih fabrika stanova.

1. aprila 1972. godine počeo je da radi kao asistent na Građevinskom fakultetu Univerziteta u Beogradu, gde

Prof. **Sekula Zivkovic**, Ph.D. a graduated civil engineer and retired full professor from the Faculty of Civil Engineering, was born on May 4th, 1944, in the village of Makce - the municipality of Veliko Gradiste. He finished elementary school in Rabrovo in 1959 - Kučevo municipality, and the Civil Engineering Technical School in 1963 in Belgrade.

He enrolled to the Faculty of Civil Engineering, University of Belgrade, in the academic year 1963/64. He graduated in April 1968 from the Department of Structural Engineering with an average grade of 8.00 and with a grade 10 for the thesis on the Concrete Structures.

Immediately after his graduation, he was employed at GK "Komgrap" - Prefabricated Element Factory "Standard-concrete" in Belgrade, where he worked in the Engineering and Technology Bureau until 1972, at the position of an engineer for concrete technology and prefabrication of concrete elements. During this period, he enrolled to the postgraduate studies at the Faculty of Civil Engineering in Belgrade, academic year 1970/71, which he successfully finished with an average grade of 8.90. As a concrete technology engineer, he had a brief study visit to East Germany, where he visited several large prefabricated apartment factories in East Berlin and Dresden.

je ostao sve do svog penzionisanja 2009. godine.

Početkom 1983. godine odbranio je na Građevinskom fakultetu u Beogradu magistarski rad pod nazivom: „Prilog istraživanju fizičko-mehaničkih karakteristika betona male starosti”, a u oktobru 1989. godine i doktorsku disertaciju pod naslovom „Prilog istraživanju uticaja temperature i drugih relevantnih parametara na neka svojstva svežeg betona i betona male starosti”. Tokom rada na izradi doktorske disertacije, boravio je i na *University College* u Londonu (saradnja s prof. Domonom), gde je učestvovao u opsežnim laboratorijskim ispitivanjima svojstava svežeg i mladog betona.

Po odbrani doktorske disertacije, Sekula Živković je 1991. godine izabran u zvanje docenta, nakon toga 1997. godine – u zvanje vanrednog profesora, a 2003. godine i u zvanje redovnog profesora za užu naučnu oblast Građevinski materijali, tehnologija betona i ispitivanje konstrukcija. Na redovnim studijama Građevinskog fakuleta u Beogradu, sve do penzionisanja – 2009. godine, držao je predavanja iz predmeta Građevinski materijali, a nakon odlaska u penziju prof. Muravljova (2003) i predmet Tehnologija betona. U okviru poslediplomske nastave na Građevinskom fakuletu u Beogradu, prof. Živković izvodio je nastavu iz predmeta: Savremene metode ispitivanja materijala, Odabrana poglavlja tehnologije betona i betonskih konstrukcija, kao i Specijalne vrste betona. U periodu između 1997. i 2002. godine, držao je nastavu i bio mentor u izradi magistarskih radova u okviru Specijalnog poslediplomskog kursa na engleskom jeziku za studente iz Libije pod nazivom: *Structural Materials and Technology of Concretein Hot Climates*.

Osim na svom matičnom fakultetu u Beogradu, prof. Živković držao je predavanja iz predmeta Građevinski materijali i na Građevinskom fakultetu Univerziteta u Podgorici (školske 1991/92. i 1992/93. godine), a od školske 1997/98. do 2000/01. godine, iz dva predmeta na Arhitektonsko-građevinskom fakultetu Univerziteta u Banja Luci (Građevinski materijali na Građevinskom odseku i Građevinski materijali i fizika na Arhitektonskom odseku ovog fakulteta).

Prof. dr Sekula Živković bio je mentor ili član komisija za odbranu brojnih diplomskih radova, magistarskih teza i doktorskih disertacija.

Može se slobodno reći da je prof. Živković najveći deo svoje radne energije i svog pregalaštva u nastavi na Građevinskom fakultetu upotrebio upravo u oblasti unapređivanja nastavnog procesa i publikovanja literature iz predmeta na kojima je bio angažovan. Tako, već 1975. godine, kao mlad asistent, bio je inicijator izrade prvog Priručnika za vežbanja iz predmeta Građevinski materijali. Početkom devedesetih godina prošlog veka, on je uložio veliki trud na izradi prve Zbirke rešenih testova iz oblasti Građevinskih materijala, koja je studentima značajno olakšala rad na laboratorijskim vežbama i pripremu kolokvijuma iz ovog predmeta. U okviru svoje nastavne delatnosti, prof. dr Sekula Živković bio je autor sedam odrednica udžbeničke literature (priručnika, praktikuma, zbirki zadataka i zbirki testova), kao i Građevinskog englesko-srpskog i srpsko-engleskog rečnika, koji je doživeo veći broj izdanja. Na području naučnoistraživačkog i stručnog rada, uglavnom se bavio problematikom građevinskih materijala i tehnologije betona. Iz ovih oblasti ima velik broj stručnih

On April 1st, 1972, he started working as teaching assistant at the Faculty of Civil Engineering, University of Belgrade, where he remained until his retirement in 2009.

At the beginning of 1983, he defended his master thesis "*A Contribution to the Investigation of the Properties of Concrete at Early Ages*", and in October 1989, the doctoral dissertation "*A Contribution to the Investigation of the Effects of Temperature on the Properties of Fresh Concrete and Concrete at Early Ages*" at the Faculty of Civil Engineering in Belgrade. During research work on his PhD thesis, he also attended the University College of London (in co-operation with Prof. Domon), where he participated in extensive laboratory tests of the properties of fresh and early age concrete.

After defending the doctoral dissertation, Sekula Živković was elected Assistant Professor in 1991, then in 1997 Associate Professor, and in 2003, Full Professor for a specific scientific field of *Building materials, concrete technology and testing of structures*. He delivered lectures for teaching subject *Building Materials* at graduate studies of the Faculty of Civil Engineering in Belgrade until his retirement in 2009. After the retirement of prof. Muravljov (2003), he also taught the teaching subject *Concrete Technology*. In the framework of postgraduate studies at the Faculty of Civil Engineering in Belgrade, prof. Živković taught the subjects *Contemporary methods of material testing, Selected chapters of concrete technology and concrete structures,* and *Special types of concrete*. In the period between 1997 and 2002, he delivered lectures and was a mentor in the preparation of master theses in the Special Postgraduate Course in English for students from Libya: *Structural Materials and Technology of Concrete in Hot Climates*.

Apart from his principal faculty in Belgrade, prof. Živković also taught *Building Materials* at the Faculty of Civil Engineering of the University of Podgorica (academic years 1991/92 and 1992/93). From the academic year 1997/98 until 2000/01, he taught two subjects at the Faculty of Architecture and Civil Engineering, University of Banja Luka (*Building Materials* at the Civil Engineering Department and *Building Materials and Physics* at the Architecture Department of this faculty).

Prof. Sekula Živković, Ph.D. was a mentor or member of the committee for defence of a large number of graduation theses, master theses and doctoral dissertations.

Prof. Živković spent most of his working energy and diligence for teaching at the Faculty of Civil Engineering in the area of improving teaching process and literature publishing in the domain of teaching subjects on which he was engaged. So, as early as in 1975, as a young assistant, he was the initiator of the first *Practice Manual for the Building Materials*. At the beginning of the 1990s, he made great efforts to create the first *Collection of solved tests in the field of Building Materials*, which significantly facilitated laboratory work exercises and colloquium preparation at this subject. As part of his teaching activity, prof. Sekula Živković, Ph.D. was the author of seven textbooks (manuals, practice books, test and task collections), as well as the English-Serbian and Serbian-English dictionary, which had a number of reprints. In the field of scientific and professional work, ostvarenja, značajan broj objavljenih radova u naučnim i stručnim časopisima i učešća na nizu skupova u zemlji i inostranstvu.

Profesor Živković je aktivno učestvovao kao istraživač, a kasnije i kao rukovodilac projekta, u realizaciji ukupno 14 naučnoistraživačkih projekata i projekata tehnološkog razvoja, koje su finansirala resorna ministarstva SFRJ, SR Jugoslavije, odnosno Republike Srbije.

Među njegovim doprinosima nauci i struci, posebno zaslužuje da bude istaknuta monografija „Beton u žarkim klimatima – svojstva, trajnost i tehnologija". Ova monografija, štampana u Beogradu 1997. godine, kao uostalom i navedena doktorska disertacija, nastala je kao rezultat višegodišnjeg angažovanja profesora Živkovića na kapitalnim projektima naših velikih građevinskih firmi u zemljama tzv. žarkog klimata. Knjiga o kojoj je reč predstavlja zaista vrednu monografiju, korisnu i danas – i za građevinsku praksu i za studente, ali i za brojne istraživače na fakultetima i u naučnim institutima. Ova interesantna materija izložena je znalački, koncizno i jasno, pri čemu su pojedini stavovi potkrepljeni vlastitim istraživanjima i iskustvom, što je predmetnoj monografiji, osim nesumnjive naučne, dalo i istinsku aplikativnu vrednost.

Značajan doprinos vrednosti same monografije daje i činjenica da je u njoj našao mesta i višegodišnji rad autora (1987–1994) u RILEM-ovom Tehničkom komitetu *SNS 94 – Concrete in Hot Climates,* gde su proučavani i analizirani svi bitni aspekti konkretnog problema i gde su razmenjivana praktična iskustva stručnjaka i naučnih radnika iz većeg broja zemalja Evrope, Azije i Afrike.

Kao što je već napomenuto, u predmetnoj monografiji došla su posebno do izražaja i lična iskustva autora, stečena tokom njegovog praktičnog angažovanja (počev od 1976. pa sve do 1990. godine) na većem broju objekata koje su naša građevinska preduzeća izvodila u Kuvajtu i Iraku, kao i na njegovim ekspertskim angažovanjima u Libiji 1990. godine.

Od brojnih projekata u inostranstvu na kojima je učestvovao, ovde ćemo navesti samo najznačajnije: u Iraku („KOL 1", „262" i „MCP1" u Bagdadu, kao i „Project 1100" u Numaniji), zatim u Kuvajtu (*Water Tower sand Ground Level Reservoirs*) – na svim ovim projektima bio je u svojstvu rukovodioca laboratorije za beton ili eksperta za tehnologiju betona. Na sve navedene poslove upućivan je kao ekspert Građevinskog fakulteta u Beogradu, a angažovali su ga tadašnji giganti domaće građevinske industrije: GP „Rad", KMG „Trudbenik" i „Ratko Mitrović".

Takođe, učestvovao je u pisanju studija, stručnih mišljenja, ili kao konsultant i na projektima u Libiji (bolnica u Agedabiji), Alžiru (brana Seklafa), Makedoniji (brana Sveta Petka). Kada je u pitanju domaće građevinarstvo, prof. dr Sekula Živković ostavio je značajan trag svojim učešćem u brojnim projektima, od kojih bismo pomenuli najznačajnije: vodotoranj „Letnjikovac" u Šapcu, više objekata i konstrukcija u okviru TENT-A u Obrenovcu (npr. turbostolovi, temelji mlinova i napojnih pumpi, dimnjaci, postrojenja za dopremu i odlaganje uglja), zatim u okviru termoelektrana „Kostolac" i „Kolubara", postrojenja fabrike vode „Makiš 2" u Beogradu, most preko Ade u Beogradu, više mostova u okviru obilaznice oko Beograda. Na ovim poslovima, najčešće je bio he mainly dealt with the problems of building materials and concrete technology. There is a large number of professional achievements in these areas, a considerable number of published works in scientific and professional journals and participation in a number of events in the country and abroad.

Professor Živković actively participated as a researcher, and later as a project manager, in implementation of 14 research projects and technological development projects funded by competent ministries of the SFRY, FR Yugoslavia and the Republic of Serbia.

Among his contributions to science and profession, his monograph "*Concrete in hot climates - properties, durability, technology*" deserves a prominent place. This monograph, published in Belgrade in 1997, as well as the aforementioned doctoral dissertation, was the result of a long-term engagement of Professor Živković on the capital projects of our major construction companies in the countries of the so-called hot climate region. The book in question is a truly valuable monograph, useful today, both for construction practice and for students, as well as for numerous researchers at faculties and scientific institutes. This interesting subject matter is laid outskillfully, concisely and clearly, whereby some attitudes are reinforced by his own research and experience, which, in addition to the irrefutable scientific, has also a true applicative value.

A significant contribution to the value of the monograph itself is provided by the fact that it also features many years of work of the author (in the period 1987-1994) at the RILEM Technical Committee *SNS 94 - Concrete in Hot Climates*, where all the important aspects of the concrete problems were studied and analyzed and where the practical experience of experts and scientists from a wide range of countries in Europe, Asia and Africa were exchanged.

As already mentioned, the subject monograph features in particular the author's personal experience gained, especially during his practical engagement (from 1976 until 1990), on a number of facilities that our construction companies built in Kuwait and Iraq , as well as during his expert engagements in Libya in 1990.

From among many projects abroad in which prof. Živković participated, we here list only the most significant: in Iraq ("KOL 1", "262" and "MCP 1" in Baghdad, and "Project 1100" in Numania), then in Kuwait ("Water Towers and Ground Level Reservoirs") - on all these projects he was the head of a concrete laboratory or a concrete technology expert. He was referred to all these tasks as an expert of the Faculty of Civil Engineering in Belgrade, engaged by the then giants of the domestic construction industry: GP "Rad", KMG "Trudbenik" and "Ratko Mitrovic".

He also participated in writing studies, expert opinions, or as a consultant on the projects in Libya (hospital in Agedabia), Algeria (SeklafaDam), Macedonia (Sveta PetkaDam). As far as domestic construction is concerned, prof. Sekula Živković, PhD left a significant mark with his participation in a number of projects, the most important of which are water tower "Letnjikovac" in Šabac, several facilities and constructions within TENT-A plant in Obrenovac (turbine bearing structures, mills and feed pumps foundation, chimneys, coal transport and storage facilities etc.), then within the thermal power angažovan na istražnim radovima radi ispitivanja kvaliteta ugrađenih materijala i utvrđivanja stanja konstrukcija, ali i na poslovima projektovanja i tehničkim rešenjima izvođenja sanacionih intervencija, kao i na ispitivanjima komponentnih materijala i izradi projekata betona.

Prof. dr Sekula Živković bio je istaknuti član brojnih domaćih i međunarodnih naučnostručnih udruženja, od kojih ćemo ovom prilikom pomenuti samo najznačajnija:

- JUDIMK – Jugoslovensko društvo za ispitivanje i istraživanje materijala i konstrukcija, kasnije DIMK Srbije,
- SDGKJ – Savez društava građevinskih konstruktora Jugoslavije,
- DiT – Društvo inženjera i tehničara Jugoslavije,
- RILEM – International Union of Testing and Research Laboratories for materials and Structures (1974–1992),
- RILEM Technical Committee 94 CHC – Concrete in Hot Climates (1987–1992).

Sekula Živković preminuo je prerano, za sve one koji su ga voleli i poštovali – iznenada i potpuno neočekivano, u ponedeljak, 4. marta 2019. godine, u 75. godini života.

Kažu da svaki čovek i njegovo delo žive dokle god su živa sećanja i uspomene onih koji ostaju iza njega. Ako je to tačno, prof. dr Sekula Živković još dugo će biti s nama, njegovim kolegama i prijateljima, koji ćemo ga rado pominjati i pamtiti dok smo živi, s njegovim studentima koji će učiti iz njegovih knjiga i sećati se njegovih predavanja, a najviše sa svojim najmilijima – suprugom Jelicom, sinovima Nebojšom i Dušanom, snajama Ivanom i Magdalenom, unucima Jelenom i Nikolom, koje je bezgranično voleo. Ovom prilikom, izjavljujemo im naše najdublje saučešće.

U Beogradu, maja 2019. godine

Prof. dr Dragica Jevtić, dipl. inž.teh.
Prof. dr Dimitrije Zakić, dipl. građ.inž.

plants Kostolac and Kolubara, the water treatment facility "Makiš 2" in Belgrade, the bridge across the Ada in Belgrade, several bridges constituting the bypass highway route around Belgrade, and others. At these jobs, he was most often engaged in investigative works examining the quality of the built-in materials and determining the state of the structures. He was also engaged on the design tasks and on providing technical solutions for the implementation of remedial interventions, as well as on the testing component materials and on making concrete designs.

Prof. Sekula Živković, Ph.D. was a prominent member of numerous national and international professional associations, and we are going to mention just the most important ones:

- JUDIMK – Yugoslav Society for Testing and Research of Materials and Structures, later on DIMK of Serbia,
- SDGKJ – Union of the Societies of Structural Engineers of Yugoslavia,
- DiT – Society of Engineers and technicians of Yugoslavia,
- RILEM - International Union of Testing and Research Laboratories for materials and Structures (1974 - 1992),
- RILEM Technical Committee 94 CHC - Concrete in Hot Climates (1987-1992).

Sekula Živković passed away too early, for all of those who loved and respected him – suddenly and completely unexpectedly, on Monday, March 4th 2019, at the age of 74.

It is said that every man and his work continue to live, as long as the memories of those who remain behind him are alive. If this is true, prof. Sekula Živković will be with us, his colleagues and friends for a long time, for we will keep him in our conversations and memory as long as we live; he will be with his students, who will study from the books he wrote and who remember his lectures; he will be with his dearest – his wife Jelica, his sons Nebojša and Dušan, daughters-in-law Ivana and Magdalena, grandchildren Jelena and Nikola, whom he loved endlessly. We, hereby, express our deepest condolences to them.

In Belgrade, May 2019

Prof. Dragica Jevtić, Ph D grad.eng.tech.
Assoc. prof. Dimitrije Zakić, Ph.D. grad.civ.eng.

IN MEMORIAM

Akademik profesor dr **Boško D. Petrović**, dipl.inž.građ.
Academician, Professor **Bosko D. Petrovic**, Ph.D., Eng.civ.

(26.10.1926 – 6.5.2019.)

S dubokom tugom, primili smo vest da nas je 6. maja 2019. godine zauvek napustio akademik dr Boško Petrović, redovni profesor u penziji Fakulteta tehničkih nauka Univerziteta u Novom Sadu. Akademik Boško Petrović ostvario je zavidnu karijeru naučnika, pedagoga i humaniste i nesumnjivo će ostati zapamćen kao veliko ime u oblasti graditeljskog konstrukterstva.

Rođen je 25. oktobra 1926. godine, u Aleksincu. Školovao se u Beogradu, gde se 1945. godine upisao na Građevinski odsek Tehničke velike škole, a diplomirao je 19. januara 1951. na Odseku za konstrukcije, kod profesora Mijata Trojanovića, na predmetu Betonski mostovi. Doktorirao je na Građevinskom fakultetu u Beogradu 1971. s tezom „Ponašanje skeletnih zgrada od armiranog i prethodno napregnutog betona pod dejstvom seizmičkih sila”. Mentor mu je bio profesor dr Vlatko Brčić. Ceneći zapažene rezultate i domete u naučnom i stručnom radu, 1985. godine Srpska akademija nauka i umetnosti izabrala ga je za dopisnog, a 2006. godine – za njenog redovnog člana.

Stručnu i istraživačku karijeru započeo je kao istraživač-pripravnik 1950. godine u Grupi za prednapregnuti beton u Saveznom građevinskom institutu, kao saradnik profesora Branka Žeželja, na

With deep sorrow we received the news that the academician Bosko Petrovic, Ph.D., full professor retired from the Faculty of Technical Sciences, University of Novi Sad, left us forever on May 6, 2019. Bosko Petrovic accomplished an outstanding career of scientist, pedagogue and humanist and will undoubtedly remain remembered as a great name in the field of civil engineering.

He was born on October 25, 1926 in Aleksinac. He studied in Belgrade, where he enrolled to the Engineering Department of the Technical High School in 1945, and graduated on January 19, 1951 at the Department of Structures, as a student of Professor Mijat Trojanovic, on the subject of Concrete Bridges. He received his doctorate at the Faculty of Civil Engineering in Belgrade in 1971 defending the thesis "Behaviour of skeletal buildings from reinforced and pre-stressed concrete under the influence of seismic forces". His mentor was Professor Dr. Vlatko Brčić. Appreciating the remarkable results and achievements in scientific and professional work of professor Petrovic, in 1985, the Serbian Academy of Sciences and Arts awarded him a corresponding membership, and in 2006 a full membership.

razvoju i uvođenju prednapregnutog betona u jugoslovensku građevinsku praksu, radeći na ispitivanju, projektovanju i teorijskoj razradi konstrukcija od prethodno napregnutog betona. Kada je 1952. godine osnovan Institut za ispitivanje materijala SRS (IMS), priključen je novom institutu u kome je radio skoro 40 godina. Tokom tih godina, bio je šef Odeljenja sa beton i asfalt, šef Odeljenja za zgradarstvo i konstrukcije, savetnik u Odeljenju za razvoj. Radio je na problemima industrijskog građenja, seizmičkog građevinarstva, a u jednom periodu i na razvoju reaktorskih sudova od prethodno napregnutog betona. S tim u vezi, boravio je i radio šest meseci u Svenska Atom Energy u Stokholmu, gde je sarađivao na projektovanju i izradi jednog suda krupnih razmera. Bio je stalni član Naučnog veća IMS-a i u dva mandata i njegov predsednik. Iz IMS-a, s položaja direktora za naučni rad, 1989. godine prešao je na Fakultet tehničkih nauka Univerziteta u Novom Sadu, gde je – kao redovni profesor – ostao do penzionisanja 1993. godine.

Svoju pedagošku aktivnost dr Boško Petrović počeo je 1960. godine i od tada, pa sve do odlaska u penziju, bio je uključen u mnoge vidove nastave na više univerziteta – u nas i u svetu. Od 1964. godine, tj. od osnivanja specijalističkih i poslediplomskih studija na Građevinskom fakultetu (GF) u Beogradu počeo je svoju dugogodišnju saradnju s tim fakultetom. Pošto je, 1965. godine, habilitirao kod prof. dr Vlatka Brčića, izabran je za docenta, a 1978. godine – za vanrednog profesora. U isto zvanje izabrao ga je i Institut za zemljotresno inženjerstvo i inženjersku seizmologiju (IZIIS) Univerziteta u Skoplju, gde je držao predavanja iz oblasti prethodno napregnutog betona na poslediplomskim studijama i međunarodnim kursevima koji su organizovani u Institutu. Od 1965. godine sve do odlaska u penziju, držao je predavanja na poslediplomskim studijama iz grupe predmeta iz oblasti prethodno napregnutog betona, a poslednjih godina i iz oblasti zemljotresnog inženjerstva na GF u Beogradu i IZIIS-u. U dugom nizu godina od 1977. godine, drži predavanja iz predmeta Odabrana poglavlja iz teorije betonskih konstrukcija na poslediplomskim studijama na Fakultetu tehničkih nauka u Novom Sadu, a jedne školske godine držao je predavanja i iz predmeta Montažne betonske konstrukcije.

Kao rezultat višegodišnjeg angažovanja na poslediplomskim studijama, objavljena je u stručnoj i naučnoj javnosti izuzetno cenjena knjiga „Odabrana poglavlja iz zemljotresnog inženjerstva”, koja je doživela dva izdanja (prvo – 1985. i drugo – 1989. godine). Pored toga, napisao je skripta na engleskom jeziku *Prestressed Concrete Structures* za poslediplomske studije na IZIIS-u Skoplje, a na srpskom jeziku iz oblasti prethodno napregnutih konstrukcija – površinski nosači (ravni i zakrivljeni). Skripta su korišćena u poslediplomskoj nastavi na Građevinskom fakultetu u Beogradu i na Fakultetu tehničkih nauka u Novom Sadu.

Pedagoški kvaliteti dr B. Petrovića na svim univerzitetima visoko su ocenjivani, pa je pored neprekidnog angažovanja na poslediplomskim studijama često držao i predavanja po pozivu. Tako je na Univerzitetu u Helsinkiju održao ciklus od pet predavanja o jugoslovenskim istraživanjima u zemljotresnom inženjerstvu, 1979. godine. Vodio je veći broj kandidata prilikom izrade diplomskih i magistarskih radova iz

He started his professional and research career as a junior researcher in 1950. He was working on the development and introduction of pre-stressed concrete in Yugoslav engineering practice, testing, designing and theoretically developing structures from pre-stressed concrete in the group for pre-stressed concrete at the Federal Institute of Civil Engineering as associate of professor Branko Žeželj. In 1952, he joined the newly formed Institute for Testing of Material SRS (IMS), in which he worked for almost 40 years. During these years, he was the Head of the Department of Concrete and Asphalt, the Head of the Department for Building and Structures, and advisor at the Department for Development. He worked on problems of industrial construction, seismic construction, and once also on the development of reactor vessels from pre-stressed concrete. On that occasion, he stayed and worked for six months in the Svenska Atom Energy in Stockholm contributing on the design and development of large-scale vessels. He was a permanent member of the Scientific Council of IMS and served as its Head in two terms. From the position of the Head of Scientific Work at the IMS, in 1989, he moved to the Faculty of Technical Sciences of the University of Novi Sad, from where he retired in 1993 as a full professor.

Dr. Bosko Petrovic. began his pedagogical activity in 1960, and from that time until his retirement, he was involved in many forms of teaching activity at several universities in our country and abroad. In 1964, the year of introduction of specialist and postgraduate studies at the Faculty of Civil Engineering (GF) in Belgrade, he began his long-term cooperation with this Faculty. After habilitation with Professor Dr. Vlatko Brčić, in 1965, he was elected assistant professor and in 1978, associate professor. In addition, he was elected associate professor at the Institute for Earthquake Engineering and Engineering Seismology (IZIIS) of the University of Skopje, where he delivered lectures in the field of pre-stressed concrete at postgraduate studies and international courses organized by the Institute. From 1965 until his retirement, he delivered lectures at post-graduate studies for the group of teaching subjects from the field of pre-stressed concrete, and in recent years also from the field of earthquake engineering at Faculty of Civil Engineering in Belgrade and IZIIS. Over a long period of time since 1977, he delivered lectures for teaching subjects *Selected Chapters from the Theory of Concrete Structures* at postgraduate studies at the Faculty of Technical Sciences, University of Novi Sad, and over one academic year for the teaching subject *Precast Concrete Structures.*

As a result of many years of involvement in postgraduate studies, he published the book "Selected Chapters from Earthquake Engineering", which is highly respected in the professional and scientific community, and had two editions (in 1985 and 1989). In addition, he was the author of a textbook in English – Pre-stressed Concrete Structures - for postgraduate studies at IZIS (Skopje), and in Serbian, from the field of pre-stressed structures - surface carriers (flat and curved), that was used at postgraduate studies at the Faculty of Civil Engineering in Belgrade and at the Faculty of Technical Sciences in Novi Sad.

Pedagogical qualities of Dr. B. Petrović were highly appreciated at many universities, and in addition to oblasti prethodno napregnutog betona i zemljotresnog inženjerstva. Bio je član komisija prilikom odbrana doktorskih disertacija iz istih oblasti na većem broju univerziteta u Jugoslaviji.

Dve oblasti u kojima je aktivno radio i – kombinujući teorijska i eksperimentalna istraživanja – postigao izvanredne domete jesu Prethodno napregnuti beton i Zemljotresno inženjerstvo. Tome je doprineo njegov dugogodišnji rad na razvoju i uvođenju Prednapregnutog betona budući da je bio najbliži saradnik akademika Branka Žeželja, sa izvanrednim poznavanjem Teorije konstrukcija i njenih primena. Zbog toga je smatran nezaobilaznim ekspertom u toj oblasti. U svojstvu eksperta za Zemljotresno inženjerstvo (ZI), aktivno se angažovao na proceni objekata stradalih u Skopskom zemljotresu 26. jula 1963, pa je svoje analize publikovao u knjizi: Solovjev, Đ. i Petrović, B.; „Dejstvo zemljotresa na građevinske objekte u Skoplju" u knjizi Skopski zemljotres 26. jula 1963. Beograd, Savez jugoslovenskih laboratorija, 1968. Značajno je pomenuti i to da je koautor prve knjige posvećene zemljotresnom inženjerstvu u našoj zemlji: Hiba, Ž., Drašković, R., Solovjev, Đ., Petrović, B.: „Priručnik za projektovanje seizmički otpornih građevina". Građevinska uprava DSNO, Beograd, 1966; drugo izdanje je publikovano 1972. godine. Prva celovita monografija „Odabrana poglavlja iz zemljotresnog građevinarstva" imala je dva izdanja: Građevinska knjiga, Beograd, 1985. i 1991. Koautor je celovite monografije „Zemljotresno građevinarstvo – visokogradnja", Građevinska knjiga, Beograd 1990. godine. Boško Petrović dao je značajan doprinos izradi i unapređivanju tehničke regulative u oblasti zemljotresnog inženjerstva. Potvrda za to jeste i to što je bio član radne grupe koja je napisala prve jugoslovenske propise za građenje u seizmičkim područjima, a učestvovao je i u izradi svih kasnijih verzija ovog dokumenta. Bitno je spomenuti i njegov rad na ojačanju stambenih i drugih zgrada u Petropavlovsku na Kamčatki, gde je metodom prednaprezanja, bez iseljavanja i ometanja funkcije ojačano vise desetina zgrada.

Zapaženi su njegovi radovi objavljeni u monografijama, časopisima, na kongresima i simpozijumima iz šire oblasti građevinskog konstrukterstva u kojima je pokazao samo deo svoje naučne i stručne sposobnosti. Vodio je niz naučnih projekta, ne samo u našoj zemlji već i na međunarodnom nivou. Naučni rad prof. B. Petrovića bio je najtešnje povezan sa zahtevima prakse, što se može ilustrovati mnoštvom primera. Uspešno je radio na razvoju mernih instrumenata i eksperimentalne tehnike, što je rezultiralo prvim spregnutim analitičko-eksperimentalnim opitima u našoj praksi. Tako se eksperimentalnim istraživanjem i u praksi potvrdio smelim projektima rešetkastih nosača u Splitu, Zadru i na Dvorani sportova u Zagrebu.

Konstruktor je mnogih objekata, sam ili kao koprojektant prof. Žeželja - velika Hala 1 na Beogradskom sajmu, drumski most preko Tise u Titelu, drumsko-železnički most u Novom Sadu. Od samostalnih projekata, ističu se kompleks od šest velikih lučkih skladišta u luci Bar, konstrukcija Fabrike aluminijskih odlivaka u Vlasenici, most preko Neretve u Čapljini, sportska – košarkaška dvorana u kompleksu Doma sportova u Zagrebu, veći broj raznih zgrada u IMS

continuous involvement at postgraduate studies, he often delivered lectures upon invitation. Thus, in 1979 he held a cycle of five lectures at the University of Helsinki on Yugoslav research in earthquake engineering. He taught and was a mentor for many students preparing master theses in the field of pre-stressed concrete and earthquake engineering. He was a member of the committee for defense of doctoral dissertations from the above fields at several universities in Yugoslavia.

He actively worked in two fields and by combining theoretical and experimental research, he accomplished outstanding results in pre-stressed concrete and earthquake engineering. It was the outcome of a long work on the development and introduction of pre-stressed concrete as the closest associate of academician Branko Žeželj, along with excellent knowledge of the Theory of Structures and its applications. Therefore, he was considered an outstanding expert in the field. As an expert in earthquake engineering, he was actively involved in assessing the structures that fell victims of the Skopje earthquake on July 26, 1963, publishing his analysis in the book Solovjev, Đ. and Petrović, B.; "*Effects of Earthquake on Objects of Civil Engineering in Skopje*" in book The Skopje Earthquake on July 26, 1963 Belgrade, Association of Yugoslav Laboratories, 1968. It is important to mention that he co-authored the first book dedicated to earthquake engineering in our country: Hiba, Z., Draskovic, R., Solovjev, Đ., Petrovic, B.: "*Manual for Designing Seismic Resistant Structures"*. Građevinska Uprava DSNO, Beograd, 1966. The second edition of this manual was published in 1972. Two editions of his book, is the first comprehensive monograph "*Selected Chapters from Earthquake Engineering*", Građevinska knjiga, Beograd, 1985 and 1991. He co-authored the comprehensive monograph *Earthquake Engineering – Building Construction*, Građevinska knjiga, Beograd 1990, 642 p. In addition, Bosko Petrović made a significant contribution to the development and improvement of technical regulations in the field of earthquake engineering. This is confirmed by the fact that he was a member of a work group that wrote the first Yugoslav codes for building in seismic areas, and participated in the drafting of all subsequent versions of this document. It is also important to mention his work on strengthening the housing and other buildings in Petropavlovsk in Kamchatka, where more than a dozen buildings were strengthened using the method of pre-stressing, without moving out the people and obstructing the operation.

He published papers in monographs, journals, at conferences and symposiums from the wider field of construction design, where he showed only part of his scientific and professional capabilities. He led a number of scientific projects both in our country and on the international level. The scientific work of Professor B. Petrović was closely related with the demands of practice, which can be illustrated by many examples. He has successfully worked on the development of measuring instruments and experimental techniques, which resulted in the first coupled analytical-experimental researches in our practice. His theoretical and practical expertise was confirmed by bold projects of truss systems projects in Split, Zadar and at the Sports Hall in Zagreb.

sistemu itd.

Boško Petrović je autor ili koautor velikog broja studija, projekata i ekspertiza, a radio je i na proračunu reaktorskih sudova, pod pritiskom, od prednapregnutog betona. Posebno se angažovao na projektovanju, razvoju i istraživanju montažnih zgrada sistema IMS, koji je, sa prof. B. Žeželjem, razvio u jedan od seizmički najotpornijih konstruktivnih sistema zgrada. Ovaj sistem je široko primenjivan kod nas i u svetu (Kuba, Etiopija, Italija, Austrija, Rusija, Gruzija i Kina). Pored eksperimentalne provere, ovaj sistem izdržao je zemljotrese u Banja Luci 1969. i 1980. godine. Na zgradama koje je instrumentizirao IZIIS Skoplje, dokazane su njegove teorijske postavke o njihovom ponašanju za vreme zemljotresa. To je doprinelo promeni mišljenja o kvalitetu prednapregnutog betona kao materijala za seizmički otporno građenje.

Od 1974. godine do kraja osamdesetih, vodio je niz projekata unutar Yu-SAD naučne saradnje. Kao izuzetan eksperimentator, Boško Petrović vodio je celokupan program ispitivanja IMS zgrada i njihovih elemenata. Ovaj dugogodišnji uspešan rad doneo mu je prestižno međunarodno priznanje 1970. godine - izbor za člana FIP-ove (Međunarodne federacije za prednaprezanje) komisije u kojoj je bio aktivan do odlaska u penziju. Isticao se kao predsednik Jugoslovenskog društva za prednaprezanje od 1984. godine; predstavnik je Jugoslavije u Savetu Međunarodnog udruženja za prednaprezanje (FIP) sve do 1998. godine, a bio je i njegov potpredsednik. Radio je kao član FIP-ove Seizmičke komisije. Član je radne grupe WG-4 (prefabrikovane zgrade u seizmičkim područjima) projekta „Smanjenje seizmičkog rizika balkanskog regiona". Kao ekspert UNTA izradio je predlog Propisa za projektovanje zidanih zgrada do visine od pet spratova u seizmičkim područjima Irana, 1975. godine. Istraživao je i radio je na obradi dva zajednička YU-SAD projekta. Aktivno je sarađivao sa *Svenska Atom Energy* na polju nuklearnih reaktora, Univerzitetom u Berkliju Kalifornija, na polju seizmički sigurnih zgrada i Međunarodnim udruženjem za prednapregnuti beton (FIP).

Za svoj rad bio je odlikovan Ordenom rada III stepena i Ordenom rada sa zlatnim vencem. Dodeljeno mu je zvanje zaslužnog i počasnog člana Društva inženjera i tehničara Srbije i Jugoslavije i Jugoslovenskog društva za prednapregnuti beton. Bio je počasni član Društva inženjera i tehničara Novog Sada. Plaketu za životno delo Jugoslovenskog društva građevinskih konstruktera dobio je 2000. godine. Zajedno sa prof. Žeželjom i inž. D. Ćertićem, 1957. godine odlikovan je nagradom Saveta za nauku i kulturu NR Srbije za rad na rešetkastim nosačima od prednapregnutog betona. Sa prof. Žeželjem dobio je takođe Oktobarsku nagradu Beograda i Sedmojulsku nagradu.

Profesor Boško Petrović visoko je cenjen pedagog i istraživač koji je strpljivo prenosio svoja znanja na saradnike i studente. I posle odlaska u penziju, živeći u Londonu, pri dolasku u Beograd, često je posećivao Novi Sad i svoje kolege i saradnike i neretko bi održao „nezvanična predavanja", u opuštenoj atmosferi, koja bi obogatila svakog slušaoca. Ipak, u svakom nastupu delovao je kao veliki pedagog i njegov uticaj na mlađe kolege iskazan je podatkom da Novi Sad u Srbiji

He designed many structures, alone or as a co-designer with Professor Žeželj, such as the large Hall 1 at the Belgrade Fair, the road bridge over the Tisa river in Titel, the road-rail bridge in Novi Sad. Important among his independent projects are the complex of six large port warehouses in the port of Bar, the structure of the Aluminum Castings Factory in Vlasenica, the bridge over the Neretva river in Čapljina, the Sports Basketball Hall in the complex of Home of Sports in Zagreb, as well as a number of various buildings in the IMS system,

Bosko Petrovic was the author or co-author of a large number of studies, projects and expertise, and he also worked on calculation of reactor vessels, under pressure, from pre-stressed concrete. He was particularly involved in the design, development and research of prefabricated buildings of the IMS system. Together with Professor B. Žeželj, he developed these buildings into one of the most seismic resistant structural building systems. This system was widely applied both in our country and abroad (Cuba, Ethiopia, Italy, Austria, Russia, Georgia and China). In addition to the experimental testing, it was able to withstand earthquakes in Banja Luka in 1969 and 1980. On buildings instrumented by IZIS Skopje, his theoretical findings on their behaviour during the earthquake were proven. It contributed to changing the opinion on the quality of pre-stressed concrete as a material for seismic-resistant construction.

From 1974 to the end of the 1980s, he led a series of projects from the Yu-USA scientific cooperation. As an exceptional experimenter, Bosko Petrovic conducted the entire program of testing the IMS buildings and their elements. This long-lasting successful work brought him a prestigious international recognition: in 1970, he was chosen a member of the FIP (International Federation for Pre-stressing) committee, where he was active until retirement. Since 1984, he stood as the President of the Yugoslav Association for Pre-stressing and was a representative of Yugoslavia at the International Federation of Pre-stressing (FIP) until 1998, where he also served as the vice-president. He worked as a member of the FIP Seismic Commission. He was a member of the WG-4 work group (prefabricated buildings in seismic areas) on the project "Reducing seismic risk in the Balkan region". In 1975, as an expert of UNTA, he drafted the proposal for the "Regulation for the design of masonry buildings up to a height of five floors in the seismic regions of Iran". He was researching and working on two joint YU-USA projects. He actively collaborated with Svenska Atom Energy in the field of nuclear reactors, with the University of Berkeley, California, in the field of seismically safe buildings, and with the International Federation of Pre-stressing (FIP).

He was decorated with the Medal of work, 3th level and Medal of work with golden wreath for his work and contribution. He was awarded the title of a deserving and honorable member of the Society of Engineers and Technicians of Serbia and Yugoslavia and the Yugoslav Association for Pre-stressed Concrete. He was honorary member of the Society of Engineers and Technicians of Novi Sad. In 2000, the Yugoslav Society of Structural Engineering awarded him the plaque for his lifework. In 1957, together with Professor Žeželj and the engineer D. Ćertić, he was awarded the prize of the Council for prednjači u oblasti zemljotresnog inženjerstva. On je dao nesumnjiv doprinos razvoju oblasti konstrukcija od prethodno napregnutog betona i seizmičkog građevinarstva i time ostavio trajni trag u našem građevinskom konstrukterstvu, ali će pre svega ostati upamćen kao izuzetno neposredan i topao čovek.

Boško Petrović bio je posvećen porodici, supruzi Zagi, kćerki Svetlani i njenoj porodici, naročito unucima Marku i Đorđu, kojima je bio i bliski učitelj i nežni deda. Bio je čovek široke kulture s bogatim poznavanjem književnosti i klasične muzike. Iskazivao se kao nesebičan prijatelj koji je rado prihvatao različite teme razgovora i bio izuzetan sagovornik. Njegova smrt je najveći gubitak za njegovu porodicu, kojoj izražavamo iskreno i duboko saučešće, ali i za sve nas koji smo ga znali i imali sreće da nam bude učitelj i prijatelj.

Juna, 2019. godine

Profesor emeritus dr Radomir Folić, dipl. inž. građ.

Science and Culture of the People's Republic of Serbia for his work on truss system from pre-stressed concrete. Together with Professor Žeželj, he received the October Award of Belgrade and the Prize of the Seventh of July.

Professor Bosko Petrovic is a highly respected pedagogue and researcher, who patiently transferred his knowledge to his associates and students. Living in London after retirement, he often visited Belgrade and Novi Sad, as well his colleagues and associates, often delivering "unofficial lectures" in a relaxed atmosphere, enriching thereby the whole audience. Nevertheless, in every presentation he was as a great pedagogue, and his influence on his younger colleagues was shown by the fact that Novi Sad is the leader in earthquake engineering in Serbia. He gave a huge contribution to the development in the field of pre-stressed concrete structures and seismic engineering, leaving a permanent trace in our structural design, but will be remembered as an extremely spontaneous and warmhearted Person.

Bosko Petrovic was dedicated to his family, his wife Zaga, his daughter Svetlana and her family, especially to his grandchildren Marko and Đorđe. He was both a dear teacher and gentle grandfather. He was a Man of broad culture with a rich knowledge of literature and classical music. He was an unselfish friend, who gladly accepted various topics of conversation and was an outstanding interlocutor. His death is the greatest loss for his family to whom we express our sincere and deep condolence, but also for all of us who knew him and was lucky to have him as a teacher and friend.

June, 2019.

Professor Emeritus Dr. Radomir Folić, B.Sc. Eng. civ.

UPUTSTVO AUTORIMA*

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U časopisu Materijli i konstrukcije štampaće se neobjavljeni radovi ili članci i konferencijska saopštenja sa određenim dopunama, iz oblasti građevinarstva i srodnih disciplina (geodezija i arhitektura). Vrste priloga autora i saradnika koji će se štampati su: originalni naučni radovi, prethodna saopštenja, pregledni radovi, stručni radovi, prikazi objekata i iskustava (studija slučaja), kao i diskusije povodom objavljenih radova.

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Prethodno saopštenje sadrži prva kratka obaveštenja o rezultatima istraživanja ali bez podrobnih objašnjenja, tj. kraće je od originalnog naučnog rada.

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Ostali prilozi su prikazi objekata, tj. njihove konstrukcije i iskustava-primeri u građenju i primeni različitih materijala (studije slučaja).

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Od broja 1/2010, prema odluci Upravnog odbora Društva i Redakcionog odbora, radovi sa pozitivnim recenzijama i prihvaćeni za štampu, publikovaće se na srpskom i engleskom jeziku, a za inostrane autore na engleskom (izuzev autora sa govornog područja srpskog I hrvatskog jezika).

Svaka stranica treba da bude numerisana, a optimalni obim članka na jednom jeziku, je oko 16 stranica (30000 slovnih mesta) uključujući slike, fotografije, tabele i popis literature. Za radove većeg obima potrebna je saglasnost Redakcionog odbora.

* U**putstvo autorima je modifikovano i treba ga, u pripremi radova, slediti.**

GUIDELINES TO AUTHORS

Acceptance and types of contributions

The Building Materials and Structures journal will publish unpublished papers, articles and conference reports with modifications in the field of Civil Engineering and similar areas (Geodesy and Architecture).The following types of contributions will be published: original scientific papers, preliminary reports, review papers, professional papers, objects describe / presentations and experiences (case studies), as well as discussions on published papers.

Original scientific paper is the primary source of scientific information and new ideas and insights as a result of original research using appropriate scientific methods. The achieved results are presented briefly, but in a way to enable proficient readers to assess the results of experimental or theoretical numerical analyses, so that the research can be repeated and yield with the same or results within the limits of tolerable deviations, as stated in the paper.

Preliminary report contains the first short notifications on the results of research but without detailed explanation, i.e. it is shorter than the original scientific paper.

Review paper is a scientific work that presents the state of science in a particular area as a result of analysis, review and comments, and conclusions of published papers, on which the necessary data are presented clearly and critically, including the own papers. Any reference units used in the analysis of the topic are indicated, as well as papers that may contribute to the results of further research. If the reference data are methodically systematized, but not analyzed and discussed, such review papers are classified as technical papers.

Technical paper is a useful contribution which outlines the known insights that contribute to the dissemination of knowledge and adaptation of the results of original research to the needs of theory and practice.

Other contributions are presentations of objects, i.e. their structures and experiences (examples) in the construction and application of various materials (case studies).

In order to speed up the acceptance of papers for publication, authors need to take into account the Instructions for the preparation of papers which can be found in the text below.

Instructions for writing manuscripts

The manuscript should be typed one-sided on A-4 sheets with margins of 31 mm (top and bottom) and 20 mm (left and right) in Word, font Arial 12 pt. The entire paper should be submitted also in electronic format to e-mail address provided here, or on CD. The author is obliged to keep one copy of the manuscript.

As of issue 1/2010, in line with the decision of the **Management Board of the Society and the Board of Editors, papers with positive reviews, accepted for publication, will be published in Serbian and English, and in English for foreign authors (except for authors coming from the Serbian and Croatian speaking area).**

Each page should be numbered, and the optimal length of the paper in one language is about 16 pages (30.000 characters) including pictures, images, tables and references. Larger scale works require the approval of the Board of Editors.

Naslov rada treba sa što manje reči (poželjno osam, a najviše do jedanaeset) da opiše sadržaj članka. U naslovu ne koristiti skraćenice ni formule. U radu se iza naslova daju ime i prezime autora, a titule i zvanja, kao i ime institucije u podnožnoj napomeni. Autor za kontakt daje telefon, adresu elektronske pošte I poštansku adresu.

Uz sažetak (rezime) od oko 150-250 na srpskom i engleskom jeziku daju se ključne reči (do sedam). To je jezgrovit prikaz celog članka i čitaocima omogućuje uvid u njegove bitne elemente.

Rukopis se deli na poglavlja i potpoglovlja uz numeraciju, po hijerarhiji, arapskim brojevima. Svaki rad ima uvod, sadržinu rada sa rezultatima, analizom i zaključcima. Na kraju rada se daje popis literature.

Kod svih dimenzionalnih veličina obavezna je primena međunarodnih SI mernih jedinica.

Formule i jednačine treba pisati pažljivo vodeći računa o indeksima i eksponentima. Autori uz izraze u tekstu definšu simbole redom kako se pojavljuju, ali se može dati i posebna lista simbola u prilogu.

Prilozi (tabele, grafikoni, sheme i fotografije) rade se u crno-beloj tehnici, u formatu koji obezbeđuje da pri smanjenju na razmere za štampu, po širini jedan do dva stupca (8 cm ili 16,5 cm), a po visini najviše 24,5 cm, ostanu jasni i čitljivi, tj. da veličine slova i brojeva budu najmanje 1,5 mm. Originalni crteži treba da budu kvalitetni i u potpunosti pripremljeni za presnimavanje. Mogu biti i dobre, oštre i kontrastne fotokopije. Koristiti fotogrfije, u crno-beloj tehnici, na kvalitetnoj hartiji sa oštrim konturama, koje omogućuju jasnu reprodukciju.

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U Popisu literature se navode prezime i inicijali imena autora, zatim potpuni naslov citiranog članka, iza toga sledi ime časopisa, godina izdavanja i početna i završna stranica (od - do). Za knjige iza naslova upisuje se ime urednika (ako ih ima), broj izdanja, prva i poslednja stranica poglavlja ili dela knjige, ime izdavača i mesto objavljivanja, ako je navedeno više gradova navodi se samo prvi po redu. Kada autor citirane podatke ne uzima iz izvornog rada, već ih je pronašao u drugom delu, uz citat se dodaje «citirano prema...».

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The title should describe the content of the paper using a few words (preferably eight, and up to eleven). Abbreviations and formulas should be omitted in the title. The name and surname of the author should be provided after the title of the paper, while authors' title and position, as well as affiliation in the footnote. The author should provide his/her phone number, e-mail address and mailing address.

The abstract (summary) of about 150-250 words in Serbian and English should be followed by key words (up to seven). This is a concise presentation of the entire article and provides the readers with insight into the essential elements of the paper.

The manuscript is divided into chapters and subchapters, which are hierarchically numbered with Arabic numerals. The paper consists of introduction and content with results, analysis and conclusions. The paper ends with the list of references. All dimensional units must be presented in international SI measurement units. The formulas and equations should be written carefully taking into account the indexes and exponents. Symbols in formulas should be defined in the order they appear, or alternatively, symbols may be explained in a specific list in the appendix. Illustrations (tables, charts, diagrams and photos) should be in black and white, in a format that enables them to remain clear and legible when downscaled for printing: one to two columns (8 cm or 16.5 cm) in height, and maximum of 24.5 cm high, i.e. the size of the letters and numbers should be at least 1.5 mm. Original drawings should be of high quality and fully prepared for copying. They also can be high-quality, sharp and contrasting photocopies. Photos should be in black and white, on quality paper with sharp contours, which enable clear reproduction.

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Izdavanje časopisa "Građevinski materijali i konstrukcije" finansijski su pomogli:

REPUBLIKA SRBIJA
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GRAĐEVINSKI FAKULTET

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MATEST
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iT TECH

INELAS ERECO d.o.o

MATEST "iT TECH" KONTROLNA JEDINICA

JEDNA TEHNOLOGIJA MNOGO REŠENJA

IT Touch Techlogy je Matestov najnoviji koncept koji ima za cilj da ponudi inovativna i user-friendly tehnologiju za kontrolu i upravljanje najmodernijom opremom u domenu testiranja građevinskih materijala

Ova tehnologija je srž Matestove kontrolne jedinice, software baziran na Windows platformi i touch screen sistem koji je modularan, fleksibilan i obavlja mnoge opcije

iT TECH pokriva

- I INOVATIVNOST
- I INTERNET KONEKCIJA
- I INTERFEJS SA IKONICAMA
- I INDUSTRIJALNA TEHNOLOGIJA

SISTEM JEDNOG RAZMIŠLJANJA JEDNOM SHVATIŠ - SVE TESTIRAŠ

NAPREDNA TEHNOLOGIJA ISPITIVANJA ASFALTA

- I GYROTRONIC - Gyratory Compactor
- I ARC - Electromechanical Asphalt Roller Compactor
- I ASC - Asphalt Shear Box Compactor
- I SMARTRACKER™ - Multiwheels Hamburg Wheel Tracker, DRY + WET test environment
- I SOFTMATIC - Automatic Digital Ring & Ball Apparatus
- I Ductilometers with data acquisition system

MULTIFUNKCIONALNI RAMOVI ZA TESTIRANJE

- I CBR/Marshall digital machines
- I Universal multispeed load frames
- I UNITRONIC 50kN or 200kN Universal multipurpose compression/flexural and tensile frames

OPREMA ZA GEOMEHANIČKO ISPITIVANJE

- I EDOTRONIC - Automatic Consolidation Apparatus
- I SHEARLAB - AUTOSHEARLAB - SHEARTRONIC Direct / Residual shear testing systems
- I Triaxial Load Frame 50kN

MIXMATIC - Automatic Programmable Mortar Mixer

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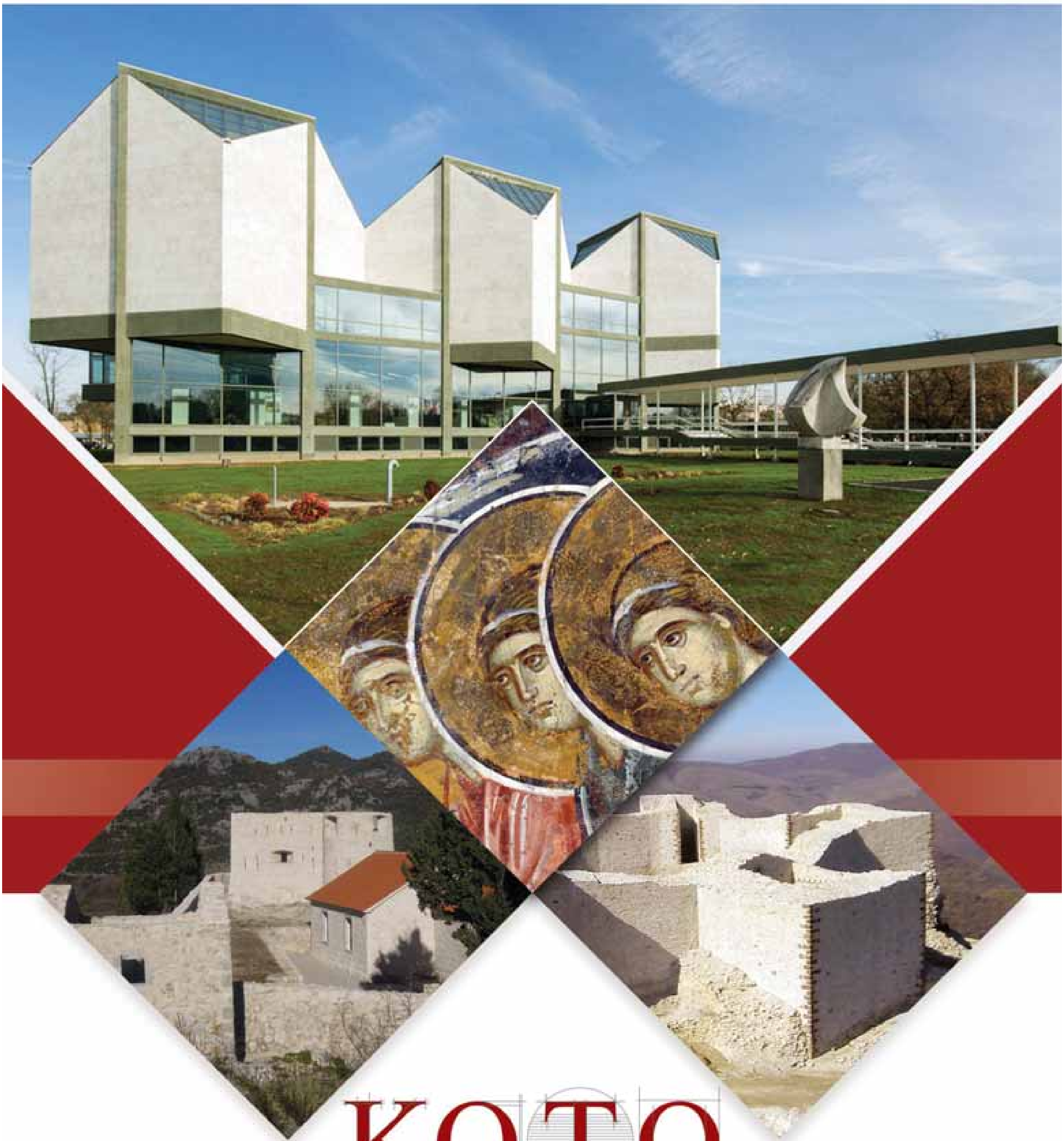
Tošin bunar 274a, 11070 , Novi Beograd tel. +381 11 2284 574 email. info@inelasereco.rs



Institut za ispitivanje materijala IMS, sa tradicijom od 1929. godine, predstavlja najstariju naučno-istraživačku instituciju u Srbiji. Osnovna ideja prilikom osnivanja bila je potreba za jedinstvenom institucijom koja bi se osim istraživanja bavila i kontrolom građevinske industrije.

Delatnost Instituta IMS obuhvata laboratorijska ispitivanja građevinskih materijala, sertifikaciju proizvoda, nadzor nad izvođenjem radova i ispitivanje različitih tipova konstrukcija, izradu projektne dokumentacije, kao i naučno - istraživački rad u svim oblastima građevinarstva.

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- Centar za konstrukcije i prednaprezanje

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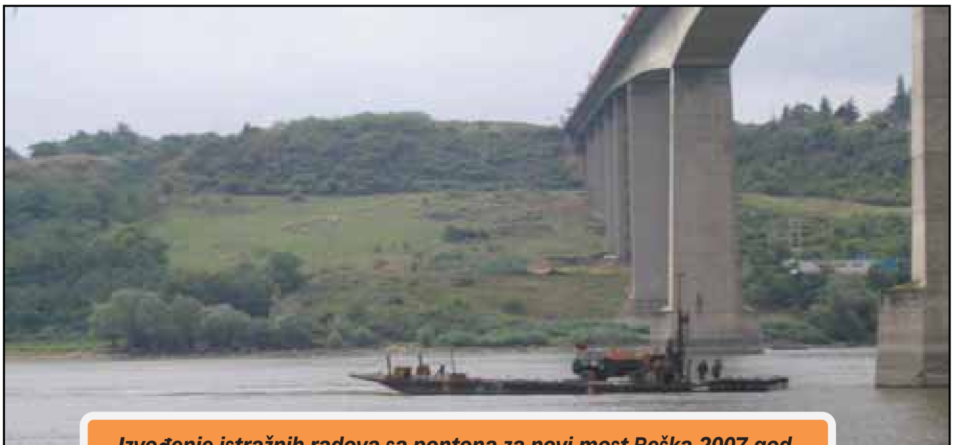


Izvođenje istražnih radova sa pontona za novi most Beška,2007.god.

Geotehnička istraživanja i ispitivanja – in situ

Od terenskih istražnih radova izdvajamo izvođenje istražnih bušotina (IB), standardnih penetracionih opita (SPT), statičkih penetracionih opita (CPT i CPTU), opita dilatometarskom sondom (DMT i SDMT), ispitivanja vodopropustljivosti tla različitim terenskim metodama (VDP), ugradnja pijezometara i dr.
Terenske metode ispitivanja šipova zauzimaju značajno mesto u našoj delatnosti, a na tržištu se izdvajamo kao lideri u toj oblasti u protekloj deceniji.

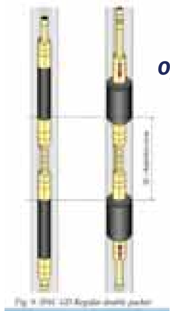

DLT-dinamičko ispitivanje šipova

Ispitivanje šipova

SLT metoda (Static load test) ispitivanje nosivosti šipova statičkim opterećenjem;
DLT metoda (Dynamic load test) ispitivanje nosivosti šipova dinamičkim opterećenjem;
PDA metoda (Pile driving analysis) omogućava praćenje i optimizaciju procesa pobijanja prefabrikovanih betonskih i čeličnih šipova u tlo;
PIT (SIT) metoda (Pile(Sonic) integrity testing) koristi se za ispitivanje integriteta izvedenih šipova (dužine, prekida, suženja ili proširenja).


CPT/CPTU opiti


Aktivno klizište



oprema za ispitivanje vodopropusnosti stena pod pritiskom do 10 bar-a metodom LIŽONA

Laboratorija za puteve i geotehniku

Laboratorija za puteve i geotehniku akreditovana je kod Akreditacionog tela Srbije – ATS prema SRPS ISO/IEC 17025:2006. U njoj se vrše ispitivanja tla (identifikaciono–klasifikaciona ispitivanja, fizičko–mehanička modelska ispitivanja), kamenog agregata i brašna, bitumena i bitumenskih emulzija, asfaltnih mešavina. U okviru laboratorijskih ispitivanja na terenu vrši se kontrola kvaliteta ugrađenog materijala i izvedenih radova (prethodna, tekuća, kontrolna ispitivanja i izvođenja opita in situ).

Projektovanje puteva i sanacija klizišta

U okviru projektovanja značajno mesto u radu zauzimaju geotehnička istraživanja terena i projekti sanacije klizišta - nestabilnih kosina useka i nasipa puteva i prirodno nestabilnih padina . Značajna su i projekovanja svih vrsta fundiranja specijalnih geotehničkih konstrukcija. Ističe se i iskustvo u oblasti putarstva, na projektovanju novih, rehabilitacija i rekonstrukcija postojećih puteva svih rangova sa pratećim objektima i dimenzionisanjem kolovoznih konstrukcija.

Nadzor

Naši inženjeri imaju veliko iskustvo u kontroli i proveri kvaliteta izvođenja svih vrsta radova, kontroli građevinske dokumentacije i praćenju radova u skladu sa njom, kao i rešavanju novonastalih situacija tokom izvođenja radova.

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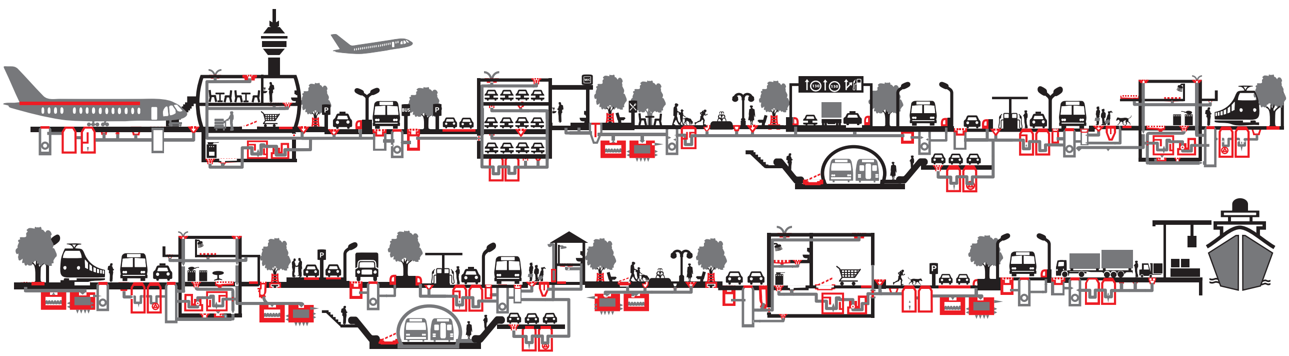


aco.rs

BUILDING TRUST

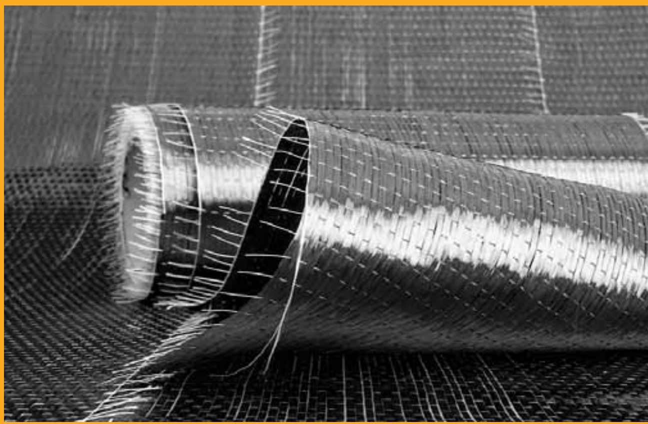
NAPREDNA SIKA REŠENJA U OBLASTI STRUKTURALNIH OJAČANJA

Kompanija Sika pruža trajnu dodatnu vrednost vlasnicima građevinskih objekata, njihovim konsultantima i izvođačima, kao i tehničku podršku tokom svih faza projekta, od ispitivanja uslova i razvoja inicijalnog koncepta ojačanja pa sve do uspešnog završetka i primopredaje projekta

SIKA – VAŠ PARTNER NA GRADILIŠTU

- Globalni lider na tržištu građevine i građevinske hemije
- Najbolja tehnička ekspertiza i praksa za sanaciju betona i strukturalna ojačanja
- Odlična reputacija kod vodećih izvođača i ugovarača posla

SIKA VREDNOSTI I INOVACIJE U GRAĐEVINI



- Integrisani proizvodi i sistemi visokih performansi koji mogu da povećaju i poboljšaju kapacitet, efikasnost, trajnost i estetiku zgrada i drugih objekata – u korist naših klijenata i boljeg održivog razvoja
- Sika mreža obučenih i iskusnih građevinskih stručnjaka

JEDINSTVENA SIKA REŠENJA U ZAHTEVNIM USLOVIMA

- Rešenja za gotovo sve uslove apliciranja
- Kontrolisano vreme rada, vreme sazrevanja i očvršćavanja za različite vremenske uslove
- Posebna rešenja završnih ojačanja za korišćenje kod betona slabije jačine i drugih podloga

POTVRĐENI SIKA SISTEMI I TEHNIKE APLICIRANJA

- Preko 40 godina iskustva u strukturalnim ojačanjima, sistemima i tehnikama
- Proizvodi i sistemi sa brojnim testovima i procenama kako internim tako i eksternim
- Najviši međunarodni standardi proizvodnje i kontrole kvaliteta

PUT INŽENJERING

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.

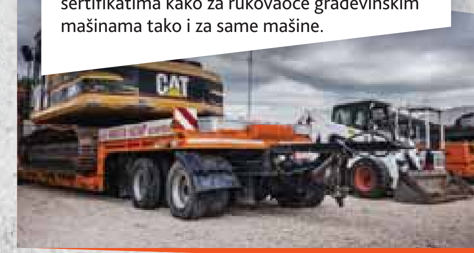
Obradu armature vršimo brzo, stručno i kvalitetno, sa kompjuterskom preciznošću i dimenzijama po projektu.

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

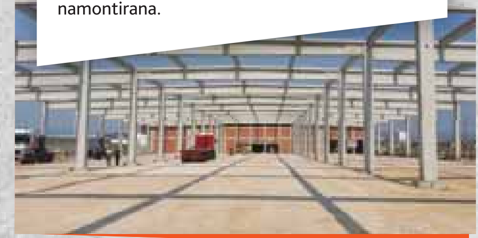
Koristeći inovativne tehnike i kvalitetan građevinski materijal iz sopstvenih resursa, spremni smo da odgovorimo na mnoge zahteve naših klijenata iz oblasti niskogradnje.

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.

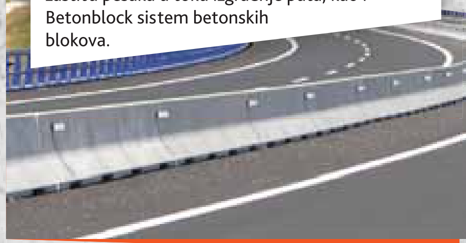


Prednapregnute šuplje ploče su konstruktivni elementi visokog kvaliteta, proizvedeni u fabrički kontrolisanim uslovima.

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.

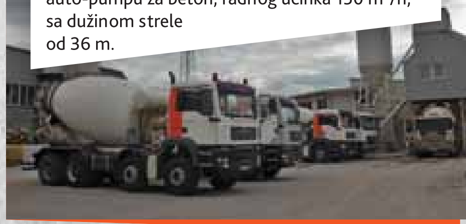
Izrađujemo betonske "New Jersey profile" koji se u svetu koriste za preusmeravanje saobraćaja i zaštitu pešaka u toku izgradnje puta, kao i Betonblock sistem betonskih blokova.



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 m^3 do 10 m^3 betonske mase. Za ugradnju betona posedujemo auto-pumpu za beton, radnog učinka 150 m^3/h, sa dužinom strele od 36 m.



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