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1

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

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Editor-in-Chief

GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

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PRIKAZ ANALITIČKIH POSTUPAKA ODREĐIVANJA SEIZMIČKI INDUKOVANIH UTICAJA U TUNELSKOJ KONSTRUKCIJI PRI INTERAKCIJI KONSTRUKCIJE I OKOLNE SREDINE

PRESENTATION OF ANALYTICAL SOLUTIONS FOR SEISMICALLY INDUCED TUNNEL LINING FORCES ACCOUNTING FOR SOIL–STRUCTURE INTERACTION EFFECTS

Elefterija ZLATANOVIĆ
Dragan Č. LUKIĆ
Vlatko ŠEŠOV

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1 UVOD

Realna procena dinamički indukovanih presečnih sila u tunelskoj konstrukciji predstavlja veoma važan aspekt aseizmičkog projektovanja tunelskih objekata. U analizi odgovora tunelskih konstrukcija na seizmičke uticaje, koriste se dva postupka: **metoda slobodnih deformacija tla** i **metoda interakcije konstrukcije i tla** [6, 23].

Najjednostavniji pristup u analizi seizmičkog odgovora tunelskih konstrukcija jeste *metoda slobodnih deformacija tla*. Termin *slobodna deformacija tla* opisuje deformaciju usled prostiranja seizmičkih talasa u tlu bez tunelske konstrukcije ili iskopa. Ideja ovog pojednostavljenog postupka jeste sračunavanje seizmički indukovanih deformacija tla u odsustvu tunelskog objekta, nanošenje tako sračunatih deformacija tla na tunelsku konstrukciju u statičkim uslovima, i projektovanje tunelske konstrukcije za tako definisano opterećenje. Samim tim, ovim postupkom krutost tunelske konstrukcije nije uzeta u obzir, čime je interakcija konstrukcije i okolnog tla zanemarena. Ipak, ovom metodom je moguća veoma jednostavna i brza procena reakcije konstrukcije na seizmička dejstva. Veličine deformacija tunelske konstrukcije

Asistent Elefterija Zlatanović, dipl.inž.građ.
Univerzitet u Nišu, Građevinsko-arhitektonski fakultet,
Aleksandra Medvedeva 14, 18000 Niš, Srbija,
e-mail: elefterija2006@yahoo.com
Prof. dr Dragan Č. Lukić, dipl.inž.građ., Univerzitet u Novom Sadu, Građevinski fakultet u Subotici, Kozaračka 2a, 24000 Subotica, Srbija, e-mail: drlukic.lukic@gmail.com
Vanr. prof. dr Vlatko Šešov, dipl.inž.građ., Univerzitet u Skoplju, Institut za zemljotresno inženjerstvo i inženjersku seizmologiju – IZIIS, Salvador Aljende 73, Poštanski fah 101, 1000 Skoplje, Makedonija, e-mail: vsesov@gmail.com

1 INTRODUCTION

The estimation of dynamic internal forces in the lining structure is a key procedure in the seismic design of tunnels. The seismic response of tunnel structures may be assessed using two approaches: the **free-field deformation approach** and the **soil–structure interaction approach** [6, 23].

The simplest approach is the so called *free-field ground deformation approach*. The term *free-field deformation* describes ground strains caused by seismic waves in the absence of structures or excavation. Accordingly, the free-field ground deformations due to a seismic event are estimated, and the underground structure is designed to accommodate these deformations. Therefore, this approach ignores the interaction of the underground structure with the surrounding ground, but can provide a first-order estimation of the anticipated deformation of the structure. This approach may overestimate or underestimate structure deformations depending on the rigidity of the structure relative to the ground, so thus, it is satisfactory when low levels of shaking are anticipated, or the underground facility is in

Assistant Elefterija Zlatanović, BCE, University of Niš, Faculty of Civil Engineering and Architecture, Aleksandra Medvedeva 14, 18000 Niš, Serbia, e-mail: elefterija2006@yahoo.com
Prof. Dr. Dragan Č. Lukić, BCE, University of Novi Sad, Faculty of Civil Engineering in Subotica, Kozaračka 2a, 24000 Subotica, Serbia, e-mail: drlukic.lukic@gmail.com
Assoc. Prof. Dr. Vlatko Šešov, BCE, Institute of Earthquake Engineering and Engineering Seismology – IZIIS, University of Skopje, Salvador Aljende 73, P.O.Box 101, 1000 Skopje, Macedonia, e-mail: vsesov@gmail.com

sračunate ovim postupkom mogu biti ili precenjene ili potcenjene, što zavisi od odnosa krutosti konstrukcije i tla. Primena ovog postupka je opravdana u slučaju zemljotresa slabijeg intenziteta, kao i u slučaju fleksibilne tunelske konstrukcije u tlu dobrih karakteristika ili u steni, kada krutost konstrukcije nema uticaja na deformaciju okolnog medijuma. U mnogim situacijama, a posebno u slučaju krute tunelske konstrukcije u sloju tla slabih karakteristika, ova metoda ne daje korektne rezultate, s obzirom na to što su seizmički indukovane deformacije tla slabih karakteristika veoma velike [21].

U analizi dinamičkog odgovora tunnelskih objekata, efekti interakcije konstrukcije i tla ne smeju biti zanemareni. Efekti sadejstva konstrukcije sa okolnim tlom veoma često mogu rezultovati znatno većim opterećenjima kojima je tunelska konstrukcija izložena.

Imajući u vidu da su tunnelski objekti ukopani u tlo (stenu), u uslovima zemljotresnih dejstava, reakcija tunela na seizmičko opterećenje će umnogome zavisi od ponašanja okolnog medijuma [18, 19], što u krajnjoj liniji rezultuje dinamičkim sadejstvom tunela sa okolnom sredinom, poznatim kao *interakcija konstrukcije i tla*.

Tunelska konstrukcija svojom krutošću u velikoj meri modifikuje kretanje tla, uzrokujući time znatno drugačiju reakciju na seizmička dejstva, usled kombinovanog efekta **kinematičke interakcije** i **inercijalne interakcije**. Kinematička interakcija, koja je od krucijalne važnosti, uslovljena je krutošću tunelske konstrukcije, i predstavlja nemogućnost konstrukcije da se prilagodi deformacijama okolnog tla, odnosno, konstrukcija svojom krutošću onemogućava pojavu slobodnih deformacija tla, svojstvenih prirodnom tlu bez prisustva podzemnih objekata. Inercijalna interakcija je uslovljena masom tunelske konstrukcije koja proizvodi efekat inercije konstrukcije na reakciju okolnog tla. Naime, dinamičke sile indukovane u tunelskoj konstrukciji uslovljavaju da tunel deformiše okolno tlo, što rezultuje pojavom radijacionog prigušenja, kada seizmički talasi menjaju smer prostiranja i propagiraju, udaljavajući se od tunelske konstrukcije. (Postoje dve vrste prigušenja. *Materijalno prigušenje* uslovljeno je neelastičnim svojstvima tla i predstavlja apsorpciju energije seizmičkih talasa koja se pretvara u toplotnu energiju, što za posledicu ima slabljenje amplituda seizmičkih talasa. U slučaju *radijacionog prigušenja*, poznatog i kao *geometrijsko prigušenje* ili *geometrijska atenuacija*, ne dolazi do konverzije energije seizmičkih talasa u druge oblike energije, ali uprkos tome, ipak se javlja redukcija amplituda seizmičkih talasa zbog zapreminskog širenja energije. Veličina materijalnog prigušenja zavisi od nivoa smičućih deformacija indukovanih u tlu – u slučaju velikih deformacija, vrednost materijalnog prigušenja je visoka, i obrnuto, u slučaju malih deformacija, vrednost materijalnog prigušenja je zanemarljivo mala. S druge strane, radijaciono prigušenje je isključivo geometrijski efekat koji se javlja i u slučaju malih i u slučaju velikih deformacija).

Kada je reč o tunnelskim objektima, inercijalna interakcija ima daleko manji značaj u odnosu na kinematičku, zato što je masa konstrukcije zanemarljivo mala u odnosu na masu okolnog tla. Prema tome, inercija tunelske konstrukcije može biti izostavljena u numeričkim modelima, jer je zanemarljiva u poređenju sa inercijom okolnog tla. Dakle, odgovor tunela na seizmička dejstva je u osnovi uslovljen deformacijama okolnog tla, i veličina deformacija konstrukcije će zavisi od odnosa

a stiff medium such as rock, or the structure is flexible compared to the surrounding medium, for example, for tunnels in rock where the stiffness of the structure is unlikely to affect the stiff surrounding deformation considerably. In many cases, however, especially in soft soils, the method gives overly conservative designs, because free-field ground distortion in soft soils is generally large [21].

In analyzing dynamic response of a structure, the effects of soil–structure interaction cannot be overlooked. The interaction effects between a structure and surrounding ground layers sometimes cause larger external forces to the structure.

Considering the fact that tunnel structures are embedded in a soil (rock) medium, the reaction of the vibrating structure depends to a great extent on the behaviour of the surrounding medium [18, 19], resulting in dynamic interaction between the structure and the ground layers, which is called *soil–structure interaction (SSI)*.

The presence of the structure sensibly modifies the free-field ground motion leading to a different structural response of the tunnel lining. This is due to a combined effect of **kinematic interaction** and **inertial interaction**. The former, primarily important one, is caused by the stiffness of the structure (inability of a structure to follow ground motion due to greater stiffness in comparison with ground stiffness), whereas the latter is caused by existence of structural mass (the effect of inertia force of the structure on the response of the surrounding soil). The mass of the structure transmits the inertial force to the soil causing further deformation in the soil, thus producing stress waves that travel away from the structure, when radiation damping occurs. (While *material damping*, caused by inelastic behaviour of the soil, absorbs some of the elastic energy of the stress wave, converting it to heat, and thus decreases the amplitude of the wave, in case of *radiation damping*, also known as *geometric damping* or *geometric attenuation*, even though elastic energy is conserved (no conversion to other forms of energy takes place), there is a reduction in amplitude of the stress wave due to spreading of the energy over a greater volume of material. The amount of material damping will depend on the level of strain induced in the soil – if the strains are high, material damping can be substantial, but if they are low, the material damping may be negligible. In contrast, radiation damping is purely geometric effect that exists at low as well as high strain amplitudes).

For a hollow structure like a tunnel, the inertia interaction is often less important than the kinematic interaction, because the mass of the structure is negligible in comparison with the mass of the surrounding ground. Thus, the tunnel section inertia could be ignored in numerical model, as it is negligible as compared to that of the surrounding ground. The tunnel basically responds to the ground deformations and the extent of the tunnel deformation will depend on the stiffness of the tunnel relative to that of the surrounding soil.

Tunnel–ground interaction under seismic impact is to a great extent more complex in comparison with the one considering surface structures, when only foundations are exposed to soil–structure interaction and vibrations of soil particles imposed to foundations are transmitted

krutosti tunelske konstrukcije i krutosti tla.

U dinamičkim analizama konstrukcija, efekti interakcije s tlom imaju veoma istaknutu ulogu, jer mogu rezultovati znatno većim opterećenjima kojima je konstrukcija izložena. Sadejstvo tunela s tlom u uslovima zemljotresne aktivnosti umnogome je složenije u poređenju s nadzemnim konstrukcijama, kod kojih su samo fundamenti izloženi efektima interakcije s tlom i u čijem se slučaju vibracije čestica tla putem temelja prenose na konstrukciju iznad tla. S druge strane, kada je reč o tunelskim objektima, interakcija konstrukcije s tlom je indukovana duž čitave konture tunelskog objekta i njena forma umnogome zavisi od tehnologije građenja, odnosno, načina iskopa i izgradnje podgradnog sistema [25].

Efekti interakcije zavise od brojnih parametara, kao što su maksimalno ubrzanje tla, intenzitet i trajanje zemljotresa, a od posebne važnosti je odnos krutosti konstrukcije i krutosti okolnog tla. U slučaju krute tunelske konstrukcije u rastresitom tlu, deformacije tla neće prouzrokovati deformacije konstrukcije, međutim, u slučaju fleksibilne tunelske obloge, interakcija sa okolnim tlom biće veoma izražena.

Još jedan veoma važan faktor, koji u značajnoj meri utiče na reakciju tunela, jeste priroda kontakta konstrukcije sa okolnim tlom. Istraživačke metode su uglavnom bazirane na pretpostavkama linearno-elastičnog ili visko-elastičnog ponašanja tla, i *idealne veze konstrukcije i tla u zoni kontakta (no-slip)*. U praksi, međutim, kontakt konstrukcije i tla nije idealan, već je daleko realnija pojava smicanja (klizanja) u kontaktnoj zoni, pa čak i odvajanja tunelske konstrukcije od tla. Takođe, zona tla u neposrednom kontaktu sa konstrukcijom može pretrpeti ekstremno velike deformacije, uslovljavajući na taj način nelinearno ponašanje sistema tunel-tlo. Ovakav slučaj se daleko realnije simulira pretpostavkom *kontakta s potpunim smicanjem (full-slip)*, koja je validna u slučajevima tla veoma slabih karakteristika i zemljotresa snažnog intenziteta. Pojava *delimičnog smicanja tunelske konstrukcije i tla u kontaktnoj zoni (partial slip)* najrealnija je u praksi, pri čemu je, s povećanjem deformacija u tlu i relativnih pomeranja između konstrukcije i tla, interakcija konstrukcije sa okolnom sredinom sve manje izražena. S ciljem sagledavanja svih mogućih slučajeva, preporučuje se da rešenja obuhvate ekstremne kontaktne uslove i da analizama bude obuhvaćen nepovoljniji slučaj. U situacijama kada ne postoji kontinuitet pomeranja tunelske konstrukcije i tla u kontaktnoj zoni, ili kada u konstrukciji postoji lokalno koncentrisana masa, efekat inercije u proračunima ne sme biti izostavljen.

Kada je reč o vrstama analiza, postoje tri kategorije: **pseudo-statička analiza, uprošćena dinamička analiza i dinamička analiza**, koje se međusobno razlikuju u pogledu kompleksnosti analitičkih modela, parametara kojima se opisuju svojstva lokalnog tla i načina prikaza seizmičkog opterećenja. Detaljan pregled analiza može se naći u istraživanju Billote i drugih [2].

- Primenom *pseudo-statičkih metoda*, analize tunelske konstrukcije i okolnog tla su razdvojene. Seizmičko opterećenje kao ulazni parametar redukovano je na veličinu maksimalne smičuće deformacije tla, koja je sračunata jednostavnim analitičkim izrazima baziranim na pretpostavci prostiranja harmonijskog smičućeg S-talasa u homogenom, izotropnom, elastičnom tlu, a potom naneta na tunelsku konstrukciju kao statičko

to the structure above the ground. On the contrary, as to tunnel structures, soil-structure interaction is induced along an overall contour of the structure, and a shape of interaction depends mainly on a type of a construction procedure, i.e., on methodology of excavation and installing of a tunnel support system [25].

Effect of an earthquake on tunnel-ground interaction depends on various parameters including peak acceleration, intensity and duration of the earthquake, and the relative rigidity between the tunnel and the ground. In case of rigid lining in soft ground, no deformation can be produced by the ground, but for a flexible lining, on the other hand, interaction between lining and ground exists.

A number of approaches are available to account for dynamic soil-structure interaction, but they are usually based on the assumption that the soil behaviour is governed by the law of linear elasticity or visco-elasticity, and the soil is perfectly bonded to the structure. In practice, however, the bonding between the soil and the structure is rarely perfect, and slippage or even separation often occur in the contact area. Furthermore, the soil region immediately adjacent to the tunnel structure can undergo a large degree of straining, which cause the soil-tunnel system to behave in a nonlinear manner. This solution is based on a *full-slip* assumption between lining and ground. This assumption is valid only for the case of very soft soil or earthquake of high intensity. In many situations, a condition of *partial slip* (due to large ground deformations, the soil-structure interaction decreases as the relative displacements between the soil and structure increase), or *no-slip* exists. So, it is rational to compute both extreme cases and apply the one that is more critical. If no continuity of displacement on the contact of the structure-environment is assumed, and if there is locally concentrated mass in the structure, the effect of inertia must be taken into account.

Concerning the types of analyses they may be grouped into three categories: **pseudo-static, simplified dynamic, and full (detailed) dynamic analysis**, regarding increasing levels of complexity in analytical models, soil characterization, and description of seismic input. All of the above mentioned analyses are reviewed by Billota et al. [2].

- In *pseudo-static methods*, the ground-tunnel analysis is uncoupled. The seismic input is reduced to the peak strain amplitude, computed by simplified formulas based on simple assumptions of harmonic plane S-wave propagation in a homogeneous, isotropic, elastic medium, and then considered acting on the tunnel lining in static conditions. By that, the effects of a tunnel shape and stiffness on the seismic ground behaviour are ignored;

- In a *simplified dynamic analysis*, the soil straining in the range of depths corresponding to the tunnel section, between the tunnel crown and the invert, is computed through a free-field one-dimensional SSR analysis, and then applied to the tunnel lining, again in pseudo-static conditions. In such a way, both the acceleration time history and the site characteristics are taken into account, whereas the kinematic soil-structure interaction is still neglected. Moreover, the effects of compressional waves are also neglected, as only the shear waves are considered, which propagate in vertical

opterećenje. Time su efekti oblika tunelskog profila i krutosti tunelske konstrukcije na ponašanje tla u seizmičkim uslovima u potpunosti zanemareni;

- U *uprosćenoj dinamičkoj analizi*, deformacije tla na dubini tunelskog objekta, između temena tunelskog i podnožnog svoda, sračunate su jednodimenzionalnom analizom odgovora tla na seizmičke uticaje (numeričkim postupkom), i potom nanete na tunelsku konstrukciju u statičkim uslovima. Na taj način, i karakteristike zemljotresa i svojstva lokalnog tla uzeti su u obzir, dok je interakcija konstrukcije i tla kao i u slučaju pseudo-statičke analize zanemarena. Takođe su ignorisani i efekti longitudinalnih seizmičkih P talasa, s obzirom na to što se jednodimenzionalne analize seizmičkog odgovora tla zasnivaju na pretpostavci vertikalnog prostiranja samo smičućih seizmičkih S talasa;

- U *dinamičkoj analizi*, priraštaj unutrašnjih sila u tunelskoj konstrukciji usled dejstva zemljotresa jeste direktna izlazna veličina kompleksnih numeričkih analiza (primenom metode konačnih elemenata ili metode konačnih razlika), u kojima se tunelska konstrukcija i okolno tlo tretiraju kao jedinstveni sistem tunel–tlo. Time su parametri zemljotresa i tla s jedne strane, i kinematička i dinamička interakcija s druge strane, u potpunosti obuhvaćeni analizama.

U prvom delu ovog rada, dat je prikaz analitičkih postupaka određivanja seizmički indukovanih uticaja u tunelskoj konstrukciji pri interakciji konstrukcije i okolne sredine, prema Wang-u, 1993 [20] i Penzien-u, 2000 [14]. Ova rešenja bazirana su na teoriji elastične grede na elastičnoj podlozi i uzimaju u obzir efekte interakcije konstrukcije s tlom u kvazi-statičkom smislu, ignorišući na taj način inercijalnu interakciju. Prikazani analitički postupci odnose se na tunelske konstrukcije kružnog poprečnog preseka, kod kojih smicanje (ovalizacija) kružnog tunelskog profila predstavlja najkritičniji vid deformacije. Ovaj vid deformacije izazvan je propagacijom smičućih seizmičkih S-talasa koji se prostiru u ravnima upravnim na pravac podužne tunelske ose. S ciljem računskog određivanja seizmički indukovane ovalizacije kružnog tunelskog profila, analitički postupci su bazirani na pretpostavci ravnog stanja deformacija iz sledećih razloga [14]:

- dimenzije poprečnog preseka tunelskih konstrukcija su zanemarljivo male u poređenju s talasnim dužinama seizmičkih talasa koji uzrokuju ovalnu deformaciju kružnog tunelskog preseka;

- inercijalni efekti u tunelskoj konstrukciji i okolnoj sredini u uslovima dinamičke interakcije konstrukcije sa okolnom sredinom, relativno su mali.

Postupci prikazanog analitičkog tretiranja interakcije takođe podrazumevaju da se tunelska konstrukcija nalazi na dovoljnoj dubini ispod površine terena i na dovoljnom rastojanju od stenske podine, kako bi efekat ovih granica na interakciju konstrukcije sa okolnom sredinom bio minimiziran. Prema Wang-ovim preporukama [20], uticaj slobodne površine terena i stenske podloge može se smatrati zanemarljivo malim ukoliko je odnos h/d najmanje 1.5, gde h predstavlja rastojanje od površine terena (odnosno stenske podloge) do centra kružne tunelske konstrukcije, a d je prečnik kružnog tunelskog iskopa. Takođe treba napomenuti da su prezentovani analitički postupci namenjeni analizi konvencionalnih betonskih tunelskih konstrukcija, kao i montažnih obloga koje se sukcesivno

planes inducing shear strain;

- In a *full dynamic analysis*, the force increments in the lining due to an earthquake are directly obtained as an output of the numerical modelling (such as dynamic finite element or finite difference methods) adopted for the simulation of the shaking of the coupled ground–tunnel system. By that, besides the acceleration time history and the site characteristics, both the kinematic and dynamic interactions are also taken into consideration.

The first part of this paper deals with a presentation of analytical solutions for seismically induced tunnel lining forces accounting for soil–structure interaction effects, proposed by Wang in 1993 [20] and Penzien in 2000 [14]. The solutions are based on the theory of an elastic beam on an elastic foundation, which takes into account the soil–structure interaction (SSI) effects in a quasi-static manner, ignoring any inertial interaction effect. In addition, they are proposed for the response of circular tunnels to an earthquake action where the most critical deformation pattern is the shearing (ovalization) of the tunnel cross section. The ovaling deformation of a circular tunnel is caused by shear S-waves propagating in planes perpendicular to the tunnel axis, and is commonly modelled as a two-dimensional plane-strain condition. This is done for two reasons [14]:

- the dimensions of a typical lining cross-section are small in comparison with the wavelengths of the dominant ground motion producing the ovaling;

- the inertia effects in both the lining and the surrounding ground as produced by dynamic soil–structure interaction effects are relatively small.

The analysis procedures presented herein are applicable to tunnel linings with burial depths sufficiently large, so that the free-surface boundary condition at the top of the soil, as well as the bedrock at the bottom of the soil have small effect on the soil–structure interaction. As shown by Wang [20], these boundary effects are negligible for circular tunnel linings when the ratio of h (a distance from the ground surface, i.e. the bedrock, to mid-height of the lining) to the outside diameter d of the lining, h/d , is greater than 1.5. In addition, the solutions are applicable for cast-in-place concrete tunnel linings, or linings of a TBM driven (shield) tunnel that are assembled of precast concrete segments, and which are embedded in soil sites whose values of elasticity modulus are lower in comparison with a modulus of a tunnel lining, i.e., for the case of tunnel lining in soft soils, when the soil–structure interaction effects are pronounced.

Even though the above mentioned solutions were suggested around 15 years ago, they still present the most frequently used analytical solutions nowadays. The main reason lies in the fact that the problem concerning the soil – tunnel structure interaction under an earthquake action is not fully researched and well known so far, and to the best of the authors' knowledge, during the past decade or two no evident progress has been accomplished with regard to simplified and applicable analytical solutions. Moreover, literature dealing with the impact of the earthquake on underground structures is quite rare, and therefore, a presentation of analytical solutions that are based on the theory of elasticity is fruitful, in order to perceive the basic assumptions and limitations that are essential in these solutions. Lastly,

izvode s napredovanjem iskopne mašine „krtice“, a za tunele koji se nalaze u sredini koja ima znatno manji modul deformabilnosti od modula deformabilnosti tunelske obloge, to jest u slučaju tunelskih konstrukcija u tlu slabijih karakteristika, kada su efekti interakcije konstrukcije sa okolnim tlom najizraženiji.

Iako pomenuti analitički postupci datiraju od pre petnaest i više godina, oni i danas predstavljaju najčešće primenjivane metode analize seizmički indukovanih uticaja u tunelskoj konstrukciji pri interakciji konstrukcije sa okolnom sredinom. Jedan od razloga je svakako činjenica da je problem interakcije tunelske konstrukcije sa okolnim tlom u seizmičkim uslovima još uvek nedovoljno istražen, i prema saznanjima autora, u pogledu pojednostavljenih i primenljivih analitičkih postupaka ovog problema proteklih deceniju-dve nije načinjen značajniji pomak. Takođe, literatura koja se bavi problemima uticaja zemljotresa na podzemne objekte dosta je retka, pa je prikaz analitičkih postupaka zasnovanih na teoriji elastičnosti koristan radi sagledavanja osnovnih pretpostavki i ograničenja koja su u osnovi tih postupaka. Na kraju, treba istaći da su savremene sveobuhvatne numeričke analize veoma kompleksne i dugotrajne, te su stoga ograničene na istraživanja konkretnog slučaja, dok s druge strane, pomenuti analitički postupci imaju opštu primenu i daju zadovoljavajuće rezultate sa inženjerske tačke gledišta.

U cilju istraživanja efekata interakcije tunelske konstrukcije i okolnog tla, u drugom delu rada sprovedene su analize koje su se bazirale na primeni analitičkih izraza za proračun seizmički indukovanih presečnih sila u tunelskoj konstrukciji iz aspekta sadejstva tunela i tla, predloženih od strane gore navedenih autora. Analizirana su dva karakteristična slučaja tla – čvrsto tlo dobrih karakteristika i meko zasićeno tlo slabih karakteristika, kao i oba ekstremna slučaja kontaktnih uslova tunelske konstrukcije sa okolnim tlom – slučaj smičućeg kontakta i slučaj idealnog kontakta tunela i tla. Komparativnom analizom rezultata za sve navedene slučajeve sagledani su najznačajniji faktori sadejstva tunela i tla u seizmičkim uslovima.

2 PSEUDO-STATIČKA ANALIZA INTERAKCIJE KONSTRUKCIJE I TLA (ANALITIČKI IZRAZI ZA TUNELE KRUŽNOG POPREČNOG PRESEKA)

S ciljem nalaženja što realnijeg odgovora tunelske konstrukcije na dejstva zemljotresa, neophodno je sprovesti analizu interakcije konstrukcije i okolnog tla kojom će biti obuhvaćene krutosti tunelske konstrukcije i tla.

U istraživanjima smičućih deformacija, na osnovu prethodnih studija Burns-a i Richard-a [4] i Hoeg-a [8], Peck i drugi [12] predložili su analitičke izraze za proračun normalnih sila, transverzalnih sila i momenata savijanja u tunelskoj konstrukciji usled dejstva zemljotresa. Odgovor tunelske konstrukcije je u funkciji krutosti konstrukcije na pritisak i savijanje, vertikalnog pritiska nadsloja tla (γh) i odnosa primarnih napona u tlu (K_0). Da bi se seizmičko opterećenje indukovano smičućim S-talasima simuliralo na adekvatan način, u uslovima čistog smicanja tla, vertikalni pritisak nadsloja tla je zamenjen smičućim naponima u tlu, dok je odnos primarnih napona pomnožen sa (-1). S druge strane, smičući naponi u tlu su u funkciji smičućih deformacija tla.

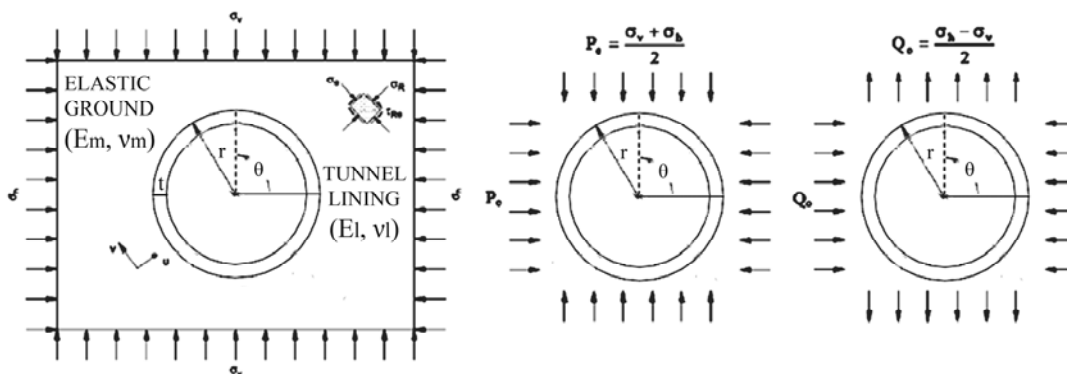
contemporary advanced numerical analyses are quite complex and time consuming, and thus, they are aimed at case-specific studies, whereas the aforementioned simplified approaches give reasonable results from an engineering point of view.

With an aim to study the effects of tunnel-ground interaction, a number of analyses were carried out in the second part of the paper, based on the most frequently used analytical expressions for evaluation of seismically induced stress increment in a tunnel lining accounting for the soil-structure interaction effects, proposed by the aforementioned authors. Various levels of analysis have been undertaken on different soil conditions, considering representative of two main soil classes – stiff soil of good conditions and soft saturated soil of poor conditions, as well as two extreme cases of tunnel-ground interface – the full-slip and the no-slip conditions. Finally, the results for all the considered cases have been evaluated and compared, and the significant mutual differences regarding tunnel-ground interaction have been emphasized.

2 SOIL-STRUCTURE INTERACTION PSEUDO-STATIC ANALYSIS (CLOSED-FORM ELASTIC SOLUTIONS FOR CIRCULAR TUNNELS)

Analysis of tunnel-ground interaction, that considers both the tunnel stiffness and the ground stiffness, is necessary in finding the true tunnel response.

In early studies of racking deformations, Peck et al. [12], based on earlier work by Burns and Richard [4] and Hoeg [8], proposed closed-form solutions in terms of thrusts, bending moments, and displacements under external loading conditions. The response of a tunnel lining is a function of the compressibility and flexibility ratios of the structure, and the in-situ overburden pressure (γh) and at-rest coefficient of earth pressure (K_0) of the soil. To adapt to seismic loadings caused by shear waves, the free-field shear stress replaces the in-situ overburden pressure and the at-rest coefficient of earth pressure is assigned a value of (-1) to simulate the field simple shear condition. The shear stress can be further expressed as a function of shear strain.



Slika 1. Geometrija problema, hidrostaticka i devijatorska komponenta opterećenja [5]
Figure 1. Problem geometry, hydrostatic and deviatoric components of the solution [5]

Na slici 1 ilustrovana je postavka problema, gde σ_v i σ_h predstavljaju primarni vertikalni i horizontalni napon u tlu, $\sigma_h = K_o \sigma_v$, gde je K_o koeficijent bočnih pritisaka tla. Primarni naponi u tlu su razdvojeni na hidrostaticku komponentu $P_o = (\sigma_v + \sigma_h)/2$ i devijatorsku komponentu $Q_o = (\sigma_h - \sigma_v)/2$. Moment i normalne sile su u funkciji centralnog ugla θ merenog od horizontalne tunelske ose u smeru suprotnom od smera kretanja kazaljke na satu.

Tokom zemljotresnog dejstva dolazi do **smicanja (distorzije, odnosno, ovalizacije)** kružnog tunelskog profila usled smičućih napona u okolnom tlu (posmatranih u ravni tunelskog preseka), koji su indukovani vertikalnim prostiranjem horizontalnih smičućih seizmičkih talasa.

U pseudo-statičkoj analizi opterećenje zemljotresom simulira se nanošenjem smičućih napona ili smičućih deformacija tla na granice modela u statičkim uslovima. Ovim postupkom vrši se aproksimacija (pojednostavljenje) realnog problema, koja je opravdana samo u slučaju kada je talasna dužina smičućih talasa znatno veća od prečnika tunelskog iskopa.

U pseudo-statičkoj analizi tunela kružnog poprečnog preseka, prosečna vrednost smičuće deformacije tla u ravni, γ_{ave} , koja predstavlja prosečnu vrednost seizmički indukovanih deformacija tla na delu od nivoa svoda do nivoa kinete tunelske konstrukcije, može se odrediti na osnovu numeričkih analiza odgovora tla na seizmičke uticaje usled vertikalnog prostiranja smičućih talasa, primenom odgovarajućih softvera. Na osnovu poznate veličine smičuće deformacije, vrednost napona smicanja tla u posmatranoj ravni može se sračunati prema izrazu:

$$\tau_{ff} = \gamma_{ave} \cdot G_m = \gamma_{ave} \frac{E_m}{2(1 + \nu_m)} \quad (1)$$

gde je γ_{ave} smičuća deformacija sračunata u seizmičkoj analizi odgovora tla, ν_m je Poisson-ov koeficijent tla, E_m predstavlja modul deformabilnosti tla, dok je G_m modul smicanja tla, koji se određuje na osnovu jednačine koja daje zavisnost brzine prostiranja transverzalnih S-talasa C_s i modula klizanja tla G_m (ρ_m predstavlja gustinu tla):

$$G_m = \rho_m C_m^2 \quad (2)$$

$$E_m = 2G_m (1 + \nu_m) \quad (3)$$

Figure 1 shows the problem geometry, where σ_v and σ_h denote the initial vertical and horizontal stresses in the ground, respectively, $\sigma_h = K_o \sigma_v$, and K_o is the coefficient of lateral earth pressure. For the solution, the initial in situ stress field is separated into a hydrostatic component, $P_o = (\sigma_v + \sigma_h)/2$ and a deviatoric component, $Q_o = (\sigma_h - \sigma_v)/2$. The moment and thrust are determined in terms of the angle θ measured counterclockwise with respect to the spring line axis of the tunnel.

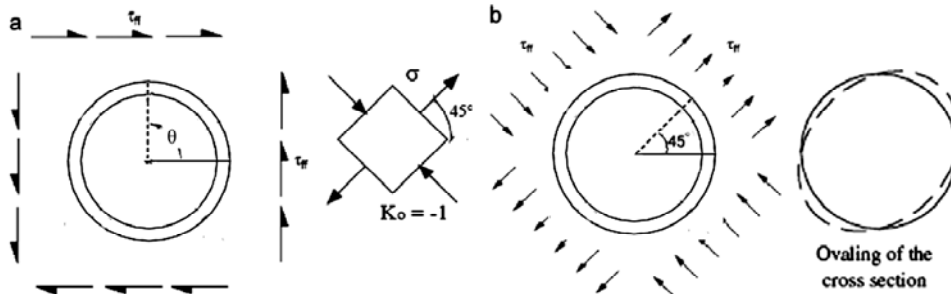
During an earthquake, **shearing (racking, i.e., ovaling) deformation** in a circular tunnel can be developed due to the in-plane shear stresses caused by vertically propagating horizontal shear waves.

In a pseudo-static approach, the earthquake loading is simulated by applying a static far-field shear stress or shear strain at the model boundaries to represent the seismic effects. This approach is approximate and can be applied only if the shear wavelength is significantly larger than the tunnel diameter.

For pseudo-static analysis of circular tunnels, the average free-field shear strain of the soil in the transverse plane, γ_{ave} , that represent the average soil shear strain in the range of depths between the tunnel crown and the invert, is obtained from ground-response analysis using the vertically propagating shear wave model, with the aid of corresponding softwares. The resultant in-plane shear stress is thus:

Dakle, na osnovu Jednačina (2) i (3) može se zaključiti da je reč o dinamičkim vrednostima modula deformabilnosti, odnosno, modula smicanja tla.

Pod pretpostavkom da je vrednost napona smicanja u tlu konstantna na dubini tunela, tako sračunati napon se unosi u model kao opterećenje u uslovima čistog smicanja, što je ilustrovano na slici 2(a), a što je ekvivalentno stanju napona prikazanom na slici 2(b).



Slika 2. (a) Smičući naponi u tlu, indukovani dejstvom zemljotresa i (b) ekvivalentni glavni naponi [5]
Figure 2. (a) Earthquake-induced shear stresses and (b) equivalent principle stresses [5]

Stanje seizmički indukovanih smičućih napona u tlu, prikazano na slici 2(b), identično je devijatorskoj komponenti napona (slika 1) koji su zarotirani za $\pi/4$. Samim tim, momenti i normalne sile u tunelskoj konstrukciji u uslovima zemljotresnih dejstava mogu se odrediti zamenom vrednosti Q_0 vrednošću τ_{ff} i transformacijom (rotacijom) koordinatnog sistema zamenom ugla θ uglom $(\theta+3\pi/4-\lambda)$, gde λ predstavlja ugao u odnosu na horizontalu u vertikalnoj ravni prostiranja smičućih seizmičkih talasa, kao što je prikazano na slici 3. Ovo je slučaj karakterističan za duboko položene tunelske konstrukcije, odnosno za manje udaljenosti od žarišta zemljotresa. U slučaju plitko položenih tunela [22], odnosno većih udaljenosti od žarišta, sa druge strane, ugao λ iznosi $\pi/2$ tj. 90° , imajući u vidu da tokom prostiranja talasa kroz horizontalne slojeve tla do površine terena dolazi do njihove višestruke refrakcije, što u krajnjoj liniji rezultuje gotovo vertikalnim pravcem propagacije talasa ka slobodnoj površini tla. Obe prethodno pomenute varijante važe u slučaju zemljotresa s većim dubinama žarišta (*far-field earthquakes*).

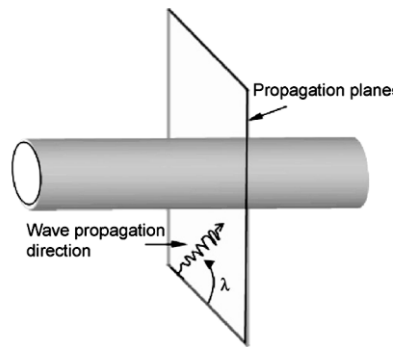
Međutim, pretpostavka o vertikalnom prostiranju smičućih seizmičkih talasa u slučaju zemljotresa s plićom lokacijom žarišta (*near-field earthquakes*) predstavljala bi samo aproksimaciju realne situacije, s obzirom na to što bi zbog male dubine lokacije žarišta prostiranje seizmičkih talasa bilo ostvareno pod nekim uglom u odnosu na vertikalni pravac, tj. upadni ugao talasa bi bio manji od 90° . Razlog tome je što seizmički talasi moraju preći izvesnu distancu od žarišta do površine terena pre nego što u potpunosti budu formirani, što predstavlja efekte blizine površine terena (*near-field effects*). Analogno tome, s druge strane, postoje efekti blizine žarišta (*far-field effects*), za koje je karakteristična mala jačina signala u poređenju s jačinom pozadinskog šuma.

According to Equations (2) and (3), it is evident that E_m and G_m represent dynamic values of the corresponding soil moduli.

Assuming that the in-plane shear stress is constant at the depth of the tunnel, the free-field shear stress is then typically applied as a far-field stress in analytical solutions. Thus, the corresponding state of stress is the pure shear type shown in Figure 2(a), which is equivalent to the state of stress shown in Figure 2(b).

The earthquake-induced far-field shear stress shown in Figure 2(b) is identical to that of the deviatoric component of the liner solution but rotated by $\pi/4$. Consequently, the moment and thrust due to the earthquake component can be derived by substituting the value of τ_{ff} for Q_0 , and transforming (rotating) the coordinate system by replacing θ with $(\theta+3\pi/4-\lambda)$, where λ is the angle relative to the horizontal plane and the shear waves travel in the vertical plane as shown in Figure 3. This is the case typical for deeply embedded tunnel structures, i.e., shorter distances from the source. For the case of shallow-laid tunnel structures [22], i.e., larger distances from the source, on the other hand, λ is normally taken $\pi/2$ or 90° , since by the time the waves reach the ground surface, multiple refractions have often bent them to a nearly vertical direction.

Both of the aforementioned cases are valid for *earthquake events with deeper location of the source (far-field earthquakes)*. The consideration of vertically propagating shear waves constitutes a simplification of the actual phenomenon, especially in the case of *earthquake events with shallower location of the source (near-field seismic events)* involving complex wave fields when a significant portion of the seismic waves arrived at some angle with the vertical direction, i.e. the incident angle of the earthquake waves was not 90° . This is due to the fact that seismic waves must travel a minimum distance from the source before being fully formed, which is termed as *near-field effects*. Conversely, low signal-to-noise ratios can be present at large distances from the source relative to the wavelength (*far-field effects*).



Slika 3. Upadni ugao seizmičkih talasa u slučaju duboko položenih tunelskih konstrukcija [5]
Figure 3. The incidence angle of seismic waves in the case of deeply embedded tunnel structures [5]

Da bi se uočio značaj krutosti tunelske konstrukcije, uvedeni su bezdimenzioni parametri, koji predstavljaju odnos krutosti tunelske konstrukcije i tla, tzv. koeficijenti kompresibilnosti i fleksibilnosti (C i F). Ovi koeficijenti su mera otpornosti tunelske konstrukcije na distorziju poprečnog preseka, u vidu odnosa krutosti na pritisak (odnosno, smicanje) tla i odgovarajuće krutosti tunelske konstrukcije.

Koeficijenti kompresibilnosti i fleksibilnosti, dakle, jesu mera relativne krutosti tla u poređenju s krutošću tunelske konstrukcije u uslovima *simetričnog*, odnosno, *asimetričnog opterećenja*. **Koeficijent kompresibilnosti C** jeste mera relativne krutosti sistema tlo–tunel u uslovima uniformnog, tj. simetričnog opterećenja (horizontalni naponi u tlu jednaki su vertikalnim naponima u tlu). Koeficijent kompresibilnosti predstavlja krutost sistema tunel–tlo u tangencijalnom pravcu (otpornost na pritisak). S druge strane, **koeficijent fleksibilnosti F** je mera relativne krutosti sistema tlo–tunel u uslovima asimetričnog opterećenja (horizontalni naponi u tlu jednaki su vertikalnim naponima po intenzitetu, ali su suprotnog znaka). Koeficijent fleksibilnosti reflektuje krutost sistema tunel–tlo u radialnom pravcu (otpornost na smicanje).

Rezultati brojnih studija sugerišu da je od ova dva pokazatelja, koeficijent fleksibilnosti od znatno većeg značaja, jer je upravo ovim parametrom okarakterisana sposobnost tunelske konstrukcije da se odupre distorziji nametnutoj od strane okolnog tla.

U literaturi ne postoje jedinstveni izrazi kojima se definišu ovi parametri. Brojnim istraživanjima izvedeni su različiti izrazi za koeficijente kompresibilnosti i fleksibilnosti [3, 10, 17]. Prema istraživanju Merritt-a i drugih [10], izrazi su oblika:

$$C = \frac{E_m(1-\nu_l^2)r}{E_l t(1+\nu_m)(1-2\nu_m)} \quad (4)$$

$$F = \frac{E_m(1-\nu_l^2)r^3}{6E_l I(1+\nu_m)} \quad (5)$$

gde E_m predstavlja modul elastičnosti tla, I je moment inercije kružnog tunelskog preseka (po jedinici širine), a r i t su radijus, odnosno, debljina tunelske konstrukcije.

Veličina ovih koeficijenata za neke uobičajene vrednosti modula i Poisson-ovog koeficijenta tla, modula deformabilnosti tunelske konstrukcije i dimenzija

To understand importance of the lining stiffness, the stiffness of a tunnel relative to the surrounding ground is quantified by dimensionless parameters, so called the compressibility and flexibility ratios (C and F), which are measures of the compressional stiffness and the shearing stiffness, respectively, of the medium relative to the lining.

The compressibility and flexibility ratios are a measure of the relative stiffness of the ground with respect to the supporting system (i.e., the lining) under a *symmetric* and *antisymmetric loading* respectively. The **Compressibility Ratio C** is a measure of the relative stiffness of the tunnel–ground system under a uniform or symmetric loading condition (horizontal ground stress = vertical ground stress) in the free field, i.e., it reflects the circumferential stiffness of the system (resistance to compression). The **Flexibility Ratio F** is a measure of the relative stiffness of the tunnel–ground system under an anti-symmetric loading condition (horizontal ground stress equal to, but of the opposite sign of the vertical ground stress in the free-field), i.e., it reflects the radial stiffness of the system (resistance to shearing).

Regarding these two ratios, it is often suggested that the flexibility ratio is more important because it is related to the ability of the lining to resist distortion imposed by the ground.

In the technical literature, there is no unique definition of these parameters. The definitions were proposed by different authors [3, 10, 17]. According to the research of Merritt et al. [10], the relations are:

where E_m is modulus of elasticity of the medium, I is moment of inertia of the tunnel lining (per unit width) for circular lining, and r and t are radius and thickness of the tunnel lining, respectively.

Values of these coefficients for various parameters of lining and ground, in order to cover a wide range of

tunelskih obloga analizirane su u Wang-ovom radu [20].

Budući da realni granični uslovi na kontaktu tunela i tla nisu poznati, radi sagledavanja svih mogućih slučajeva, preporučuje se da rešenja obuhvate ekstremne kontaktne uslove: slučaj potpunog smicanja tunela i tla u kontaktnoj zoni, i slučaj idealnog kontakta tunela s tlom.

Glatki kontakt ili kontakt s potpunim smicanjem (full-slip) podrazumeva jednake vrednosti radijalnih pomeranja i nejednake vrednosti tangencijalnih pomeranja tunelske konstrukcije i tla u zoni kontakta (nema pojave smičućeg napona i transverzalnih sila, odnosno, nema kontinuiteta pomeranja u kontaktnoj zoni). Pretpostavka ovakvog graničnog uslova rezultuje maksimalnim vrednostima momenata i transverzalnih sila u tunelskoj konstrukciji, i opravdana je u slučaju mekih rastresitih tla slabih karakteristika ili u slučaju zemljotresa jakog intenziteta. Međutim, u mnogim situacijama izvesnija je pojava delimičnog smicanja tunela i tla u kontaktnoj zoni.

Idealni kontakt, kruti kontakt ili kontakt bez smicanja (no-slip) podrazumeva jednake veličine pomeranja tunelske konstrukcije i okolnog tla u zoni kontakta (u graničnoj zoni nema relativnih transverzalnih pomeranja između konstrukcije i tla, i postoji kontinualnost napona i pomeranja). Pretpostavka ovakvog kontaktnog uslova rezultuje maksimalnim vrednostima normalnih sila u tunelskoj konstrukciji.

2.1 Analitički izrazi prema Wang-u [20]

Pod pretpostavkom **smicanja na kontaktu tunela i okolnog tla**, bez pojave odvajanja tunelske konstrukcije od tla, kao i bez pojave tangencijalnih sila, radijalna deformacija, normalna sila i moment savijanja mogu biti izraženi u sledećem obliku:

$$\frac{\Delta d}{d} = \pm \frac{1}{3} K_1 F \cdot \gamma_{ave} \quad (6)$$

$$T(\theta) = \pm \frac{1}{6} K_1 G_m d \cdot \gamma_{ave} \cos \left[2 \left(\theta + \frac{\pi}{4} \right) \right] \Rightarrow T_{\max} = \pm \frac{1}{6} K_1 \frac{E_m}{1 + \nu_m} r \cdot \gamma_{ave} \quad (7)$$

$$M(\theta) = \pm \frac{1}{12} K_1 G_m d^2 \cdot \gamma_{ave} \cos \left[2 \left(\theta + \frac{\pi}{4} \right) \right] \Rightarrow M_{\max} = \pm \frac{1}{6} K_1 \frac{E_m}{1 + \nu_m} r^2 \cdot \gamma_{ave} \quad (8)$$

$$K_1 = \frac{12(1 - \nu_m)}{2F + 5 - 6\nu_m} \quad (9)$$

gde K_1 predstavlja koeficijent seizmičkog odgovora tunelske konstrukcije za slučaj kontakta s potpunim smicanjem, $d=2r$ je prečnik tunelskog iskopa, r je radijus tunelskog iskopa, a γ_{ave} je prosečna vrednost smičućih deformacija koje se ostvaruju u okolnoj sredini na delu tunelske konstrukcije između temena svoda i inverta.

Presečne sile u tunelskoj konstrukciji prikazane su na slici 4.

possible effects of lining-ground interaction, have been analyzed in the study of Wang [20].

Since the real boundary conditions at the soil-structure interface are unknown, simplified solutions available in literature for evaluation of seismic actions generally consider two extreme cases which bound the real situation: the full-slip and the no-slip condition.

Full-slip condition (smooth contact or sliding contact) considers equal normal displacements and unequal tangential displacements of medium and lining at common interface (i.e., no shear stress transmission, no tangential shear force and no continuity of displacements exist). It is adopted to obtain the extreme values of bending moment and shear in tunnel lining. This assumption is valid only for the case of very soft soil or earthquake with high intensity. In many situations, however, there is a partial or no-slip condition.

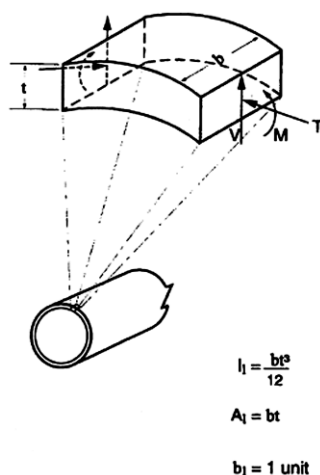
No-slip condition (perfect contact, rigid contact or rough interface) considers equal displacements of medium and lining at the common interface (i.e., no relative shear displacements, continuity of stresses and displacements exist). It is used to find the maximum values of the thrust acting on the lining.

2.1 Solution after Wang [20]

Assuming **full-slip conditions**, without normal separation and therefore, no tangential shear force, the diametric strain, the maximum thrust, and bending moment can be expressed as:

where: K_1 is the full-slip lining response coefficient, $d=2r$ is tunnel diameter, r is tunnel radius, and γ_{ave} is the average value of the free-field shear strain in the range of depths between the tunnel crown and the invert.

These forces and moments are illustrated in Figure 4.



Slika 4. Unutrašnje sile u poprečnom preseku tunelske konstrukcije indukovane prostiranjem seizmičkih talasa u pravcu upravnom na podužnu osu tunela [15]

Figure 4. Induced circumferential forces and moments caused by waves propagating perpendicular to tunnel axis [15]

Pojava smicanja u kontaktnoj zoni tunela i okolnog tla svojstvena je tunelskim konstrukcijama u mekom rastresitom tlu slabih karakteristika, a može se javiti i pod dejstvom zemljotresa veoma snažnog intenziteta. Međutim, u najvećem broju slučajeva, kontaktni uslovi tunela i okolnog tla su negde između kontakta s potpunim smicanjem i idealnog kontakta. Stoga se preporučuje da u seizmičkim analizama tunelskih konstrukcija budu razmotrena oba granična slučaja kontakta radi što tačnijeg određivanja kritičnih presečnih sila i deformacija tunelske konstrukcije.

S obzirom na to što pretpostavka kontaktnog uslova s potpunim smicanjem rezultuje potcenjenim vrednostima maksimalne normalne sile u tunelskoj konstrukciji, preporuka je da se njena vrednost odredi iz uslova **idealnog kontakta tunela i okolnog tla** [8, 17]:

$$T(\theta) = \pm \frac{1}{2} K_2 G_m d \cdot \gamma_{ave} \cos \left[2 \left(\theta + \frac{\pi}{4} \right) \right] \Rightarrow T_{max} = \pm K_2 \tau_{ave} r = \pm K_2 \frac{E_m}{2(1+\nu_m)} r \cdot \gamma_{ave} \quad (10)$$

$$K_2 = 1 + \frac{F[(1-2\nu_m) - (1-2\nu_m)C] - \frac{1}{2}(1-2\nu_m)^2 + 2}{F[(3-2\nu_m) + (1-2\nu_m)C] + C \left(\frac{5}{2} - 8\nu_m + 6\nu_m^2 \right) + 6 - 8\nu_m} \quad (11)$$

gde je K_2 koeficijent seizmički indukovane normalne sile za slučaj idealnog kontakta tunela i okolnog tla.

Kao što se može videti na osnovu prethodnih izraza, za uslov idealnog kontakta tunelske konstrukcije i tla nisu izvedene relacije za proračun radijalne deformacije i maksimalnog momenta savijanja u tunelskoj konstrukciji, odnosno, preporučuje se da njihove vrednosti budu sračunate iz uslova smičućeg kontakta. Iako se na taj način dobijaju nešto veće vrednosti za M_{max} i Δd , ipak, dobijeni rezultati su na strani sigurnosti, jer se time postiže kompenzacija greške aproksimiranja dinamičkog opterećenja pseudo-statičkim modelom. S druge strane, zbog prirode kontakta s potpunim smicanjem, vrednost T_{max} je potcenjena. Upravo zbog tog razloga se preporučuje da maksimalna normalna sila u tunelskoj konstrukciji bude određena iz uslova idealnog kontakta

According to various studies, slip at the interface is only possible for tunnels in soft soils or cases of severe seismic loading intensity. For most tunnels, the interface condition is between full-slip and no-slip, so both cases should be investigated for critical lining forces and deformations.

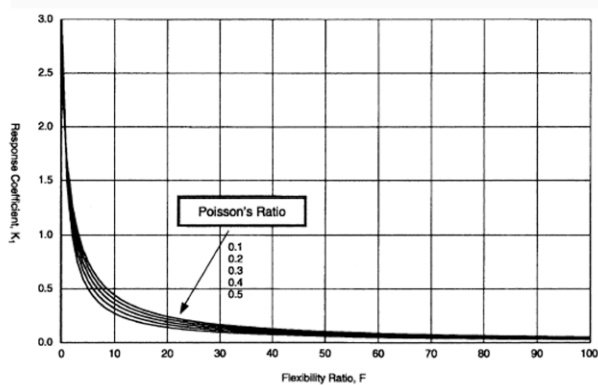
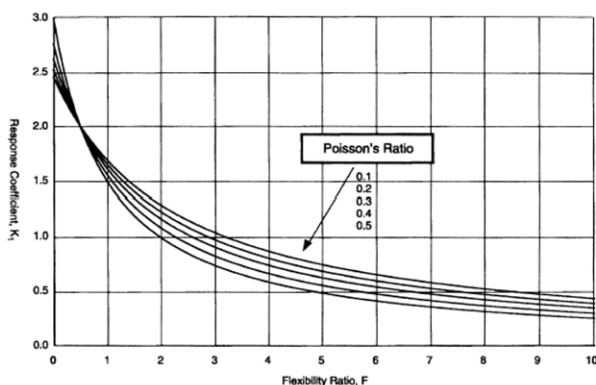
However, full-slip assumptions under simple shear may cause significant underestimation of the maximum thrust, so it has been recommended that the **no-slip assumption** of complete soil continuity be made in assessing the lining thrust response [8, 17]:

where K_2 stands for the thrust response coefficient.

It should be noted that no solution is developed for calculating diametric strain and maximum moment under no-slip condition. It is recommended that the solutions for full-slip condition are used for no-slip condition. Although assumption of full slip creates conservative results for M_{max} and Δd , it compensates for errors related to using pseudo-static model for dynamic earthquake loading. In a full-slip condition, because of the consideration of the slip, T_{max} is underestimated. It is recommended to use non-slip formula to find it. A finite difference reference solution shows that M_{max} and Δd_{lining} calculated considering the full-slip, and T_{max} considering the no slip, give the most accurate result. So, it is recommended to calculate Δd_{lining} , M_{max} and T_{max} using Equations (6), (8), and (10), respectively.

tunela sa okolnim tlom. Analize bazirane na primeni metode konačnih razlika ukazuju da vrednosti M_{max} i Δd_{lining} sračunate za uslove kontakta s potpunim smicanjem (Jednačine (6) i (8)), i vrednost T_{max} sračunata iz uslova idealnog kontakta tunela i tla (Jednačina (10)) predstavljaju najtačnije rešenje.

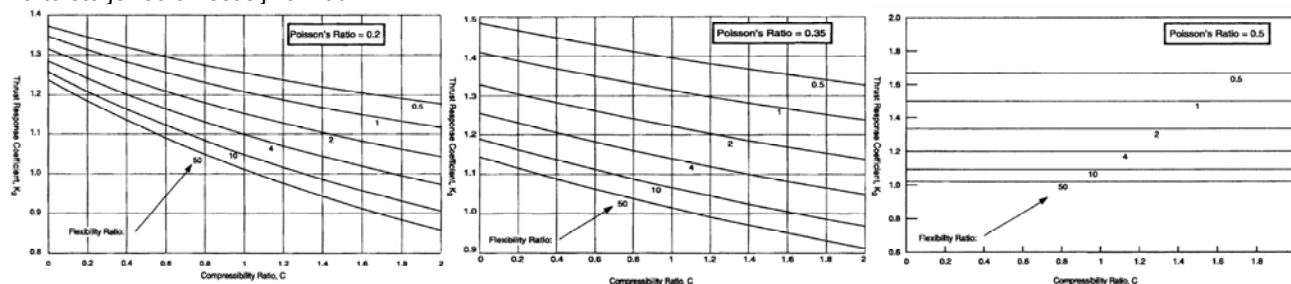
Zavisnost koeficijenta seizmičkog odgovora tunelske konstrukcije za slučaj kontakta s potpunim smicanjem (K_1) i koeficijenta fleksibilnosti (F) prikazana je na slici 5.



Slika 5. Zavisnost koeficijenta seizmičkog odgovora tunelske konstrukcije i koeficijenta fleksibilnosti za slučaj smičućeg kontakta tla i tunela kružnog poprečnog preseka [20]

Figure 5. Lining response coefficient vs. flexibility ratio, full-slip interface, and circular tunnel [20]

Sa slike 6 može se uočiti tendencija povećanja vrednosti seizmički indukovane normalne sile u tunelskoj konstrukciji sa smanjenjem koeficijenta kompresibilnosti i fleksibilnosti, za vrednosti Poisson-ovog koeficijenta tla manje od 0.5. Kada je vrednost Poisson-ovog koeficijenta tla 0.5 (dakle, slučaj zasićenog tla, poput nedrenirane gline), vrednost normalne sile postaje nezavisna od koeficijenta kompresibilnosti (C), s obzirom na to što je reč o nestišljivoj tlu.



Slika 6. Zavisnost koeficijenta seizmički indukovane normalne sile i koeficijenta kompresibilnosti za slučaj idealnog kontakta tla i tunela kružnog poprečnog preseka [20]

Figure 6. Thrust response coefficient vs. compressibility ratio, no-slip interface, and circular tunnel [20]

Odnos radijalne deformacije tunelske konstrukcije Δd_{lining} (Jednačina (6)) i kompatibilne radijalne deformacije prirodnog tla bez tunelskog iskopa $\Delta d_{free-field}$, usled seizmičkih uticaja, predstavlja normalizovanu deformaciju tunelske konstrukcije, koja ukazuje na značaj koeficijenta fleksibilnosti F u seizmičkom odgovoru tunelske konstrukcije, i može se izraziti u obliku:

The relationship between the full-slip lining response coefficient (K_1) and flexibility ratio (F) is shown in Figure 5.

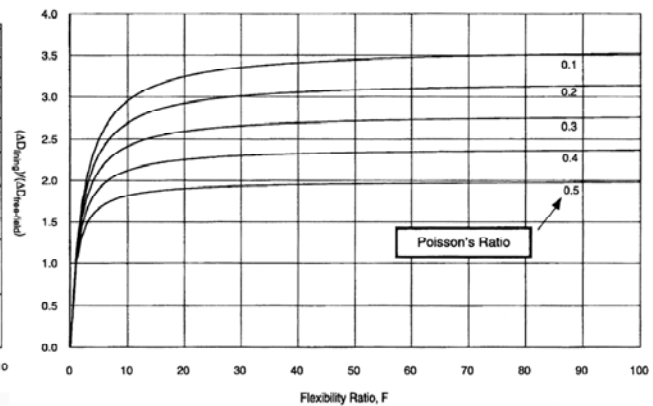
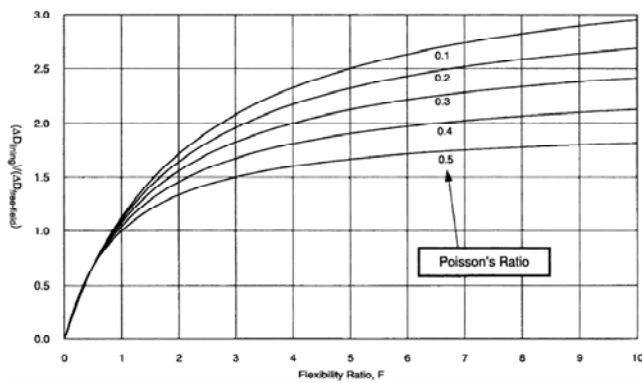
As Figure 6 shows, seismically-induced thrusts increase with decreasing compressibility and flexibility ratios when the Poisson's ratio of the surrounding ground is less than 0.5. As Poisson's ratio approaches 0.5 (i.e., saturated undrained clay), the thrust response is independent of compressibility because the soil is considered incompressible.

The normalized lining deflection, which represents the ratio of the diametric strain of a lining Δd_{lining} (Equation (6)) and the free-field soil-compatible diametric strain $\Delta d_{free-field}$, induced by an earthquake action, provides an indication of the importance of the flexibility ratio in lining response, and is defined as:

$$\frac{\Delta d_{\text{lining}}}{\Delta d_{\text{free-field}}} = \frac{\frac{1}{3} K_1 F \cdot \gamma_{\text{ave}} \cdot d}{\frac{\gamma_{\text{ave}}}{2} \cdot d} = \frac{2}{3} K_1 F \quad (12)$$

Prema Jednačini (12) i slici 7, deformacija tunelske konstrukcije biće manja od slobodne deformacije tla kada je vrednost koeficijenta fleksibilnosti manja od 1.0 (slučaj krute tunelske konstrukcije u tlu slabih karakteristika). S povećanjem vrednosti koeficijenta fleksibilnosti, deformacije tunelske konstrukcije postaju veće od deformacija prirodnog tla bez tunelskog otvora ($\frac{\Delta d_{\text{free-field}}}{d} = \pm \frac{\gamma_{\text{ave}}}{2}$), i mogu dostići maksimalne vrednosti koje su ekvivalentne deformacijama tla s tunelskim otvorom ($\frac{\Delta d_{\text{free-field}}}{d} = 2\gamma_{\text{ave}}(1-\nu_m)$). Tendencija rasta deformacija tunelske konstrukcije nastavlja se sve dok koeficijent fleksibilnosti ne dostigne beskonačno veliku vrednost (slučaj idealno fleksibilne tunelske konstrukcije).

According to this equation and Figure 7, a tunnel lining will deform less than the free-field when the flexibility ratio is less than 1.0 (the case of a stiff lining in a soft soil). As the flexibility ratio increases, the lining deflects more than the free-field ($\frac{\Delta d_{\text{free-field}}}{d} = \pm \frac{\gamma_{\text{ave}}}{2}$), and may reach an upper limit equal to the deformations of a ground with a tunnel cavity ($\frac{\Delta d_{\text{free-field}}}{d} = 2\gamma_{\text{ave}}(1-\nu_m)$). This condition continues as the flexibility ratio becomes infinitely large (the case of a perfectly flexible lining).



Slika 7. Zavisnost normalizovane deformacije tunelske konstrukcije i koeficijenta fleksibilnosti za slučaj smičućeg kontakta tla i tunela kružnog poprečnog preseka [20]
Figure 7. Normalized lining deflection vs. flexibility ratio, full slip interface, and circular lining [20]

Traba naglasiti da povećanje dimenzija tunelske konstrukcije ne predstavlja uvek pouzdanu meru ublažavanja efekata zemljotresnih dejstava, budući da s povećanjem debljine konstrukcije dolazi do povećanja njene krutosti (dakle, mala vrednost koeficijenta fleksibilnosti F), što u krajnjoj liniji rezultuje pojačanim efektom seizmičkih uticaja [11]. U slučaju tunelskih konstrukcija u slabo nosivim terenima, jedna od najefikasnijih mera ublažavanja seizmičkih uticaja bila bi primena tehnika stabilizacije i ojačanja tla [24].

Prema rezultatima nekih istraživanja [12], ukoliko je $F < 20$, interakcija tunelske konstrukcije i okolnog tla mora biti uzeta u obzir, dok se za $F > 20$, proračun može sprovesti po metodi slobodnih deformacija tla, s obzirom na to što su u tom slučaju efekti interakcije tunela i tla zanemarljivo mali.

Increasing structural dimensions of lining in static design can not always be a reliable method against earthquake loading, because this would increase the rigidity of lining (low F) and therefore, would increase the effect of earthquake loading [11]. For the case of tunnel structures in soft soil conditions, one of the most efficient measures for seismic impact mitigation would be application of the techniques of soil conditions improvement [24].

According to Peck et al. [12], if $F < 20$, interaction between lining and ground must be taken into account, otherwise free-field approach explained above can be used, i.e., ignoring overall the SSI is sound engineering approach.

2.2 Analitički izrazi prema Penzien-u [14]

Penzien i Wu [13] takođe su izveli analitičke izraze za normalne sile, transverzalne sile i momente savijanja u tunelskoj konstrukciji usled smičućih deformacija. Nešto kasnije, Penzien [14] je u dopuni prethodnih istraživanja predložio izraze za određivanje smičućih deformacija tunela pravougaonog i kružnog poprečnog preseka.

S ciljem određivanja seizmički indukovane smičuće deformacije tunelske konstrukcije, uveden je koeficijent smicanja tunelske konstrukcije i tla, koji predstavlja odnos smičuće deformacije tunelske konstrukcije i smičuće deformacije prirodnog tla bez tunelskog iskopa, i može se prikazati u obliku:

$$R = \frac{\Delta_{structure}}{\Delta_{free-field}} = \frac{\Delta_{il}}{\Delta_{il} + \Delta_{is}} \quad (13)$$

$$4(1 - \nu_m)$$

pri čemu je Δ_{il} smičuće pomeranje tunelske konstrukcije izazvano interakcijom sa okolnim tlom, dok $\Delta_{is} = (k_l/k_{so})\Delta_{il}$ predstavlja redukciju smičućeg pomeranja tla na mestu konture tunelskog iskopa koja je uzrokovana sadejstvom tla s konstrukcijom.

U slučaju kružnog tunelskog profila, R predstavlja odnos radijalne deformacije tunelske konstrukcije i radijalne deformacije prirodnog tla bez tunelskog iskopa. Generalizovana krutost tunelske konstrukcije k_l definiše se kao ona vrednost napona τ_{ff} prikazanog na slici 2(b) za uslove čistog smicanja, koja će izazvati jediničnu radijalnu deformaciju tunelske konstrukcije duž glavnih pravaca ($\theta = \pm \pi/4$). S druge strane, generalizovana spoljašnja krutost tla k_{so} definiše se kao ona vrednost napona τ_{ff} koji je nanet duž konture tunelske šupljine u suprotnom smeru od smera prikazanog na slici 2(b), i koji će usloviti jediničnu radijalnu deformaciju tla na mestu konture tunelskog iskopa duž istih glavnih pravaca, kao u slučaju krutosti tunelske konstrukcije, ali suprotnog znaka. Takođe, postoji i generalizovana unutrašnja krutost tla k_{si} koja se definiše kao onaj intenzitet napona τ_{ff} koji će izazvati jediničnu radijalnu deformaciju tla na mestu konture tunelskog iskopa, čiji se smer poklapa sa smerom deformacije tunelske konstrukcije. Odnos krutosti tla k_{so} i k_{si} zavisi od definicije konturnog uslova na spoju tunelske konstrukcije i okolne sredine, dok odnos k_{si}/k_l predstavlja koeficijent krutosti koji je ekvivalentan koeficijentu fleksibilnosti F sugerisanom od strane prethodno pomenutih autora [10].

Pod pretpostavkom pojave **potpunog smicanja na kontaktu tunela i tla**, prema Penzien-ovom rešenju [14], izrazi za normalnu silu, transverzalnu silu i moment savijanja u tunelskoj konstrukciji su:

$$\Delta d_{lining}^n(\theta) = R^n \frac{d}{2} \gamma_{ave} \cos \left[2 \left(\theta + \frac{\pi}{4} \right) \right] \Rightarrow \pm \Delta d_{lining}^n = \pm R^n \Delta d_{free-field} = R^n \frac{d}{2} \gamma_{ave} \quad (14)$$

$$T(\theta) = \frac{12E_l I \Delta d_{lining}^n(\theta)}{d^3(1 - \nu_l^2)} \Rightarrow T_{max} = \pm \frac{12E_l I \Delta d_{lining}^n}{d^3(1 - \nu_l^2)} = \pm \frac{6E_l I R^n \gamma_{ave}}{d^2(1 - \nu_l^2)} \quad (15)$$

2.2 Solution after Penzien [14]

Penzien and Wu [13] developed similar closed-form elastic solutions for thrust, shear, and moment in the tunnel lining due to racking deformations. Penzien [14] provided an analytical procedure for evaluating racking deformations of rectangular and circular tunnels that supplemented the previous publication.

In order to estimate the distortion of the structure, a lining-soil racking ratio is defined as:

in which Δ_{il} is the lining racking displacement produced by its interaction with the soil and $\Delta_{is} = (k_l/k_{so})\Delta_{il}$ is the decrease in soil cavity racking displacement produced by its interaction with the lining.

In the case of circular tunnel, R is the ratio of lining diametric deflection and free-field diametric deflection. The generalized lining stiffness k_l is defined as that value of τ_{ff} shown for the pure-shear loading in Figure 2(b) which will produce unit diameter changes of the lining in the principal directions ($\theta = \pm \pi/4$), and the generalized outside-soil stiffness k_{so} is defined as that value of τ_{ff} when applied in the opposite directions to the cylindrical cavity wall will produce unit cavity-wall diameter changes of opposite signs in the same principal directions. In addition, there is the corresponding inside-soil generalized stiffness k_{si} defined as that intensity of τ_{ff} that will produce unit cavity-wall diameter changes in the same directions as for the lining. A relation of the soil stiffnesses k_{so} and k_{si} depends on a defined contact condition at the conjunctive surface of a medium and a lining, whereas k_{si}/k_l denotes the stiffness ratio that is equivalent with a flexibility coefficient F defined by the previously referenced authors [10].

Assuming **full slip condition**, according to Penzien [14], solutions for thrust, moment, and shear in circular tunnel linings caused by soil-structure interaction during a seismic event are expressed as:

$$M(\theta) = \frac{6E_l I \Delta d_{\text{lining}}^n(\theta)}{d^2(1-\nu_l^2)} \Rightarrow M_{\max} = \pm \frac{6E_l I \Delta d_{\text{lining}}^n}{d^2(1-\nu_l^2)} = \pm \frac{3E_l I R^n \gamma_{\text{ave}}}{d(1-\nu_l^2)} \quad (16)$$

$$V(\theta) = \frac{24E_l I \Delta d_{\text{lining}}^n(\theta)}{d^3(1-\nu_l^2)} \operatorname{tg} \left[2 \left(\theta + \frac{\pi}{4} \right) \right] \Rightarrow V_{\max} = \pm \frac{24E_l I \Delta d_{\text{lining}}^n}{d^3(1-\nu_l^2)} = \pm \frac{12E_l I R^n \gamma_{\text{ave}}}{d^2(1-\nu_l^2)} \quad (17)$$

pri čemu $\Delta d_{\text{lining}}^n$ predstavlja radijalnu deformaciju tunelske konstrukcije samo usled normalnog opterećenja.

Koeficijent smicanja tunela i tla u uslovima normalnog opterećenja definisan je sledećim izrazom:

$$R^n = \pm \frac{4(1-\nu_m)}{\alpha^n + 1} \quad (18)$$

gde je:

$$\alpha^n = \frac{k_l}{k_{so}} = (5-6\nu_m) \frac{k_l}{k_{si}} = (5-6\nu_m) \frac{\frac{72E_l I}{d^4(1-\nu_l^2)}}{\frac{6G_m}{d}} = \frac{12E_l I (5-6\nu_m)}{d^3 G_m (1-\nu_l^2)} \quad (19)$$

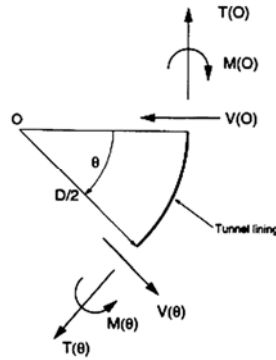
Konvencija znaka presečnih sila u tunelskoj konstrukciji prikazana je na slici 8.

where $\Delta d_{\text{lining}}^n$ is lining diameter deflection under normal loading only.

The lining-soil racking ratio under normal loading only is defined as:

where:

The sign convention for the above force components in circular lining is shown in Figure 8.



Slika 8. Konvencija znaka presečnih sila u tunelskoj konstrukciji kružnog poprečnog preseka [14]
Figure 8. Sign convention for force components in circular lining [14]

Za slučaj **idealnog kontakta tunela i tla**, jednačine imaju oblik:

In the case of **no slip condition**, the formulations are presented as:

$$\Delta d_{\text{lining}}(\theta) = R \frac{d}{2} \gamma_{\text{ave}} \cos \left[2 \left(\theta + \frac{\pi}{4} \right) \right] \Rightarrow \pm \Delta d_{\text{lining}} = \pm R \Delta d_{\text{free-field}} = R \frac{d}{2} \gamma_{\text{ave}} \quad (20)$$

$$T(\theta) = \frac{24E_l I \Delta d_{\text{lining}}(\theta)}{d^3(1-\nu_l^2)} \Rightarrow T_{\max} = \pm \frac{24E_l I \Delta d_{\text{lining}}}{d^3(1-\nu_l^2)} = \pm \frac{12E_l I R \gamma_{\text{ave}}}{d^2(1-\nu_l^2)} \quad (21)$$

$$M(\theta) = \frac{6E_l I \Delta d_{\text{lining}}(\theta)}{d^2(1-\nu_l^2)} \Rightarrow M_{\max} = \pm \frac{6E_l I \Delta d_{\text{lining}}}{d^2(1-\nu_l^2)} = \pm \frac{3E_l I R \gamma_{\text{ave}}}{d(1-\nu_l^2)} \quad (22)$$

$$V(\theta) = \frac{24E_l I \Delta d_{\text{lining}}(\theta)}{d^3(1-\nu_l^2)} \operatorname{tg} \left[2 \left(\theta + \frac{\pi}{4} \right) \right] \Rightarrow V_{\max} = \pm \frac{24E_l I \Delta d_{\text{lining}}}{d^3(1-\nu_l^2)} = \pm \frac{12E_l I R \gamma_{\text{ave}}}{d^2(1-\nu_l^2)} \quad (23)$$

gde je:

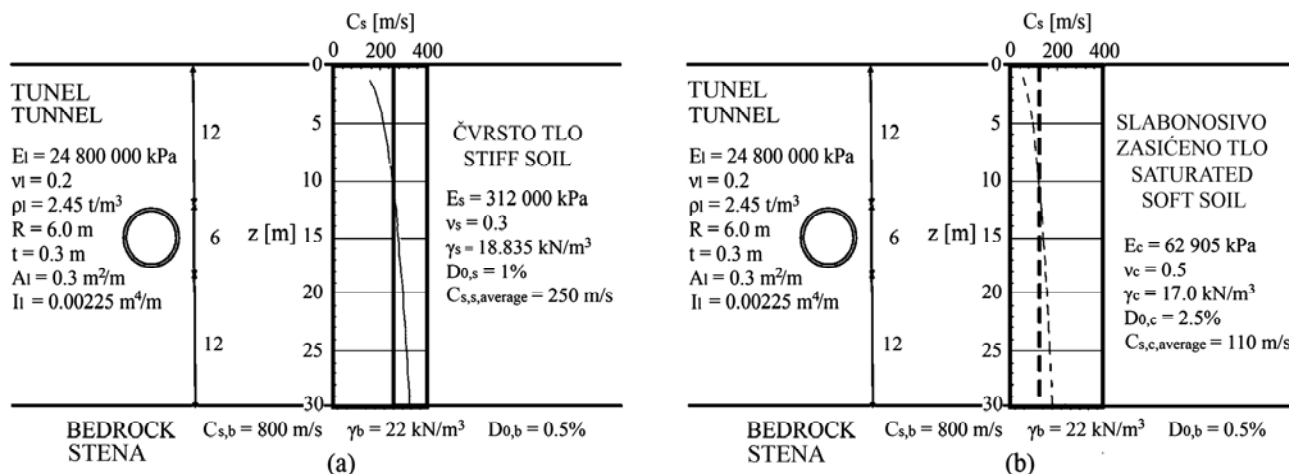
where:

$$R = \pm \frac{4(1 - \nu_m)}{\alpha + 1} \quad (24)$$

$$\alpha = \frac{k_l}{k_{so}} = (3 - 4\nu_m) \frac{k_l}{k_{si}} = (3 - 4\nu_m) \frac{48E_l I}{\frac{2G_m}{d}} = \frac{24E_l I(3 - 4\nu_m)}{d^3 G_m (1 - \nu_l^2)} \quad (25)$$

3 PRIMER PRORAČUNA ZA TUNEL U ČVRSTOM TLU I SLABONOSIVOM VODOZASIĆENOM TLU

3 A DESIGN EXAMPLE OF A TUNNEL IN STIFF SOIL AND SOFT SATURATED SOIL



Slika 9. Karakteristike tunela i tla: (a) čvrsto tlo, (b) vodozasićeno slabonosivo tlo
Figure 9. Tunnel characteristics and soil properties: (a) stiff soil, (b) soft saturated soil

Tunelska konstrukcija kružnog poprečnog preseka nalazi se u sloju tla debljine 30 m, koji leži na sloju relativno krute stenske mase. Visina nadsloja je 12 m, dok se tunelska osa nalazi na dubini od 15 m. Poluprečnik kružnog tunelskog iskopa je 3.0 m, a debljina betonske tunelske konstrukcije 0.3 m.

Fizička svojstva tunelske konstrukcije i okolnog tla prikazana su na slici 9. Imajući u vidu činjenicu da efekti interakcije tunela sa okolnim tlom zavise od odnosa krutosti konstrukcije i tla, u predmetnom istraživanju analizirana su dva karakteristična slučaja tla: čvrsto tlo kao primer tla sa dobrim karakteristikama, i slabonosivo vodozasićeno tlo kao primer tla sa slabim karakteristikama.

Prethodno prezentovana analitička rešenja priraštaja napona u tunelskoj konstrukciji usled dejstva zemljotresa u funkciji su smičuće deformacije tla, koja uzrokuje ovalizaciju tunela kružnog poprečnog preseka. Maksimalne vrednosti smičućih deformacija tla mogu biti sračunate na bazi jednodimenzionalnih analiza odgovora tla na seizmičke uticaje, u kojima se polazi od pretpostavke vertikalnog prostiranja horizontalnih smičućih (SH) talasa kroz horizontalne slojeve tla.

Profili brzine prostiranja smičućih talasa u čvrstom tlu i slabonosivom vodozasićenom tlu $C_s(z)$, takođe su prikazani na pomenutoj slici, gde je vertikalnom linijom (puna linija u slučaju čvrstog tla (slika 9(a)) i isprekidana u slučaju mekog zasićenog tla (slika 9(b))) prikazana tzv.

A tunnel structure of circular cross-section is placed within a 30-m-thick soil deposit overlying a relatively stiff bedrock, with an overburden cover of 12 m and axis depth to 15 m. An external tunnel radius of 3.0 m has been used, whereas the thickness of the lining is 0.3 m.

Physical properties of a tunnel lining and a ground material surrounding the tunnel are reported in the Figure 9. With regard to dependent nature of the soil-structure interaction effects upon the ratio between the relative stiffness of the ground with respect to the lining, in the given analyses representative of two main soil classes – stiff soil and soft saturated soil deposits have been considered.

Above presented closed-form solutions to compute the earthquake-induced stress increment in the lining are functions of the shear strain field, which is the cause of the ovaling of the circular tunnel cross-section. The peak ground strains could be computed based on one-dimensional seismic site response analyses for solving the problem of vertical propagation of horizontal shear (SH) waves through a horizontally layered soil deposit.

The shear wave velocity profiles $C_s(z)$ are also illustrated in the given figure. The vertical line (solid one in the case of the stiff soil (Figure 9(a)) and dashed one in the case of the soft saturated soil (Figure 9(b))) represents the value of the so called *equivalent velocity*, which is an average value of the shear wave velocity within the soil medium necessary for the purpose of one-

ekvivalentna brzina, koja predstavlja srednju vrednost brzine prostiranja seizmičkih talasa u tlu. Prosečna brzina prostiranja smičućih talasa bila je neophodna za potrebe sprovođenja linearnih analiza odgovora tla na seizmička dejstva, koje su realizovane primenom softvera EERA. U slučaju čvrstog tla, prosečna brzina prostiranja S-talasa iznosi 250 m/s, dok je u slučaju mekog zasićenog tla srednja vrednost brzine talasa 110 m/s.

Kada je reč o modulu smicanja tla, u linearnoj analizi odgovora tla na seizmičke uticaje njegova vrednost je nepromenljiva, usled pretpostavke konstantne brzine prostiranja smičućih talasa u tlu. Za slučaj čvrstog tla $G=G_{max}=120000$ kPa, dok je u slabom vodozasićenom tlu $G=G_{max}=20968$ kPa (prema Jednačini (2)).

Na osnovu Jednačine (3), odgovarajući moduli deformabilnosti su $E=312000$ kPa za tlo dobrih karakteristika i $E=62905$ kPa za vodozasićeno tlo slabih karakteristika.

Vrednost koeficijenta prigušenja tla takođe je konstantna, i iznosi $D=D_0=1$ % za čvrsto tlo i $D_0=2.5$ % za slabonosivo tlo.

4 ANALIZA ODGOVORA TLA NA SEIZMIČKE UTICAJE (SOFTVER EERA)

Seizmičke analize odgovora tla izvršene su primenom softvera EERA (Equivalent-linear Earthquake site Response Analysis) [1]. Ovaj softver je baziran na primeni dobro poznatog koncepta ekvivalentne linearne analize seizmičkog odgovora tla, koji je prvi put bio primenjen u programu SHAKE [16]. Unos podataka i obrada rezultata integrisani su s programom MS-Excel.

Softver je namenjen za seizmičke analize slojevitih terena s linearnim i ekvivalentnim linearnim svojstvima ponašanja, u frekventnom domenu (FD). Jednodimenzionalna analiza odgovora tla na seizmičke uticaje bazirana je na pretpostavci vertikalnog prostiranja horizontalnih smičućih (SH) talasa kroz horizontalne slojeve tla. Ponašanje horizontalnih slojeva tla simulirano je Kelvin-Voigt-ovim modelom, čije su osnovne karakteristike modul smicanja i koeficijent prigušenja tla.

Stenska masa ispod sloja tla u softveru EERA može biti simulirana kao čvrsta stena biranjem opcije *inside*, ili kao elastična stena izborom opcije *outcrop*. S ciljem transformisanja akceleroograma sa dela stenske mase na površini terena (outcropping rock) do dela stene s nadslojem tla (bedrock), u softveru su definisane odgovarajuće transformacijske funkcije.

U predmetnim analizama razmatrane su dve vrste tla čije su karakteristike naznačene na slici 9. U proračunu deformacija tla izazvanih propagacijom seizmičkih talasa, pošlo se od pretpostavke da se ponašanje tla bazira na zakonima linearne elastičnosti (modul smicanja i koeficijent prigušenja tla imaju konstantne vrednosti i ne zavise od veličine smičućih deformacija tla).

U radu je za potrebe dinamičkih analiza korišćen zapis zemljotresa Kobe u Japanu (1995. godine), zato što je ovaj potres imao najrazornije posledice po tunnelske konstrukcije ikad zabeležene u istoriji. Zbog nepostojanja zapisa zemljotresa u okolini bilo kakvog podzemnog objekta, originalni akcelerogram zabeležen na površini terena skaliran je na veličinu maksimalne

dimensional (1D) linear seismic site response (SSR) analyses completed by means of the software EERA. For the given soil column, an average value of the shear wave velocity profile of 250 m/s for the stiff soil and 110 m/s for the soft saturated soil have been used throughout the analyses.

When it comes to a soil shear modulus, in linear analysis it is of constant nature, regarding the constant value of the shear wave velocity. The modulus value in the case of the stiff soil is $G=G_{max}=120000$ kPa, whereas for the soft undrained soil $G=G_{max}=20968$ kPa (according to Equation (2)).

Based on Equation (3), the corresponding moduli of elasticity are $E=312000$ kPa in the case of stiff soil and $E=62905$ kPa for the soft saturated soil.

A value of damping ratio is also constant, and for the stiff soil material is taken as $D=D_0=1$ %, whereas for the soft undrained soil deposit $D=D_0=2.5$ %.

4 ONE-DIMENSIONAL SSR ANALYSIS (CODE EERA)

The SSR analyses have been performed by means of the code EERA [1] that stands for Equivalent-linear Earthquake site Response Analysis. It is a modern implementation of the well-known concepts of the equivalent linear site response analysis that was first implemented in the SHAKE code [16]. The input and output are fully integrated with the spreadsheet program MS-Excel.

This code is intended to perform analyses for linear and equivalent linear stratified subsoils. One-dimensional site response analysis is used to solve the problem of vertical propagation of horizontal shear waves (SH waves) through a horizontally layered soil deposit. Horizontal soil layer behaviour is approximated as a Kelvin-Voigt solid whereby elastic shear moduli and viscous damping characterize soil properties. Solution of wave propagation equations is performed in the frequency domain (FD).

The bedrock can be modelled as rigid, if the option *inside* is selected in the Profile spreadsheet of the program, or as elastic, by assigning its properties to the last layer and selecting the option *outcrop* in the Profile spreadsheet. In order to transform the signal from the outcropping rock to the bedrock, placed at the bottom of the soil layer, EERA applies a suitable transfer function to the input signal.

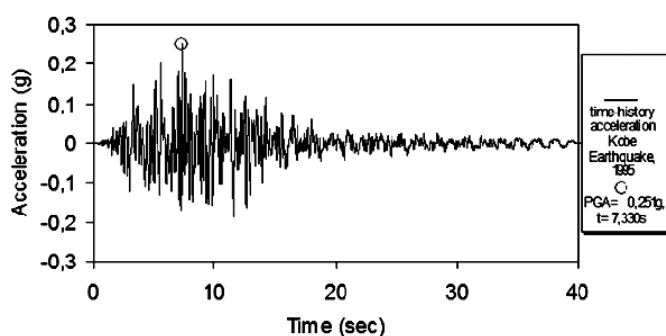
In the present analyses, ground conditions and the soil behaviour were modelled according to the Figure 9. The free-field soil deformations caused by wave propagation are calculated under the assumption that the soil behaviour is described by a linear elastic model (shear modulus and damping coefficient of the soil are constant and do not depend on a shear strain level throughout analysis).

The source of dynamic excitation in this project was the acceleration record for the 1995 Kobe, Japan earthquake. This earthquake data was used because this earthquake event was the most devastating to civil

akceleracije od 0.25g, imajući u vidu prirodu slabljenja ubrzanja tla s većim dubinama [9]. Tako dobijeni akcelerogram nanet je duž granice čvrste stenske podloge i posmatranog sloja tla.

Slika 10 ilustruje modifikovani akcelerogram zemljotresa Kobe (acceleration time history) koji je korišćen u seizmičkim analizama odgovora tla. Maksimalna vrednost ubrzanja tla (peak ground acceleration) iznosi 0.251g (2.46 m/s²) i javlja se približno 7.3 s od početka potresa.

Analize odgovora tla na seizmičke uticaje omogućile su sračunavanje maksimalnih smičućih deformacija tla za oba razmatrana slučaja – čvrsto tlo i vodosasićeno slabonosivo tlo, što je prikazano na slici 11. Takođe je sračunata i prosečna vrednost smičuće deformacije tla, γ_{ave} , na dubini tunelskog objekta, između temena tunelskog i podnožnog svoda.



Slika 10. Modifikovani akcelerogram zemljotresa Kobe u Japanu (1995)

Figure 10. Scaled accelerogram of 1995 Kobe Earthquake in Japan

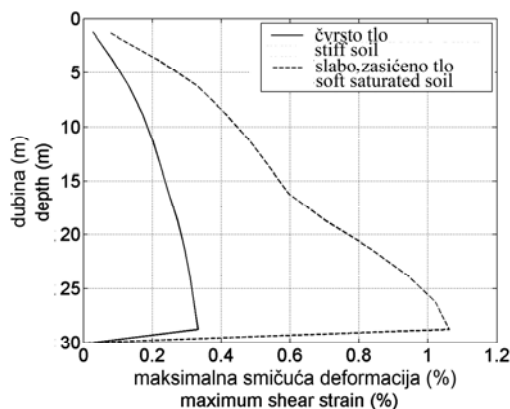
Za uneti akcelerogram, maksimalna deformacija smicanja u čvrstom tlu je 0.33 %, dok je njena srednja vrednost na dubini tunela (između temena gornjeg i podnožnog svoda) 0.24 %. Kada je reč o zasićenom mekom zemljanom materijalu, maksimalna smičuća deformacija tla iznosi 1.06 %, dok je u nivou tunelskog objekta njena prosečna vrednost 0.57 %. Na osnovu ovih rezultata može se zaključiti da su maksimalne smičuće deformacije tla daleko veće po svojoj vrednosti za slučaj mekog zasićenog tla, što je posledica njegovih slabijih karakteristika usled prisustva vode, jačeg prigušenja vibracija, i samim tim, apsorpcije većeg dela seizmičke energije.

Sračunata prosečna vrednost smičuće deformacije tla, γ_{ave} , uslovljena zemljotresnim dejstvom, iskorišćena je za sračunavanje seizmički indukovanih presečnih sila u tunelskoj konstrukciji prema analitičkim izrazima po Wang-u [20] i Penzien-u [14].

infrastructure in recorded history. Since there is no bedrock strong motion record in the vicinity of tunnels, the surface accelerogram was scaled to 0.25g to account for strong motion attenuation with depth [9]. The acceleration input from the earthquake was applied to the bottom boundary of the soil column model.

Figure 10 illustrates the scaled acceleration time history that was employed in SSR analyses. The peak value of the input acceleration time history is 0.251g (2.46 m/s²) occurring approximately 7.3 s after the onset of the excitation.

Shear strain induced by the seismic waves at the tunnel depth have been calculated through a free-field one-dimensional SSR analysis for both stiff soil and saturated soft soil deposits, as illustrated in Figure 11. A value of the average soil shear strain, γ_{ave} , in the range of depths between the tunnel crown and the invert has been computed.



Slika 11. Profili maksimalnih smičućih deformacija u čvrstom tlu i slabonosivom zasićenom tlu dobijeni 1D analizom odgovora tla na seizmičke uticaje

Figure 11. Maximum shear strain profiles for stiff soil and soft saturated soil obtained by 1D SSR linear analysis

Linear EERA analysis has shown that for the given stiff soil profile and input excitation, maximum soil shear strain is 0.33 %, and its average value at tunnel location (at depths between the crown and invert) is 0.24 %. In case of saturated soft soil, maximum soil shear deformation is 1.06 %, whereas its average value at tunnel location is 0.57 %. Accordingly, soil shear strains for the case of soft saturated soil are significantly higher, considering weaker ground conditions due to a presence of water, higher damping values, and higher portion of seismic wave energy absorption.

The calculated average soil shear straining, γ_{ave} , induced by earthquake excitation, is then applied to the aforementioned analytical expressions after Wang [20] and Penzien [14].

5 PRORAČUN PRESEČNIH SILA U TUNELSKOJ KONSTRUKCIJI INDUKOVANIH ZEMLJOTRESNIM DEJSTVOM

5.1 Čvrsto tlo ($\gamma_{ave}=0.0024$)

5.1.1 Rešenje prema analitičkim izrazima po Wang-u [20]

a) Smičući kontakt tunelske konstrukcije i okolnog tla:

1) Određivanje koeficijenta fleksibilnosti (F), koeficijenta seizmičkog odgovora tunelske konstrukcije za slučaj smičućeg kontakta tunela i tla (K_1) i koeficijenta smicanja ($\Delta d_{lining}/\Delta d_{free-field}$) prema Jednačinama (5), (9) i (6):

$$F = \frac{E_m(1-\nu_l^2)r^3}{6E_lI(1+\nu_m)} = \frac{312000(1-0.2^2)3^3}{6 \cdot 24.8 \cdot 10^6 \cdot 0.00225(1+0.3)} = 18.5806$$

$$K_1 = \frac{12(1-\nu_m)}{2F+5-6\nu_m} = \frac{12(1-0.3)}{2 \cdot 18.5806 + 5 - 6 \cdot 0.3} = 0.20812$$

$$\frac{\Delta d_{lining}}{\Delta d_{free-field}} = \frac{2}{3} K_1 F = \frac{2}{3} \cdot 0.20812 \cdot 18.5806 = 2.578$$

2) Proračun maksimalnih vrednosti normalne sile (T) i momenta savijanja (M) indukovanih prostiranjem smičućih S-talasa prema Jednačinama (7) i (8):

$$T_{max} = \frac{1}{6} K_1 \frac{E_m}{1+\nu_m} r \cdot \gamma_{ave} = \frac{1}{6} \cdot 0.20812 \cdot \frac{312000}{1+0.3} \cdot 3 \cdot 0.0024 = 59.9 \text{ kN}$$

$$M_{max} = \frac{1}{6} K_1 \frac{E_m}{1+\nu_m} r^2 \cdot \gamma_{ave} = \frac{1}{6} \cdot 0.20812 \cdot \frac{312000}{1+0.3} \cdot 3^2 \cdot 0.0024 = 179.8 \text{ kNm}$$

3) Proračun napona u tunelskoj konstrukciji (σ):

$$\sigma = \frac{T}{A} + \frac{M \cdot y}{I} = \frac{59.9}{0.3} + \frac{179.8 \cdot 0.15}{0.00225} = 12187.4 \text{ kPa}$$

b) Idealni kontakt tunelske konstrukcije i okolnog tla:

Prema Jednačinama (4), (11) i (10) redom:

$$C = \frac{E_m(1-\nu_l^2)r}{E_l \cdot t(1+\nu_m)(1-2\nu_m)} = \frac{312000(1-0.2^2) \cdot 3}{24.8 \cdot 10^6 \cdot 0.3(1+0.3)(1-2 \cdot 0.3)} = 0.2323$$

5 DETERMINATION OF SEISMICALLY INDUCED TUNNEL LINING FORCES

5.1 Stiff soil ($\gamma_{ave}=0.0024$)

5.1.1 Solution according to the formulations of Wang [20]

a) Full-slip condition:

1) Determination of the flexibility ratio (F), full-slip lining response coefficient (K_1), and racking ratio ($\Delta d_{lining}/\Delta d_{free-field}$) according to Equations (5), (9), and (6), respectively:

2) Determination of the maximum tangential thrust (T) and moment (M) due to S-waves according to Equations (7) and (8), respectively:

3) Determination of the combined stress (σ) from thrust and bending moment:

b) No-slip condition:

According to Equations (4), (11), and (10), respectively:

$$K_2 = 1 + \frac{F[(1-2\nu_m) - (1-2\nu_m)C] - \frac{1}{2}(1-2\nu_m)^2 + 2}{F[(3-2\nu_m) + (1-2\nu_m)C] + C(\frac{5}{2} - 8\nu_m + 6\nu_m^2) + 6 - 8\nu_m} =$$

$$= 1 + \frac{18.5806[(1-2 \cdot 0.3) - (1-2 \cdot 0.3) \cdot 0.2323] - \frac{1}{2}(1-2 \cdot 0.3)^2 + 2}{18.5806[(3-2 \cdot 0.3) + (1-2 \cdot 0.3) \cdot 0.2323] + 0.2323(\frac{5}{2} - 8 \cdot 0.3 + 6 \cdot 0.3^2) + 6 - 8 \cdot 0.3}$$

$$= 1.1523$$

$$T_{\max} = K_2 \tau_{\max} r = K_2 \frac{E_m}{2(1+\nu_m)} r \cdot \gamma_{ave} = 1.1523 \frac{312000}{2(1+0.3)} 3 \cdot 0.0024 = 995.6 \text{ kN}$$

$$\sigma = \frac{T}{A} + \frac{M \cdot y}{I} = \frac{995.6}{0.3} + \frac{179.8 \cdot 0.15}{0.00225} = 15305.3 \text{ kPa}$$

5.1.2 Rešenje prema analitičkim izrazima po Penzien-u [14]

5.1.2 Solution according to the formulations of Penzien [14]

a) Smičući kontakt tunelske konstrukcije i okolnog tla:

a) Full-slip condition:

1) Određivanje α^n , koeficijenta smicanja (R^n) i radialne deformacije tunelske konstrukcije ($\Delta d_{\text{lining}}^n$) prema Jednačinama (19), (18) i (14):

1) Determining the α^n , racking ratio (R^n), and the diametric strain of the lining ($\Delta d_{\text{lining}}^n$) according to Equations (19), (18), and (14), respectively:

$$\alpha^n = \frac{12E_l I (5 - 6\nu_m)}{d^3 G_m (1 - \nu_l^2)} = \frac{12 \cdot 24.8 \cdot 10^6 \cdot 0.00225 (5 - 6 \cdot 0.3)}{6^3 \cdot 120000 (1 - 0.2^2)} = 0.086111$$

$$R^n = \frac{4(1 - \nu_m)}{\alpha^n + 1} = \frac{4(1 - 0.3)}{0.086111 + 1} = 2.5780$$

$$\Delta d_{\text{lining}}^n = R^n \Delta d_{\text{free-field}} = R^n \frac{d}{2} \gamma_{ave} = 2.5780 \frac{0.0024 \cdot 6}{2} = 0.01856$$

2) Proračun maksimalnih vrednosti normalne sile (T), momenta savijanja (M) i transverzalne sile (V) prema Jednačinama (15), (16) i (17):

2) Determination of the maximum tangential thrust (T), moment (M), and shear force (V) according to Equations (15), (16), and (17), respectively:

$$T_{\max} = \frac{12E_l I \Delta d_{\text{lining}}^n}{d^3 (1 - \nu_l^2)} = \frac{12 \cdot 24.8 \cdot 10^6 \cdot 0.00225 \cdot 0.01856}{6^3 (1 - 0.2^2)} = 59.9 \text{ kN}$$

$$M_{\max} = \frac{6E_l I \Delta d_{\text{lining}}^n}{d^2 (1 - \nu_l^2)} = \frac{6 \cdot 24.8 \cdot 10^6 \cdot 0.00225 \cdot 0.01856}{6^2 (1 - 0.2^2)} = 179.8 \text{ kNm}$$

$$V_{\max} = \frac{24E_l I \Delta d_{\text{lining}}^n}{d^3 (1 - \nu_l^2)} = \frac{24 \cdot 24.8 \cdot 10^6 \cdot 0.00225 \cdot 0.01856}{6^3 (1 - 0.2^2)} = 119.9 \text{ kN}$$

3) Proračun napona u tunelskoj konstrukciji (σ):

3) Determination of the combined stress (σ) from thrust and bending moment:

$$\sigma = \frac{T}{A} + \frac{M \cdot y}{I} = \frac{59.9}{0.3} + \frac{179.8 \cdot 0.15}{0.00225} = 12187.4 \text{ kPa}$$

b) Idealni kontakt tunelske konstrukcije i okolnog tla:

b) No-slip condition:

Prema Jednačinama (25), (24), (20), (21), (22) i (23) redom:

According to Equations (25), (24), (20), (21), (22), and (23), respectively:

$$\alpha = \frac{24E_l I(3 - 4\nu_m)}{d^3 G_m (1 - \nu_l^2)} = \frac{24 \cdot 24.8 \cdot 10^6 \cdot 0.00225(3 - 4 \cdot 0.3)}{6^3 \cdot 120000(1 - 0.2^2)} = 0.096875$$

$$R = \frac{4(1 - \nu_m)}{\alpha + 1} = \frac{4(1 - 0.3)}{0.096875 + 1} = 2.553$$

$$\Delta d_{\text{lining}} = R \Delta d_{\text{free-field}} = R \frac{d}{2} \gamma_{\text{max}} = 2.553 \frac{6}{2} \cdot 0.0024 = 0.01838$$

$$T_{\text{max}} = \frac{24E_l I \Delta d_{\text{lining}}}{d^3 (1 - \nu_l^2)} = \frac{24 \cdot 24.8 \cdot 10^6 \cdot 0.00225 \cdot 0.01838}{6^3 (1 - 0.2^2)} = 118.7 \text{ kN}$$

$$M_{\text{max}} = \frac{6E_l I \Delta d_{\text{lining}}}{d^2 (1 - \nu_l^2)} = \frac{6 \cdot 24.8 \cdot 10^6 \cdot 0.00225 \cdot 0.01838}{6^2 (1 - 0.2^2)} = 178.1 \text{ kNm}$$

$$V_{\text{max}} = \frac{24E_l I \Delta d_{\text{lining}}}{d^3 (1 - \nu_l^2)} = \frac{24 \cdot 24.8 \cdot 10^6 \cdot 0.00225 \cdot 0.01838}{6^3 (1 - 0.2^2)} = 118.7 \text{ kN}$$

$$\sigma = \frac{T}{A} + \frac{M \cdot y}{I} = \frac{118.7}{0.3} + \frac{178.1 \cdot 0.15}{0.00225} = 12266.1 \text{ kPa}$$

Rezultati za slučaj tunela u čvrstom tlu prezentovani su u Tabeli 1.

The results regarding the tunnel in the stiff soil are summarized in Table 1.

Tabela 1. Rezultati maksimalnih vrednosti seizmički indukovanih presečnih sila u tunelskoj konstrukciji za slučaj tunela u čvrstom tlu

Table 1. Results of maximum values of seismically induced internal lining forces for the case of a tunnel in a stiff soil

ČVRSTO TLO STIFF SOIL	Wang [20]		Penzien [14]		% Razlika Difference	
	Smičući kontakt Full slip	Idealni kontakt No slip	Smičući kontakt Full slip	Idealni kontakt No slip	Smičući kontakt Full slip	Idealni kontakt No slip
Koeficijent smicanja Racking ratio	2.58	2.58	2.58	2.55	0	1.2
Maksimalna normalna sila Maximum thrust (T_{max} [kN])	59.9	995.6	59.9	118.7	0	738.7
Maksimalni moment savijanja Maximum moment (M_{max} [kNm])	179.8	179.8	179.8	178.1	0	0.9
Maksimalna transverzalna sila Maximum shear force (V_{max} [kN])	–	–	119.9	118.7	–	–
Napon Stress (σ [kPa])	12187.4	15305.3	12187.4	12266.1	0	24.8

5.2 Vodozasićeno slabonosivo tlo ($\gamma_{ave}=0.0057$)

Postupak proračuna prezentovan u tački 5.1 ponovljen je za slučaj slabonosivog tla zasićenog vodom. Kada je reč o izrazu za koeficijent kompresibilnosti (Jednačina (4)), Poisson-ov koeficijent treba uzeti s vrednošću 0.49, jer vrednost 0.5 rezultuje beskonačno velikim koeficijentom kompresibilnosti.

Prema Wang-ovom postupku, sračunate vrednosti koeficijenata su $F=3.2467$ i $K_1=0.7064$ (smičući kontakt tunela i tla), odnosno $C=0.8117$ i $K_2=1.231$ (idealni kontakt tunela i tla).

Prema Penzien-ovom postupku, $\alpha^n=0.308$ (glatki spoj obloge i tla) i $\alpha=0.3203$ (puni kontinuitet između obloge i okolnog tla).

Rezultati proračuna su sumirani u Tabeli 2.

5.2 Saturated soft soil ($\gamma_{ave}=0.0057$)

The above calculation is repeated for the case of soft water-saturated medium. With regard to the expression for compressibility ratio given by Equation (4), the Poisson ratio should be set equal to 0.49, since the value of 0.5 results in the infinite value of the compressibility ratio.

Regarding the Wang's methodology, the calculated values of the corresponding coefficients are $F=3.2467$ and $K_1=0.7064$ (full-slip condition), and $C=0.8117$ and $K_2=1.231$ (no-slip condition).

Referring to the Penzien's approach, $\alpha^n=0.308$ (smooth lining-ground contact) and $\alpha=0.3203$ (perfect lining-ground contact).

The results are summarized in Table 2.

Tabela 2. Rezultati maksimalnih vrednosti seizmički indukovanih presečnih sila u tunelskoj konstrukciji za slučaj tunela u zasićenom tlu male nosivosti

Table 2. Results of maximum values of seismically induced internal lining forces for the case of a tunnel in a saturated soft soil

VODOZASIĆENO SLABONOSIVO TLO SATURATED SOFT SOIL	Wang [20]		Penzien [14]		% Razlika Difference	
	Smičući kontakt Full slip	Idealni kontakt No slip	Smičući kontakt Full slip	Idealni kontakt No slip	Smičući kontakt Full slip	Idealni kontakt No slip
Koeficijent smicanja Racking ratio	1.53	1.53	1.53	1.54	0	0.6
Maksimalna normalna sila Maximum thrust (T_{max} [kN])	84.4	440.2	84.4	170.2	0	158
Maksimalni moment savijanja Maximum moment (M_{max} [kNm])	252.6	252.6	253.3	255.3	0.2	1.0
Maksimalna transverzalna sila Maximum shear force (V_{max} [kN])	–	–	168.9	170.2	–	–
Napon Stress (σ [kPa])	17121.3	18307.3	17168.0	17587.3	0.2	3.9

6 DISKUSIJA REZULTATA

Poređenjem rezultata prezentovanih u Tabelama 1 i 2, može se uočiti da su za slučaj smicanja tunelske konstrukcije i okolnog tla u kontaktnoj zoni obe formulacije rezultovale jednakim maksimalnim vrednostima presečnih sila u tunelskoj konstrukciji, kako za slučaj čvrstog tla, tako i za slučaj slabonosivog zasićenog tla.

Takođe se može zapaziti da za slučaj smičućeg kontakta tunela i tla, vrednosti momenata savijanja imaju izraženiji uticaj na naponsko stanje u tunelskoj konstrukciji u uslovima zemljotresnih dejstava od uticaja normalnih sila. S druge strane, u slučaju idealnog kontakta tunela i tla, normalne sile imaju dominantniji uticaj na seizmički indukovane napone u tunelskoj konstrukciji u poređenju sa uticajem momenata savijanja.

Pod pretpostavkom idealnog kontakta tunelske

6 DISCUSSION ON RESULTS

Referring to the previously summarized results in Tables 1 and 2, it is evident that in the case of full-slip condition, the two formulations give the same values of the force components, for both stiff and soft soil conditions.

Furthermore, it can be observed that magnitude of the moment has a much stronger influence than thrust over the stresses experienced by the tunnel lining for the case of full-slip conditions. On the other hand, for the no-slip assumption, magnitude of the thrust has a more dominating influence than moments on the stresses in the lining induced by earthquake excitation.

Calculation also shows that under the no-slip assumption, the formulation of Penzien [14] highly underestimates the thrust in comparison with the Wang's

konstrukcije sa okolnim tlom, analitički izrazi prema Penziju [14] rezultuju daleko manjim vrednostima normalne sile od izraza prema Wangu [20], i to za oba razmatrana slučaja tla. Razlika ova dva rešenja je znatno manje izražena u slučaju zasićenog slabonosivog tla. Ovo zapažanje potvrđuje zaključke istraživanja Hashasha i drugih [6, 7] o tome da ne postoji značajnija razlika u vrednostima normalnih sila sračunatih prema izrazima Penziena za ekstremne slučajeve kontaktnih uslova tunela i tla.

Ovo nepoklapanje rezultata izaziva veliku konfuziju i nedoumicu, imajući u vidu da oba razmatrana postupka polaze od identičnih postavki problema baziranih na teoriji elastičnosti. Preliminarni rezultati numeričkih analiza, koje su još uvek u toku i koje autori ovog rada sprovode u cilju istraživanja razloga nepoklapanja rezultata Penzien-ovog i Wang-ovog pristupa, ukazuju na činjenicu da rešenje po Wang-u daje merodavnije vrednosti normalnih sila u tunelskoj konstrukciji u uslovima punog kontinuiteta između obloge i okolnog tla. Razlozi za iskazane razlike u veličinama koje daju ova dva analitička postupka, još uvek se istražuju. Međutim, neka generalna sagledavanja ovih analitičkih izraza navode na zaključak da mogući razlog, zbog kojeg izraz prema Penzien-ovom postupku rezultuje znatno manjim vrednostima seizmički indukovanih normalnih sila u tunelskoj oblozi za slučaj idealnog kontakta tunela sa okolnim tlom, jeste nepostojanje adekvatnog koeficijenta kompresibilnosti. Naime, u Wang-ovom izrazu za proračun normalnih sila u oblozi u uslovima punog kontinuiteta konstrukcije i tla, osim koeficijenta fleksibilnosti F koji reflektuje krutost sistema tunel–tlo u radijalnom pravcu (otpornost na smicanje), uključen je i koeficijent kompresibilnosti C . Ovaj koeficijent ima veoma važnu ulogu u određivanju normalnih sila, s obzirom na to što predstavlja krutost sistema tunel–tlo u tangencijalnom pravcu (otpornost na pritisak). S druge strane, Penzien u svom izrazu za proračun seizmički indukovanih normalnih sila u tunelskoj konstrukciji, u uslovima punog kontinuiteta obloge i okolnog tla, uzima u obzir samo krutost sistema tunel–tlo na smicanje koja je iskazana koeficijentom α . Upravo zbog ovih razloga, preporučuje se da u proračunu seizmički indukovanih normalnih sila u tunelskoj konstrukciji ne treba koristiti analitički izraz prema Penzien-u za slučaj idealnog kontakta tunelske konstrukcije i okolnog tla.

7 ZAKLJUČAK

U radu je dat prikaz analitičkih postupaka za proračun seizmički indukovanih presečnih sila u tunelskoj konstrukciji prema Wang-u [20] i Penzien-u [14], koji su bazirani na aspektu interakcije konstrukcije i tla. Ovi uprošćeni analitički postupci daju zadovoljavajuće rezultate sa inženjerskog stanovišta. Određivanje naponskog stanja u tunelskoj konstrukciji kružnog preseka u uslovima zemljotresnih dejstava bazirano je na pretpostavci ravnog stanja deformacija. Okolno tlo, odnosno stenska masa, tretiraju se kao beskonačna, elastična, homogena i izotropna sredina. Efekti interakcije konstrukcije sa okolnim tlom zavise od odnosa krutosti tunelske konstrukcije i krutosti okolne sredine. Drugi veoma važan faktor, koji u značajnoj meri utiče na odgovor tunela na seizmička dejstva, jeste priroda kontakta konstrukcije sa okolnim tlom. Radi

solution [20], considering both representative soil deposits. Discrepancy with the Penzien's solution is, however, slightly lower in the case of soft soil medium. This observation confirms the conclusions of the studies by Hashash et al. [6, 7] that the Penzien's approach does not seem to be very sensitive to the assumed condition along the interface between the ground and the lining.

This discrepancy is a source of confusion, having in mind that both of the considered approaches are based on the same assumptions governed by the theory of elasticity. Some preliminary results of numerical investigations by the authors of this paper that are in progress, with regard to the discrepancy of the Penzien's and Wang's approaches, indicate that the Wang's solution provides a realistic estimate of the thrust in tunnel linings for the no-slip condition. The reason for the difference is still under investigation. However, some general observations of these analytical expressions imply the conclusion that the possible reason for the underestimation of seismically induced lining thrust according to the Penzien's expression for the no-slip condition might be nonexistence of the corresponding compressibility coefficient. Namely, in the Wang's expression for the tunnel lining thrust under the no-slip assumption, beside the flexibility ratio F related to the ability of the lining to resist distortion imposed by the ground, the compressibility ratio C is also taken into consideration. This coefficient has an effect on the lining thrust response, since it reflects the circumferential stiffness of the system (resistance to compression). On the other hand, in the Penzien's expression for the seismically induced tunnel lining thrust under the assumption of rigid lining–ground contact, only the flexibility ratio represented by the coefficient α is taken into account. For that reason, it is recommended that in estimation of the seismically induced tunnel lining thrust the analytical expression after Penzien for the no-slip condition should be avoided.

7 CONCLUSION

In this work, analytical solutions for seismically induced internal lining forces by Wang [20] and Penzien [14], accounting for soil–structure interaction effects, have been presented. Aforementioned simplified approaches give reasonable results from an engineering point of view. These solutions are based on the computation of the state of stress in the cross section of a lined circular tunnel in plane strain conditions. The soil or rock is considered to be an infinite, elastic, homogeneous, isotropic medium. The soil–structure interaction effects depend upon the ratio between the relative stiffness of the ground with respect to the lining. Another aspect which significantly affects the response of the tunnel is represented by the shear stress transmission at the ground–lining interface. The solutions are usually derived for two extreme contact

sagledavanja svih mogućih slučajeva, preporučuje se da rešenja obuhvate ekstremne kontaktne uslove: glatki kontakt ili kontakt s potpunim smicanjem (*full-slip*) kada nema pojave smičućeg napona u kontaktnoj zoni, i idealni kontakt ili puni kontinuitet obloge i okolnog tla (*no-slip*) u čijem slučaju u graničnoj zoni nema relativnih transverzalnih pomeranja između konstrukcije i tla. Najkritičniji vid deformacije jeste smicanje (ovalizacija) kružnog tunelskog profila, izazvano propagacijom smičućih seizmičkih S-talasa koji se prostiru u ravnima upravnim na pravac podužne ose tunela. U razmatranim pseudo-statičkim postupcima, seizmičko opterećenje definiše se merodavnom seizmički indukovanom deformacijom smicanja koja se u okolnom tlu javlja na rastojanju od nivoa tunelskog svoda do nivoa kinete.

Postupci prikazanog analitičkog tretiranja interakcije takođe podrazumevaju da se tunelska konstrukcija nalazi na dovoljnoj dubini ispod površine terena i na dovoljnom rastojanju od stenske podine, kako bi efekat ovih granica na interakciju konstrukcije sa okolnom sredinom bio minimiziran. Takođe treba napomenuti da su prezentovani analitički postupci namenjeni analizi konvencionalnih betonskih tunelskih konstrukcija, kao i montažnih obloga koje se sukcesivno izvode s napredovanjem iskopne mašine „krtice“, a za tunele koji se nalaze u sredini koja ima znatno manji modul deformabilnosti od modula deformabilnosti tunelske obloge, to jest u slučaju tunelskih konstrukcija u tlu slabijih karakteristika, kada su efekti interakcije konstrukcije sa okolnim tлом najizraženiji. Rezultantni naponi u tlu koji stvaraju sile u tunelskoj konstrukciji ne mogu biti naponi zatezanja, i moraju zadovoljiti granične vrednosti napona smicanja. U slučaju tunelskih obloga koje su ankerima povezane sa okolnom sredinom, a koje nisu bile predmet razmatranja u prikazanim analitičkim postupcima, treba napomenuti da se mogu očekivati sile vučenja obloge od konture obloge prema okolnoj sredini, što bi, osim pojave prenosa sila smicanja po obodu tunelske konstrukcije, u izvesnoj meri rezultovalo i prenosom dilatacija zatezanja u okolnoj sredini.

U radu je takođe izvršeno poređenje ova dva analitička postupka na primeru tunela i tla idealne geometrije i karakteristika. Imajući u vidu činjenicu da efekti sadejstva tunela sa okolnim tлом zavise od odnosa krutosti konstrukcije i tla, u predmetnom istraživanju analizirana su dva karakteristična slučaja tla: čvrsto tlo kao primer tla sa dobrim karakteristikama, i zasićeno tlo male nosivosti kao primer tla sa slabim karakteristikama. Vrednosti presečnih sila u tunelskoj konstrukciji prema analitičkim izrazima sračunate su na osnovu prosečne vrednosti smičuće deformacije tla na delu tunelskog profila, dobijene seizmičkom analizom odgovora tla u softveru EERA. U istraživanju su razmatrana oba ekstremna slučaja kontaktnih uslova tunela sa okolnim tлом: slučaj krutog ili idealnog kontakta tunela i tla (*no-slip*), polazeći od hipoteze jednakih pomeranja tunelske konstrukcije i tla u kontaktnoj zoni, i slučaj glatkog kontakta ili kontakta s potpunim smicanjem (*full-slip*), kada u kontaktnoj zoni dolazi do pojave jednakih radijalnih i nejednakih transverzalnih pomeranja konstrukcije i tla. Na osnovu analitičkih izraza gore pomenutih autora, sračunate su maksimalne vrednosti normalne sile (T), transverzalne sile (V) i momenta savijanja (M) u tunelskoj konstrukciji kružnog poprečnog preseka.

Prema rezultatima sprovedenih istraživanja, analitički

conditions: *full-slip* (no shear stress transmission) and *no-slip* (no relative shear displacements). The seismic stress increment in the lining is accounted for in the closed-form solutions by analyzing the response of the cross-section to an imposed uniform strain field, induced by vertically propagating shear waves, using the pseudo-static approach.

The analysis procedures presented herein apply to tunnel linings having burial depths sufficiently large, so that the free-surface boundary condition at the top of the soil, as well as the bedrock at the bottom of the soil, have small effect on the soil–structure interaction. In addition, the solutions are applicable for cast-in-place concrete tunnel linings, or linings of a TBM driven (shield) tunnel that are assembled of precast concrete segments, and which are embedded in soil sites, whose values of elasticity modulus are lower in comparison with a modulus of a tunnel lining, i.e., for the case of tunnel lining in soft soils, when the soil–structure interaction effects are pronounced. The resulting soil stresses, which induce tunnel lining forces, cannot be tensile stresses, and they should not exceed the limiting values of the soil shear stress. For the case of tunnel linings attached with anchors to the surrounding ground, which have not been in the focus of the presented approaches, it should be noted that certain forces pulling the lining from the cavity toward the surrounding ground could be expected to appear. This would result not only in the shearing force distribution around the external lining surface, but also, to some extent, in the transmission of tensile strains throughout the surrounding medium.

In addition, the two presented approaches have been compared on the example of idealized tunnel geometry and ground properties. With regard to dependent nature of the soil–structure interaction effects upon the ratio between the relative stiffness of the ground with respect to the lining, in the given analyses representative of two main soil classes – stiff and soft soil deposits have been considered. Internal forces in the lining are calculated from the free-field shear strain obtained by the EERA code. The strain value considered is the average value between the depths corresponding to the crown and the invert of the tunnel. Under the hypothesis of the rough interface between the lining and the soil (assuming compatible displacements of the lining and the ground), as well as, *full-slip* condition (equal normal displacements and unequal tangential displacements of medium and lining at the common interface), the maximum values of thrust (T), shear force (V), and bending moment (M) are calculated using the expressions by the aforementioned authors.

According to the given study, the solutions of Penzien [14] result in values of thrust and moment that are very close to those of Wang [20] for *full-slip* condition and for both stiff and soft saturated soil deposits. Yet, the Penzien's solutions highly underestimate the thrust for all investigated soil conditions with the rough interface assumption. The reason for the underestimation of seismically induced lining thrust according to the Penzien's expression for the *no-slip* condition, in the authors' opinion, might be a nonexistence of the corresponding compressibility coefficient in the Penzien's expression, as it has an effect on the lining thrust response reflecting the circumferential stiffness of the system (resistance to

izrazi Penzien-a [14] i Wang-a [20], u uslovima smičućeg kontakta tunela i tla, dali su gotovo istovetna rešenja za maksimalne vrednosti seizmički indukovanih normalnih sila i momenata savijanja u tunelskoj konstrukciji, kako za slučaj čvrstog tla, tako i za slučaj zasićenog slabonosivog tla. S druge strane, rešenje prema Penzien-u u uslovima idealnog kontakta tunela sa okolnim tlom predviđa mnogo manje vrednosti normalnih sila u tunelskoj konstrukciji za oba razmatrana slučaja tla, u poređenju s rešenjem prema Wang-u. Razlog zbog kojeg Penzien-ov postupak rezultuje znatno manjim vrednostima seizmički indukovanih normalnih sila u tunelskoj oblozi za slučaj idealnog kontakta tunela sa okolnim tlom, prema mišljenju autora, mogao bi biti nepostojanje koeficijenta kompresibilnosti u analitičkom izrazu po Penzien-u, koji ima veoma važnu ulogu u određivanju normalnih sila, s obzirom na to što predstavlja krutost sistema tunel–tlo u tangencijalnom pravcu (otpornost na pritisak). Stoga se preporučuje da u proračunu seizmički indukovanih normalnih sila u tunelskoj konstrukciji ne treba koristiti analitički izraz prema Penziju za slučaj idealnog kontakta tunelske konstrukcije i okolnog tla.

S ciljem provere i eventualne potvrde gorenavedenog uzroka odstupanja ova dva rešenja u pogledu seizmički indukovanih normalnih sila u tunelskoj konstrukciji za slučaj idealnog kontakta tunela i okolnog tla, trebalo bi sprovesti dalja istraživanja, što se može postići kompleksnim i sveobuhvatnim numeričkim analizama, primenom metode konačnih elemenata, a što predstavlja tekuću istraživačku aktivnost autora.

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compression). For that reason, it is recommended that in estimation of the seismically induced tunnel lining thrust the analytical expression after Penzien for the no-slip condition should not be used.

However, the above stated remark should be investigated further by comprehensive numerical finite element analyses, which is the ongoing research activity of the authors of this paper.

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REZIME

PRIKAZ ANALITIČKIH POSTUPAKA ODREĐIVANJA SEIZMIČKI INDUKOVANIH UTICAJA U TUNELSKOJ KONSTRUKCIJI PRI INTERAKCIJI KONSTRUKCIJE I OKOLNE SREDINE

Elefterija ZLATANOVIĆ
Dragan Č. LUKIĆ
Vlatko ŠEŠOV

U novije vreme, uporedo sa uvođenjem savremenog pristupa projektovanja na bazi ponašanja konstrukcija (*performance design*), aseizmičko projektovanje iz aspekta interakcije konstrukcije i tla dobija na sve većoj važnosti. U analizi dinamičkog odgovora tunelskih objekata, efekti interakcije konstrukcije i tla imaju veoma važnu ulogu. Efekti sadejstva konstrukcije sa okolnim tlom veoma često mogu rezultovati znatno većim opterećenjima kojima je tunelska konstrukcija izložena. Rezultati brojnih studija ukazuju na to da je odnos krutosti tunelske konstrukcije i krutosti tla najdominantniji faktor koji utiče na sadejstvo tunela sa okolnim tlom. S ciljem istraživanja efekata interakcije tunelske konstrukcije i tla, u radu su sprovedene analize koje su se bazirale na primeni analitičkih izraza za proračun seizmički indukovanih presečnih sila u tunelskoj konstrukciji iz aspekta sadejstva tunela i tla. Analitička rešenja priraštaja napona u tunelskoj konstrukciji usled dejstva zemljotresa u funkciji su smičuće deformacije tla, koja uzrokuje ovalizaciju tunela kružnog poprečnog preseka. Vrednosti presečnih sila u tunelskoj konstrukciji prema analitičkim izrazima sračunate su na osnovu prosečne vrednosti smičuće deformacije tla na delu tunelskog profila, između temena tunelskog i podnožnog svoda, koja je određena seizmičkom analizom odgovora tla u softveru EERA. Analizirana su dva karakteristična slučaja tla – čvrsto tlo dobrih karakteristika i vodozasićeno tlo slabih karakteristika, kao i oba ekstremna slučaja kontaktnih uslova tunelske konstrukcije sa okolnim tlom – slučaj smičućeg kontakta i slučaj idealnog kontakta tunela i tla. Komparativnom analizom rezultata za sve navedene slučajeve sagledani su najznačajniji faktori sadejstva tunela i tla u seizmičkim uslovima.

Ključne reči: kružni tunelski profil, čvrsto tlo, slabonosivo vodozasićeno tlo, zemljotres, analitička rešenja, sile u tunelskoj konstrukciji, interakcija tunela i tla, idealni kontakt, glatki kontakt

SUMMARY

PRESENTATION OF ANALYTICAL SOLUTIONS FOR SEISMICALLY INDUCED TUNNEL LINING FORCES ACCOUNTING FOR SOIL-STRUCTURE INTERACTION EFFECTS

Elefterija ZLATANOVIĆ
Dragan C. LUKIĆ
Vlatko SESOV

Recently, as the structural design has shifted to the performance design, seismic design of tunnel structures considering soil-structure interaction becomes more important. The effects of soil-structure interaction should not be overlooked for the reason that the interaction effects between a structure and surrounding ground may cause larger external forces to the structure. It has been highlighted that the relative rigidity between the soil and the structure is the predominant factor influencing the soil-structure interaction effects. With an aim to study the effects of tunnel-ground interaction, a number of analyses were carried out in this work, based on the most frequently used analytical expressions for evaluation of seismically induced stress increment in a tunnel lining accounting for the soil-structure interaction effects. These solutions are functions of the shear strain field which is the cause of the ovaling of the circular tunnel cross-section. A value of the average soil shear strain in the range of depths corresponding to the tunnel section, between the tunnel crown and the invert, has been computed through a free-field one-dimensional seismic site response analysis performed by the code EERA. Various levels of analysis have been undertaken on different soil conditions, considering representative of two main soil classes – stiff soil of good conditions and soft saturated soil of poor conditions, as well as, two extreme cases of tunnel-ground interface – the full-slip and the no-slip conditions. Finally, the results for all the considered cases have been evaluated and compared, and the significant mutual differences with regard to a tunnel-ground interaction have been underlined.

Keywords: circular tunnel, stiff soil, soft saturated soil, earthquake, analytical solution, internal lining forces, soil-structure interaction, no-slip condition, full-slip condition

HARMONIZACIJA EVROPSKOG KVALITETA KOLOSEKA

HARMONIZATION OF EUROPEAN TRACK QUALITY

Zdenka POPOVIĆ
Filip TRPČEVSKI
Isidora PANČIĆ
Luka LAZAREVIĆ

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1 UVOD

Geometrija koloseka može da se opiše kao projektovana (nominalna) geometrija i neregularnosti (odstupanja) aktuelne geometrije koloseka. Projektovana geometrija koloseka je trodimenzionalna geometrija izražena kao horizontalna i vertikalna projekcija. Horizontalna projekcija je projekcija koloseka na horizontalnu ravan i sastoji se od pravaca, kružnih krivina i prelaznih krivina. Vertikalna projekcija je projekcija koloseka na vertikalnu ravan i sastoji se od uniformnih gornjih površi glave šine u vertikalnoj ravni duž koloseka. To znači da se vertikalna projekcija sastoji od vertikalne krivine, podužnog nagiba koloseka i nadvišenja šine u koloseku.

Projektovana geometrija koloseka pruža tačne, prostorne podatke u globalnom koordinatnom sistemu koji su neophodni za polaganje koloseka, obnovu ili održavanje. Tokom eksploatacije koloseka degradacija geometrije koloseka se manifestuje kao razvoj neregularnosti geometrije koloseka. Ovaj proces je normalni deo životnog ciklusa koloseka. S obzirom na to da stvarna geometrija koloseka direktno utiče na kvalitet vožnje šinskih vozila, precizno merenje geometrije koloseka je odlučujuće za procenu bezbednosti saobraćaja. Geometrija koloseka treba da se meri i snima periodično na osnovu očekivanog propadanja geometrije koloseka. Svako geometrijsko odstupanje prostornog položaja koloseka mora biti procenjeno i korigovano na

1 INTRODUCTION

The track geometry can be described by the designed (nominal) geometry and irregularities (deviation) of the actual track geometry.

The designed track geometry is three-dimensional geometry, which is usually expressed in two separate layouts - horizontal and vertical. The horizontal layout is the track layout in the horizontal plane and it involves straight lines, curves, and track transition curves. The vertical layout is the track layout in the vertical plane and it involves the rail surface uniformities in the vertical plane along the track. So, the vertical layout consists of vertical curvature, gradient (i.e. slope) of track and cant.

The designed track geometry provides accurate spatial data in global coordinate system that is necessary for track laying, track renewal or maintenance works.

During the track exploitation, the deterioration of the track geometry is manifested as development of track geometry irregularities. This process is a common part of track life cycle. Since the actual track geometry directly influences the ride quality of a rail vehicle, an accurate measurement of track geometry is decisive for the evaluation of operating safety. The track geometry shall be measured and recorded periodically according to the expected deterioration of track geometry. Each defect of the track geometry should be assessed and corrected according to its size and wavelength. The objective is to

V. prof. dr Zdenka Popović, dipl.inž.građ., Građevinski fakultet, Bulevar kralja Aleksandra 73, Beograd, Srbija
zdenka@grf.bg.ac.rs
Asist. Luka Lazarević, master inž.građ., Građevinski fakultet, Bulevar kralja Aleksandra 73, Beograd, Srbija
Isidora Pančić, dipl.inž.građ.
Filip Trpčevski, master inž.građ.

Assoc. professor Zdenka Popović, BCE, University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, Belgrade, Serbia, zdenka@grf.bg.ac.rs
Asist. Luka Lazarević, MSc. University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, Belgrade, Serbia
Isidora Pančić, BCE
Filip Trpčevski, master BCE

osnovu njegove vrednosti i talasne dužine. Cilj je da se obezbedi udobnost vožnje sa prihvatljivim troškovima održavanja i da se spreči propadanje koloseka u uslovima koji mogu da dovedu do isključujućih, ugroženosti slobodnog profila, povreda zbog neravnomerne vožnje i sl.

Za definisanje kvaliteta geometrije koloseka neophodno je odabrati i kombinovati značajne geometrijske parametre koloseka. Nakon toga neophodno je odrediti prag bezbednosti uzimajući u obzir aspekt interakcije koloseka i vozila.

Ovaj rad razmatra parametre i metodologiju za procenu kvaliteta geometrije koloseka u skladu sa evropskim standardima.

2 PRAVNI OKVIR ZA DEFINISANJE GEOMETRIJE KOLOSEKA U REPUBLICI SRBIJI

Železnička uprava u Republici Srbiji ima interes da podigne svoje kapacitete na viši tehnički i tehnološki nivo i da obezbedi pouzdano korišćenje infrastrukture od strane svih evropskih saobraćajnih operatera [10,11]. Uzimajući u obzir da je teritorija Republike Srbije značajan deo železničkog Koridora X (30.89%), neophodna je harmonizacija parametara magistralnih železničkih pravaca u Srbiji sa parametrima evropske železničke mreže, korišćenjem savremene tehničke regulative za projektovanje i održavanje infrastrukture. Slika 1 prikazuje aktuelno stanje harmonizacije podzakonskih akata za održavanje železničke infrastrukture u Srbiji.

Tehnički uslovi interoperabilnosti (TSI) za trans-evropski konvencionalni železnički sistem – Podsystem "Infrastruktura" obuhvata održavanje sistema konvencionalne železnice sa aspekta bezbednosti, pouzdanosti, dostupnosti, zdravlja, zaštite životne sredine i tehničke kompatibilnosti opreme za održavanje konvencionalnog voznog parka [7]. Tehnička oprema i procedure koje se koriste tokom održavanja moraju da obezbede bezbedno funkcionisanje podsistema "Infrastruktura" i da ne prouzrokuju opasnost po zdravlje i bezbednost. Takođe, uticaj tehničke opreme i procedura ne sme da prekorači nivo negativnih uticaja na životnu sredinu. U skladu sa [7, 12] Upravljač infrastrukture mora da definiše plan održavanja za svaku konvencionalnu železničku prugu, koji se zasniva na ograničenjima koja zavise od brzine. Osnova za izradu plana održavanja su izmereni podaci sa železničke mreže. U tom smislu podaci sakupljeni na mreži dobijeni korišćenjem mernih vozila imaju veliki značaj. Tokom merenja se registruju podaci koji prekoračuju propisane granične vrednosti tako da je moguće, ukoliko je to potrebno, preduzeti odgovarajuće mere za osiguranje bezbednog saobraćaja. Nacionalne granične vrednosti (pragove) mora da definiše Upravljač infrastrukture. One su osnova tehničke regulative u oblasti održavanja koloseka.

Slično kao TSI, evropski standardi serije EN 13848 (delovi 1-6) [1-6] su međunarodna direktiva. Za sada, evropski standard [6] postoji samo kao nacrt.

Evropski standardi EN 13848 (delovi 1-5) [1-5] su publikovani od strane Instituta za standardizaciju Srbije kao SRPS EN (delovi 1-5) [10, 13] (slika 1). Stručni rad Instituta za standardizaciju Srbije u oblasti standardizacije "Primene na železnici" se spovodi u okviru tehničke komisije P256. Uzimajući u obzir

provide ride comfort with acceptable maintenance costs and to prevent the track deterioration into a condition which may cause derailments, unacceptable clearances, a risk of injuries due uneven rides, etc.

To define track geometry quality, it is necessary to choose and combine significant geometrical parameters of the track. Thereafter, it is necessary to determine thresholds for safety, taking into account the track/vehicle interaction aspects. This paper discusses the parameters and methodology for assessing the quality of track geometry according to European standards.

2 THE LEGAL FRAMEWORK FOR OF TRACK GEOMETRY IN THE REPUBLIC OF SERBIA

The railway administration in the Republic of Serbia is interested in developing its capacity to a higher technical and technological level and thus, ensures reliable operation of infrastructure facilities by all European operators [10,11]. Given the fact that the territory of the Republic of Serbia is an important part of the railway Corridor X (30.89%), it is necessary to harmonize the parameters of the main rail lines in Serbia with parameters of European railway network, using modern technical regulations for the design and maintenance of infrastructure. Figure 1 shows the current status of harmonization of subordinate acts for maintenance of railway infrastructure in Serbia.

The Technical Specification of Interoperability (TSI) relating to the trans-European conventional rail system - Subsystem "Infrastructure", covers the conventional railway system maintenance from the aspect of safety, reliability and availability, health, environmental protection and technical compatibility of the maintenance installations for conventional rolling stock [7]. Technical installations and procedures used during maintenance activities should ensure the safe operation of the subsystem "Infrastructure" and exclude health and safety hazards. Also, the influence of technical installations and procedures should remain in the permissible levels of nuisance regarding the local environment. According to [7] and [12], the Infrastructure Manager has to define, for each conventional rail line, a maintenance plan based on speed-related limits. The basis for creation of the maintenance plan is data measured on the railway network. In that way, the data collected from the network using track recording vehicles have great importance. During measurements, exceeding of the regulated limit values is registered and, if necessary, appropriate measures for traffic safety insurance can be taken. National limit values (thresholds) have to be defined by the Infrastructure Manager. They are the basis for technical regulations in the field of track maintenance.

Similar to the TSI, European standard EN 13848 Series (Parts 1-6) [1-6] are international directives. For now, European standard [6] exists only as a draft version.

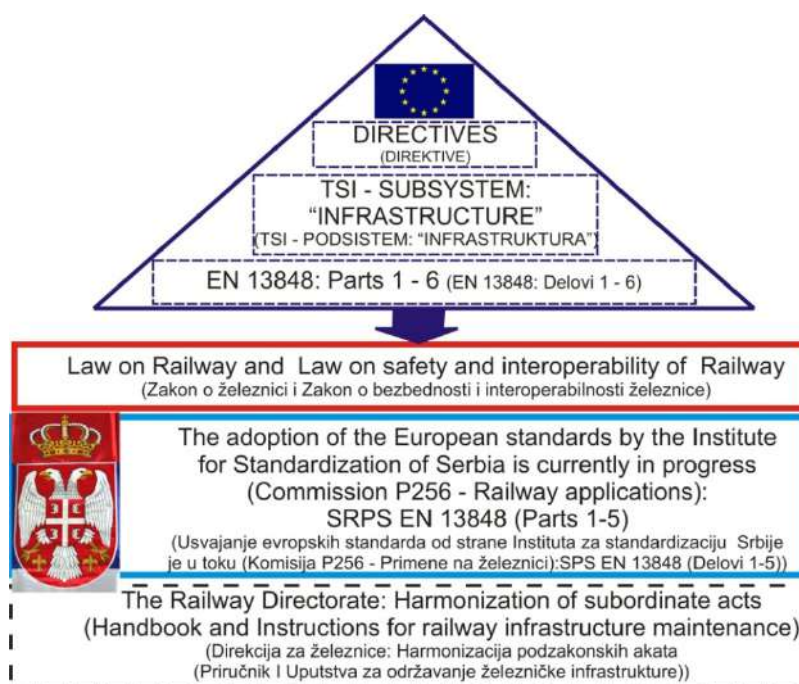
European standard EN 13848 Series (Parts 1-5) [1-5] are published by Institute for Standardization of Serbia as SRPS EN 13848 (Parts 1-5) [10,13] (Figure 1). Professional work of the Institute for Standardization of Serbia in the field "Railway Applications" is conducted within the Technical Committee P256. Considering the

kompleksnost oblasti "Primene na železnici", tehnička komisija ima dve potkomisije: za građevinski i mašinski deo. Tehnička komisija P256 okuplja predstavnike univerziteta (Građevinski i Mašinski fakultet u Beogradu), Direkcije za železnice, projektante, izvođače i predstavnike proizvođača u oblasti železnice. Ovakva struktura odgovara strukturi tehničke komisije za oblast "Primena na železnici" prema iskustvima Evropske Unije. Ipak, praktična primena usvojenih standarda SRPS EN 13848 (delovi 1-5) biće moguća na Železnicama Srbije tek nakon njihovog povezivanja sa tehničkom regulativom.

Direkcija za železnice je formirana u skladu sa Zakonom o Železnici 2005. godine da pruži profesionalne usluge u oblasti železničkog saobraćaja, propise i druge poslove utvrđene ovim zakonom. Jedan od zadataka Direkcije je da donosi tehničku regulativu u oblasti železničkog saobraćaja, kao i da predlaže mere za harmoizaciju i podizanje nivoa interoperabilnosti i modernizacije železnice u Republici Srbiji (slika 1).

complexity of the field "Railway Applications", the Technical Committee has two subcommittees: one for civil engineering and another for mechanical engineering. The Technical Committee P256 brings together representatives of the University (the Faculty of Civil Engineering and the Faculty of Mechanical Engineering in Belgrade), the Railway Directorate, designers, contractors and industry in the field "Railway Applications". According to the experience of the European Union, this structure of the Committee corresponds to the structure of Standardization committee in the field "Railway Applications". However, practical application of adopted standard SRPS EN 13848 Series (Parts 1-5) on Serbian railways will be possible only after complying with technical regulations.

The Railway Directorate was established by the Railway Act in 2005, in order to provide professional services in the field of railway transport, regulations and other tasks determined by this act. One of the tasks of the Directorate is drafting technical regulations in the field of railway transport, as well as suggesting the measures to harmonize and increase the level of interoperability and modernization of the railway in the Republic of Serbia (Figure 1).



Slika 1. Procedura za harmonizaciju tehničke regulative za održavanje železničke infrastrukture u Republici Srbiji
Figure 1. The procedure of harmonization of technical regulations for railway infrastructure maintenance in the Republic of Serbia

Stoga, TSI "Tehnički uslovi interoperabilnosti", Plan održavanja i evropski EN 13848 (delovi 1-6) obezbeđuju pravni okvir za snimanje geometrije koloseka kao dela modernog održavanja.

Evropski standard [1] određuje "uslove za homologaciju parametara kvaliteta geometrije koloseka merenih uređajima koji su instalirani u mernim kolima". Ovaj evropski standard definiše glavne parametre geometrije koloseka. On određuje uslove za merenje,

Therefore, TSI "Technical Specifications for Interoperability", Maintenance Plan and European standard EN 13848 Series (Parts 1-6) provide the legal framework for track geometry recording as a part of modern maintenance.

European standard [1] specifies "the requirements for the homologation of track geometry quality parameters as measured by measuring devices fitted on track recording vehicles". This European standard

metode analize i predstavljanje rezultata.

Evropski standard [2] određuje "minimalne uslove za principe i sisteme merenja sa ciljem da se ostvare uporedivi rezultati i definiše uslove za merenja".

Evropski standard [3] određuje "minimalne uslove koje treba da ispune merni sistemi postavljeni u mašine za građenje i održavanje koloseka kako bi se izvršila procena kvaliteta geometrije koloseka merenjem jednog ili više parametara koji su opisani EN 13848-1".

Evropski standard [4] određuje "minimalne uslove koje treba da ispune merni sistemi postavljeni u kolica za merenje geometrije koloseka i ručni merni uređaji kako bi se izvršila procena kvaliteta geometrije koloseka merenjem jednog ili više parametara koji su opisani u EN 13848-1". Treba imati na umu da korišćenjem manuelnih metoda merenja nije moguće meriti geometriju koloseka pod opterećenjem.

Evropski standard [5] definiše "minimum uslova za nivo kvaliteta geometrije koloseka i određuje granične vrednosti sa aspekta bezbednosti za svaki parametar definisan u EN 13848-1". Ovaj standard uključuje: opis nivoa kvaliteta, relativni značaj parametara, granicu preduzimanja hitnih aktivnosti i razmatranje drugih nivoa kvaliteta.

Nacrt evropskog standarda [6] pokriva sledeće teme: opis kvaliteta geometrije koloseka (postojeće stanje), klasifikaciju kvaliteta koloseka u skladu sa parametrima geometrije koloseka [1] i razmatranje na koji način ova klasifikacija može da se primeni.

3 GLAVNI PARAMETRI GEOMETRIJE KOLOSEKA

U skladu sa [1], glavni parametri geometrije koloseka su:

- širina koloseka,
- vertikalno odstupanje,
- odstupanje smera,
- nadvišenje i
- vitopernost.

Standard [1] definiše svaki od pomenutih parametara i daje uslove za merenje, metode analize i prezentaciju rezultata.

Takođe, standard [1] definiše kvalitet geometrije koloseka kao ocenu odstupanja od prosečnih ili projektovanih karakteristika propisanih parametara u vertikalnoj i poprečnoj ravni, koji utiču na bezbednost ili su u korelaciji sa kvalitetom vožnje.

Kvalitet geometrije koloseka se opisuje korišćenjem relativnog pravouglog koordinatnog sistema koji je centriran u koloseku i orjentisan u smeru kazaljke na satu. Slika 2 prikazuje odnos osa u koordinatnom sistemu XYZ u skladu sa [1]:

- X - osa je produžetak ose koloseka u smeru vožnje,
- Y - osa je paralelna ravni dodira točkova osovin-skog sklopa i šina u koloseku (u daljem tekstu ravan dodira),
- Z - osa je upravna na ravan dodira i usmerena na dole.

Dalje, slika 2 prikazuje princip rada sistema za

definiše principal track geometric parameters. It specifies the requirements for measurement, the analysis methodology and presentation of obtained results.

European standard [2] specifies "the minimum requirements for measuring principles and systems in order to produce comparable results and it defines the requirements for measurement".

European standard [3] specifies "the minimum requirements that shall be met by measuring systems fitted on track construction and maintenance machines to give an evaluation of track geometry quality when measuring one or more of the parameters described in EN 13848-1".

European standard [4] specifies "the minimum requirements that shall be met by measuring systems fitted on track geometry measuring trolleys and manually operated devices to give an evaluation of track geometry quality when measuring one or more of the parameters described in EN 13848-1". It should also be born in mind that by using the manual measuring method it is impossible to measure the track geometry under load.

European standard [5] defines "the minimum requirements for the quality levels of track geometry, and specifies the safety related limits for each parameter defined in EN 13848-1". This standard includes: description of quality levels, relative importance of parameters, immediate action limit and considerations on other quality levels.

Draft version of European standard [6] covers the following topics: description of track geometry quality (state of the art), classification of track quality according to track geometric parameters [1], and considerations on how this classification can be used.

3 PRINCIPAL TRACK GEOMETRIC PARAMETERS

According to [1], principal track geometric parameters are:

- track gauge,
- longitudinal level,
- alignment,
- cross level, and
- twist.

Standard [1] defines each mentioned parameter and specifies the requirements for measurement, the analysis methodology and presentation of obtained results.

In addition, standard [1] defines the track geometry quality "as an assessment of excursions from the mean or designed geometrical characteristics of specified parameters in the vertical and lateral planes which give rise to safety concerns or have a correlation with ride quality".

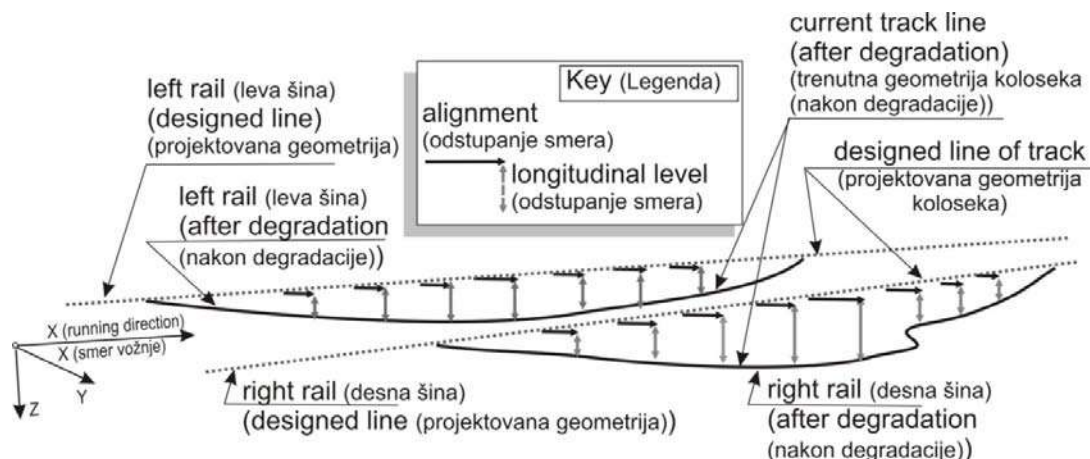
The track geometry quality is described by using a relative rectangular coordinate system centred to the track with clockwise rotation. Figure 2 shows relationship between the axes of the track co-ordinate system XYZ in accordance to [1]:

- X - axis is an extension of the track axis towards the direction of running,
- Y - axis is parallel to the running surface, and
- Z - axis is perpendicular to the running surface and it points downwards.

Furthermore, Figure 2 shows a working principle of

merenje geometrije koloseka u skladu sa serijom evropskih standarda EN 13848 [9].

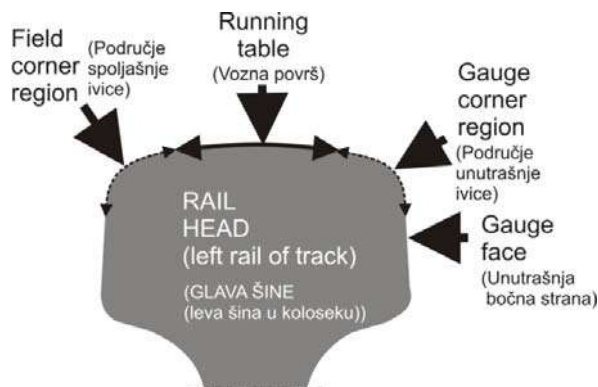
track geometry measuring system corresponding to the European standard EN 13848 Series [9].



Slika 2. Princip rada sistema za merenje geometrije koloseka i koordinatni sistem u koloseku
Figure 2. Working principle of track geometry measuring system and track coordinate system

Dodirna površ je definisana u [1] kao zakrivljena površ koja se generiše podužnim pomeranjem prave upravne na osu koloseka i tangira vozne površi obe glave šine. Vozna površ je gornja površ glave šine, kao što prikazuje slika 3.

The running surface is defined in [1] as a curved surface generated by the longitudinal displacement of a straight line perpendicular to the centre-line of the track and tangential to both running tables. Running table is upper surface of the rail head, as shown in Figure 3.



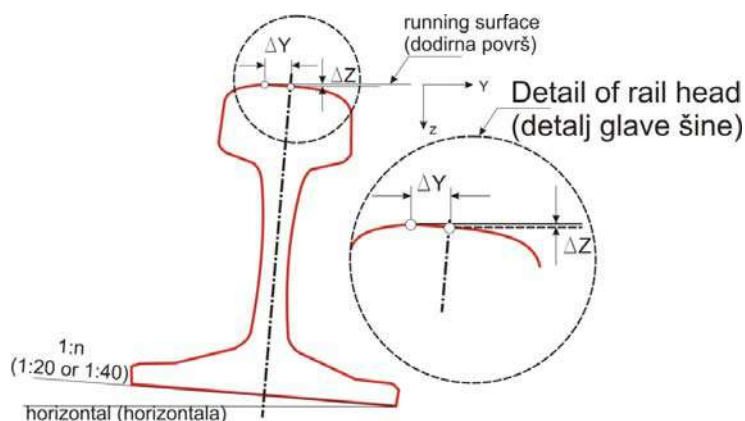
Slika 3. Gornja površ glave šine
Figure 3. Upper surface of the rail head

Koordinate ΔY i ΔZ (slika 4) određuju položaj dodirne tačke vozne površi i dodirne ravni. Ove koordinate zavise od tipa i nagiba šine u poprečnoj ravni (inklinacija) (tabela 1).

The coordinates ΔY and ΔZ (Figure 4) determine the position of the contact point between the running surface and running table. These coordinates depend on the type and inclination of rail (Table 1).

Tabela 1. Koordinate ΔY i ΔZ dodirne tačke vozne površi na glavi šine
Table 1. Coordinates ΔY and ΔZ of the contact point on running table of rail head

Inclination (inklinacija)	Coordinates ΔY and ΔZ (koordinate ΔY i ΔZ)	Rail type 60 E1 (tip šine 60 E1)	Rail type 49 E1 (tip šine 49 E1)
1:20	ΔY [mm]	11.6874	14.9812
	ΔZ [mm]	0.3496	0.3743
1:40	ΔY [mm]	7.4977	7.4977
	ΔZ [mm]	0.0937	0.0937



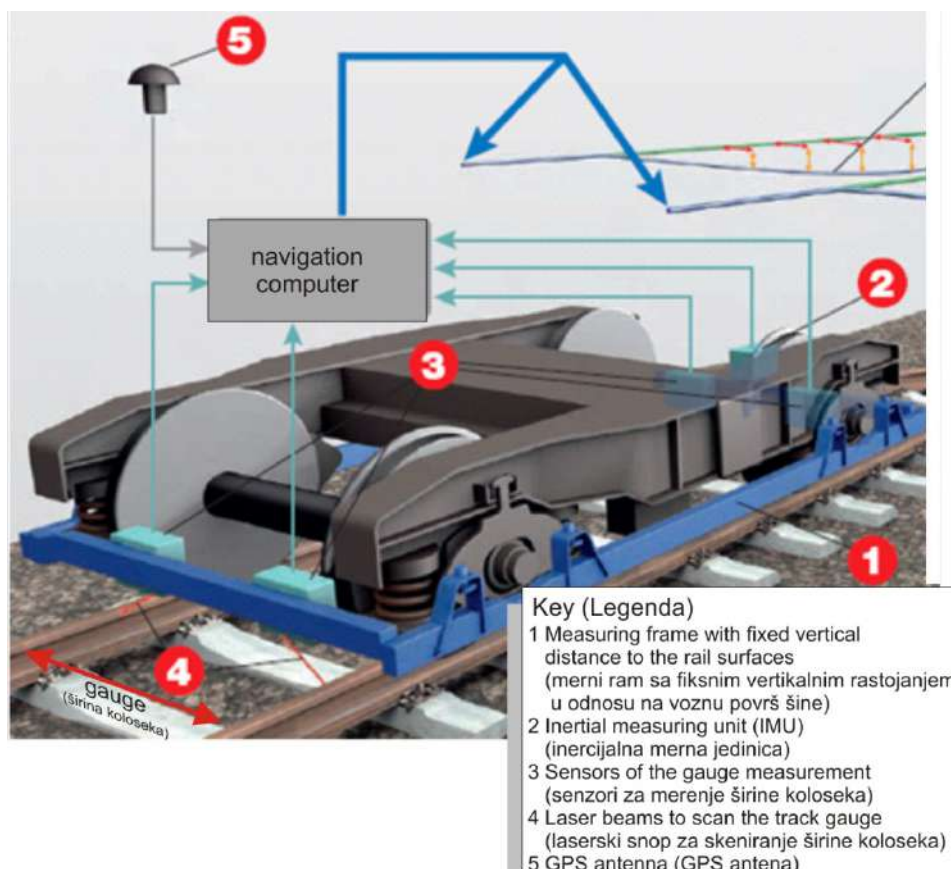
Slika 4. Položaj dodirne tačke između dodirne površi i vozne površi na glavi šine
 Figure 4. Position of the contact point between the running surface and running table

3.1 Širina koloseka

U skladu sa [1], "širina koloseka G je najmanje rastojanje između pravih koje su upravne na dodirnu površ i dodiruju obe glave šine u tački P , koja se nalazi na unutrašnjoj bočnoj površi glave, u zoni 0 do $z_p=14$ mm ispod dodirne površi" (slika 5). Širina koloseka je određena primenom formule (1).

3.1 Trak gauge

According to [1], "track gauge G is the smallest distance between lines perpendicular to the running surface intersecting each rail head profile at gauge face point P in a range from 0 to $z_p=14$ mm below the running surface" (Figure 5). Track gauge is determined using Equation (1).



Slika 5. Širina koloseka za nove šine i sistem merenja bez dodira [1, 8]
 Figure 5. Track gauge for new rails and non contact measuring system [1, 8]

$$G = G(x) = y_{p_2}(x) - y_{p_1}(x), \quad (1)$$

gde je:

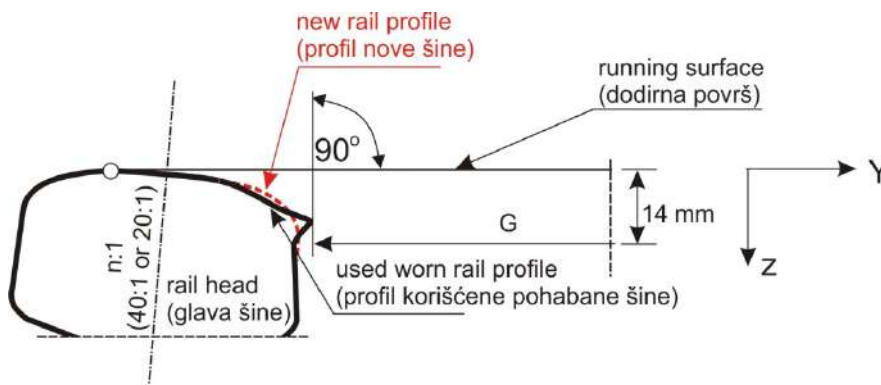
P_1, P_2 – dodirne tačke (slika 5) na unutrašnjoj bočnoj površi glave šine (slika 3).

Isti princip se primenjuje za širinu koloseka u krivini sa projektovanim nadvišenjem. Slika 6 prikazuje postupak za određivanje širine koloseka kada se koristi pohabana šina.

where:

P_1, P_2 – gauge face (inside face of the rail head, Figure 3) contact points (Figure 5).

The same principle applies to the track gauge in a curve with the cant. Figure 6 shows the procedure for determining the gauge in the situation of used worn rail.



Slika 6. Širina koloseka sa šinama koje nisu nove
Figure 6. Gauge in track with used worn rail

3.2 Vertikalno odstupanje

U skladu sa [1], "vertikalno odstupanje je odstupanje $z_{p'}$ nivoa gornje površi glave bilo koje šine u z-pravcu, izraženo kao pomeranje u odnosu na srednji vertikalni položaj (referentna linija), koje pokriva talasne dužine navedene u tekstu koji sledi i koje se sračunava na osnovu uzastopnih merenja (duž koloseka)" (slika 7). Na taj način se vertikalno odstupanje može razmatrati kao odstupanje smera koloseka u vertikalnoj podužnoj ravni.

Potrebno je razmatrati tri opsega talasnih dužina:

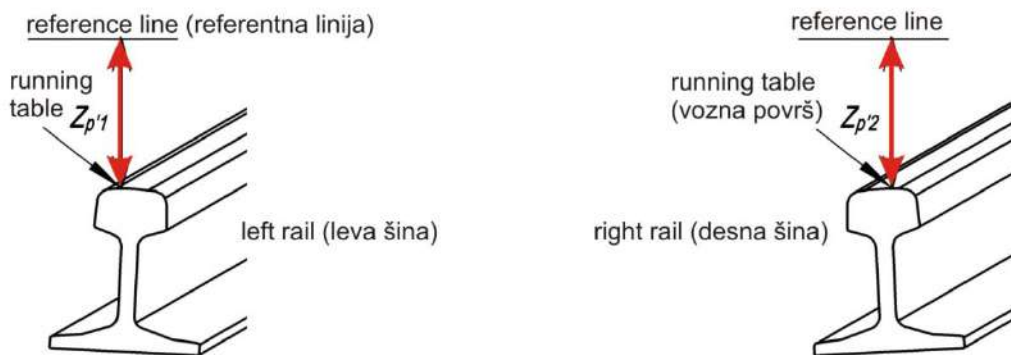
- interval talasnih dužina D1: $3 \text{ m} < \lambda \leq 25 \text{ m}$ (donju granicu za D1 treba spustiti na 1 m kako bi se detektovali defekti kratkih talasnih dužina: $1 \text{ m} < \lambda \leq 25 \text{ m}$),
- interval talasnih dužina D2: $25 \text{ m} < \lambda \leq 70 \text{ m}$, i
- interval talasnih dužina D3: $70 \text{ m} < \lambda \leq 150 \text{ m}$ (za defekte dugih talasnih dužina, brzina $> 250 \text{ km/h}$).

3.2 Longitudinal level

According to [1], "longitudinal level is deviation $z_{p'}$ in z-direction of consecutive running table levels on any rail, expressed as an excursion from the mean vertical position (reference line), covering the wavelength ranges stipulated below and is calculated from successive measurements" (Figure 7). So, longitudinal level can be considered as a vertical alignment.

Three wavelength ranges of longitudinal level shall be considered:

- wavelength range D1: $3 \text{ m} < \lambda \leq 25 \text{ m}$ (the lower limit of D1 should be reduced to 1 m in order to detect short wavelength defects: $1 \text{ m} < \lambda \leq 25 \text{ m}$),
- wavelength range D2: $25 \text{ m} < \lambda \leq 70 \text{ m}$, and
- wavelength range D3: $70 \text{ m} < \lambda \leq 150 \text{ m}$ (for long wavelength defects, speeds $> 250 \text{ km/h}$).

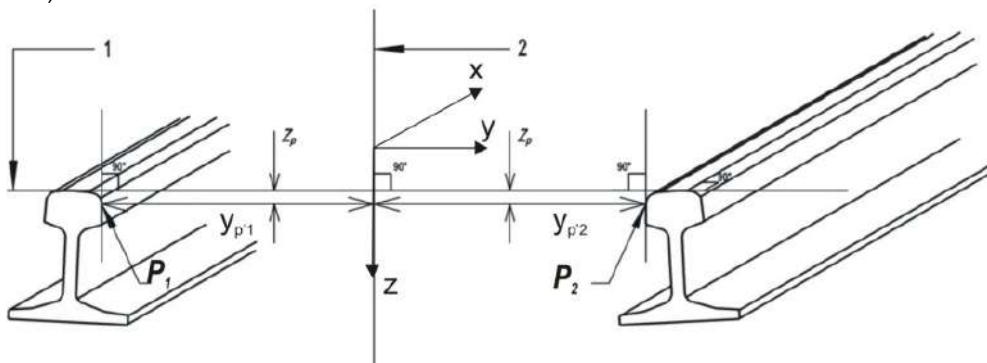


Slika 7. Merenje vertikalnog odstupanja
Figure 7. Measurement of longitudinal level

Interval talasnih dužina D1 u najvećoj meri utiče na kvalitet geometrije koloseka i ponašanje vozila. Uticaj dugačkih talasnih dužina D2 na propadanje geometrije koloseka obično nije veliki. Interval veoma dugačkih talasnih dužina D3 obuhvata greške geometrije koloseka koje su nastale usled grešaka projekta, polaganja ili održavanja koloseka.

3.3 Odstupanje smera koloseka

Odstupanje smera koloseka (bočni smer) je poprečno odstupanje obe šine mereno u tački koja se nalazi od 0 do 14 mm ispod dodirne površi. Standard [1] definiše "odstupanje smera kao odstupanje y_p uzastopne pozicije tačke P u smeru y - ose za bilo koju šinu, izraženo kao odstupanje u odnosu na srednji horizontalni položaj (referentna linija), koje pokriva talasne dužine navedene u tekstu koji sledi i koje se sračunava na osnovu uzastopnih merenja (duž koloseka)" (slika 8).

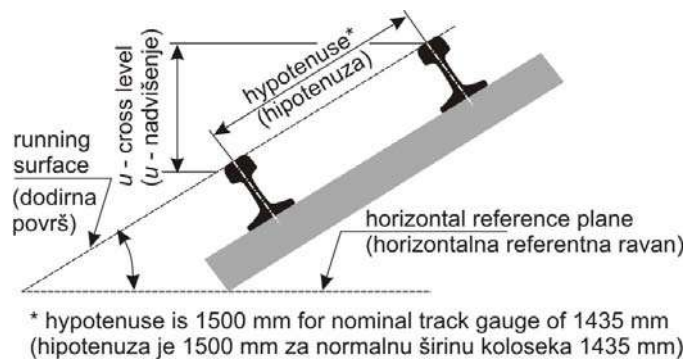


Slika 8. Odstupanje smera koloseka u skladu sa [1] (1- dodirna površ, 2- referentna linija)
Figure 8. Alignment according to [1] (1- running surface, 2- reference line)

Za odstupanje smera potrebno je razmatranje intervala talasnih dužina D1, D2 i D3.

3.4 Nadvišenje

Nadvišenje je definisano u [1] kao razlika visina naspramnih vozni površi glava šina i računa se na osnovu ugla dodirne površi i horizontalne referentne ravni (slika 9).



Slika 9. Nadvišenje
Figure 9. Cross level

Wavelength range D1 mostly affects the quality of track geometry and vehicles behaviour. Influence of the long wavelength range D2 on the deterioration of track geometry is usually small. Very long wavelength range D3 contains track geometry irregularities due to improper track design, track laying, or maintenance activities.

3.3 Alignment

Alignment (lateral alignment) is lateral deviation of both rails measured from a point between 0 to 14 mm below the running surface. Standard [1] defines "alignment as deviation y_p in y-direction of consecutive positions of point P on any rail, expressed as an excursion from the mean horizontal position (reference line) covering the wavelength ranges stipulated below and calculated from successive measurements" (Figure 8).

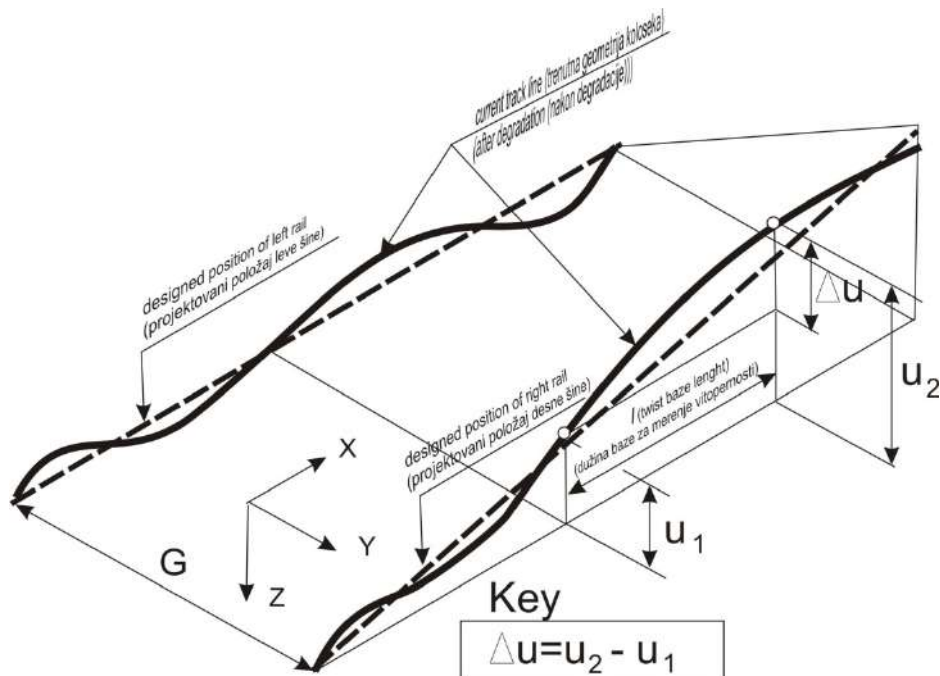
Wavelength ranges D1, D2 and D3 of alignment shall be considered.

3.4 Cross level

Cross level was defined in [1] as the difference in height of the adjacent running tables calculated from the angle between the running surface and a horizontal reference plane (Figure 9).

3.5 Vitopernost

Vitopernost je promena nadvišenja na datom rastojanju duž koloseka [1]. U skladu sa [1], vitopernost se definiše “kao algebarska razlika dva nadvišenja Δu na datom rastojanju l ”. Vitopernost je kritičan problem u prelaznim krivinama (slika 10).



Slika 10. Vitopernost u prelaznoj krivini
Figure 10. Twist in transition curve

3.5 Twist

Twist is the variation in cross level over a given distance along the track [1]. According to [1], twist was defined “as the algebraic difference between two cross levels Δu taken a given distance l ”. Twist is a critical issue in transition curves (Figure 10).

4 KVALITET GEOMETRIJE KOLOSEKA

Praktična i teorijska razmatranja su pokazala da svi parametri geometrije koloseka imaju uticaj na odziv vozila. Drugim rečima, da bi se definisao kvalitet geometrije koloseka, neophodno je izabrati i kombinovati značajne geometrijske parametre za kolosek i odrediti pragove za bezbednost uzimajući u obzir aspekt interakcije koloseka i vozila. Posledica toga je da bi neke kombinacije parametara geometrije koloseka mogle imati jak uticaj na odziv vozila.

4.1 Nivoi kvaliteta geometrije koloseka

EN standard [5] razmatra uticaje individualnih parametara geometrije koloseka (odstupanje visine, odstupanje smera, vitopernost i širina koloseka) koji su definisani u [1] na odziv vozila i definiše tri glavna nivoa kvaliteta geometrije koloseka kao funkciju brzine, kao što sledi: granica hitne intervencije (IAL), granica intervencije (IL) i granica upozorenja (AL). Prekoračenje propisane IAL – vrednosti zahteva preduzimanje mera za redukovanje rizika od iskliznuća na prihvatljiv nivo. Ove mere uključuju zatvor pruge, redukovanje brzine ili korigovanje geometrije koloseka.

4 TRACK GEOMETRY QUALITY

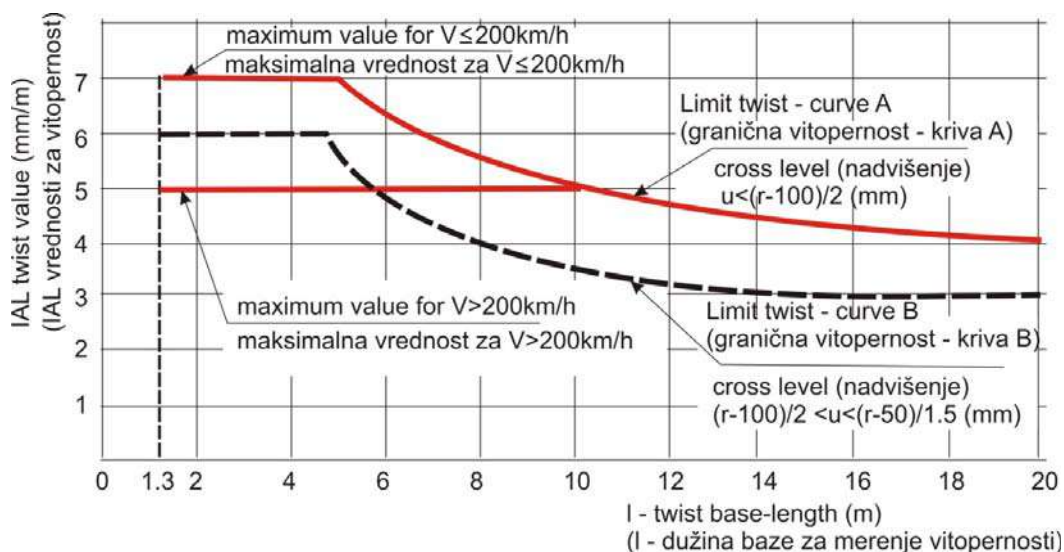
Experience and theoretical considerations have shown that all track geometry parameters have an influence on the vehicle response. In other words, to define track geometry quality, it is necessary to choose and combine significant geometrical parameters of the track and determine thresholds for safety taking into account the track/vehicle interaction aspects. Consequently, certain combinations of track geometry parameters may have a strong influence on the vehicle response.

4.1 Track geometric quality levels

EN standard [5] considers the influences of individual track geometry parameters (longitudinal level, alignment, twist and track gauge) which are defined in [1] on vehicle response and defines three main levels of the track geometric quality as a function of speed, as follows: Immediate Action Limit (IAL), Intervention Limit (IL) and Alert Limit (AL). Exceeding of normative IAL – values requires taking measures to reduce the risk of derailment to an acceptable level. These measures include closing the line, reducing speed or correction of track geometry.

Standard [5] ne daje IAL vrednosti za nadvišenje zato što je rizik u vezi nadvišenja tesno povezan sa vitopernošću i nedostatkom nadvišenja (slika 11).

Standard [5] fails to provide IAL values for cross level because the risk associated with a cross level defect is closely tied to twist and cant deficiency (Figure 11).



Slika 11. Vitopernost – IAL – izolovane greške [5]
Figure 11. Twist – IAL – Isolated defects [5]

Informativni aneks A standarda [5] razmatra uticaj parametara geometrije koloseka na odziv vozila i bezbednost (Tabela 2).

An informative Annex A of standard [5] considers the influence of track geometry parameters on vehicle response and safety (Table 2).

Tabela 2. Odnos odziva vozila i parametara geometrije koloseka [5]
Table 2. Relationship between vehicle response and track geometry parameters [5]

Vehicle response Odziv vozila (forces and accelerations) (sile i ubrzanja)	Track parameters according to EN 13848-1 [1] Parametri koloseka u skladu sa EN 13848-1 [1]			
	Track gauge Širina koloseka	Longitudinal level Vertikalno odstupanje	Twist/cross level Vitopernost/nadvišenje	Alignment Odstupanje smera koloseka
Lateral forces Y Bočne sile Y	●		●	●
Vertical forces Q Vertikalne sile Q		●	●	●
Lateral accelerations Bočna ubrzanja			●	●
Vertical accelerations Vertikalna ubrzanja		●		
Y/Q	●	●	●	●

Odnos Y/Q (Tabela 2) utiče na rizik iskliznuća u vezi sa penjanjem venca točka na glavu šine. Tabela 2 pokazuje da odnos bočne i vertikalne sile zavisi od svih parametara koloseka prema [1].

Treba napomenuti da Tehnički uslovi interoperabilnosti za podsistem "Infrastruktura" trans-evropskog konvencionalnog železničkog sistema [8] definiše, takođe, IAL-vrednosti za vitopernost, širinu koloseka, nadvišenje, ali ne i za odstupanje visine i

The ratio Y/Q (Table 2) influences the risk of derailment, corresponding to wheel flange climbing onto the rail head. Table 2 shows that relationship between lateral and vertical forces depends on all the track parameters according to [1].

It should be noted that Technical specification for interoperability relating to the subsystem "Infrastructure" of the trans-European conventional rail system [8] also sets IAL-values for twist, track gauge, cant, but not for

odstupanje smera. Dalje, TSI [8] definiše IAL-vrednosti za srednju širinu koloseka na dužini od 100 m za kontrolu ekvivalentne koničnosti.

Tabela koja sledi (Tabela 3) upoređuje granične vrednosti za širinu koloseka definisane u [5] i [8]. Tabela 3 prikazuje sledeći paradoks: TSI - vrednosti su strožije od EN - vrednost za IALmin. To je zato što su EN 13848-5: IAL vrednosti zasnovane na evropskoj anketi praktičnih iskustava o granicama bezbednosti.

longitudinal level and alignment. Further, TSI [8] sets IAL-values for mean track gauge over 100 m of track for controlling equivalent conicity.

The following table (Table 3) compares limit values of gauge defined in [5] and [8]. Table 3 shows the following paradox: TSI – values are stricter than EN-values for IALmin. It is because EN 13848-5 IAL values are based on a European survey of the practice in terms of safety limits.

Tabela 3. Poređenje IAL vrednosti za širinu koloseka u skladu sa EN 13848-5 [5] sa graničnim vrednostima prema TSI [8]

Table 3. Comparison of track gauge – IAL according to EN 13848-5 [5] with limit values according to TSI [8]

Speed Brzina [km/h]	Track gauge – IAL IAL za širinu koloseka Nominal to peak value of isolated defects according to EN 13848-5 [5] Vrednosti od nominalne do vršne vrednosti izolovane greške prema EN 13848-5 [5]		Track gauge – IAL IAL za širinu koloseka Nominal to peak value of isolated defects according to TSI [8] Vrednosti od nominalne do vršne vrednosti izolovane greške prema TSI [8]	
	Minimum [mm]	Maximum [mm]	Minimum [mm]	Maximum [mm]
V≤80	-11	+35	-9	+35
80<V≤120	-11	+35	-9	+35
120<V≤160	-10	+35	-8	+35
160<V≤200	-7	+28	-7	+28
200<V≤230				
230<V≤300	-5	+28		

EN 13848-5 utvrđuje informativne IL i AL - vrednosti koje su date u intervalu (min-max) za sve glavne parametre geometrije koloseka koji su definisani u [1]. Informativne IL i AL-vrednosti koje su definisane u [5] odražavaju uobičajenu praksu većine evropskih uprava infrastrukture. Ipak, svaki Upravljač infrastrukture treba da definiše sopstvene granične vrednosti za parametre koloseka na mreži uzimajući u obzir karakteristike gometrijskog oblika projektovane trase i način građenja, kao i karakteristike saobraćaja. Upravljač infrastrukture definiše politiku održavanja za svoju mrežu. Ta politika može biti usmerena:

- samo na osiguranje bezbednosti saobraćaja,
- na dostizanje dobrog kvaliteta vožnje i/ili manje troškove životnog ciklusa i/ili atraktivne usluge uz obavezno osiguranje bezbednosti u svakoj od pomenutih opcija.

Usvojena politika održavanja utiče na kvalitet vožnje vozila i dinamičko opterećenje koloseka, radove i troškove održavanja, kao i troškove održavanja vozila.

Glavni problem je što standard [5] razmatra izolovane greške bez razmatranja njihovog oblika, brzine promene, njihovog rasporeda (redosled i položaj), kombinacije defekata različitih tipova, projektovanu geometriju koloseka (horizontalna zakrivljenost, manjak ili višak nadvišenja), uticaj opterećenja od vetra, geometriju dodira točka i šine (uključujući ekvivalentnu koničnost), tip i konstrukciju vozila i strategiju održavanja. Nabrojani uticaji bi mogli da pogoršaju reakciju vozila na lokalne greške geometrije koloseka.

EN 13848-5 sets informative IL- and AL-values that are given as a range (min-max) for all principal track geometric parameters defined in [1]. Informative IL- and AL-values that were given in [5] reflect the common practice of the most European infrastructure managers. However, each infrastructure manager should set his own limit values of track parameters for network taking into account characteristics of alignment design and construction rules, as well as the characteristics of the traffic. The infrastructure manager establishes maintenance policy for his own network. This policy can be directed at:

- ensuring traffic safety alone, or
- achieving good ride quality, and/or lower life cycle costs, and/or more attractive services, all in addition to safety.

Established maintenance policy influences vehicle ride quality and dynamic loading of the track, track geometry maintenance works and costs, as well as vehicle maintenance cost.

The main issue is that standard [5] considers isolated defects without considering their shape, rate of change, sequence (order and location), combinations of defects of different type, designed track geometry (horizontal curvature, cant deficiency or cant excess), influence of wind loads, wheel-rail contact geometry (including equivalent conicity), type and condition of vehicles, and maintenance strategy. Listed influences may worsen the reaction of the vehicle on a local defect of track geometry.

4.2 Ocena kvaliteta geometrije koloseka

Neophodno je da se standardizuje procedura za ocenu kvaliteta geometrije koloseka širom evropske železničke mreže. Sve standardizovane procedure za ocenu moraju biti zasnovane na parametrima u skladu sa EN 13848-1 [1], zato što su izlazni rezultati merenja pomoću vozila za snimanje geometrije koloseka zasnovani na tim parametrima.

Nacrt standarda prEN 13848-6 [6] je u razvoju i definiše indeks kvaliteta (TQI) kao vrednost koja određuje kvalitet koloseka na deonici koloseka u skladu sa parametrima i metodama merenja u skladu sa serijom standarda EN 13848 [1-5]. Indeksi kvaliteta koloseka i brzina na deonici koloseka su osnova za definisanje klase kvaliteta koloseka (TQC).

Standardna devijacija je jedan od najrasprostranjenijih TQI koji se koristi na železničkoj mreži. Ona pokazuje raspodelu signala na deonici koloseka u odnosu na srednju vrednost signala na posmatranoj deonici. Standardna devijacija je mera rasipanja skupa izmerenih podataka (vrednosti signala) u odnosu na njihovu srednju vrednost. Ovaj indeks kvaliteta je definisan poznatom formulom (2).

$$SD = \sqrt{\frac{\sum_{i=1}^N (x_i - \bar{x})^2}{N-1}} \quad (2)$$

gde je:

SD – standardna devijacija,
 x_i – trenutna vrednost mernog signala,

\bar{x} – srednje vrednost signala,
 N – broj vrednosti.

Standardna devijacija se normalno koristi kao indeks kvaliteta za vertikalno odstupanje (interval talasnih dužina D1: $3 \text{ m} < \lambda \leq 25 \text{ m}$) i odstupanje smera (interval talasnih dužina D1). Pored toga, može da se koristi za vitopernost, širinu koloseka, nadvišenje, vertikalno odstupanje (interval talasnih dužina D2: $25 \text{ m} < \lambda \leq 70 \text{ m}$), i odstupanje smera (interval talasnih dužina D2).

Standardna devijacija za vertikalno odstupanje ili odstupanje smera se računa odvojeno za svaku šinu. To je preporučeni postupak. Pored toga, može se računati drugačije: srednja vrednost za obe šine, bolja ili gora šina, ili spoljna šina u krivini.

Takođe, broj grešaka po jedinici dužine koloseka se obično koristi kao indeks kvaliteta za sledeće parametre: odstupanje visine (interval talasnih dužina D1), odstupanje smera (interval talasnih dužina D1), vitopernost, širina koloseka i nadvišenje. Pored toga, može se koristiti za odstupanje visine (interval talasnih dužina D2) i odstupanje smera (interval talasnih dužina D2).

Jedan od indeksa kvaliteta TQI železničke mreže se može računati kao kombinacija standardnih devijacija individualnih geometrijskih parametara pomnoženih težinskim koeficijentima. Ovaj indeks kvaliteta se definiše formulom (3).

$$CoSD = \sqrt{w_{AL} \cdot SD_{AL}^2 + w_G \cdot SD_G^2 + w_{CL} \cdot SD_{CL}^2 + w_{LL} \cdot SD_{LL}^2} \quad (3)$$

4.2 Evaluation of track geometry quality

It is necessary to standardize the procedure for evaluation of track geometry quality across the European rail network. All standardized evaluation procedures should be based on the parameters according to EN 13848-1 [1], because the outputs of the track geometry recording vehicles are based on these parameters.

Draft version of standard prEN 13848-6 [6] is under development and defines a track quality index (TQI) as a value that characterizes the track quality of a track section according to parameters and measurement methods according to EN 13848 Series [1-5]. Track quality indices and speed on the track section are a basis for defining the track quality class (TQC).

The standard deviation is one of the most widely used TQIs of railway networks. It shows the distribution of the signal on the track section, in relation to the average value of the signal over the observed section. Standard deviation is the measure of dispersion of a set of measured data (values of the signal) from its mean value. This quality index is defined by well-known Equation (2).

where:

SD - standard deviation,
 x_i - the current value of the measured signal,

\bar{x} - the average value of the signal,
 N - number of measured values.

Standard deviation as track quality index is commonly used for longitudinal level (wavelength range D1: $3 \text{ m} < \lambda \leq 25 \text{ m}$) and alignment (wavelength range D1). Further, it can be used for twist, track gauge, cross level, longitudinal level (wavelength range D2: $25 \text{ m} < \lambda \leq 70 \text{ m}$), and alignment (wavelength range D2).

Standard deviation of longitudinal level or alignment is calculated separately for each rail. This is the recommended procedure. Furthermore, it can be calculated differently: by mean value of two rails, best or worst of the rails, or outer rail in curves.

In addition, the number of defects per unit of track length as track quality index is usually used for the following parameters: longitudinal level (wavelength range D1), alignment (wavelength range D1), twist, track gauge, and cross level. In addition, it can be used for longitudinal level (wavelength range D2) and alignment (wavelength range D2).

One of TQI values for railway network can be calculated as combination of weighted standard deviations of individual geometric parameters. This quality index is defined by Equation (3).

gde je:

$CoSD$ – kombinacija standardnih devijacija pomnoženih težinskim koeficijentima za svaki individualni geometrijski parametar (srednja vrednost odstupanja smera (AL) za obe šine, širina koloseka (G), nadvišenje (CR) i srednja vrednost odstupanja visine (LL) za obe šine),

w – težinski koeficijent za individualni geometrijski parametar.

Slično, standardna devijacija kombinacije parametara geometrije koloseka može da se koristi za ocenu kvaliteta geometrije koloseka. Ovo je zasnovano na opažanju da bi vrednost kombinovanog signala mogla bolje da odražava ponašanje vozila u odnosu na individualni signal.

Postupak ubrzanja materijalne tačke (PMA), analiza odziva vozila (VRA), direktno merenje odziva vozila i analiza gustina spektra snage (PSD) se takođe koriste kao TQI za železničku mrežu.

prEN 13848-6 definiše klase kvaliteta koloseka (TQC) kao standardnu devijaciju za odstupanje visine i odstupanje smera za pet intervala brzina: $V \leq 80$ km/h, 80 km/h $< V \leq 120$ km/h, 120 km/h $< V \leq 160$ km/h, 160 km/h $< V \leq 230$ km/h, 230 km/h $< V \leq 300$ km/h.

Za svaki interval brzina daje se pet klasa rangiranih A (dobar) do E (loš) za odstupanje visina i odstupanje smera.

U ovom standardu [6], definicije TQC-a su zasnovane na kumulativnoj raspodeli frekvencije prosečnih vrednosti pomnoženih težinskim koeficijentima za svaku od 13 mreža koje su učestvovala u evropskoj anketi kvaliteta. U evropskoj anketi kvaliteta koloseka učestvovala su sledeće mreže: Austrija (ÖBB-Infrastruktur), Belgija (Infrabel), Republika Češka (SŽDC), Danska (Rail Net Denmark), Finska (FTA), Francuska (SNCF / RFF), Nemačka (DB Netz), Italija (RFI), Holandija (ProRail), Portugalija (REFER), Švajcarska (SBB), Švedska (Trafikverket) i Velika Britanija (NetworkRail).

Pet TQC-a je definisano, kao što sledi:

- klasa A – najboljih 10% raspodele evropskog kvaliteta koloseka,
- klasa B - između 10% i 30% raspodele evropskog kvaliteta koloseka,
- klasa C - između 30% i 70% raspodele evropskog kvaliteta koloseka,
- klasa D - između 70% i 90% raspodele evropskog kvaliteta koloseka,
- klasa E – predstavlja najgorih 10% raspodele.

Slika 12 ilustruje metod za određivanje klase kvaliteta koloseka TQC, koji je zasnovan na TQI, u opštem slučaju. Referentni TQI se mogu odnositi na standardnu devijaciju za odstupanje smera ili standardnu devijaciju za odstupanje visine (slika 13).

Na slici 13 su takođe predstavljeni nivoi kvaliteta geometrije koloseka QN1 i QN2 koji su zasnovani na kriterijumima održavanja koloseka. U skladu sa [7], nivo kvaliteta QN1 zahteva osmatranje deonice koloseka ili preduzimanje mera održavanja u okviru normalnog plana rada, dok nivo kvaliteta QN2 zahteva primenu kratkoročnih mera održavanja. Propisane vrednosti za nivo kvaliteta geometrije koloseka QN2 su definisane u [7].

where:

$CoSD$ - combination of weighted standard deviations of individual geometric parameters (average value of alignment (AL) of both rails, gauge (G), cross level (CR) and average value of longitudinal level (LL) of both rails),
 w - weighted factor of the individual geometrical parameter.

Similarly, the standard deviation of the combination of track geometry parameters can be used for evaluation of track geometry quality. This is based on the observation that the size size of the combined signals may better reflect the vehicle behaviour than the individual signals.

Point mass acceleration process (PMA), vehicle response analysis (VRA), direct measurements of the vehicle response, and power spectral density (PSD) analysis are also used for TQIs of railway networks.

prEN 13848-6 defines Track Quality Classes (TQC) as standard deviation for both longitudinal level and alignment according to five speed ranges: $V \leq 80$ km/h, 80 km/h $< V \leq 120$ km/h, 120 km/h $< V \leq 160$ km/h, 160 km/h $< V \leq 230$ km/h, 230 km/h $< V \leq 300$ km/h.

For each speed range five classes are given, ranging from A (good) to E (poor) for both, longitudinal level and alignment.

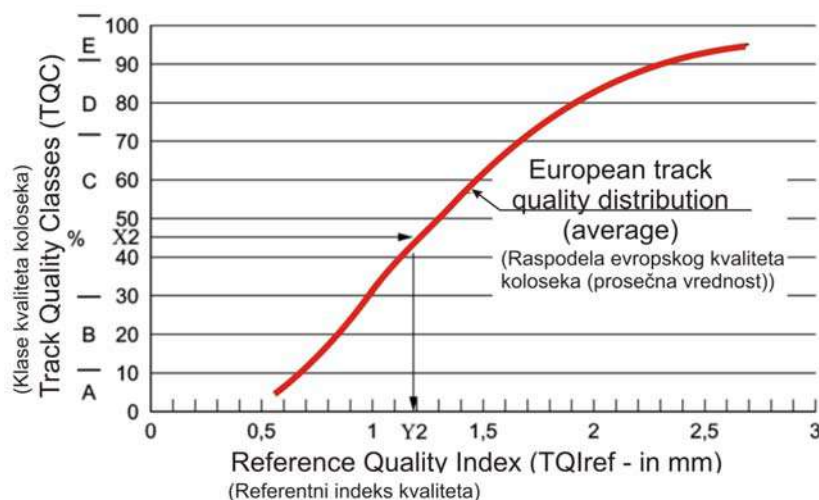
In this standard [6], the definitions of the TQCs are based on the cumulative distribution of frequency of the weighted average of 13 networks that have participated in the European track quality survey. Participating networks in European track quality survey were: Austria (ÖBB-Infrastruktur), Belgium (Infrabel), Czech Republic (SŽDC), Denmark (Rail Net Denmark), Finland (FTA), France (SNCF / RFF), Germany (DB Netz), Italy (RFI), Netherlands (ProRail), Portugal (REFER), Switzerland (SBB), Sweden (Trafikverket), and United Kingdom (NetworkRail).

The five TQCs are defined as follows:

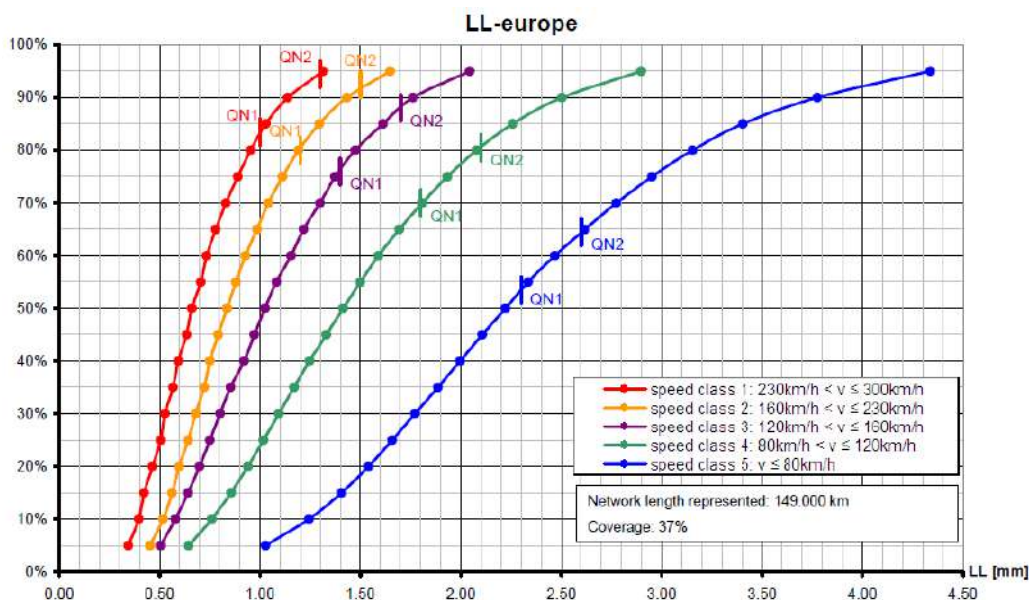
- Class A – the best 10% of the distribution of European track quality,
- Class B - between 10% and 30% of the distribution of European track quality,
- Class C - between 30% and 70% of the distribution of European track quality,
- Class D - between 70% and 90% of the distribution of European track quality,
- Class E - represents the worst 10% of the distribution.

Figure 12 illustrates the method of determining the track quality classes TQCs based on the reference TQI in the general case. Reference TQIs may refer to standard deviation for alignment or standard deviation for longitudinal level (Figure 13).

Figure 13 also presents track geometry quality level QN1 and QN2 based on track maintenance criteria. According to [7], quality level QN1 requires observation of a track section or taking maintenance measures within the frame of normal operations schedule, while quality level QN2 requires taking short-term maintenance measures. Values of track geometry quality level QN1 and QN2 to be respected are defined in [7].



Slika 12. Određivanje klasa kvaliteta koloseka TQC [6]
 Figure 12. Determination of the track quality classes TQCs [6]



Slika 13. Kumulativna raspodela frekvencije za standardnu devijaciju vertikalnog odstupanja
 Figure 13. Longitudinal level – cumulative distribution of frequency of standard deviation

5 ZAKLJUČAK

Harmonizacija tehničke regulative u oblasti železničke infrastrukture u Republici Srbiji i usvajanje evropskih standarda za oblast "Primene na železnici" su u toku. Evropski komitet za standardizaciju je kreirao grupu standarda EN 13848 - Primene na železnici - Kolosek - Kvalitet geometrije koloseka, koju čini šest delova. Cilj kreiranja ove grupe standarda je definisanje jedinstvenog pristupa za ocenu kvaliteta geometrije koloseka različitih evropskih železničkih infrastruktura. Institut za standardizaciju Srbije je usvojio i publikovao pet od šest delova ove grupe standarda. Sa druge strane, postojeća železnička regulativa u Srbiji nije harmonizovana sa regulativom Evropske Unije. Očekuje se povezivanje tehničke regulative za održavanje želez-

5 CONCLUSION

Harmonization of the technical regulation in the area of railway infrastructure in the Republic of Serbia and the adoption of the European standards in sector "Railway applications" are in progress. European Committee for Standardization has created a group of standards EN 13848 - Railway applications – Track – Track geometry quality, which consists of six parts. The objective of creation of this group of standards is to define a unique approach for the evaluation of track geometry quality of various European railway infrastructures. The Institute for Standardization of Serbia has adopted and published five out of six parts of this group of standards. On the other hand, the existing Serbian railway regulations are still uncoordinated with those of the European Union. The

ničke infrastrukture u Srbiji sa standardima iz serije EN 13848 u što skorije vreme. U suprotnom, usvojeni standardi SRPS EN 13848 ne bi mogli da se primenjuju u praksi.

Razmatranja u ovom radu su deo napora da se kvantifikuje evropski kvalitet koloseka.

ZAHVALNICA

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Serbian technical regulation for the railway infrastructure maintenance is expected to be linked with the standard EN 13484 Series, as soon as possible. On the contrary, adopted standards SRPS EN 13848 are inapplicable in practice.

Considerations in this paper are a part of effort to quantify the European track quality.

ACKNOWLEDGEMENT

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REZIME

HARMONIZACIJA EVROPSKOG KVALITETA KOLOSEKA

Zdenka POPOVIĆ
Luka LAZAREVIĆ
Isidora PANČIĆ
Filip TRPČEVSKI

Ovaj rad se bavi definisanjem geometrije koloseka, sistemima merenja i snimanja koloseka, kao i nivoima kvaliteta geometrije koloseka. Predstavljani su pravni okvir i trenutno stanje u oblasti harmonizacije tehničke regulative za geometriju koloseka u Republici Srbiji. Naročito, rad razmatra seriju evropskih EN 13848 (deo 1-6). Analizirani su glavni parametri geometrije koloseka prema EN 13848-1, nivoi kvaliteta geometrije koloseka prema EN 13848-5, kao i ocenjivanje kvaliteta geometrije koloseka u skladu prEN 13848-6. Cilj izrade serije evropskih standarda EN 13848 (delovi 1-6) je definisanje jedinstvenog pristupa za ocenu kvaliteta geometrije koloseka različitih železničkih infrastruktura u Evropi. Institut za standardizaciju Srbije je usvojio pet od šest delova ovog standarda. Ovaj rad je deo napora ka harmonizaciji železničke tehničke regulative u Srbiji sa regulativom Evropske Unije.

Ključne reči: železnica, geometrija koloseka, nivoi kvaliteta geometrije, interoperabilnost, harmonizacija, održavanje.

SUMMARY

HARMONIZATION OF EUROPEAN TRACK QUALITY

Zdenka POPOVIC
Luka LAZAREVIC
Isidora PANCIC
Filip TRPCEVSKI

This paper deals with characterisation of track geometry, track measuring and recording system, as well as geometric quality levels. The legal framework and state of the art in the field of harmonization of track geometry technical regulation in the Republic of Serbia were presented. In particular, the paper discusses the European standard EN 13848 Series (Parts 1-6). Principal track geometric parameters were analyzed according to EN 13848-1. Track geometric quality levels were examined according to EN 13848-5. The evaluation of track geometry quality according to prEN 13848-6 was analyzed as well. The objective of creation of the European standard EN 13848 Series (Parts 1-6) was defining a unique approach for the evaluation of track geometry quality of various European railway infrastructures. The Institute for Standardization of Serbia has adopted and published five of six parts of this standard. This paper is a part of an effort to harmonize the Serbian railway technical regulations with those of the European Union.

Key words: railway, track geometry, geometric quality levels, interoperability, harmonization, maintenance.

STATISTIČKA ANALIZA REZULTATA ISPITIVANJA KVALITETA BETONA

STATISTICAL ANALYSIS OF CONCRETE QUALITY TESTING RESULTS

Dragica JEVTIĆ
Dimitrije ZAKIĆ
Aleksandar SAVIĆ
Aleksandar RADEVIĆ

STRUČNI RAD
PROFESSIONAL PAPER
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1 UVOD

Ocena kvaliteta betona ugrađenog u neku betonsku, odnosno armirano-betonsku konstrukciju, zahteva kontinualno ispitivanje osnovnih fizičko-mehaničkih karakteristika kontrolnih betonskih uzoraka, uzetih na mestu proizvodnje (u fabrici betona) i na mestu ugradnje (na gradilištu). Cilj ovog istraživanja jeste da se izvrši kritička ocena kvaliteta betona koji se proizvodi i ugrađuje na tržištu Republike Srbije, uzimajući u obzir osnovne parametre kvaliteta - čvrstoću pri pritisku (marku betona) i izmerenu zapreminsku masu u očvrslom stanju. Ispitivanja o kojima je reč obavljena su na ukupno 4.420 kontrolnih betonskih uzoraka, oblika kocke ivice 15 ili 20 cm. Predmetne kocke uzorkovane su na petnaest betonskih baza, odnosno na 50 gradilišta različitih armirano-betonskih konstrukcija koje je izvodilo dvadeset dva izvođača, u periodu od januara do decembra 2012. godine. Od ukupnog broja ispitanih uzoraka, 75% dostavljeno je s gradilišta koja se nalaze na široj teritoriji grada Beograda, a 25% iz drugih delova Srbije (Vojvodina, Zlatiborski okrug, Pirotski okrug, Raška oblast i Podrinje). Imajući u vidu da se najveći deo građevinskih radova u našoj zemlji izvodi baš na široj teritoriji grada Beograda, možemo smatrati da ispitivani uzorci daju prilično reprezentativnu sliku o kvalitetu betona koji se proizvodi i ugrađuje u Republici Srbiji.

Prof. dr Dragica Jevtić, dipl.inž.tehn.
Univerzitet u Beogradu, Građevinski fakultet, Bulevar kralja Aleksandra 73, Beograd, Srbija
Doc. dr Dimitrije Zakić, dipl.inž.građ.
Univerzitet u Beogradu, Građevinski fakultet, Bulevar kralja Aleksandra 73, Beograd, Srbija
Aleksandar Savić, dipl.inž.građ.
Univerzitet u Beogradu, Građevinski fakultet, Bulevar kralja Aleksandra 73, Beograd, Srbija
Aleksandar Radević, dipl.inž.građ.
Univerzitet u Beogradu, Građevinski fakultet, Bulevar kralja Aleksandra 73, Beograd, Srbija

1 INTRODUCTION

Quality assessment of concrete placed in an ordinary concrete or reinforced concrete structure, requires continuous testing of basic physical-mechanical properties on control specimens taken either at the production site (i.e. concrete plant) or at the placement site (i.e. construction site). The main objective of this research is to perform a critical quality assessment of concrete which is produced and built in at the Serbian market, considering the basic quality parameters – such as compressive strength (i.e. class of concrete) and density measured in the hardened state. This investigation was carried out by testing a total number of 4420 control concrete specimens in the shape of cubes with 15 or 20 cm sides. Sampling of these cubic specimens was performed at 15 concrete plants, i.e. at 50 different reinforced concrete construction sites managed by 22 contractors, covering a period from January until December 2012. From the total number of tested samples, 75% were delivered from sites in Belgrade city and surrounding area, and the rest of 25% from other Serbian regions (Vojvodina, Zlatibor county, Pirot county, Raška province and Podrinje region). Considering the fact that most of the construction work in our country is performed in the city of Belgrade and its outskirts, we can assume that the tested samples can

Prof. dr Dragica Jevtic, B.E.Tech.
University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, Belgrade, Serbia
Doc. dr Dimitrije Zakic, BCE.
University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, Belgrade, Serbia
Aleksandar Savic, BCE.
University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, Belgrade, Serbia
Aleksandar Radevic, BCE.
University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, Belgrade, Serbia

U svim razmatranim slučajevima, beton je na gradilište dopreman automikserima, nakon čega je ugrađivan u kalupe oblika kocke ivice 15 cm ili 20 cm. Uzorkovanje i negu betonskih uzoraka sprovodili su izvođači betonskih radova, u skladu s važećim standardima i to: SRPS EN 12350-1:2009, SRPS EN 12390-2:2009 i SRPS ISO 2736-2:1997. Uzorci su pri starosti od dvadeset osam dana (ili nešto ranije, u kom slučaju je negovanje nastavljano u laboratoriji), isporučivani Laboratoriji za materijale Građevinskog fakulteta u Beogradu, gde su obavljena predmetna ispitivanja. Čvrstoća pri pritisku betonskih uzoraka ispitivana je u skladu sa standardom SRPS ISO 4012:2000, a određivanje zapremine mase prema odredbama SRPS ISO 6275:1997. Tokom statističke analize podataka, akcent je stavljen na vrednosti čvrstoće pri pritisku betonskih uzoraka, koje predstavljaju svakako najbitniji parametar za ocenu kvaliteta betona.

2 EKSPERIMENTALNO ISTRAŽIVANJE

U ovom istraživanju, centralno mesto zauzima statistička analiza zasnovana na rezultatima ispitivanja čvrstoće pri pritisku i zapremine mase kontrolnih betonskih uzoraka (kocki ivice 15 cm ili 20 cm). Iako se dokaz marke betona, po još uvek važećem Pravilniku za beton i armirani beton - PBAB'87 [6], zasniva na karakterističnoj dvadesetosmodnevnoj čvrstoći pri pritisku betonskih uzoraka oblika kocke ivice 20 cm, danas je evidentna dosta češća upotreba uzoraka oblika kocke ivice 15 cm. Ovakav trend, s jedne strane, jeste posledica usklađivanja tehničke regulative u oblasti građevinarstva sa evropskim zemljama - u kojima se (po standardu EN 12390) klasa čvrstoće betona dokazuje na cilindrima Ø15/H30cm ili na kockama ivice 15 cm. S druge strane, ovakav trend takođe je posledica i dugogodišnjih teškoća u pogledu manipulacije s kalupima, negovanjem i ispitivanjem betonskih kocki ivice 20 cm. Iz tih razloga, čak 90% ispitanih uzoraka tokom 2012. godine činile su kocke ivice 15 cm. Kako bi omogućili adekvatnu analizu rezultata ispitivanja, kao i njihovo poređenje s projektovanim markama betona, čvrstoće betona pri pritisku dobijene na uzorcima oblika kocke ivice 15 cm prethodno su preračunate na čvrstoću pri pritisku kocki ivice 20 cm, množenjem sračunatih vrednosti koeficijentom konverzije $k=0.95$ (u skladu s Pravilnikom BAB'87).

Uzorkovanje betona vršeno je manjim delom u fabrikama betona (20%), a većim delom na gradilištima (80%). U ovu drugu grupu svrstavaju se objekti različite namene: 92.0% uzoraka s poslovno-stambenih objekata, 3.8% s mostovskih konstrukcija i 4.2% sa industrijskih objekata.

3 REZULTATI ISPITIVANJA I DISKUSIJA

Iako se i prema domaćim i prema inostranim standardima marka betona (odnosno klasa čvrstoće) dokazuje na uzorcima starosti dvadeset osam dana, često postoji potreba da se i nešto ranije dođe do određenih pokazatelja kvaliteta ugrađenog betona. Svakako, prvi takav pokazatelj - koji se može dobiti praktično odmah nakon spravljanja i ugrađivanja betona - jeste zapreminska masa betona u svežem stanju. Po

give a fairly representative picture regarding the quality of concrete which is produced and placed in Serbia. In all the cases, concrete was transported by mixer trucks and afterwards placed in 15 or 20 cm cubic moulds. Sampling and curing concrete specimens was done by the contractors, according to the valid standards, such as: SRPS EN 12350-1:2009, SRPS EN 12390-2:2009 and SRPS ISO 2736-2:1997. The samples were delivered at the age of 28 days (or earlier, in which case the curing was continued in the lab) and afterwards tested in the Laboratory for materials at the Faculty of Civil Engineering, University of Belgrade. Compressive strength of concrete was tested according to SRPS ISO 4012:2000, and density according to SRPS ISO 6275:1997. During the statistical analysis, the accent was put on the concrete specimens' compressive strength values, which certainly represent the most important parameter for concrete quality assessment.

2 EXPERIMENTAL RESEARCH

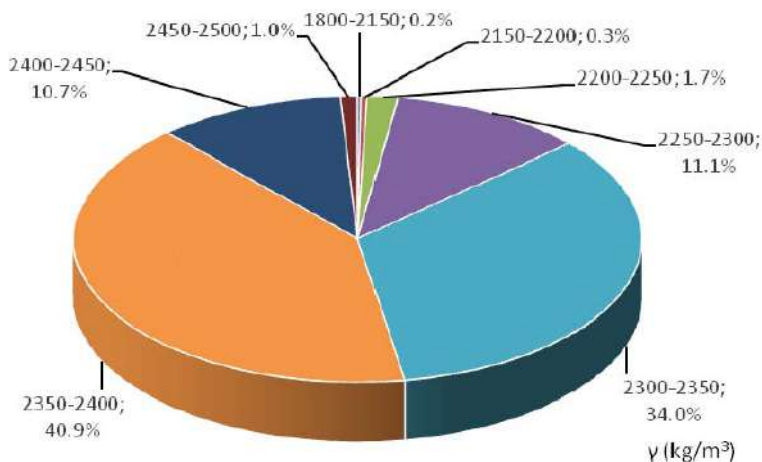
The central point in this research has the statistical analysis based on compressive strength and density testing results of control concrete samples (cubes with 15 or 20 cm sides). Although the still valid Rulebook for concrete and reinforced concrete (PBAB'87) prescribes using characteristic 28-day compressive strength obtained by testing the 20cm cubic specimens in order to prove the class of concrete, it is evident that today the application of 15cm specimens has become more frequent. Such a trend is a consequence of the technical regulation harmonization process with the European countries in the field of Civil engineering – in which (according to EN 12390) the concrete class should be proved on Ø15/H30cm cylindrical specimens or 15cm cubic specimens. On the other hand, this trend is also caused by the long-lasting difficulties related to mould manipulation, curing and testing of 20cm cubic specimens. Therefore, even 90% of specimens tested during 2012 were 15cm cubes. In order to make possible an adequate analysis of testing results, as well as their comparison with designed concrete classes, concrete compressive strengths obtained using 15cm cubic specimens were recalculated to 20cm cubic specimens, multiplying the experimental values by conversion coefficient $k=0.95$ (according to the Rulebook BAB'87). Sampling of concrete was performed partially at the concrete plants (20%), but to a greater extent at the construction sites (80%). The latter group consists of structures with different purposes, such as: 92.0% samples coming from commercial/residential buildings, 3.8% from bridge structures and 4.2% from industrial facilities.

3 RESEARCH RESULTS AND DISCUSSION

Although, according to both domestic and international standards, the proof of concrete class has to be established by testing 28-day old specimens, sometimes it is necessary to make an early assessment of in situ concrete quality. Certainly, one of the first parameters that could be obtained right after mixing and placing concrete is its density in the fresh state. By the rule, the increment of concrete's density also implies its

pravilu, povećanje zapremine mase betona podrazumeva i veće čvrstoće pri pritisku i zatezanju, kao i poboljšanu otpornost na dejstvo mraza i otpornost na dejstvo mraza i soli, zatim viši stepen vodonepropustljivosti, itd. Isti trend važi i kada je reč o odnosu između vrednosti zapremine mase betona u očvrslom stanju i navedenih parametara kvaliteta betona. Na slici 1 prikazani su rezultati ispitivanja zapremine masa na 4.420 kontrolnih uzoraka betona u očvrslom stanju.

higher compressive and tensile strength, as well as better freeze/thaw and deicing salts resistance, higher degree of water tightness, etc. The same tendency exists when the relations between concrete's density in hardened state and its other above mentioned quality parameters are concerned. Figure 1 show the experimental results obtained after testing density in hardened state on 4420 control concrete samples.



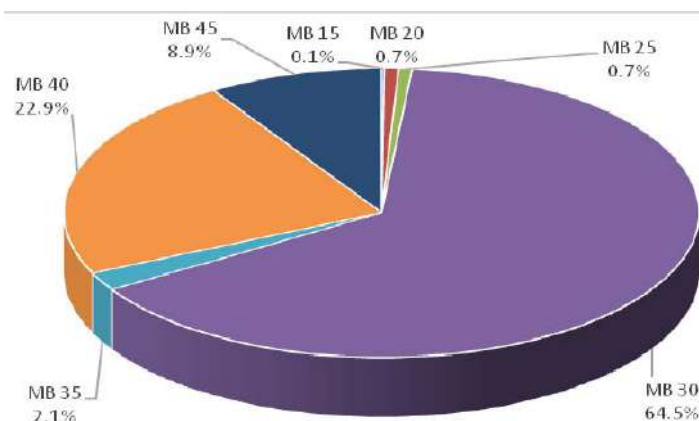
Slika 1 – Zapremine mase betona u očvrslom stanju
Figure 1 – Densities of hardened concrete

Prikazani rezultati ispitivanja pokazuju da su kod 14.4% uzoraka izmerene zapremine mase niže od 2.300 kg/m³. Znatno veći procenat uzoraka - 34.0% - imao je zapremine masu između 2.300 i 2.350 kg/m³, a kod čak 40.9% uzoraka ovo svojstvo variralo je između 2.350 i 2.400 kg/m³. Kod 10.7% uzoraka izmerene su relativno visoke vrednosti zapremine mase od preko 2.400 kg/m³. Imajući u vidu da se uobičajene vrednosti zapremine mase kod klasičnih betona kreću u granicama od 2.300 do 2.400 kg/m³, dobijene vrednosti ukazuju na potencijalno dobar kvalitet betona.

The presented experimental results show that 14.4% of samples had densities lower than 2300 kg/m³. Much higher percentage of samples (34.0%) had the density values ranging between 2300 and 2350 kg/m³, and for another 40.9% of samples this property varied between 2350 and 2400 kg/m³. At 10.7% of samples, relatively high values of density were measured - exceeding 2400 kg/m³. Considering the fact that the usual values of density of normal concrete range between 2300 and 2400 kg/m³ the obtained values suggest potentially good concrete quality.

Na slici 2 prikazana su procentualna učešća pojedinih projektovanih (zahtevanih) marki betona u ispitivanom statističkom uzorku.

Figure 2 shows the contribution of different designed (required) concrete classes, expressed in percent of the analyzed statistical sample.



Slika 2 – Učešća projektovanih marki betona (%)
Figure 2 – Contribution of designed concrete classes (%)

Imajući u vidu da se za većinu građevinskih objekata upotrebom betona projektovanih marki MB30 i MB40 mogu zadovoljiti uslovi graničnog stanja nosivosti i graničnog stanja upotrebljivosti nosećih elemenata konstrukcije, navedena procentualna učešća imaju očekivane vrednosti. Naime, na osnovu prikazanih rezultata vidi se da su najzastupljenije projektovane marke betona MB 30 i MB 40, sa učešćima od 64.5% i 22.9%, respektivno.

Prosečne vrednosti čvrstoće pri pritisku f_p , standardne devijacije σ , koeficijenta varijacije u , karakteristične čvrstoće pri pritisku σ_{kar} i ostvarene marke betona prikazane su u tabeli 1, za svaku od projektovanih marki betona koje su bile zastupljene u predmetnom istraživanju (od MB15 do MB45).

Having in mind the fact that for most of the concrete structures the application of designed concrete classes MB30 and MB40 can satisfy the conditions of load-bearing limit state and serviceability limit state of bearing structural elements, the above mentioned percentages have expected values. Namely, according to the presented experimental results, it is obvious that the most frequent designed concrete classes are MB 30 and MB 40, with contributions of 64.5% and 22.9%, respectively.

The average values of compressive strength f_p , standard deviation σ , variation index u , characteristic compressive strength σ_{kar} and achieved concrete class are given in the Table 1, for each of the designed concrete classes that were analyzed in this research (from MB15 to MB45).

Tabela 1 – Prosečne vrednosti analiziranih statističkih parametara u funkciji marke betona
Table 1 – Average values of analyzed statistical parameters related to the concrete class

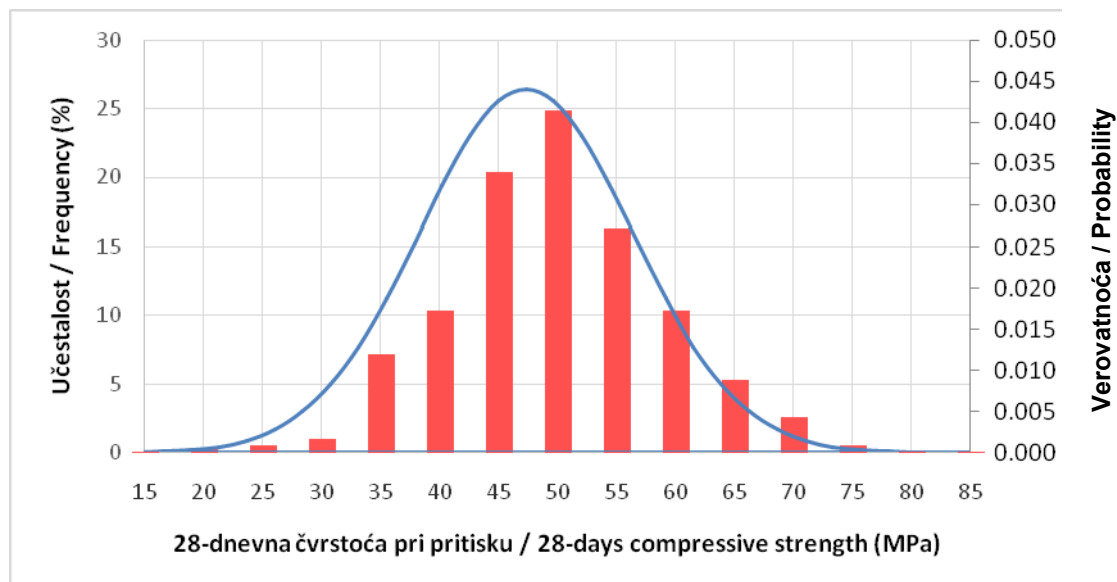
Projektovana marka betona Designed concrete class	f_p (MPa)	σ (MPa)	u (-)	σ_{kar} (MPa)	Ostvarena marka betona Achieved concrete class
MB 15	33.4	16.6	0.498	12.1	MB 10
MB 20	24.9	5.9	0.237	17.4	MB 15
MB 25	30.7	5.5	0.177	23.8	MB 20
MB 30	44.9	7.8	0.175	34.8	MB 30
MB 35	48.2	8.3	0.172	37.6	MB 35
MB 40	51.1	7.3	0.142	41.8	MB 40
MB 45	57.6	7.7	0.134	47.7	MB 45

S ciljem dokazivanja marke betona, a s obzirom na broj rezultata ispitivanja, pri proračunu karakteristične čvrstoće pri pritisku (σ_{kar}), korišćena je normalna raspodela slučajne promenljive s fraktilom $p=10\%$ (prema Pravilniku za beton i armirani beton PBAB '87). Iz tabele 1 jasno se uočava da prosečne vrednosti čvrstoća pri pritisku znatno premašuju projektovane marke betona, dok su razlike u vrednostima karakterističnih čvrstoća pri pritisku i projektovanih marki betona znatno manje. Naime, u slučaju projektovanih marki MB 15, MB 20 i MB 25 nisu zadovoljeni kriterijumi prema Pravilniku BAB '87, koji se odnose na zahtevanu marku betona. Razlog ovako velikih odstupanja karakteristične vrednosti u odnosu na prosečnu čvrstoću pri pritisku ispitivanih betonskih uzoraka leži u velikom rasipanju rezultata, što dovodi do relativno visokih vrednosti standardne devijacije. Ipak, treba napomenuti da su ovakve vrednosti standardne devijacije delom i posledica statističke obrade podataka koja je vršena na svim uzorcima jedne iste projektovane marke betona, ne uzimajući u obzir druge uticajne činioce (vrstu betona, njegov sastav, proizvođača, uslove transporta, termo-higrometrijske parametre, itd.). Baš zbog očekivanog značajnog rasipanja rezultata, ozbiljniji proizvođači betona pribegavaju proizvodnji betona koji će zadovoljiti neophodne uslove, ne samo za projektovanu marku, već i za jednu marku ili čak dve marke više od projektovane. Osim poboljšanih mehaničkih karakteristika, neka druga svojstva takvih betona (kao što su otpornost na dejstvo mraza, otpornost na dejstvo mraza i soli, vodonepropustljivost itd.) takođe će time svakako biti

Considering the number of testing samples, in order to establish the class of concrete the characteristic value of compressive strength (σ_{kar}) was calculated using normal distribution with 10% fractile (according to BAB '87). Analyzing the results from the Table 1, it is obvious that the average values of compressive strength (f_p) clearly exceed the designed concrete classes, whereas the differences between characteristic values of compressive strength (σ_{kar}) and designed concrete classes are less pronounced. To be exact, in the case of designed classes MB 15, MB 20 and MB 25 even the minimal required criteria according to the Rulebook BAB '87 are not satisfied. The reason for such difference between the characteristic values and average values of compressive strength originates from the significant dispersion of testing results and consequently relatively high values of standard deviation (σ). However, it should be mentioned that such values of standard deviation also occur as a result of statistical data processing which was performed on the samples with the same designed concrete class, but without taking into consideration other important influential parameters (such as: type and mix design of concrete, production and transportation conditions, thermo-hygrometric parameters, etc.). Due to expecting significant dispersion of results, respected concrete producers usually tend to make concrete which satisfies not only the required conditions, but also the conditions needed for one or two classes above the designed value. Besides better mechanical characteristics, other properties of such concrete (like freeze-thaw resistance, water tightness, resistance to

bolja. Dobijeni rezultati ispitivanja ukazuju na to da su kod deset od ukupno dvadeset dva tretirana proizvođača betona (što čini 45%) zadovoljeni uslovi za prvu ili drugu višu marku betona u odnosu na projektovanu marku u čak 90% slučajeva. Pritom, treba napomenuti da je preko 70% ispitivanih betonskih kocki uzorkovano na gradilištima koja su se snabdevala betonom iz jedne od pomenutih deset fabrika betona.

Na slici 3 prikazana je učestalost rezultata ispitivanja dvadesetosmodnevne čvrstoće pri pritisku na analiziranom statističkom uzorku od 4.420 betonskih kocki.



Slika 3 – Učestalost ispitivanja i kriva verovatnoće
Figure 3 – Testing results frequency and probability curve

Pritom, prosečna čvrstoća svih ispitanih uzoraka iznosila je 47.0 MPa, standardna devijacija 9.1 MPa, a koeficijent varijacije 0.194. Na osnovu raspodele čvrstoća pri pritisku (prikazanoj na slici 3), može se zaključiti da čvrstoće prate normalnu raspodelu slučajne promenljive koja je definisana izrazom:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right) \quad (1)$$

gde je:

- σ – standardna devijacija;
- μ – srednja vrednost;
- x – nezavisno promenljiva.

where:

- σ – standard deviation,
- μ – mean value,
- x – independent variable.

Zavisnost između zapremine mase u očvrslom stanju i čvrstoće pri pritisku prikazana je na slici 4.

Iako ovakav dijagram ukazuje na veliko rasipanje rezultata, ipak blago nagnuta linearna korelacija pokazuje da se s povećanjem zapremine mase, u principu, poboljšavaju i mehaničke karakteristike betona.

Imajući u vidu da su svi uzorci ispitivani pri starosti od dvadeset osam dana, na osnovu učestalosti ispitivanja po mesecima, može se sa određenim stepenom preciznosti proceniti i učestalost izvođenja betonskih radova u toku 2012. godine, koja je prikazana

deicing salts, etc.) will also be improved. The obtained testing results indicate that 10 out of 22 analyzed concrete producers (or 45%) satisfy the conditions for first or second concrete class above the designed value in 90% of cases. At the same time, more than 70% of tested concrete cubes were sampled at the sites supplied by one of the 10 above mentioned concrete plants.

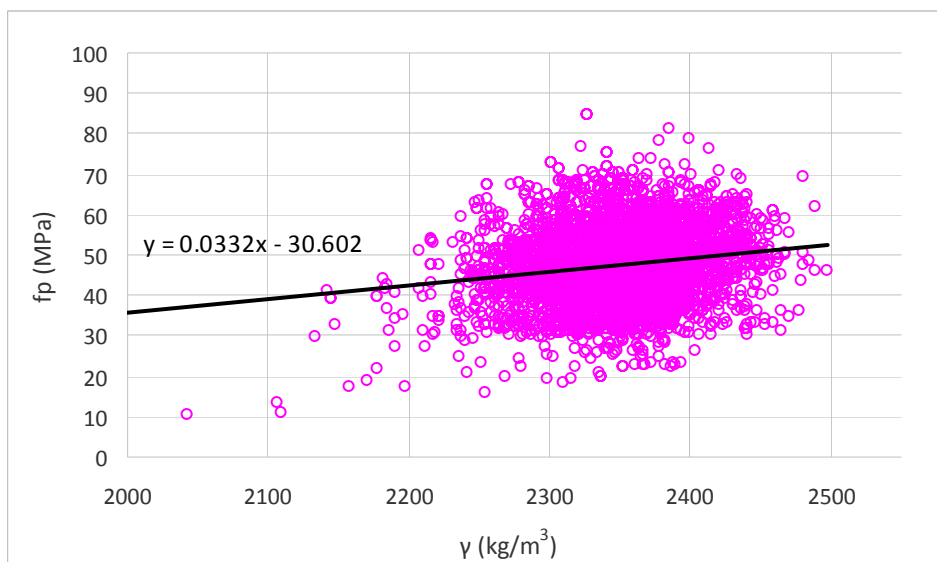
Figure 3 shows the frequency of 28-day compressive strength testing results and probability curve based on statistical sample of 4420 concrete cubes.

In addition, the mean value of compressive strength of all tested specimens amounted to 47.0 MPa, standard deviation 9.1 MPa, and variation index 0.194. Based on the distribution of compressive strengths (see Figure 3), the conclusion can be made that these strengths correspond with the normal distribution of independent variable which is defined as follows:

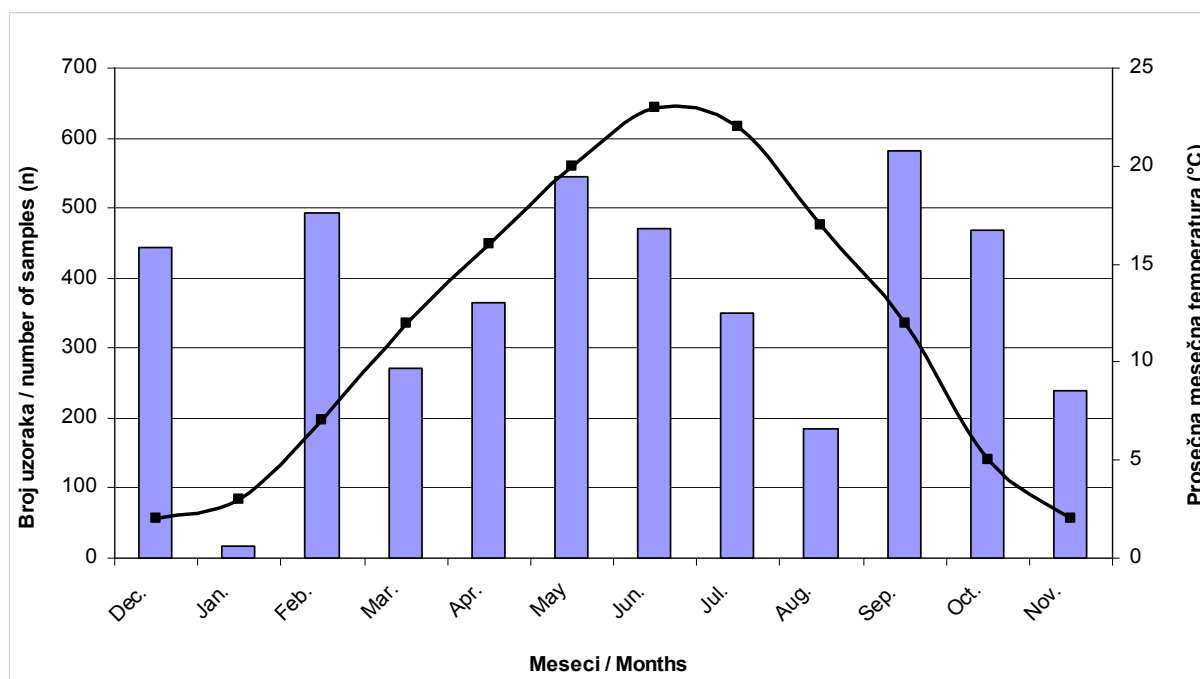
The correlation between the density of hardened concrete and its compressive strength is presented at Figure 4.

Although this diagram shows large dissipation of experimental results, nevertheless the slightly inclined linear correlation suggests that the increase of density generally leads to better mechanical properties of concrete.

Having in mind the fact that all samples were tested at the age of 28 days and taking into account the testing frequency per month, we can make a fairly accurate as-



Slika 4 – Zavisnost zapremine mase u očvrslom stanju i čvrstoće pri pritisku
Figure 4 – Correlation between the density in hardened state and compressive strength



Slika 5 – Učestalost ispitivanja uzoraka po mesecima i prosečne mesečne temperature vazduha na teritoriji Beograda
Figure 5 – Frequency of testing per month versus average monthly air temperatures in Belgrade area

na slici 5. Na istom dijagramu, prikazane su i prosečne mesečne temperature vazduha na teritoriji grada Beograda [5] u periodu od decembra 2011. do novembra 2012. godine, kada su uzimani uzorci betona koji su ispitivani u Laboratoriji za materijale Građevinskog fakulteta Univerziteta u Beogradu tokom 2012. godine. Na slici 5 jasno se uočava da dinamika izvođenja betonskih radova, u periodu od marta do avgusta 2012. godine, prati promenu prosečne mesečne temperature vazduha. U letnjim mesecima izraženiji je pad aktivnosti na izvođenju betonskih radova zbog povišenih

sumption about the frequency of concrete works execution during 2012 (see Figure 5). At the same diagram, average monthly temperatures in Belgrade area for the period December 2011 – November 2012 are presented [5], which corresponds with the period of sampling and testing of specimens in the Laboratory for materials at the Faculty of Civil Engineering – University of Belgrade. Looking at Figure 5, it is obvious that the dynamics of concrete works execution during the period March-August 2012 corresponds with the change of average monthly temperatures. Throughout the summer

maksimalnih dnevnih temperatura koje su za mesece jun, jul i avgust 2012. godine iznosile 35, 40 i 38 °C, respektivno [5]. S druge strane, u zimskim mesecima predmetne godine nisu belezene ekstremno niske temperature, pa su betonski radovi, uz korišćenje specijalnih postupaka (upotreba aditiva za zimsko betoniranje, zagrevanje komponentnih materijala, termička izolacija oplata, grejanje ugrađenog betona, itd.), obavljani nesmetano.

4 ZAKLJUČAK

U radu je prikazana statistička analiza rezultata ispitivanja kvaliteta betonskih uzoraka u Laboratoriji za materijale Građevinskog fakulteta Univerziteta u Beogradu, u toku 2012. godine. Ispitivanja su obavljena na ukupno 4.420 kontrolnih betonskih uzoraka. Rezultati istraživanja o kome je reč pokazuju sledeće:

- 74.9% uzoraka imalo je vrednosti zapremine mase u očvrslom stanju koje su se kretale između 2.300 i 2.400 kg/m³, a 10.7% uzoraka zapreminsku masu veću od 2.400 kg/m³;
- najzastupljenije projektovane, a ujedno i ostvarene marke betona, jesu MB 30 i MB 40, sa ukupnim učešćima od 64.5% i 22.9%, respektivno;
- prosečne vrednosti čvrstoća pri pritisku znatno premašuju projektovane marke betona, dok su razlike u vrednostima karakterističnih čvrstoća pri pritisku i projektovanih marki betona znatno manje, što je posledica značajnog rasipanja rezultata;
- prosečna čvrstoća svih 4.420 ispitanih betonskih uzoraka iznosila je 47.0 MPa, standardna devijacija 9.1 MPa, a koeficijent varijacije 0.194;
- povećanje zapremine mase uzoraka u očvrslom stanju, u načelu, prati povećanje čvrstoće pri pritisku betona;
- upotrebom odgovarajućih hemijskih dodataka i adekvatnim izvođenjem betonskih radova u posebnim uslovima, a za termohigrometrijske uslove koji važe u Republici Srbiji, može se reći da termin „građevinska sezona” postepeno gubi smisao, jer se betonski radovi – kao što je pokazano - mogu izvoditi praktično tokom cele godine, gotovo nesmetano.

ZAHVALNOST

U radu je prikazan deo istraživanja koje je pomoglo Ministarstvo prosvete, nauke i tehnološkog razvoja Republike Srbije, u okviru tehnoloških projekata TR 36048 pod nazivom „Ispitivanje stanja i metoda unapređenja građevinskih konstrukcija sa aspekta upotrebljivosti, nosivosti, ekonomičnosti i održavanja” i TR 36017 pod nazivom „Istraživanje mogućnosti primene otpadnih i recikliranih materijala u betonskim kompozitima, sa ocenom uticaja na životnu sredinu, u cilju promocije održivog građevinarstva u Srbiji”.

months the reduction of concreting activities is more evident, due to higher daily temperatures, which maximum values in June, July and August of 2012 amounted to 35, 40 and 38 °C, respectively. On the other hand, during the winter months of the same year extremely low temperatures were not recorded, so the concrete works supplemented by special procedures (such as: application of winter concreting admixtures, preheating constituent materials, thermal insulation of formwork, heating the placed-in concrete, etc.) could be continued without problems.

4 CONCLUSION

Statistical analysis of the results obtained after testing compressive strength and density of concrete samples in the Laboratory for materials at the Faculty of Civil Engineering – University of Belgrade during 2012, are presented in this paper. The research included a total number of 4420 control concrete samples. The following conclusions can be derived from the obtained results:

- 74.9% of samples had values of density in the hardened state varying between 2300-2400 kg/m³ and 10.7% of samples had density higher than 2400 kg/m³;
- the most frequent designed classes of concrete, as well as the achieved classes of concrete, were MB 30 and MB 40 – with total contributions of 64.5% and 22.9%, respectively;
- the average values of compressive strength significantly exceed the designed concrete classes, whereas the differences between characteristic values of compressive strength and designed concrete classes are less pronounced, which is a consequence of a considerable dispersion of testing results;
- the average compressive strength of all 4420 tested concrete samples amounted to 47.0 MPa, standard deviation was 9.1 MPa, and variation index 0.194;
- the increase of density in the hardened state is generally followed by improvement of concrete's compressive strength;
- with application of special chemical admixtures and adequate execution of concrete works, at the same time respecting characteristic thermo-hygrometric conditions in Serbia, the term „construction season” is gradually losing its traditional meaning; as it is shown in this paper, concreting can be performed practically during the whole year, almost without any problems.

ACKNOWLEDGEMENTS

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REZIME

STATISTIČKA ANALIZA REZULTATA ISPITIVANJA KVALITETA BETONA

*Dragica JEVTIĆ
Dimitrije ZAKIĆ
Aleksandar SAVIĆ
Aleksandar RADEVIĆ*

U radu je prikazana statistička analiza rezultata ispitivanja čvrstoće pri pritisku i zapremne mase kontrolnih betonskih uzoraka dobijenih u Laboratoriji za materijale Građevinskog fakulteta Univerziteta u Beogradu, u toku 2012. godine. Ispitivanja su obavljena na ukupno 4.420 betonskih uzoraka, uzorkovanih na više lokacija - na mestu proizvodnje (u fabrikama betona) i na mestu ugradnje (na različitim gradilištima). Naime, reč je o uzorcima proizvedenim na petnaest betonskih baza, odnosno o uzorcima betona ugrađenih u 50 armirano-betonskih konstrukcija različite namene, koje je tokom 2012. godine izvodilo dvadeset dva različita izvođača. Poznata je činjenica da su ostvarene vrednosti čvrstoće pri pritisku betona veoma važne, kako za procenu kvaliteta i trajnosti betona u konstruktivnim elementima, tako i za proračun njihove granične nosivosti. Osim rezultata ispitivanja čvrstoće pri pritisku, analizirani su i podaci o zahtevanim (projektovanim) markama betona, o tome koliko se ostvarene marke poklapaju s projektovanim markama betona, zatim o dobijenim vrednostima zapremne mase, kao i o učestalosti izvođenja betonskih radova u toku predmetne - 2012. godine.

Ključne reči: čvrstoća pri pritisku, zapremna masa, statistička analiza

SUMMARY

STATISTICAL ANALYSIS OF CONCRETE QUALITY TESTING RESULTS

*Dragica JEVTIC
Dimitrije ZAKIC
Aleksandar SAVIC
Aleksandar RADEVIC*

This paper statistically investigates the testing results of compressive strength and density of control concrete specimens tested in the Laboratory for materials, Faculty of Civil Engineering, University of Belgrade, during 2012. The total number of 4420 concrete specimens were tested, which were sampled on different locations – either on concrete production site (concrete plant), or concrete placement location (construction site). To be exact, these samples were made of concrete which was produced on 15 concrete plants, i.e. placed in at 50 different reinforced concrete structures, built during 2012 by 22 different contractors. It is a known fact that the achieved values of concrete compressive strength are very important, both for quality and durability assessment of concrete inside the structural elements, as well as for calculation of their load-bearing capacity limit. Together with the compressive strength testing results, the data concerning requested (designed) concrete class, matching between the designed and the achieved concrete quality, concrete density values and frequency of execution of concrete works during 2012 were analyzed.

Key words: compressive strength, density, statistical analysis

Prof. MIKLOS IVANYI, Ph.D.

31 Oct. 1940 – 21. Dec. 2013

“Nihil est anni velocius” (Nothing runs as fast as the time)



With great sadness we received the announcement that on December 21, 2013 Prof. Miklos Ivanyi from the University of Pecs, passed away.

He was born in Endrőd, Hungary, on October 31, 1940. In 1963 he graduated from the Technical University of Building, Civil and Transport Engineering in Budapest. In 1974 he became Doctor Technicus, at Technical University of Budapest. He was successively: 1963 Assistant Professor, 1974 Associate Professor and from 1984 Full Professor of Structural Engineering. At the same time he held high administrative positions: 1982-88 Vice rector, Technical University of Budapest and in the period of 1986-99 he was the Head of Department of Steel Structures, TUB. His teaching experience is very rich and various: steel bridges and structures, theory of strength and stability of steel structures, dimensioning theory, experimental structural testing. Also his fundamental and research activity comprises various studies in stability of frames, trusses, spatial behavior of members, plate stability, 3D structures; various studies in plastic analysis: collapse load of frames, inelastic instability of steel structures. Various studies in bridges: design of plate girders and trusses, effect of erection on the structures. Likewise, he carried out many experimental research projects connected to the preparation of a new version of the Hungarian specification for plastic design. He was member in different professional associations, like IABSE, Structural Stability Research Council (USA), Academic Council of International Center of Mechanical Sciences (Udine), Editorial Board of the Journal of Constructional Steel Research (United Kingdom), Editorial Board of the "Costruzioni Metalliche" (Milano, Italy), American Society of Civil Engineering (ASCE), Editorial Board of the "Steel and Composite Structures" (Yusong, Korea). Furthermore, it can be mentioned his important technical activities, like: Secretary of Scientific and Organizing Committees of Regional Colloquium on "Stability of Steel Structures", he was *Chairman* of many International Scientific Committees, he performed visiting Professorships in Tokyo, Mushasi Institute of Technology, in Helsinki University of Technology etc. He was initiator of many international projects as coordinator or contractor: TEMPUS, LEONARDO; PECO

etc. As Editor of the "Pollack Periodica" he was continuously raising the level and prestige of this International Journal where prestigious researchers published their contributions. Prof. Ivanyi was *initiator* of the "Danube Bridges Conference" in 1992 and it was held every third year. The 8. Conference was held in Timisoara and Belgrade in October 2013; at that time he already had health problems and all the participants regretted his absence. Professor Ivanyi has an impressive *list of publications* Scientific Works & Books. He was Honorary Professor of the "POLITEHNICA" University Timisoara. As one of the good friends of Prof. Ivanyi, I want to express some personal thoughts about him.

I remember that in 1990, immediately after the changes in Romania when we were still confused, Prof. Ivanyi contacted me from Budapest, proposing the organization of a conference about the Danube bridges aiming to build a bridge of friendship and collaboration among the Danubian countries. We, the Danubian people, are bounded by culture, history and traditions. It was a generous initiative in the right historical moment, which proved the visionary way of thinking of Prof. Ivanyi. I remember also with emotion the first contact with him, in 1991 in Budapest, during the organization of the first Danube Conference. It was my first participation at an international scientific symposium; he with more experience in this direction guided me discretely. I am also very grateful for this. We became good friends and our scientific cooperation developed in time. I would like to thank prof. Miklos Ivanyi, expressing my gratitude for his sense of humanism, collaboration and friendship. I believe that Professor M. Ivanyi was a man of vision, who had the ability to set forces in movement to realize the practical implications of those visions.

He was the prime mover and the "main architect" of the International Conferences "Bridges on the Danube" where he constantly set up and guided the collaboration of highly qualified experts of the European and more largely international level.

He also set up and played a major role in the preparation of the synthesizing publication "Bridges on the Danube Catalogue", being Editor-in-Chief.

Undoubtedly Professor M. Ivanyi had an important contribution to the progress of research in the field of steel construction, promoting the international cooperation in science and techniques of civil engineering.

I consider that, the whole activity of Prof. Miklos Ivanyi was an example of fine academic behaviour. His name persists, he will be missed by all his friends and the recent and future generations of researchers will never forget him! Professor Ivanyi lives in our hearts.

Our deepest condolences to his family.

Prof. Dr. Ing. Radu Băncilă
University "POLITEHNICA" Timișoara

January, 2014.

UPUTSTVO AUTORIMA*

Prihvatanje radova i vrste priloga

.....A.A.A.....A.....A.....A.....A.....

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