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# SJCE
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SS CYRIL AND METHODIUS UNIVERSITY
FACULTY OF CIVIL ENGINEERING

# EDITORIAL - Preface to Volume 9 Issue 1 of the Scientific Journal of Civil Engineering (SJCE)

**Marijana Lazarevska EDITOR - IN - CHIEF**

Dear Readers,

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

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

As an editor-in-chief of the Scientific Journal of Civil Engineering, it is my great pleasure to present the First Issue of Volume 9, an issue that is mainly devoted to **MASE**, the **M**acedonian **A**ssociation of **S**tructural **E**ngineers.

MASE has been present in the engineering world for 44 years. During this time MASE has successfully organized 18 symposia and it has been recognized as a forum where domestic and foreign researches can exchange knowledge and ideas for the state-of-the-art scientific achievements and practices, allowing the participants to set up guidelines for future research and collaboration in the field of civil engineering.

The last Symposium of MASE was held in October 2019 and it was dedicated to monitoring, assessment and rehabilitation of structures. Invited lecturers from USA, Germany, Switzerland, Slovenia, Croatia, Serbia and North Macedonia had a unique opportunity to share their rich and valuable professional experience, contributing significantly for the great success of the Symposium.

The main concept behind this Issue is to honour the importance of MASE for the professional growth of many national and international engineers, researches and scientists. That is the reason why the editorial board invited the guest speakers of the 18th MASE Symposium to participate in this Journal by publishing their recent scientific-research work.

This Issue of the Journal presents the results of thirteen scientific research papers. It includes the introduction part that gives a short overview of the successful story of MASE, followed by seven papers written by the invited speakers of the 18th MASE Symposium and six papers in different topics of civil engineering and geodesy.

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

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

*Faith and joy are just as contagious as fear and panic. Choose well and embrace the "new normal".*

Sincerely Yours,

Assoc. Prof. Dr. Marijana Lazarevska

July, 2020

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# 18TH SYMPOSIUM OF MASE

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As a President of the **M**acedonian **A**ssociation of **S**tructural **E**ngineers, wellknown as **MASE**, I can proudly say that in the past 44 years of existence MASE has slowly grown into a "Brand". This is due to the hard and successful engaging of all uptodate presidencies. To continue in same manner is a challenge, but also a commitment, so I hope that in my two mandates as a president of MASE I managed to continue the tradition and preserve the brand of the best and the most active engineering association.

In 44 years of existence MASE successfully realized 18 symposia whose main goal was to provide a forum where domestic and foreign participants will be able to exchange knowledge and ideas for the latest scientific achievements and practices, setting guidelines for future research and cooperation in the field of construction. The symposia have always been dedicated to current topics such as: new methods for design and construction; actual standards and regulations for design of structures; modern materials and technologies as a prerequisite for the construction of sustainable structures, the impact of natural disasters on structures and other topics related to construction.

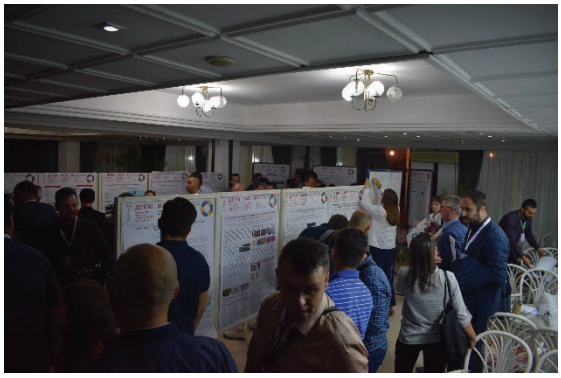
The 18th Symposium of the MASE was held in October 2019. The Symposium was attended by over 300 participants from 18 countries in the region and all over the world and once again successfully was realized one of the most important program goals of MASE, which is affirmation of scientific achievements and their implementation in practice.

The main topic of the Symposium was monitoring, assessment and rehabilitation of structures. The reason why we decided to treate this topic was the current state of our civil infrastructure from aspect of its age.

The most of our constructions already exceed the prescribed service life. Despite the appropriate design and construction procedures, the continuous and at the same time safe and economical operation of buildings mostly depends on their proper maintenance and management. The building structures

“suffer” in time. Deterioration of structure which can lead to even collapse, is due to various reasons, such as: cyclic loading, environmental influences (eg corrosion of steel, carbonization of concrete); aging of building materials; extreme events such as earthquakes, strong winds, floods and fires, etc. All these factors are time dependent and with a certain probability to occur. Therefore, it is necessary to accurately assess the current and future condition of the structures, ie to define the current and future capacity of the structure to ensure the predicted level of behavior when exposed to expected loads and possible hazards throughout the service life.

The answers to all these questions were found in a total of 173 papers presented within the 18th MASE Symposium.



A special note on the success of the symposium was given by the invited lecturers from USA, Germany, Switzerland, Slovenia, Croatia, Serbia and North Macedonia. They gave an overview of modern methods and techniques for monitoring, assessment and rehabilitation of bridge structures, buildings, facilities which represent cultural heritage, as well as an overview and possibility for application of modern materials in rehabilitation, reconstruction and rehabilitation of civil infrastructure.

One successful symposium is behind us, but not only from the aspect of presentation of scientific and professional works dedicated to construction, but also from the aspect of presentation of the achievements of construction companies from the country and the region which as partners and sponsors supported the organisation and contributed for its success.

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# INFLUENCE OF CONCRETE AGE ON A COMPOSITE CONCRETE SECTION BEHAVIOR

Composite concrete beams made of prefabricated prestressed element and cast in place reinforced concrete slab become very popular in nowadays bridge engineering. The two concrete composite parts are cast at different times, under different conditions. As a consequence, they have different concrete strengths and moduli of elasticity, as well as different rheological properties. The latter is responsible for stress redistribution within the composite section. The objective of the paper is to evaluate the influence of different concrete age between the girder and the in-situ slab on the behavior of composite concrete beams. Numerical study was performed on a real example of composite beams that are part of a multi-span continuous highway bridge. Four age differences between the precast girder and the in-situ slab were considered: 30, 90, 365 and 730 days. The numerical results indicate that different creep and shrinkage properties arising from the age difference between the concrete parts can significantly affect the stress redistribution, as well as the final deflections of the composite concrete beams.

Keywords: composite beams, creep and shrinkage, construction stages, redistribution

## 1. INTRODUCTION

Composite concrete beams become very popular in present-day bridge construction, especially for short- and medium-span bridges [1]. They are usually made of two concrete elements, a precast pretensioned beam and an in-situ deck slab (Fig.1). These two concrete parts are bonded together and form stiff composite bridge deck. Apart from providing a significant increase to the strength and stiffness of the prestressed girder, the in-situ slab can also provide continuity and lateral stability to the precast elements.

However, there are some specific aspects arising from the construction process that should be carefully considered in the design. The two concrete elements that form the composite section are cast at different times and under different conditions. Inevitably they

have different concrete strengths, different moduli of elasticity and different creep and shrinkage properties (Fig.1).

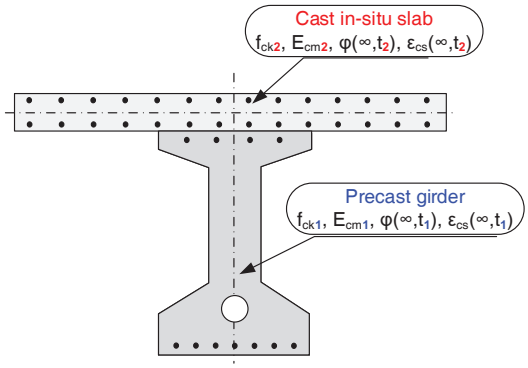


Figure 1. Typical concrete composite section

The concrete in the precast element is generally of better quality than the concrete in the cast in-situ element. It usually has higher specified target strength and experiences better quality control during construction [2]. Furthermore, the precast part is cast sometime prior the deck slab. With the concrete in the precast element being older and of better quality than the in-situ concrete, a redistribution of stresses within the composite section arises. Certain shrinkage amount of the girder occurs before setting the in-situ slab. Therefore, the subsequent shrinkage of the girder will be less than the shrinkage of the slab. Since the two concrete parts are bonded, the bigger shrinkage in the slab part is restrained by the girder. This internal restrain often results in tensile stresses in the deck slab, redistribution of the girders' stresses and increase in curvature of a section.

At the design stage, the designer has little control over the precast girders' age at which the in-situ slab will be placed. Usually, it is assumed that the age difference between the girder and the slab is within 60 and 90 days. However, there are many cases in practice when this difference is even more pronounced and less cases when it is less pronounced. For instance, in multi-span bridges, the age difference between the slab and the girders placed in the first span sometimes can be significantly large.

## 2. STRESSES AT STAGES OF LOADING

The history of construction and service stages influences the ultimate and serviceability limit states of composite concrete elements [3]. It is very likely that some intermediate construction stage can even be decisive in the choice of concrete strength class, tendon layout or reinforcement area in the deck slab. Therefore, the construction stages should be carefully treated in the design.

For composite sections, usually the following loading stages need to be considered [2]:

- Transfer of the initial prestress to the precast element – involves calculation of the elastic stresses due to the initial prestress and the self-weight of the precast element (line 1 in Fig.2).
- Period before casting the in-situ slab – requires time analysis to calculate the redistribution of the stresses caused by creep and shrinkage in the precast element (line 2 in Fig.2).
- Casting the in-situ concrete before composite action - requires short-term analysis of the precast element to calculate the instantaneous effects of the superimposed dead load prior to the composite action (line 3 in Fig.2).
- Immediately after the establishment of the composite action - involves short-term analysis of the composite cross section to determine the stresses for all the remaining loads (e.g. additional dead loads, live loads etc.) (line 4 in Fig.2).
- Period after the establishment of the composite action: - involves time analysis for the composite cross section until time infinite.

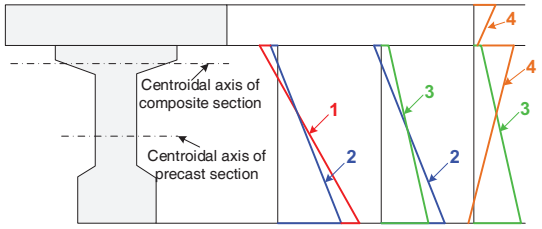


Figure 2. Concrete stresses at various load stages

## 3. ANALYSIS OF COMPOSITE CONCRETE BRIDGE BEAM – CASE STUDY

The influence of different age of precast concrete girder and in-situ concrete slab is evaluated through a numerical study on a real example of composite bridge beams. Five-span continuous bridge with composite concrete deck was analysed. This structure is chosen to

demonstrate the above-mentioned effect due to the type of its structural system that is sensitive to the effects caused by creep and shrinkage. In addition to this, the age difference between the prestressed girder and the cast in-situ slab was approximately 2 years that is beyond the age difference treated in the design.

In the paper, the analysis was performed for various age differences: 30, 90, 365 and 730 days.

## 3.1 GENERAL BRIDGE DATA

The subject structure is five-span continuous bridge (41.89 m + 3 x 42.79 m + 41.89 m) with composite concrete deck. The superstructure is composed of five precast pretensioned girders (C40/50) continued in a second phase through deck slab reinforcement, cross girders above the piers (C35/45) and cast in-situ concrete slab (C35/45). The characteristic distance between the main girders is 2.20 m (fig. 3).

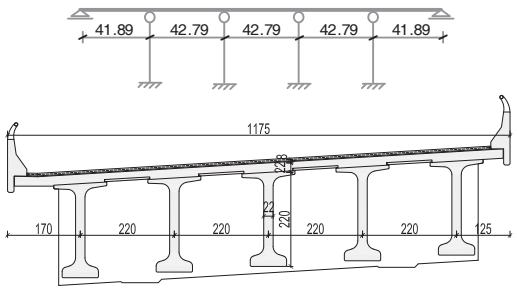


Figure 3. Structural system and typical cross-section

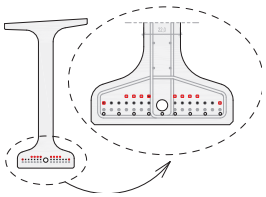

Figure 4. Prestressing arrangement

The main girders are pretensioned with 36 straight strands and additionally with one parabolic tendon (1600/1860MPa). The arrangement of the prestressing reinforcement is presented in Fig.4.

## 3.2 NUMERICAL MODEL

The numerical model, built in FEM software Sofistik, consists of composite girders from two adjacent spans. The model considers the composite action between the precast girder and the in-situ slab, as well as the transformation of the structural system from simple supported to continuous beam. The analysis is done for two phases:

- Phase I: the prestressed girders are acting independently with a structural system simple supported beam (Fig.5 top).
- Phase II: the prestressed girders and the cast in-situ slab are acting compositely with a structural system continuous beam (Fig.5 bottom).

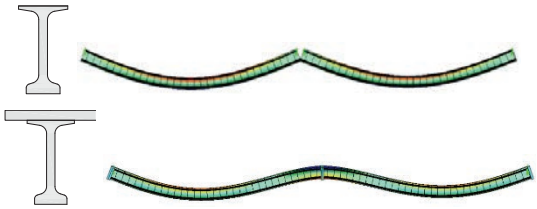

Figure 5. Phase I (top) and Phase II (bottom)

Construction Stage Manager (CSM) tool in Sofistik was used to simulate the history of construction and service stages of the bridge. The following stages were considered in the analysis (Fig.6):

**1.** Prestress of the 36 strands at $t_1 = 3$ days

**2.** Self-weight activation of the girders at $t_1 = 3$ days

**3.** Creep and shrinkage until stressing the parabolic tendon (duration $\Delta t_1 = 11$ days)

**4.** Prestress of the tendon at $t_2 = 14$ days

**5.** Creep and shrinkage until casting the in-situ slab (duration $\Delta t_2 = 16/76/351/716$ days)

**6.** Casting the in-situ slab at $t_3 = 30/90/365/730$ days

**7.** Hardening of the slab

**8.** Removing the temporary and placing the final supports

**9.** Creep and shrinkage until placing the asphalt and other additional loads (duration $\Delta t_3 = 90$ days)

**10.** Application of additional dead loads at $t_4 = 120/180/455/820$ days

**11.** Creep and shrinkage until the end of service life.

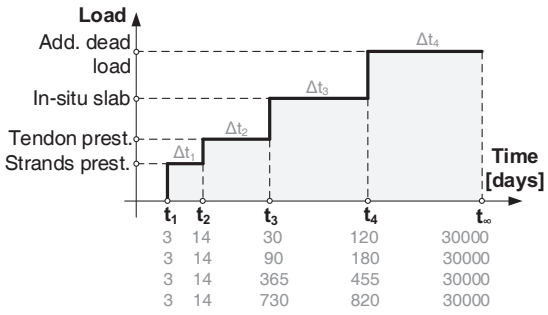


Figure 6. History of construction stages

In order to calculated the concrete stresses in the stages that require time analysis (stages

3.,5.,9. and 11.), different creep and shrinkage sources were used:

- Real creep and shrinkage strains based on measurements performed on concrete test specimens (provided by the Contractor)
- Available creep and shrinkage models implemented in Sofistik (Eurocode 2, fib Model Code 2010, CEB-FIP Model Code 1990 and Rusch's summation model)

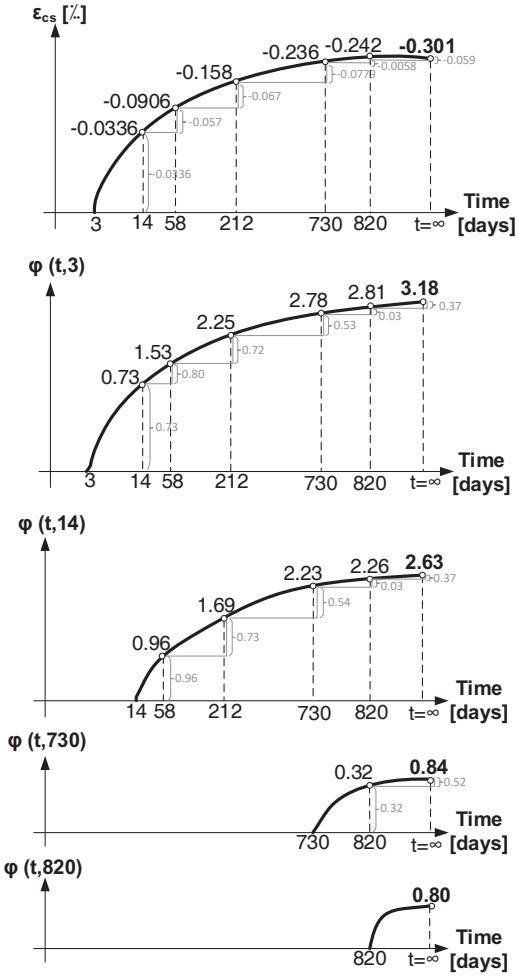


Figure 7. Creep and shrinkage curves for the prestressed girder (age difference 730 days)

For each concrete age at which immediate change in stresses appeared, different creep curves were developed separately for the girder and the slab:

- Creep coefficient for the precast element when the prestressing of the strands is applied: $\varphi(\infty, t_1) = \boldsymbol{\varphi}(\infty, \mathbf{3})$;
- Creep coefficient for the precast element when the prestressing of the tendon is applied: $\varphi(\infty, t_2) = \boldsymbol{\varphi}(\infty, \mathbf{14})$;
- Creep coefficient for the precast element when the in-situ slab is casted: $\varphi(\infty, t_3) = \boldsymbol{\varphi}(\infty, \mathbf{30/90/365/730})$);
- Creep coefficient for the precast element when additional dead loads are applied: $\varphi(\infty, t_4) = \boldsymbol{\varphi}(\infty, \mathbf{120/180/455/820})$;
- Creep coefficient for the deck slab when the additional dead loads are applied: $\varphi(\infty, t_4) = \boldsymbol{\varphi}(\infty, \mathbf{90})$;

Fig.7 presents the creep and shrinkage curves calculated according to one of the above-mentioned methods and indicates the values necessary for each load step.

On-site measurements of the girders deflections nearly before setting the in-situ slab were on disposal. The measured deflections are result of prestressing forces, self-weight of the girder and creep and shrinkage effects before placing the in-situ slab.

Table 1 contains comparison between the measured and the calculated deflections for the section at the middle of a span.

Table 1. Mid-span deflection of the girders

| | | Mid-span deflection at t=730 days [mm] |
|---|---|---|
| **Measured** | | 117.3 |
| **Calculated** | Real creep and shr. | 108.1 |
| | EC2 | 100.4 |
| | CEB-FIP MC90 | 107.4 |
| | fib MC10 | 108.0 |
| | Rusch's method | 115.2 |

The comparison between the measured and the calculated deflections shows relatively good agreement. It suggests that the numerical model is sufficiently accurate to be used for the further analyses.

## 3.3 RESULTS OF THE ANALYSIS

### *3.3.1 Stresses*

Stresses at different stages of loading were calculated for the varied age differences between the concrete parts (30, 90, 365 and 730 days). An age difference of 90 days was chosen as a reference, since it is usually assumed in the design. Figs. 8 and 9 summarize the calculated stresses in two characteristic cross-sections for the reference age difference. In the presented diagrams, sign "-" corresponds to compression. For the sake of clarity, the stresses are presented for each subsequent load stage. However, only the results in the stages affected by the considered effect will be discussed.

Right after the establishment of the composite action between the concrete parts (II.2), tensile

stresses developed in the in-situ slab. They are more pronounced for the section at the intermediate support (Fig.9). The reason for development of such stresses without any external load effect lies in different creep and shrinkage properties between the concrete parts. Having in mind that 50% of the total shrinkage generally takes place in the first 3 months after casting, it will be clear that at this stage the in-situ slab is shrinking at a faster rate than the precast girder. On the other hand, the precast concrete at the element interface is creeping more than the in-situ slab due to the higher initial compressive stresses. This complex interaction between the concrete parts is responsible for the tensile stresses in the slab and redistribution of the stresses in the girder.

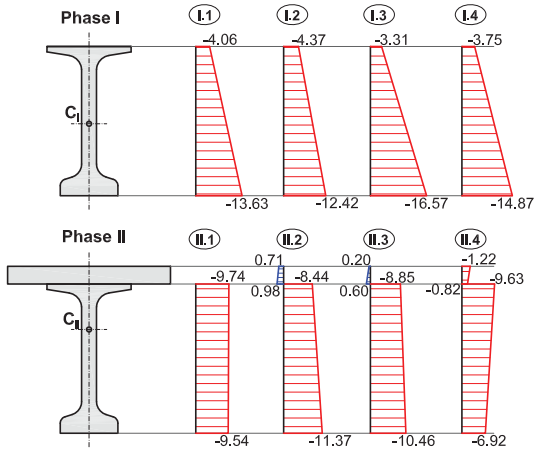


Figure 8. Concrete stresses for age difference of 90 days at mid-span section

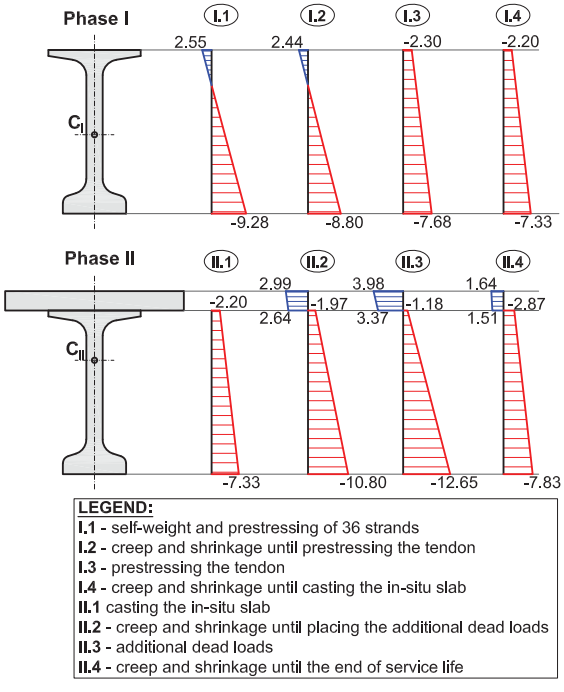


Figure 9. Concrete stresses for age difference of 90 days at intermediate support

The comparison between the stresses in the last two stages (II.3 and II.4) indicates that the creep and shrinkage effects increase the compressive stresses at the top fibre of the girder and reduce the stresses at the bottom one.

The influence of various age differences on the redistribution of concrete stresses is presented in Figs. 10-12. The stresses are normalized in terms of the chosen reference age difference. Figs. 10 and 11 show the relation between the age difference in the concrete parts and stresses in the precast girder, while Fig.12 presents the same relation for the stresses in the in-situ slab.

Fig.10 clearly indicates that the compressive stresses at the mid-span section has a tendency to increase with increasing the age difference. However, this tendency is less pronounced for the stresses at the top fiber. For the considered age differences in the study, the maximum increase in the girder stress can reach 25 % in terms of the reference one.

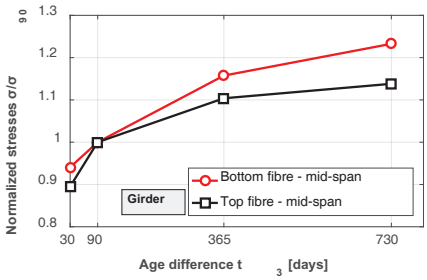


Figure 10. Relation between age difference and noramalized stresses for mid-span section

The similar trend is observed for the stresses at the bottom fibre of the support section (Fig.11). Unlike the bottom fibre, the stresses at the top fibers are reducing for up to 50 % by increasing the age difference. The reduction in concrete compressive stresses means that creep and shrinkage effects induce tensile stresses that increase with age difference.

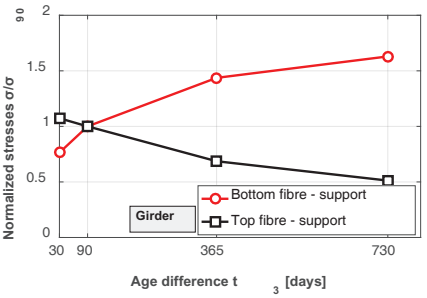


Figure 11. Relation between age difference and noramalized stresses for support section

Such variation in the prestressed girders' stresses due to different ages of the concrete

parts may lead to difficulties in satisfying the serviceability checks for stress limitations.

The age difference has the biggest impact on the development of tensile stresses in the slab part. This is especially pronounced at the intermediate support (Fig.12). The relationship presented in Fig.12 suggests that the deck slab is the most sensitive part of the composite section to the considered effect. For the analysed age differences, the tensile stresses in the slab can be even 3 times bigger than the ones calculated with the reference age difference.

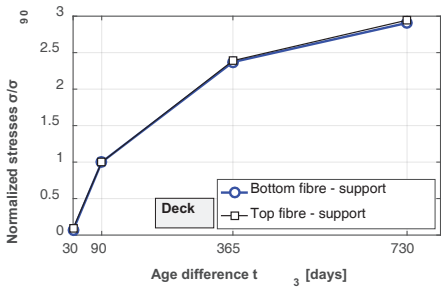


Figure 12. Relation between age difference and noramalized tensile stresses for concrete deck slab

Usually, these tensile stresses are carried by the reinforcement in the concrete slab. The results here suggest that bigger age difference than considered in the design may result in insufficient amount of reinforcement in the slab.

### *3.3.2 Deflections*

Another limit state that has been examined on the effect of different age of concrete parts was the long-term deflections.

Fig. 13 presents the relationship between the age difference between the concrete parts and the mid-span deflections for some characteristic load stages. The deflections in the diagram are presented as normalized values in terms of the deflections calculated for the reference age difference.

It has to be mentioned that the calculated deflections at the end of service life remained positive values (upwards) for all analysed cases. The relations presented in Fig. 13 show that the deflections increase proportionally to the age difference. It means that if bigger age difference than the one considered in the design occurs, then the structure will suffer bigger positive deflections. For the biggest age difference considered in the study, the increase in deflections is somewhat more than 20% during the construction and around 15% at the end of service life.

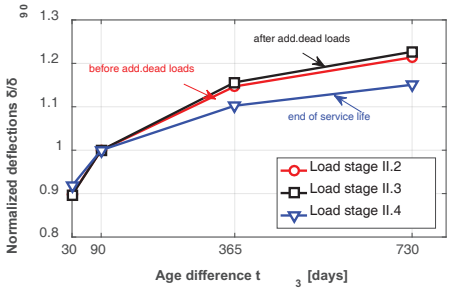


Figure 13. Relation between age difference and noramalized deflections

A misprediction of the positive deflections can be responsible for several serviceability issues. Additionally, it can contribute to many difficulties during the final activities of the bridge construction. Most common issues arising from over- or underestimation of deflections at construction stages are: disability of reaching the designed road level, difficulties in providing the designed asphalt depth and consequently providing correct waterproofing details.

## CONCLUSIONS

Based on the results from the numerical study and the on-site measurements of deflections for the composite concrete bridge beams, the following conclusions are drawn:

- The age difference between the concrete parts has significant effects on the composite beams' behavior.
- Bigger age difference between the girder and the in-situ slab produces bigger compressive stresses in the girder. An exception is the top fibre of the intermediate support section.
- In-situ slab at the intermediate support section suffers several times higher tensile stresses for increased age differences between the composite elements.
- The positive deflections at the end of a construction process, as well as, at the end of a service life are bigger as the age difference between the concrete parts is pronouncer.

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# EXISTING BRIDGES – BURDEN OR OPPORTUNITY?

It is widely accepted that safety and serviceability are primary concerns in bridge design. However, for the most of bridges' service life, these concerns are addressed indirectly by a qualitative measure, defined herein as condition state, which is based upon observable damages recorded during inspections. Condition state is at best, only loosely correlated to safety and serviceability. It would be more reasonable to address safety and serviceability in inspection process directly, using the information on bridge performance obtained during the design and construction. This seems inevitable given the ageing, deterioration, growing traffic and climate change. A vague measure for the deviation of inspected bridge from the "as new" condition i. e. condition state is simply not adequate tool to cope with these challenges. The future Bridge Management Systems (fBMS) should therefore include assessment of safety and serviceability based on inspection results and Structural Health Monitoring (SHM). This can be further enhanced by combining or merging BMS with Bridge Information Models that are currently being developed. The fBMS will thus become an invaluable decision-support tool not only for maintenance planning but also for specification of heavy vehicle corridors, risk assessment due to natural hazards, etc.

Keywords: bridges, service life, reliability, safety, serviceability, performance Indicators, maintenance, BIM

## 1. INTRODUCTION

There is a broad consensus that the benefits of transportation infrastructure for the society cannot be overestimated. The investments in transportation infrastructure raise the growth potential of a national economy, which can be fully exploited by fostering division of labor. It is difficult to quantify the economic benefit of road infrastructure but its lower bound is estimated to be between 4% [1] and 10% ([2] and [3]) of Gross Domestic Product (GDP). Apart from purely economic benefit, the road infrastructure enables road users to be involved in various activities that yield private, public, and social benefits [4]. Maintaining these benefits on the long run in economically efficient,

environmentally responsible, and socially reconcilable manner is the fundamental task of road authorities. They are bound to provide fast, safe, comfortable, and affordable travel.

The road infrastructure comprises various components such pavement structure, retaining walls, galleries, tunnels, bridges, etc. From the users' perspective, it is irrelevant whether a road is carried by a bridge or being in a tunnel or merely resting on soil, so long safe, fast, comfortable and affordable travel from origin to destination is provided. Given that bridges provide passages over otherwise hardly surmountable obstacles, their inability to accommodate the actual traffic can have significant impact on safe, fast and comfortable travel. The road authorities or more concrete the part of them responsible for bridges must specify the traffic that the existing bridges can accommodate, which is in face of ageing road infrastructure and growing traffic increasingly challenging task.

Bridges or parts of them can fail harming life and limb and inducing adverse consequences for economy and environment. A failed bridge causes detours and therefore impact travel times. In some cases, a failed bridge can render a region completely inaccessible with disastrous economic effects. It is therefore not surprising that the **structural safety** is the primary concern of bridge owners since it affects both safe and fast travel. Besides structural safety the bridge owners care about **serviceability**, which relates to **traffic safety** and user comfort that can be affected by bridge deflections and vibrations. If safety or serviceability requirements are not met, the road authorities need to post or even closed the bridge and thus affect road users and the whole economy adversely. This means that road authorities are bound avoid any traffic restriction on the bridges and, at the same time preclude bridge failure. Considering the extent and the age of the road infrastructure in developed countries, this is increasingly challenging task. The tools currently available to road authorities and their engineers to find the balance between safety/serviceability and traffic requirements seem not to be adequate and, in this paper, recommendations are made that may facilitate the management of existing bridges in future.

## 2. CHALLENGES

Safety and serviceability were always the primary concerns in bridge design. However, in course of time the requirements have changed significantly, and the existing bridges are characterized by the several generations of design codes. They have changed both with regard to actions and resistance models. It is therefore not surprising that the old, but undamaged bridge may not fulfill safety and/or serviceability requirements for the current traffic loading.

Furthermore, safety and serviceability can be jeopardized by deterioration processes or sudden events. The resistance of deteriorated bridges can in time reach a level, at which there is an immediate danger of structural failure. In addition to it, the new insights (e.g. statistical analysis) in frequencies and magnitude of sudden events can render some bridges as unsafe and require posting or even their closure.

The increasing traffic volume and traffic loads are probably the most significant challenge regarding the existing bridges. The traffic mix is shifting toward the larger share of heavy weight vehicles, effectively increasing the occurrence probability of load situations that exceed current design loads. Furthermore, there is an increasing number of special transports, i.e. the ones that exceed legal limits and require special permits. In Switzerland, passages of these transports cannot be regarded as rare occurrences as they occur on weekly or even daily basis. Similar situation is also in the USA as reported in [5]. This trend will increase in the future as the transportation industry is interested in using larger trucks with higher axle loads in order to improve their economies of scale. The stiff competition will also lead to platooning i.e. to the trains of wirelessly coupled trucks. The wireless coupling allows to significantly reduce the safety distance between the trucks. This seems to result in a particularly aggressive load situation as these trucks also break simultaneously. The break forces that are mostly neglected in road bridges must be considered in the future and existing bridges need to be assessed for this action.

Finally, climate change may lead to more frequent and intensive gravitational hazards, such as flooding, avalanches, landslides, rockfalls, etc.

In summary, the prudent road authority needs to plan and execute timely intervention in order to cope with following challenges:

- Potentially unsafe bridges that are designed using bygone codes of practice.
- Bridges with reduced resistance due to deterioration or mechanical damage.

- Bridges exposed to natural hazards that are not or not adequately considered in design.
- Increasing traffic volume and loads that can render some bridges unsafe.
- Increased exposure of bridges to natural hazards due to climate change.

In face of these challenges one is inclined to regard existing bridges as burden that requires ever increasing funding. However, the existing bridges are also an opportunity for structural engineers equipped with diagnostic knowledge to specify both economically efficient and environmentally friendly interventions. Moreover, in the same way structural engineers propelled the infrastructure boom in the 20th century, they will be instrumental in the transformation of transportation infrastructure in meeting ever-changing societal needs.

## 3. CURRENT ASSESSMENT AND DECISION-MAKING

The practice in dealing with existing bridges differs quite significantly from country to country, but the common denominator is that it relies on visual inspections. The visual inspections are – if performed by a qualified structural engineer – cost efficient and very valuable source of information. During the inspection, observations are recorded and evaluated. The result of inspections is qualitative indicator, which is named differently from country to country as condition rating, condition state, condition class, etc. Herein, the term condition state is used. Whereas in the design phase the safety and serviceability concerns are addressed directly in quantitative manner, in the service phase, based on inspection results the condition state is determined. The condition state is a vague measure for the deviation of inspected bridge from the "as new" condition. The direct assessment of safety and serviceability during an inspection is regarded as not cost efficient since it is commonly assumed that it always requires an in-depth material investigations and structural analysis.

Based on condition state, owners trigger often costly in-depth investigations or even maintenance actions. In practice, once an in-depth investigation based on condition state is triggered, the maintenance intervention is very likely to follow, even if a bridge can still be used without restrictions. The reasons are different from country to country, but one is surely the visual appearance and related perception of safety that entice authorities to remove all visible damages with appropriate maintenance interventions. In some (rare) cases, a maintenance action is triggered if a bridge fails structural safety and serviceability checks, with the load and resistance models for the design of new bridges. This is clearly inadequate and uneconomical, given the remaining service life and possibilities to reduce uncertainties on existing bridges. Thus, some countries have introduced specialized safety and serviceability checking formats for existing bridges in their code of practice (e.g. [6]).

The above-mentioned approach seems not to be very logical. In the design phase, the wealth of information about safety and serviceability for different load situations is created. This information is unstructured and mostly in paper form. After the commissioning of newly constructed bridge, the documents containing this information is handed over to the road authorities or operators that act henceforth as trustees of the bridges assigned to them. The documents are mostly in archives and in general not easily accessible. During the service life, inspections are performed with no consideration of safety and serviceability information produced during the design and construction phases. It is only within the in-depth investigation that the safety and serviceability are assessed again. There is a substantial gap during the service life of a bridge, in which decision are made based on qualitative indicators, that are sometimes unrelated to the key concerns of road authorities: safety and serviceability.

In most countries, decision/making is supported by databases, in which the results of inspections are stored, sometimes in great detail. The information from design phase i.e. critical load combinations, safety factors, assumed traffic loads is usually not stored in these databases. In some road agencies, there are load rating software that facilitate evaluation of special transports, but it is rarely used in conjunction with inspection results.

Even more surprising is that the relevant information on safety and serviceability is often not stored in the database after maintenance interventions. It can be assumed that the provisions of the current code of practice are fulfilled due to maintenance interventions, but it is not recorded if these are exceeded and by what margin.

It should be noted that within the in-depth investigations a substantial work effort is necessary to find information from the design

phase or previous maintenance activities. In some cases, the information on existing bridges is lost due to negligence or some accident (e.g. fire, flooding).

In some ways, the current bridge management undergoes amnesia because

- relevant information from the design phase and/or in-depth investigation is not stored and/or
- information is stored only in paper form and is lost due to negligence or accidents.

It is not that road authorities and operators are not aware of this deficiency, but to remedy it, they need to be provided tools and resources to efficiently store and access the elaborated information on their bridge inventory. In the time of growing awareness of data importance and big data, it is high time to establish an organizational setting for business processes to ensure the benefit of collected data on the long term.

## 4. ASSESSMENT

The challenges listed in chapter 2 can be only efficiently coped with, if the bridge owners have all necessary information at their fingertips. This means that the current databases need to be significantly improved to accommodate all relevant information either from design phase, inspections, or maintenance interventions. This transition can be quite costly and therefore needs to be performed gradually in several phases as outlined in the following chapters.

### 4.1 SAFETY AND SERVICEABILITY OF UNDAMAGED BRIDGES

The information on safety and serviceability margin of an undamaged (pristine) bridge is essential in the service phase. This information should be structured and include all relevant load cases, which would also allow owners to have a clear picture of possible failure modes and related vulnerable zones that need to be observed in more detail.

The current databases are not structured to accommodate the graphical representation of structural systems and load situations. The material properties and load effects need also to be included in the database as searchable data and coupled with graphical representations. The same apply to load models and provisions of current and previous codes of practice.

A large effort is required to obtain and store information for all existing bridges and this cannot be done within a short time period. Ideally, it could be done together with inspections or in-depth investigations and in this way, one can gradually fill the database.

Fortunately, there is a simplified method to assess safety and serviceability margins due to traffic loads. If the load model used originally for the design of a bridge is known, one can assume that the bridge is designed according to it. This means that the bridge resistance is in minimum as high as to sustain the internal forces due to the load model multiplied with the safety factor. In some sense, the originally used design load model is a proxy for the resistance (or service limit) of the bridge. However, to obtain internal forces, one still need a structural system.

Based on the experience from the load rating software (e.g. [7] and [8]) most non-landmark bridges can be simplified with a series of simply supported beams. Somewhat more sophisticated alternative is to model a bridge with a continuous girder as used in traffic simulations (e.g. [9], [10] and [11]). The simplified model considers only the load transfer in longitudinal direction. The load transfer in traverse direction i.e. across the deck is indirectly considered by defining appropriate effective widths. If the bygone codes of practice didn't require dimensioning in traverse direction, the presented simplification can be problematic. Furthermore, modelling of skew decks or girders with skewed support with simply supported beams is challenging and requires good understanding of the load carrying paths in these structures. Further details on this approach can be found in [12].

Owners can decide, based on their needs to store

- the actual structural system as used in analysis or
- a continuous girder model as used often used in simulation to obtain maximum load effect or
- a series of simply supported beams as in some load rating software.

Independent of this choice, the results of a thorough structural analysis on an adequate structural system can be used to update the resistance (or service limits) of the model stored in the database. The safety or serviceability margins against the current design loads can be expressed therein either as

- the degree of compliance, that can be evaluated for each load situation as a ratio between the available resistance/service limit and resistance/service requirements based on the total factored load effect, or as
- the traffic load capacity factor with which the traffic load can be multiplied and still fulfill the safety and serviceability requirements.

The latter seems to be more useful for road authorities to assess special transports or future traffic load.

## 4.2 RELIABILITY OF UNDAMAGED BRIDGES

The modern codes define the safety and serviceability in terms of reliability i.e. the probability that a bridge will be fit for purpose during its service life. The partial safety factors in modern codes are calibrated to satisfy these reliability requirements. In [13] the target annual reliability index $\beta$ for safety is 4.7 (corresponds to occurrence probability of $1.3 \cdot 10^{-6}$) and for serviceability 2.9 (corresponds to occurrence probability of $1.9 \cdot 10^{-3}$). The bridge is considered as safe and serviceable if specified reliability indexes are not below these target values. If for an existing bridge the degree of compliance is above 1, one can assume that the reliability index for safety exceeds the target value. However, the degree of compliance below 1 doesn't necessarily mean that the bridge doesn't meet reliability requirements. In [14] traffic load capacity factor is evaluated based on reliability assessment for 15 similar concrete bridges in UK that are constructed in 1960's and 1970's to the same design requirements. Nevertheless, the traffic load capacity factors – derived from reliability assessment - vary between 1.9 and 5.2. Given that at that time the design was based on global safety factor, the results are not surprising. For the bridges that are designed or examined using modern codes of practice the scatter is significantly smaller as demonstrated in [15]. However, even for these bridges the evaluation of reliability may be economically beneficial if existing bridges can still be used without restrictions.

Assessing the reliability of existing bridges can be tedious task as one needs to model all actions and material properties as stochastic variables. However, based on experience and available data, a simplified reliability assessment can be performed using the similar approach to the one mentioned in chapter 4.1 (see also [16]). The original design load situation can be used to assess the characteristic value of resistance $R_c^o$ against the chosen failure mode. If the percentile of the characteristic value $R_c^o$ and the type of its distribution is known, one can obtain the resistance distribution for a chosen failure mode. The recommendations for percentiles of characteristic values, standard deviations and distribution types based on construction materials can be found in literature (e.g. [17]). To compute load effects current design loading needs to be used. In general, several actions contribute to load effect and their characteristic values represent different percentiles of their distributions. In addition, the actions are modeled with different distribution types. For instance, the load effect due to the traffic load is normally modeled by extreme distributions. This is supported by measurements and simulation that are performed within research projects all over the world (e.g. [9], [10], [15], [18]). After all, traffic load models are derived from probabilistic analysis based on traffic simulations and weigh-in-motion measurements (e.g. [19]). The distributions for the self-weight and dead load as well as for other actions can be derived based on percentile assumption of their characteristic values following the recommendations from literature (e.g. [17]). The derivation of distribution for load affect *e* and resistance *r*, for the chosen failure mode is illustrated in Fig. 1.

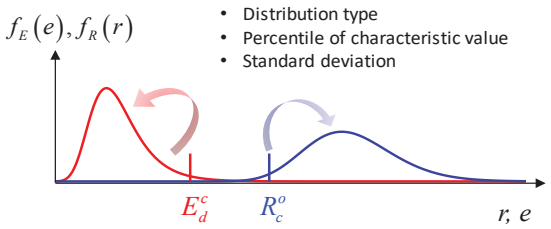


Figure 1. Estimating distribution of load effect and resistance

The load effect can be determined on the simplified structural system and the reliability index can be computed. The reliability obtained in this manner can be regarded as a rough estimate as the material and action uncertainties are modeled based on literature and experience data. These results can be improved if reliability analyses would be performed on the relevant sample of the bridges of same type. The systematic detailed reliability analyses can be also used to update assumptions regarding distribution of stochastic variables.

The traffic load capacity factor can be also obtained based on reliability assessment as in

[14]. It is a deterministic coefficient with which the stochastic effect of traffic load can be multiplied and still fulfill the reliability criterion.

## 4.3 INSPECTION AND IMPACT OF DAMAGES ON SAFETY AND SERVICEABILITY

In order to assess the impact of damages on safety and serviceability, the inspection procedures need not to change significantly. However, there is some additional information that is indispensable if the effect of deterioration and damages is to be appropriately considered in assessment of bridges:

- Based on the design documentation, relevant failure modes need to be defined. These failure modes correspond to the critical load situations used in design and
- for each failure mode, vulnerable zones (see [20] and [21]) are to be defined, in which damages have the largest impact on safety and serviceability.

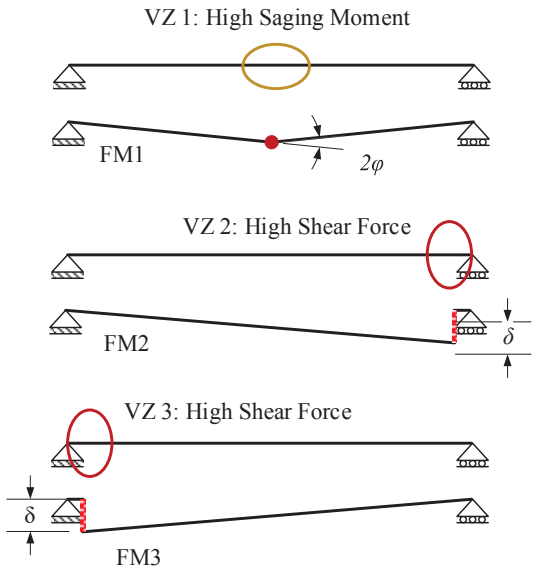


Figure 2. Vulnerable zones and failure modes

Experienced inspectors know intuitively where these zones are, but they can confirm themselves with the readily available information in [22]. The damages outside vulnerable zones can also trigger failures, but for them to occur the extent of damage needs to be significantly larger than in the vulnerable zones. If this seems likely, one needs to define an additional failure mode that can be triggered by the observed damages.

For a simply supported beam the vulnerable zones and corresponding failure modes are illustrated in Fig. 2. In this example failure modes (FM1, FM2 and FM3) are chosen to be collapse mechanisms. However, other failure modes can be selected based on owner's preferences. For instance, the extent of spalling or crack width can be chosen to be failure criteria.

For each failure mode, the corresponding degree of compliance, if semi-probabilistic format is used, or reliability, if probabilistic assessment is required, is to be evaluated for an undamaged bridge. This can be done beforehand either using the simplified approach described in chapter 4.1 and 4.2 or by in-depth examination as described in chapter 4.5.

## 4.4 REALIABILITY OF DAMAGED BRIDGE

The estimation of the impact of inspection (mostly visual) findings on reliability is up to now not seriously considered as a viable option. Visual inspections are considered to be subjective and uncertain allowing only qualitative outcome such as condition rating. Although it is undeniable that observations made during visual inspection are often fuzzy, they can be useful if their inherent uncertainty is modelled properly. In [23], a subset of observations collected in a survey are identified to have an impact on reliability. The corresponding uncertainties both regarding the inspection process and an impact on reliability are however not addressed, which remains to be the topic of a future research.

In addition, there is also useful data that is simply not collected. For instance, if merely "Corroded reinforcement" on a certain bridge element is reported, this means that the reinforcement corrosion can be anywhere on its elements i.e. its location needs to be uniformly distributed. Likewise, a spalling area and a section loss can be also modelled with slightly informative or non-informative distributions. If, however additional information is available such as that the reinforcement corrosion is located in vulnerable zone, the uncertainty with regard to its influence on reliability can be significantly reduced. The quantitative information on spalling area and section loss can further reduce uncertainty.

To consider the effect of inspection findings the Bayesian network can be used as shown in Fig. 3. In this example it is assumed that the visual inspection revealed a spalling area with the reinforcement corrosion with a section loss of e. g. 10%. This is a typical entry as shown in the survey performed by the WG1 of the COST Action 1406 (see [23]). The location of the

defect is not known and there is inherent uncertainty with regard to this section loss, which can be modelled in the node “Corroded reinforcement” in Fig. 3.

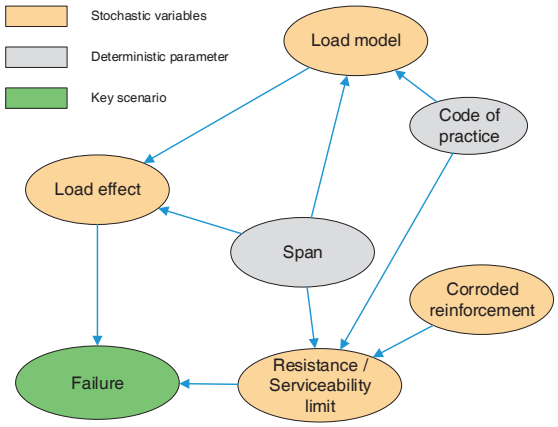


Figure 3. Reliability evaluation of damaged bridge

## 4.5 IN-DEPTH INVESTIGATIONS

If the results of an inspection raise doubts regarding acceptable safety and serviceability, the in-depth investigation should be triggered. The in-depth investigation should include all measures that may reduce uncertainties such as testing of material properties, checking of dimensions, stiffness measurements, etc. Furthermore, archive documents must be duly examined since they may reveal assumption made regarding load situations and material properties. For hidden structural elements such as foundations and reinforcements, old plans are the only source of information. Finally, the exposure of bridge to natural hazard needs to be investigated based on the newest findings (e.g. update of magnitudes, re-assessment of site condition).

The structural analysis needs to be performed with nonlinear methods that address failure modes appropriately and yield realistic failure probabilities. This doesn’t necessarily mean that sophisticated analysis is necessary, in most cases the skillful application of the limit theorems of the theory of plasticity is sufficient.

The bridge should be examined for all relevant load situations and the results need to be stored in the database in a structured form. Gradually, by means of in-depth investigations, high quality information on all bridges will be stored in the databases allowing bridge owner to manage their inventory efficiently. The growing experience will also allow for more accurate assessment of reliability based on inspections as Bayesian nets can be updated introducing new high-quality data.

## 5. MAINTENANCE PLANNING

Bridge Management is not confined to bridge assessment based on inspections and in-depth investigations but also includes maintenance planning. The short-term maintenance planning is based on in-depth investigations and structural analysis and include detailed specification of interventions that are to be taken shortly thereafter. The mid- to long-term maintenance planning is a process, in which different intervention scenarios are developed. The interventions are not specified in detail and their costs are rough estimates backed by experience. The goal is to estimate financial and other needs well in advance and avoid unpleasant surprises. Furthermore, early planning allows to choose the optimum time for interventions and reduce long-term costs.

The general approach is presented in Fig. 4, where it is assumed that an inspection is performed “today”. The results from the inspection revealed some damages that in conjunction with the actual loads lead to worsening of the safety and serviceability levels that, however, still meet the requirements for existing structures. This procedure is explained in previous chapters. For mid- to long-term maintenance, planning forecasts for serviceability and safety are performed predicting that serviceability criterion will be not fulfilled at the time instance marked “Tul”. This means that the intervention needs to be executed no later than at that point in time, if serviceability requirements are not to be violated. However, it may well be that a scenario that includes an intervention at the time instance “Top”, has lower long-term costs than the one with the intervention at the time instance “Tul”. The Fig. 4 doesn’t show any interventions after “Tul”, but normally the ensuing interventions are considered in long-term costs.

The forecasts of safety and serviceability over time defines the time instance at which, at the latest, an intervention is necessary. However, these forecasts cannot be used to define the type and the costs of the interventions. To this end afore-mentioned qualitative indicators (e.g. condition rating, condition state, condition class, etc.) are used. The application of qualitative indicators to generate interventions is well-established in current approaches to maintenance planning as explained in the following chapter. The problem lies with the usage of these qualitative indicators as proxies for safety and serviceability. The worst condition state represents herein the safety and

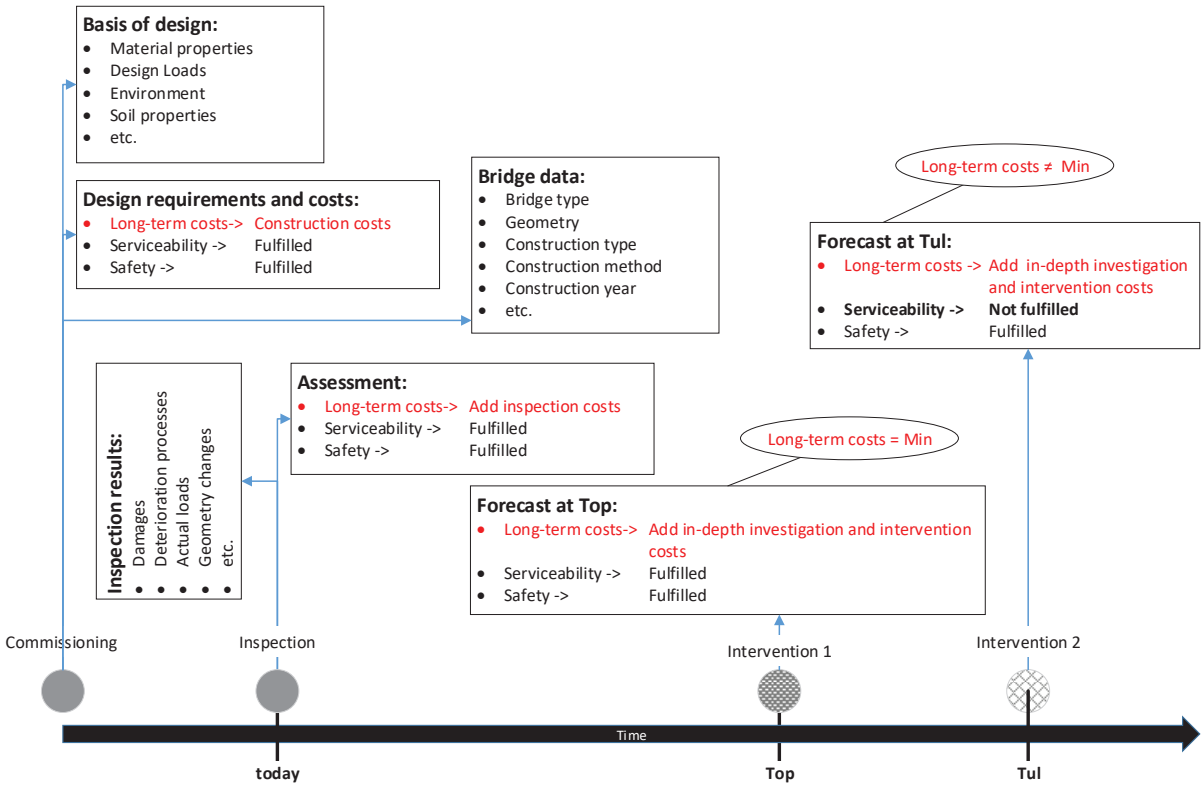


Figure 4. General approach to mid- to long-term maintenance planning

serviceability threshold. To remedy this problem of mid- to long-term maintenance planning adaptations are suggested that are explained in the following chapters.

## 5.1 CURRENT APPROACH TO MAINTENANCE PLANNING

Currently, maintenance planning is based on qualitative indicators. The procedure can vary quite significantly from country to country, but the main distinction lies in the choice of **assessment unit**. The assessment unit is the physical entity of the lowest granularity that is assessed during inspections. The assessment units can be whole bridges, bridge components (super-, substructure and equipment), bridge elements, group of damages or even single damages. All these different assessment units can be found in current BMS, based on the owner or operator's needs.

The scale of condition state is chosen in such a manner that only if an assessment unit is assigned into the worst condition state – being a proxy for safety and serviceability thresholds - immediate interventions are required. The maintenance planning considers therefore only interventions that are to be initiated before the worst condition state is reached. The optimum maintenance intervention(s) is/are the ones that minimize the long-term agency costs and in some cases user costs during these interventions, as in [24].

The interventions are performed on **planning units** that in some BMS coincide with assessment units and in the others, they differ from them. If assessment units are damages, then it is often more reasonable to plan interventions on affected bridge element or the whole bridges than on a single damage. Based on the condition state of assessment units, the intervention on planning units are determined based mostly on heuristic rules. To this end, technically feasible maintenance intervention on planning units are catalogued together with their unit costs and effectiveness. For instance, for the same condition states of assessment units, both rehabilitations on elements as well as replacement of the whole bridge are technically feasible. The choice of the optimum intervention is subject to minimization of the long-term costs as in [25].

This approach has significant drawbacks since it focuses solely on restoring damage-free bridges, without considering directly the safety and serviceability or consequences of possible failures. It is therefore that mid- to long-term maintenance planning can significantly deviate from the short-term planning and executed interventions, which are often triggered by safety and serviceability criteria.

## 5.2 DETERIORATION MODELS

The key ingredient of mid- to long term maintenance planning is a deterioration model. The deterioration model allows condition state forecasts of assessment units and determine possible maintenance interventions in the future. The deterioration models can be derived from physico-chemical deterioration processes, and there is significant research in this area (e.g. [26], [27]). Notwithstanding the undeniable progress in this research, most deterioration models in current BMS are based on statistical analysis of past condition data (e.g. [28], [29]). Many BMS use Markov chains to model deterioration as it supports the discrete scale for condition states and transition probabilities can be easily derived from condition data. Finally, with Markov decision process, which is based on Markov chains and catalogued interventions, the optimum set of intervention can be estimated with modest computational effort. However, Markov chains are mostly employed for elements or components, whereas their applicability for individual damages waits to be proven.

## 5.3 IMPROVEMENT TO THE CURRENT APPROACH

The current approach to mid- to long-term maintenance planning based on condition states of assessment units is used quite successfully to define maintenance scenarios and estimate long-term costs. However, it doesn't consider safety and serviceability directly and therefore the estimation of the optimum time instance to perform intervention can be quite wrong. A bridge with high safety and serviceability margin may be used significantly longer than suggested by an assessment unit in the worst condition state. The opposite is also true: A bridge barely meeting serviceability requirements may need to undergo an intervention even before a single assessment unit reaches the worst condition state.

On one side, it is necessary to estimate both condition states of assessment units and based on them define intervention scenarios on planning units and related costs. On the other side, the reliable forecast of safety and serviceability are needed to estimate the remaining service life without interventions. It is shown in the previous sections that meaningful safety and serviceability assessment is possible only if individual damages or damage groups underlying the same damage process are assessed. This means that the assessment unit must be either an individual damage or a damage group. Consequently, the deterioration models need to be developed for these assessment units and based on the evolution of these assessment units, the safety and serviceability can be evaluated. The current deterioration processes used for elements and components can be quite useful as baseline for calibration of deterioration models for damages and damage groups.

The improved approach is presented in Fig. 5 with damage group as assessment unit as in Swiss Bridge Management System KUBA. An individual damage can also be a damage group, so there is no loss of generality if a damage group is used as an assessment unit. The basic bridge data include apart from element data also the most probable failure modes. These failure modes can be determined from bridge design documents as explained in 4.3. A bridge can have several elements and several failure modes. A failure mode can include several vulnerable zones as shown in 4.3. For instance, a collapse mechanism of an hyperstatic structure has always several places where rupture or yielding occurs and these are vulnerable zones. These vulnerable zones can also participate in several failure modes. Hence, there is many-to-many relationship between the "Failure mode" and "Vulnerable zone".

Using the deterioration model for an assessment unit, the severity and extent of the damage group can be predicted at some time point T. The impact of the damage group on resistance and/or service limits in vulnerable zones can be estimated using heuristic rules or Bayesian nets as in [30]. Following the same procedure as in 4.1, the degree of compliance for safety and serviceability at the time T can be assessed. These steps are represented in the gray column in Fig. 5 to highlight the addition to current maintenance planning procedure.

The path from damage group forecast at time T to maintenance interventions is already implemented in KUBA and many other BMS. Herein, the catalog of technically feasible maintenance interventions for each damage type and severity of a given element type is of pivotal importance. The cataloged interventions are characterized by their unit costs and effectiveness. The cost predictions are generated by the heuristic rules and optimization depend on these data. The catalog is the result of a statistical analysis of executed preservation projects and is subject to changes reflecting developments in construction industry. The unit costs refer to a specific measurement unit. The unit for maintenance

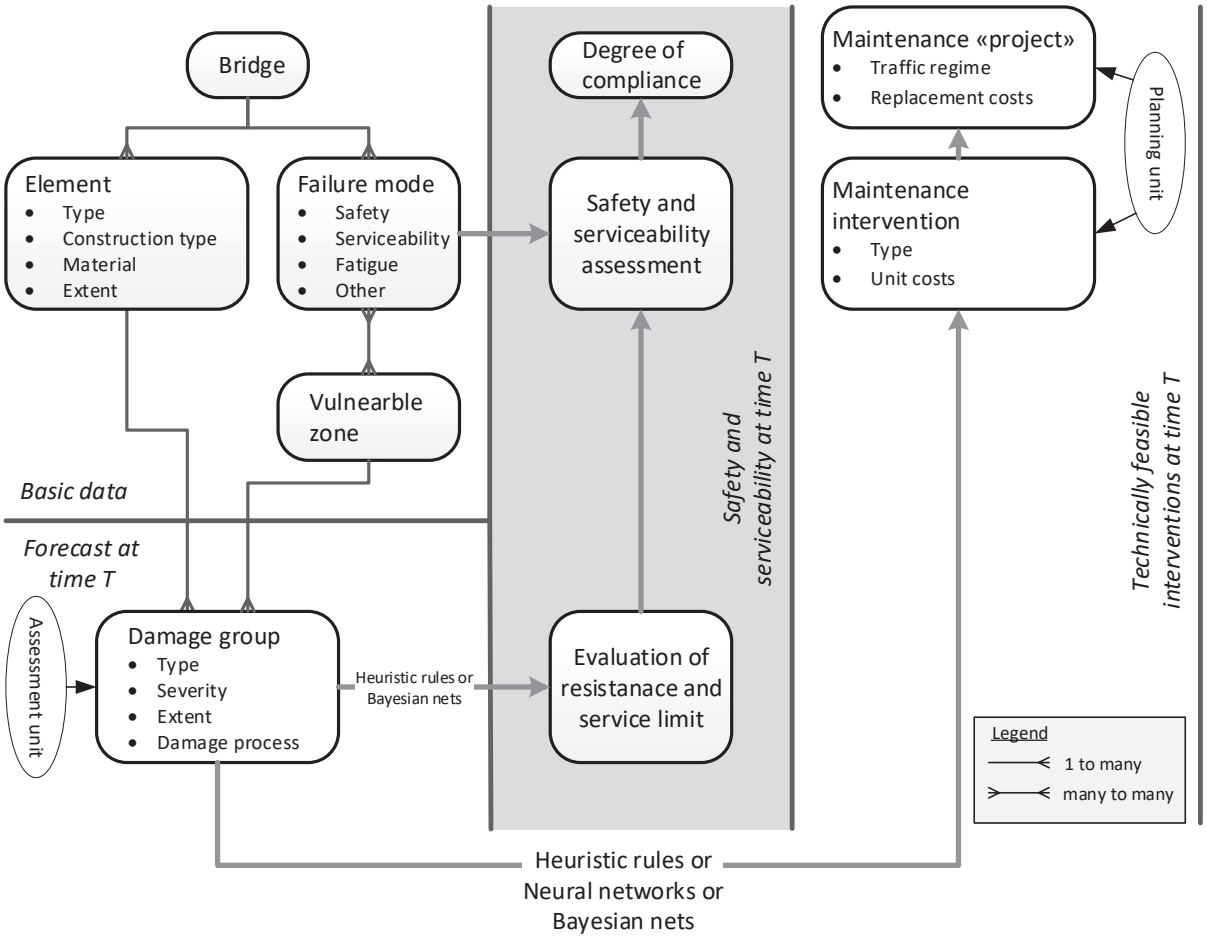


Figure 5. Assessment of degree of compliance related to safety and serviceability and determination of maintenance intervention at time T

work on steel elements, for example, is the square meter [$m^2$], i.e. the surface area. This unit must be the same unit used to measure the extent of damage groups.

The technically feasible maintenance interventions are then aggregated using heuristic rules and threshold values in maintenance "projects" on bridge level. The quotation marks indicate that these are not engineering "project", but rather a very general outline of the maintenance work that is feasible at the time T. In addition to "projects" aggregated from elements, a replacement "project" is added as an option, as it can be assumed that it is the costliest maintenance option. For the high-volume bridges, a traffic regime and eventual closures during the execution of the maintenance works needs to be defined. This information is needed to evaluate societal costs. The interested reader can find further details on estimation of user costs in [31].

A maintenance intervention results in improvement in degree of compliance regarding safety and serviceability. Currently, the estimation of this improvement is not possible, since there are only data on improvement in condition states. Also, similar refers to the effectiveness of the intervention. Still, the effectiveness of intervention related to condition states can be useful: If the condition state is restored to the best condition state after the intervention, one can assume that also the degree of compliance of the undamaged bridge is restored.

In summary, the proposed improvements allow decision maker to

- estimate remaining intervention-free service life of a bridge in rational manner, using safety and serviceability assessment,
- consider the improvement in safety and serviceability and compare with the costs to obtain this improvement, and
- plan reliable financial needs and needs for other resources on mid to long-term, considering the forecasts of traffic loads.

The data collection effort to implement these improvements is quite modest. However, the

data needs to be collected conscientiously and the procedure to evaluate degree of compliance related to safety and serviceability needs to be updated to match performed in-depth investigations.

One step further would be to replace the degree of compliance with reliability assessment at some time T as explained in 4.2. The estimation of reliability over time as in [32] is just a stepping stone to estimation of risk over time.

### 5.4 APPLICATION OF BIM

It is foreseeable that Building Information Models (BIM) of both newly built and existing bridges will be available (see e.g. [33] and [34]). These models will be included into the Bridge Management System (BMS) and will significantly enhance the quantity of useful information in future BMS (fBMS). A BIM can embed realistic structural system of a bridge as well as the relevant load situations (see [35]). The evaluation of the reliability or safety/serviceability would be therefore possible quasi, on-the-fly within the fBMS, provided that the observations and results from SHM can be adequately integrated in BIM.

In principle, the inspection results can be directly captured in the BIM using photogrammetry or some other procedure. Cracks, spalling, deformation, and other defects will be a part of a BIM, which in most cases alter the BIM geometry. An information model and a candidate binding to Industry Foundation Classes (IFC) has been developed in [36]. In [38] the damages are modelled using exiting IFC entities. Both studies demonstrated that that IFC provides sufficient functionality to serve as a basis for integrating relevant defect information and imagery.

The results of past inspections, e.g. damages are stored in a temporal BIM, which is an enhancement of the existing temporal databases. This would enhance possibility to statistically estimate deterioration models. Furthermore, the temporal BIM is a valuable data source that will allow researchers to better understand physical deterioration processes. The data stored in fBMS include also other changes that a bridge experience during its life span. This includes strengthening, widening, seismic retrofit and other structural changes. In short fBMS is similar to the 6D BIM or Asset Information Model (see [37]), which continues to be updated during the whole service life of a bridge.

For purposes of maintenance planning, the BIM with damages can be used as the basis for the deterioration simulation, which can provide significantly more accurate forecast that the current methods. The reasons for this are twofold:

- The resistance and loads i.e. probability of failure of an intact structure is duly taken into account.
- The exact location of a defect is known so that its effect on the safety and serviceability can be assessed in a more accurate manner.

The same applies to maintenance scenarios, which can be determined more accurately, both regarding their types as well as regarding their costs estimates.

## 6. CONCLUSIONS

The need for more economic utilization of transportation infrastructure is also a challenge for bridge owners. They need to have readily available, high quality information on their bridge inventory in order to cope with gradual deterioration, growing traffic demands and increased frequency and magnitude of natural hazards. To obtain and store this information is a task that cannot be accomplished in a short time. In this paper, the methods are proposed that use data that is already available and allow their gradual refinement. They allow better exploitation of inspection results to assess safety and serviceability and improved decision making regarding in-depth investigations and maintenance interventions. The assessment of safety and serviceability during the whole service life allows also more responsive processing of special permits and established corridors for heavy weight vehicles.

The presented methods are only steppingstones toward a new generation of BMS that will integrate BIM and allow more accurate and prompter information on bridge behavior. Current databases need to be enhanced with BIM. The data generated during the design and construction process must be handed to the authorities or operators that act henceforth as their trustees. It is essential that this data can be transparently and intuitively used in the exploitation phase. To this end, the owners need to set requirements that are independent of software packages used in design and construction phase.

Finally, the road authorities and/or operators must provide both financial and personal resources to maintain a fBMS and its data. Owners need in-house competence both in

structural and decision engineering as well as in information technology similar to the banking sector, where competence in information technology, financial and decision engineering is required. It is not a coincidence that in both sectors the term “Asset Management” is widely used.

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# СЛЕДЕЊЕ, ПРОЦЕНА И САНАЦИЈА НА КОНСТРУКЦИИ
# MONITORING, ASSESSMENT AND REHABILITATION OF STRUCTURES

**ДГКМ**

ДРУШТВО НА ГРАДЕЖНИ КОНСТРУКТОРИ НА МАКЕДОНИЈА

**MASE**

MACEDONIAN ASSOCIATION OF STRUCTURAL ENGINEERS

**18** МЕЃУНАРОДЕН СИМПОЗИУМ
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# INSPECTION AND MAINTENANCE OF STRUCTURES OF LARGE FAILURE CONSEQUENCE CLASS - EXAMPLE TROPICAL ISLAND

All sustainable structural maintenance and therefrom excepted Life Cycle Performance has to be based on reasonable data management, done by the owner or the user. Knowing this, a few decades ago (in 1998) the German Federal Highway Research Institute led the development of a computer based building data management system for bridges. By limiting project on the bridges, which are up to 99% public property, the demands of the such system were reduced to reasonable size. At the time, the approach would not have been suit-able for other civil engineering structures for the reasons as: high variability of structural concepts, large amount of different structural components, concepts even within one single structure, multiple contractors at one building, high owner change rate, bad database, and so on. The structural failures during the winter 2005/06 demonstrated the urgent need for clear concept of Data Management and Controlled Structural Maintenance for other civil engineering structures, besides bridges, once again. Responsible owners recognise that and react accordingly, with the fundamental demand of controlled maintenance for structures. It can be facilitated with appropriate model similar as for the bridges. As example, a-step-towards-a solution is presented on the Tropical Islands Dome south-east of Berlin, the largest free span Dome in Europe. Basic considerations herein are: practical use for the owner from today onwards, easy updating, long durability, low permanent costs, and some others. This can be done following High-Brain/Low-Tech concept knowing that such basic electronic formats are often used to deal with a very complex structures. The expected result is an easy-to-use database for the structure and controlled maintenance of structures over their service time in next decades. It is an example for treating the problem with the aim of keeping maintenance costs predictable and aims to fulfil the precondition of every structural design code in order to keep the calculated safety level – *adequate maintenance over the structure service time*.

Keywords: structures, inspection, data management, assessment, maintenance

# 1. INTRODUCTION

## 1.1 PROBLEM

The collapse of a couple of wide-span structures during the last decades and the accumulated failures during the winter 2006 (after the structural collapse of a glulam roof of an ice hall in Bad Reichenhall in Germany, a steel roof of single-story exhibition in Katowice in Poland and a reinforced concrete roof of a supermarket in Moscow in Russia with a death toll of 120 persons) forced officials to focus on the problem of scheduled and controlled maintenance on high rise structures.

Basis of all proposals is the existing system for assessing and maintaining bridges. Here, a staged schedule in intensity is given (DIN 1072). The system and according software for structural maintenance and documentation were developed by the German Federal Highway Research Institute and prescribed (BMV, 1998).

As reaction on these accumulated failures of the fundamentally well-designed structures, the new system was established, with intention to be used later for building constructions within large failure consequence classes (ARGEBAU, 2006). The task is not easy as it may seem.

## 1.2 REGULATION BY LAW

The owner of a structure is responsible by the law for obtaining sufficient safety, §3 MBO (ARGEBAU, 2002). This is also the fundamental demand of Eurocode 1 (EN 1990-1). Despite a couple of regulation on fire safety and the safety of other technical components (fume outlets, elevators etc., summarized list in (AIG, 2006)).

Until now in Germany there was nearly nothing that introduced such structural demands to the owners. An exception are some hints in (DIN 32736). Also, regarding Swiss concept of a safety and utilization plan, less or nothing is known by engineers in practice (Schneider, 1994). The demand of a sufficient structural documentation is neglected both by the owners and the officials at the moment. Anyone can prove this by simply visiting any public archive.

## 1.3 APPLICABILITY TO BUILDING CONSTRUCTIONS

It is not possible to apply unmodified the system for bridges (BMV, 1998) to building constructions, opposite to what the guideline by the ARGEBAU (ARGEBAU, 2006) suggests. There are two main reason. These are the higher diversification both in the ownership and in the structural details. Both is linked to significant higher reconstruction rates. These are some reasons which prevent the development of a special software. At the moment, the question arises if the simple sufficient inspection system can be reached easier and with less technological effort. This shall lead to a higher acceptance by the owners than a statement “it is prescribed by law” does.

# 2. SOLUTION APPROACH

## 2.1 PREPARATION OF A BUILDING DOCUMENTATION

Usually every structural work is completed with an “As Build” documentation. For building constructions this must be done by architects and engineers in the “Service Rendering Phase 9” (HOAI 2002).

In times of budget control this position is sometimes not included in the according contracts. Even if this Service Rendering Phase is done during erection of the building, documents were very seldom carried forward. So, the first step is the preparation of a building documentation. Of course, this documentation can be made in a scientific database as e. g. the one used by the Gemeinsamer Ausschuss Elektronik im Bauwesen (GAEB, 2007). But until now, these approaches are still under development and the main demands: sustainability and accessibility of the data are not fulfilled. So, the best practice approach is, in the opinion of the authors, a simple text document, accompanied by drawings in a *.pdf; *.dxf and *.dwg format. These documents shall contain all necessary information about the structural details according to (DIN 1072). Especially important are overviews As-Built drawings in an appropriate scale. From a practical point of view these blueprints shall not exceed the size DIN A1. These drawings are needed to document all future inspection results.

Table 1. Service Rendering Phase 9

| standard activity | special activity: |
|---|---|
| - Inspection for warranty covered defects<br>- Inspection of rehabilitation of warranty covered defects | - As build drawings<br>- Inspections<br>- Inspection and maintenance plans<br>- Generation of an object documentation<br>- … |

## 2.2 INSPECTION SCHEDULE

The inspection schedule is proposed in the guideline by the ARGEBAU (ARGEBAU, 2006). This guideline rules the minimum qualification of the personal and the minimum inspection amount. If this guideline is sufficient, and if it is applicable to a structure with a large consequence class in practice will be presented at an example in the succeeding chapter.

## 2.3 INSPECTION AND MAINTENANCE DOCUMENTATION

It is not described how the results have to be documented and how an assessment of the condition of the structural components has to be done. The existing grading system for bridges may be a solution. There, grades were assigned for the load carrying capacity, the usability and the durability of components or the building. This assignment is done according to the influence of a damage. These grades provide a decision support to schedule maintenance and other actions.

Table 2. Inspection Schedule

| category | type of building | inspection | detailed inspection | extensive inspection |
|---|---|---|---|---|
| | | every N years | | |
| I | Buildings with more than 5000 users | 1-2 | 2-3 | 6-9 |
| II | Structures with more than 60m height; 12m span; 6m cantilever beams and all other structures with high damage potential | 2-3 | 4-5 | 12-15 |

Table 3. Inspection Guideline

| performed | inspection | detailed inspection | extensive assessment |
|---|---|---|---|
| by whom | owner | e.g. structural engineer | e.g. check engineer |
| how | without tools | without tools | with tools, handclose |
| | checking structural elements for:<br><br>cracks, excessive deflection, corrosion,…<br><br>Check for decay promoting conditions:<br><br>like leakage | As before and e.g. additional check by structures:<br>Concrete:<br>carbonization / chlorides / concrete cover<br>corrosion / …<br><br>Steel:<br>loose screws / cracks in welds / corrosion / coating thickness / …<br><br>Timber:<br>splits / decay / moisture contend / …<br>to be continued… | |

## 3. EXAMPLE OF THE DOME OF TROPICAL ISLANDS

### 3.1 INTRODUCTION OF THE BUILDING NAMED TROPICAL ISLANDS

At the end of the 20th century a new generation of airships has been developed by the CargoLifter AG. Therefore, a new hangar for two airships has been built. With a span of 210 m, a height of 107 m and a length of 363 m it has been the largest single-story building in the world. The central part of the hangar is of a half-cylindrical shape consisting of five steel arches covered with a textile membrane. At both ends of the building are the doors which consist of two fixed and six moving elements. They form a semicircle in plan and a quarter-segment of a circle in elevation.

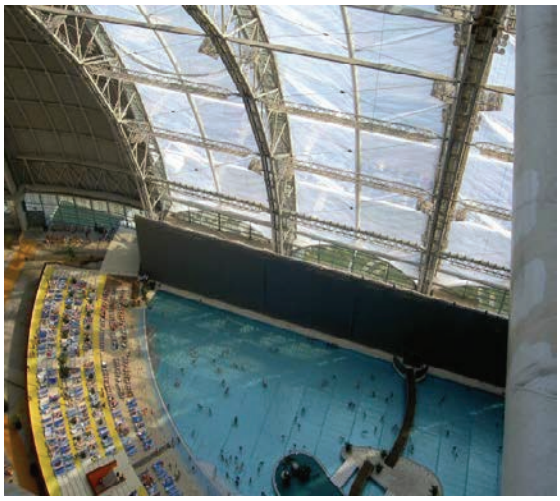

Figure 1. Tropical Islands Dome

After the breakdown of the Cargo-Lifter AG the real estate was bought by Tropical Island, and converted to a leisure facility. Within the former hangar houses in native tropical style (Fig. 1), a lagoon, a sauna and many other facilities were built-in. To generate tropical climate, raise a rain forest and gain additional sunlight it was necessary to dismantle the membrane from the southern side. Transparent ETFE cushions, supported by a cable net, were installed. The capacity is 8000 visitors, so the building belongs to the category I (Tab. 2).

### 3.2 BUILDING DOCUMENTATION / AS BUILD OVERVIEW DRAWINGS

Not only the first owner, Cargo-Lifter AG, but also some contractors went bankrupt during the last decade over this structure, so the owners changed several times. Also, the design-responsible officers were changed. This caused a significant loss of information about structural details. So, the first step was to write a summarizing building documentation As-Built overview drawings, including some standard details too (Fig.2).

### 3.3 INSPECTION SCHEDULE

The inspection guideline (Tab. 3) had to be extended and changed in detail. A couple of reasons therefore can be named. Structural parts have a different potential of structural failure. So, it will make sense to inspect the main arches more often or more in detail, than the infill of the gates. Also, it makes sense to check innovative and new structures, as the cushions, more often to detect early failure. At least, some inspection is already covered by inspection contracts like fume outlets, cushion and membrane structures, or need more often inspection like guard rails. A part of the inspection plan, here for the cushions, including technical components, is given in the Table 4.

A proper inspection plan contains also information about work preparation, inspection facilities.

### 3.4 INSPECTION RESULTS AND DOCUMENTATION

In Figure 3 two usual maintenance problems are shown. In the left picture of figure 3 the effect of the leakage in the facade is shown. Here it is linked to a basin-like structural detail. The water was already removed by vacuum cleaner. The solution is fixing of the leakage. On the right picture beginning of corrosion on the bracings is shown. It may be that the bracings have the thinnest paint cover. Here a new paint job is advisable during the next years.

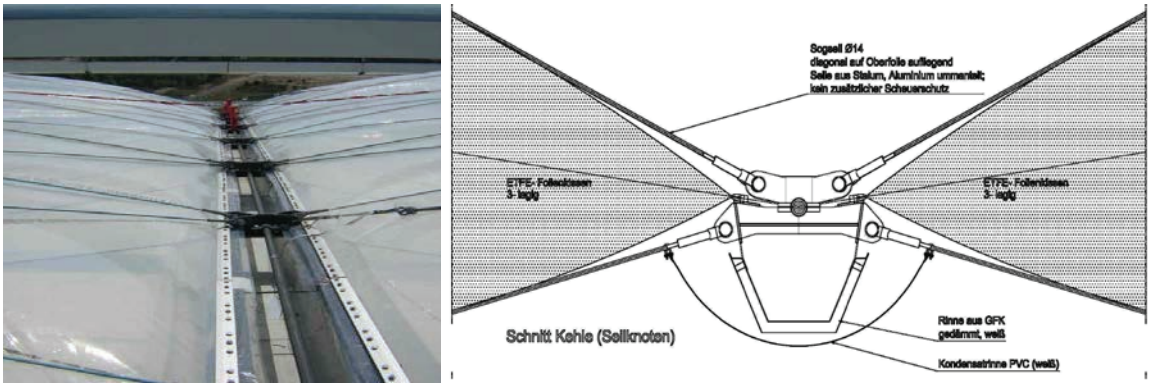


Figure 2. As-Built overview drawing: Detail of cushion

Table 4. Part of the inspection plan

| every .... year | | | | |
|---|---|---|---|---|
| 0.25 | 1 | 3 | 6 | |
| | | | | Check Service Contract (DIN EN 13269) |
| | | | | Check of emergency kit |
| | | | | simple visual inspection on water penetration / damages |
| | | | | |
| | | | | *Technical Components* |
| | | | | Check trouble indicator, document errors |
| | | | | Check suction hole |
| | | | | Open and close hatches |
| | | | | Check sensors (Pressure / Temperature) |
| | | | | Check sensor (Snow) |
| | | | | Check sensor (Wind) |
| | | | | Service Compressor VDI 3801 |
| | | | | Service Dehumidifier VDI 3801 |
| | | | | ... |

Figure 3. Some inspection results

Table 5. Documentation and assessment of inspection results

| Date | Component | Subcomponent | Position | Who | Action | Result | Safety | Durability | Usability |
|---|---|---|---|---|---|---|---|---|---|
| ??? | Crane Runway | Crane Runway | | Check Eng. Prof. Pasternak | Check Eng. Report | Acc. To report, not finished, no loads allowed | 0 | 0 | 4 |
| ??? | Fixed Gate | Weather Strip | D / 7-4 | Trop.-Islands | Visual Inspection | Weather strip, heavy leackage, water at framework knots | 0 | 2 | 1 |
| ??? | Bracings | | G-H / 4-7 | Ceno-Tec | Check Pretension | Pretension to small | 3 | 0 | 3 |
| ??? | Main Arches | Windows | All Arches | | Check | Cracks at the covering bead | 1 | 1 | 1 |
| ??? | Pillows | Inside Membran | | Ceno-Tec | Check | Cracks, where the ropes separate | 2 | 1 | 2 |
| ??? | Gate 2 | Gate Rails | B-C 6-7 | ???? | Check | Drainage defect, 10cm water | 0 | 2 | 1 |

It may be advantageous to document this inspection results (and the according maintenance) in a sufficient form. Here the grading system from the bridges may be adopted.

The three categories may be adopted: for safety, durability and usability. A grade of 0 means, similar to bridges, no effect. With a grade of 4 the component is not safe, durable and usable. An easy example for this case is the crane runway. It was not completed after the break down of CargoLifter AG. So, the cranes were not approved, but only the static load-carrying system by the check-engineer. So, it has no effect for safety, because using it is prohibited.

## 3. SUMMARY

It was possible to show, that even simple tools and limited efforts, can lead to a sufficient management of inspections and maintenance. So, the public demand on safety is accomplished. From point of view of owners, immediate advantages are arising: structural maintenance can be scheduled more efficiently, early failures are found during warranty. This win-win situation shall be communicated more aggressively to gain additional acceptance for the common aim: sustained and well maintained structures.

The retention requirements have been transferred to the owners in the newer building regulations, see e.g. §16 of the Brandenburg Building Regulation (BbgBauVorlV) of 2016. At the same time, the archiving requirements for the building authorities were reduced. Whether this increases the quality and availability of the inventory files is to be proved yet. It remains to be seen how the more recent efforts for introducing digital approval procedures (see final DBGV NRW from 2019) will ensure the long-term availability of the construction files. Together with the transfer of the retention obligation to the owners, permanent availability in this digital form is very questionable.

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# AVOIDING MISTAKES DURING ASSESSMENT, DESIGN AND CONSTRUCTION OF STRUCTURAL REPAIRS

The process of structural repairs or retrofit generally includes evaluation of an existing structure and its design or as-built capacity, design of the required repairs, and the construction of designed repairs. If the evaluation was not done properly, the design may not address the causes of the structural deficiencies and the repair will not be adequate. After proper evaluation of the existing conditions and design of appropriate repairs, it is important that the construction of the repairs be performed properly. Failure of any of these steps may result in inadequate or faulty repairs. This paper discusses, in more detail, the specific projects and the mistakes made in the repair process.

Keywords: structural repairs, mistakes, assessment, repair design

## 1. INTRODUCTION

Over the years, more existing structures are being repaired or structurally retrofitted. In times of the economic slowdowns, as the construction of new structures slows down, more and more existing structures are being reused and upgraded. The main reasons for the repairs and retrofits are deterioration of structures over time, material failures, deficient design, faulty original construction or change of use which requires increased load-carrying capacity.

The process of structural repairs or retrofit generally includes evaluation of an existing structure and its design or as-built capacity, design of the required repairs, and the construction of designed repairs. In order to complete the appropriate and necessary repair, all these three steps in the repair process need to be done properly. It is obvious that if the evaluation was not done properly, the design may not address the causes of the structural deficiencies and the repair will not be adequate. After proper evaluation of the existing conditions and design of appropriate repairs, it is important that the construction of the repairs

be performed properly. Failure of any of these steps may result in inadequate or faulty repairs.

Over the years, the author's firm has designed repairs or investigated the design of repairs performed by others for hundreds of structures and has encountered instances where there were mistakes made which resulted in ineffective repairs. The following sections will discuss, in more detail, the specific projects and the mistakes made in the repair process.

## 2. MISTAKES DURING ASSESSMENT OF EXISTING STRUCTURES

It is taken for granted that the dimensions of the existing structural members should be measured properly and that the location and size of steel reinforcing in concrete structures will be determined by non-destructive testing methods and verified by limited exploratory openings. Mistakes are more likely to be made in interpretation of observed conditions and test data during the load testing of structures.

As part of the evaluation process, exploratory openings are made in concrete structures to evaluate the condition of underlying concrete or the buried membrane, or to inspect post-tensioning tendons and then the observed conditions are extrapolated to the rest of the structure and projections are made of anticipated repair quantities. This methodology could lead to wrong results. For example, there have been instances where the actual quantities of post-tensioning tendon repairs exceeded the projected quantities by a factor of 5 to 10. It is almost impossible to apply this methodology to this type of repairs. The knowledge gained by examining exploratory openings needs to be supplemented by engineering judgment and past experience with similar projects.

Sometimes we overact when we observe significant cracking in concrete structures like the one shown in Figure 1. Before we declare an emergency and start installing temporary shoring, we should perform a simplified quick evaluation of the remaining structural capacity. In some instances, we will find that even though structural capacity is reduced and strengthening is required, there may be enough capacity remaining. This condition need not be treated as an emergency.

Figure 1. Severely cracked concrete slab

Figure 2. Cracking due to restrained volume change

It is not uncommon to misinterpret the structural distress caused by the restrained volume changes in concrete structures as shown in Figure 2. We should be aware that long pieces of concrete, when restrained at both ends, will crack because there will be large tensile stresses due to shrinkage and creep that can overcome the strength of the structural member. It is important to properly assess the cause of this distress in order to design appropriate repairs.

Figure 3. Arching of the test load

Figure 4. Test slab exposed to temperature effects

During the load testing of the structures, we have to recognize the effects of the possible arching of the test loads (Figure 3), the temperature effects on the structure during the testing (Figure 4) and the deflection of the loaded structure member under constant hydraulic load (Figure 5). We also need to design the test load application so it does not fail during the load application and that we provide adequate shoring under the structure in case of its unexpected failure under the test load (Figure 6).

Figure 5. Hydraulic test loading

Figure 6. Failure of structure during the load test

## 3. MISTAKES DURING DESIGN OF STRUCTURAL REPAIRS

Most common mistakes include inadequate attention to design of the temporary shoring required for construction of the structural repairs. This shoring needs to be strong enough to carry all vertical and lateral loads on the structure in addition to its self-weight. The footing elements of the shoring have to have adequate stiffness (Figure 7) to distribute the loads to the soil and to be properly designed for the expected soil reactions (Figure 8). Also, not taking into account temporary longitudinal loads may result in failure of the temporary shoring structure (Figure 9).

Figure 7. Deflection of shoring mudsill

Figure 8. Cracked concrete bearing pad

Figure 9. Collapsed construction gantry

During a demolition of an old steel bridge, the fascia girder was being removed by picking it up with two cranes on barges, one on each end (Figures 10 and 11). When the existing floor beams were cut to free the fascia girder from the rest of the bridge, the weight of the girder was transferred at each end to a crane (Figure 12). However, since the girder was not adequately laterally braced, it buckled under its own weight and collapsed. The two cranes also collapsed and an operator of one crane was killed (Fig. 13).

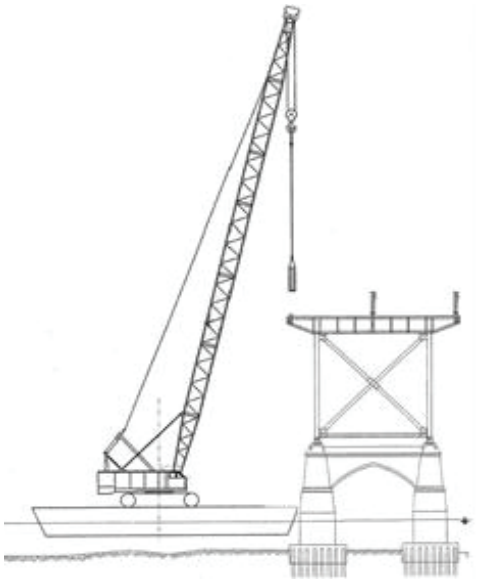

Figure 10. Removal of fascia girder from the bridge

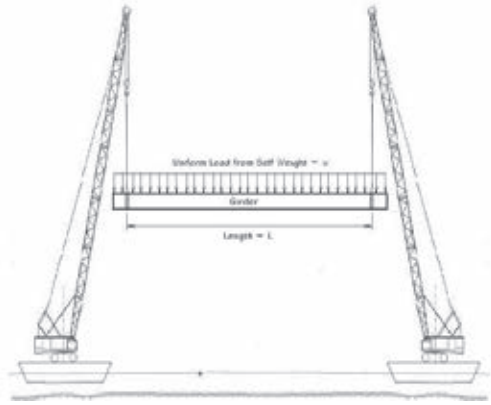

Figure 11. Pick-up points for the fascia girder

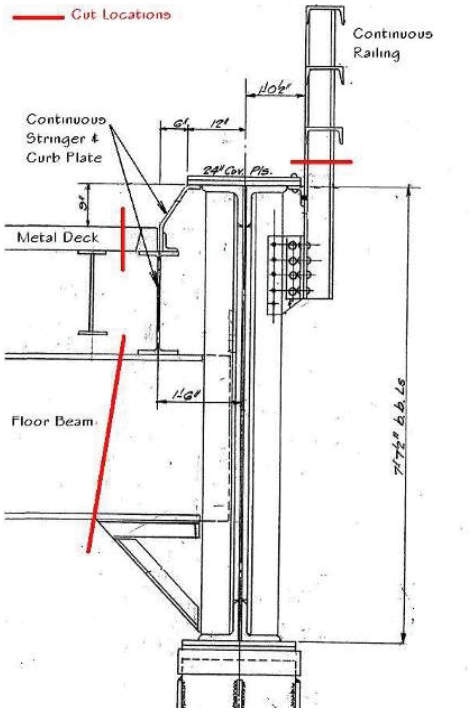


Figure 12. Cutting the connections of fascia girders

Figure 13. Collapsed girder and barges

When our repair designs include removing the concrete floor around a column, we need to recognize that this may double the unsupported length of the column which may reduce the column capacity by a factor of four (Figure 14). Our designs need to provide for adequate lateral bracing during that phase of construction or specify the proper sequencing of concrete removal.

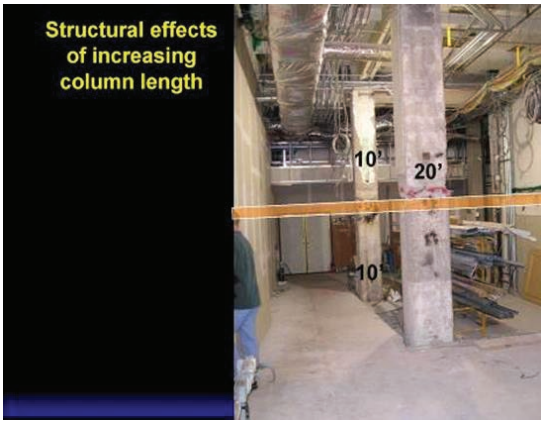


Figure 14. Increasing column length

Figure 15. Compression struts in post-tensioning repairs

Similar conditions may occur in sequencing of repairs of post-tensioning tendons. If we do not provide for leaving concrete compression struts along the line of tendon repairs, parts of the structure may move when the tensioning loads are applied to the tendon (Figure 15).

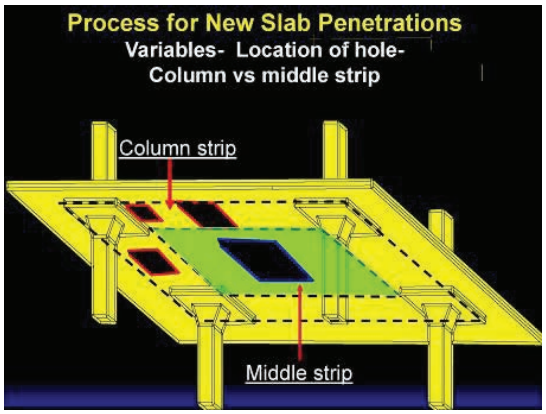


Figure 16. Cutting large openings in slab

When we need to design penetrations in the existing floor slabs, we need to make sure that they do not reduce the capacity of the slab beyond what is required. If we cut large openings in the slab (Figure 16) which interrupt the flow of tensile forces in a two-way slab system without additional reinforcing, the structure will not be able to carry even its self-weight. A similar condition may occur if we design too many penetrations in the floor close to each other (Figure 17).

Figure 17. Closely spaced slab penetrations

## 4. MISTAKES DURING CONSTRUCTION OF STRUCTURAL REPAIRS

As in the previous sections, one of the common mistakes during the construction of structural repairs includes improper bracing and shoring of structures during the repair process. We have to be alert to the fact that when we remove a connection between a wall and slab during the repair process (Figure 18), we need to provide additional supports for both the slab and the wall.

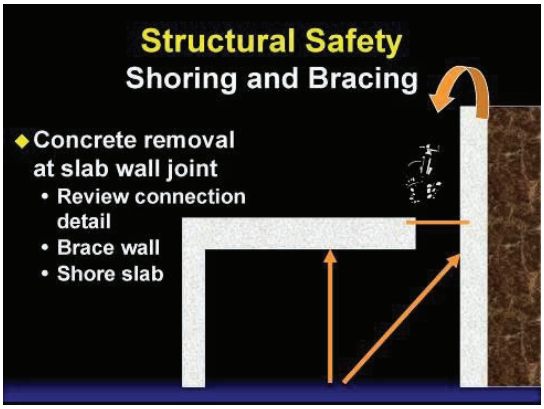


Figure 18. Removal of slab wall connection

Also, when we chip deteriorated concrete in slab around a column, the unbraced length of column doubles and it needs to be laterally supported (Figure 19).

We have to provide for additional supports to accommodate the weight of construction loads, including heavy equipment and removed debris to prevent overloading of the structure during repair (Figure 20).

Figure 19. Removing concrete around column

Figure 20. Excessive construction loads

During the strengthening of the structures we oftentimes use steel anchors into the existing concrete to attach supplemental supports. It is very difficult to predict the exact as-built location of embedded steel reinforcing and we can have great difficulties placing the anchors in the

intended locations (Figure 21). It is important to allow for this condition by placing the anchors where possible and then drilling the holes in the attachments to match the locations of the already placed anchors.

Figure 21. Irregular spacing of anchors

Figure 22. Cutting post-tensioning tendons

Special care is required during the repairs of post-tensioned concrete structures. Sometimes during the chipping of deteriorated concrete, the contractor may not be aware of the presence or location of post-tensioning cables and will cut them by mistake (Figure 22). It is important that the sequence of concrete removal does not result in undermining of the post-tensioning cable anchorages.

The contractor should be aware how the structure is constructed and how its load-bearing elements function prior to beginning the repair. The main steel reinforcing is usually at the bottom of the concrete slab but it is the opposite in cantilevered balcony slabs. If that is not recognized in advance, concrete removal may result in collapse of the balcony (Figure 23).

When repairing a loaded concrete column, we need to be aware that chipping inside the reinforced column core, we will be removing loaded concrete and overstressing the column (Figure 24). It is also important that personnel in the field do not over excavate the concrete in loaded columns (Figure 25).

Figure 23. Balcony slab failure

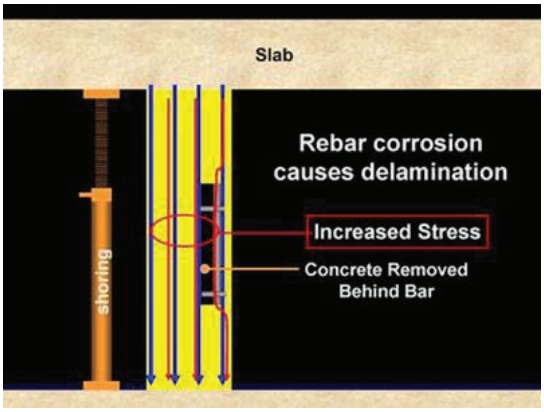


Figure 24. Overstressing column during repair

During concrete repairs, it is important that supplemental reinforcing steel be placed within required tolerances because even small variations may reduce the effectiveness of the repair. All repair materials are not always suitable for all repair applications. The repair material has to have similar characteristics to the base material including strength, modulus of elasticity and shrinkage. Otherwise, the repair will fail. It is common that concrete patches made with very high strength material with a lot of cement will crack because of large shrinkage. An example of improper material used for replacement of an expansion joint seal is shown in Figure 26.

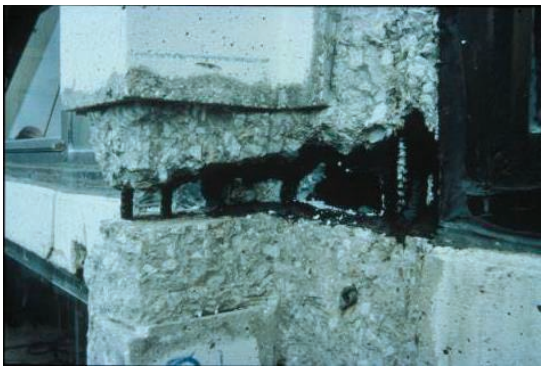

Figure 25. Excessive concrete excavation

Figure 26. Failed joint seal

When using an epoxy to inject cracks, it is necessary to confirm with tests that the epoxy has penetrated the full depth and length of the crack or an area to be epoxied (Figure 27). In concrete repair, proper surface preparation will ensure intimate bond between the patch material and the substrate. When the repair project includes installation of a protective waterproofing membrane over the repaired area, the ambient temperature and the temperature of the substrate have to be within the limits specified by the manufacturer to avoid failures of the membrane (Figure 28).

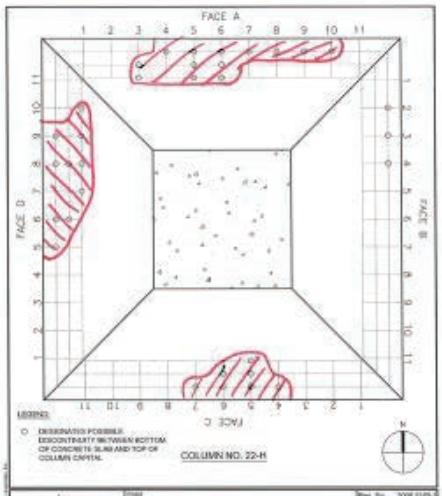


Figure 27. Testing for voids

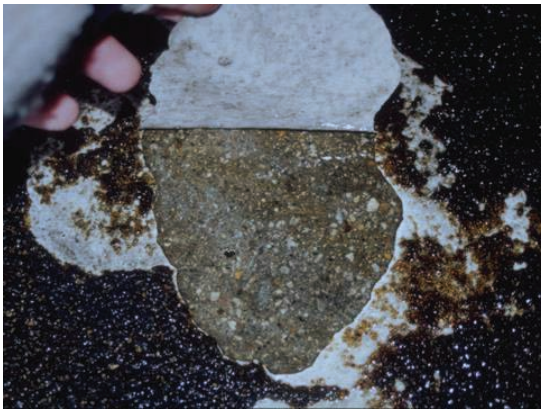

Figure 28. Failed waterproofing membrane

## 5. SUMMARY AND ACKNOWLEDGEMENTS

The above mentioned examples do not provide all possible mistakes that we encountered over the years but point out to commonly made mistakes in evaluation, of structures and design and construction of structural repairs. The examples listed in this article were, in addition to the author, investigated by engineers from author's firm and their contribution is gratefully acknowledged.

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# ASSESSMENT AND REPAIR OF EXTERIOR FACADES

Common problems in exterior facades include water infiltration, lack of proper details to accommodate volumetric changes of the façade materials and to accommodate differential movement between facade elements and the building structural system, improperly designed or built connections, deterioration of anchors and connections, material flaws and durability, and use of thinner than traditional wall systems. Safety issues during the assessment and repairs are briefly described. Methods and examples of repairs for brick masonry, granite, limestone and concrete facades are presented. Examples of retrofit of the precast concrete facade panel connections and recladding of building facades are shown.

Keywords: exterior facades, facade panels, granite, limestone, brick, concrete

## 1. INTRODUCTION

Common problems in exterior facades include water leakage, condensation at the interior of the building, cracking, displacement, fracturing, bowing and/or dislocation of facade elements.

Claddings on modern multi-story buildings usually include a combination of various materials. They include aluminum, concrete (cast-in-place, precast walls, panels), glass, masonry, mortar (joints), sealant (joints), steel and stone (granite, limestone, marble). Most of the older building facades were built by with brick masonry, terra cotta, granite and limestone. Unlike the floor structure of the building which is usually never exposed to its full design live load, cladding on buildings will most likely be exposed to the wind load, rain, and temperature ranges for which they were designed. Consequently, the cladding on a multi-story building has a greater potential to fail if it is not properly designed or constructed to meet design conditions.

The major causes of problems and failures in exterior facades include:

- Inadequate provisions for prevention of water leakage into the interior of the building
- Lack of proper details to accommodate volumetric changes of the facade materials

- Lack of proper details to accommodate differential movement between the facade and the building structure
- Improper connections and improper construction methods
- Material flaws and durability
- Use of thinner than traditional wall systems

Sometimes, corrosion of metal elements embedded in the cladding system can present a problem. Several examples of facade problems are shown in Figures 1 and 2.

Figure 1. Moisture expansion of brick

Figure 2. Bowed marble panel due to confined permanent expansion

## 2. SAFETY ISSUES IN FACADE REPAIRS

During the assessment of conditions and repair of exterior facades, access and personal safety were major issues. To access the facade, there is a need to install a suspended scaffold system which needs to be properly supported at the top of the building (Figures 3 and 4). In addition to the support, the scaffold needs to be tied to a different point at the building in case of failure of the supports. Special safety requirements exist for personal safety which includes a personal safety harness which is tied to a safety line tied to a different part at the building than the lines for the scaffold.

In the course of the investigation and repair, additional safety issues can arise. They can include finding cracks or expensive spalls that indicate structural distress in elements of the facade. That may require installing shoring before proceeding with a repair program (Figure 5).

Figure 3. Suspended scaffolds on the side of the building

Figure 4. Supports for suspended scaffold

Figure 5. Severely deteriorated balcony requires shoring

## 3. BRICK MASONRY CLADDING

Brick masonry cladding is usually installed over the building's concrete and steel framing with

common brick masonry backing. Corrodible metal shelf angles and window and door steel lintels were used as a support for brick cladding. Moisture intrusion behind the brick masonry cladding have caused the corrosion of steel shelf and lintel angles. Cracking, spalling of brick masonry, as well as bowing out of sections brick clad walls above window openings and at parapet walls were common distress due to corrosion of embedded steel.

Staining in a form of efflorescence, atmospheric deposits, staining from various causes, and other contaminants are often present at the exterior walls, and façade cleaning is usually part of the restoration project in order to improve the appearance of the building.

## 3.1 TYPES OF DISTRESS IN BRICK MASONRY CLADDING

The most common distress observed at the exterior walls clad with brick masonry consists of deteriorated mortar joints; spalled brick and bowed out wall sections due to corrosion of embedded steel shelf angles and steel window and door lintels (Figure 6); cracks observed at building corners, and sections of walls that cannot accommodate movements; displaced wall sections due to inadequate or missing anchorage to the backing wall; deteriorated sealant in expansion joints; soiled walls; freeze-thaw damaged brick and deteriorated bricks due to applied unbreathable paint coatings over them.

Figure 6. Distressed section of brick masonry wall before and after removal of brick to expose corroded embedded steel lintel

In most of the cases during the investigation work, it is challenging to determine the condition of the embedded steel angles. The best practice would be to make exploratory openings during that time. The openings should be made in several distressed areas to determine the extent and severity of deterioration. This will ultimately determine the repair approach.

## 3.2 TYPES OF REPAIRS IN BRICK MASONRY CLADDING

Repointing of mortar joints: includes grinding out deteriorated or cracked mortar joints, and installation of new mortar in order to prevent water infiltration into the building; replacement of damaged brick with new; installation of repair anchors to stabilize/secure the movement of displaced section of the brick wall; and repair/replacement of shelf angle/lintel includes removal of brick above the shelf angle/lintel as needed to expose the corroded portion of the embedded steel and repair/replace it based on percentage of section loss (Figure 7).

Additional type of repairs include resealing / repointing of crack; creating a new expansion joint; as repaired facade may also be cleaned. The type of cleaning solution to be used will depend on condition of brick, level of soil, efflorescence staining, atmospheric deposits, and other contaminants.

Figure 7. Repaired steel lintel with installed flashing over it

## 4. STONE MASONRY CLADDING

Stone masonry cladding is usually installed over the building concrete and steel framing. The limestone cladding was typically 10 cm (4 in.) thick and was supported or attached to a backing wall with corrodible angles and straps. The limestone pieces above the window openings are supported by steel angles. No flashing was installed over the steel window lintels typically. Granite panels 3 cm (1.2 in.) thick were supported with non-corrodible angles and brackets.

Moisture intrusion behind the limestone cladding caused the corrosion of concealed corrodible metal straps, dowels and shelf angles. Cracking, spalling, and displacement of limestone panels were typical forms of distress observed. Displacement of granite panels was found to be typical distress due to the inability of connections to accommodate movement of the panels, and failed epoxy adhesives. Sealant between granite panels was generally deteriorated.

## 4.1 TYPE OF DISTRESS IN LIMESTONE CLADDING

Distress observed at the exterior limestone walls consist of deteriorated mortar joints between limestone panels; spalled limestone due to corrosion of embedded strap anchors; spalled limestone due to corrosion of embedded shelf angles; cracks observed along the building corners; displaced wall sections with spalled limestone.

Figure 8. Cracked and spalled window mullions and rosettes

At locations of displaced and cracked limestone panels steel lintels, dowels and straps behind them were severely corroded. The expansion of corrosion build-up creates large expansion forces causing pressure on adjacent stone, which caused displacement and cracking of the limestone blocks at their back portion. Typical observed distress is shown in Figures 8 and 9.

Figure 9. Severely corroded steel shelf angle and cracked and spalled limestone block due to corrosion

## 4.2 TYPES OF DISTRESS AT GRANITE CLADDING

Displacement of granite panels was found to be typical distress due to the inability of connections to accommodate movement of the panels, and failed epoxy adhesives. At those instances connections installed to support granite panels were inadequate and different from designed. Instead of using metal clips at the bottom and top of the panels to provide for gravity and lateral support, panels were connected with wires glued with epoxy adhesive to the back of the panels and attached with anchors to the backing wall.

In case where epoxy was used as an attachment adhesive, it was discovered that granite inserts embedded into granite panels were debonded from the backing material, as shown in Figure 10.

Figure 10. Granite inserts attached with epoxy adhesive that has failed

## 4.3 TYPES OF REPAIRS FOR STONE CLADDING

Repointing of mortar joints includes grinding out deteriorated or cracked mortar joints and installation of new mortar; patching of spalled limestone with a hand-placed mortar patch, as shown in Figure 11.

Dutchman repair includes removal of sections of deteriorated or damaged limestone and repairing the removed area with limestone dutchman held in place with strap anchors and/or stainless steel pins, as shown in Figure 12.

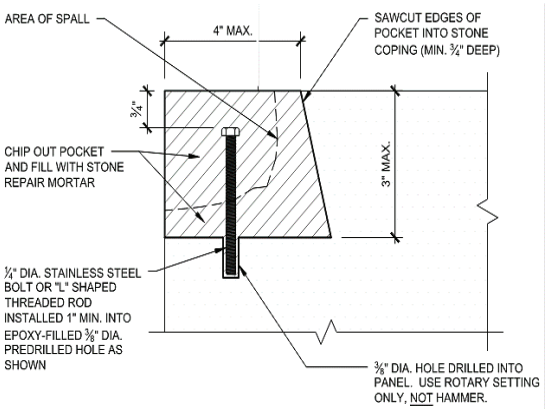


Figure 11. Limestone patch

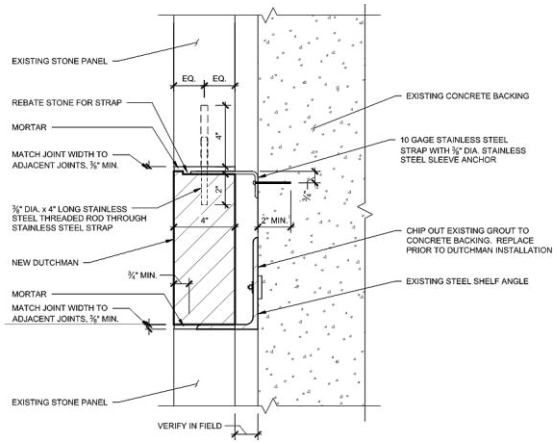


Figure 12. Dutchman limestone repair

Repair option also included replacement of damaged limestone with new; installation of repair anchor to stabilize/secure the movement of displaced stones; repair of shelf angle/lintel. Based on the percentage of section loss it is determined is shelf angle/steel lintel would be repaired or replaced; installation of flashing with a drip edge over steel lintels (Figure 13).

Figure 13. Repair of steel shelf angle/window lintel

Other repairs included replacement of deteriorated sealant in joints between stone panels; replacement of inadequate connections at granite panels included installation of new stainless steel angles anchored into the backing wall and reattaching granite panels to them; and reattaching of granite inserts using epoxy adhesives included removal of existing failed adhesives and backing material and installation of new.

## 5. PRECAST CONCRETE PANELS AND THEIR CONNECTIONS

The controlled manufacturing environment allows the facade panels to meet exacting tolerances for dimensions and anchorage placement that are generally not attainable in the structural frame to which they are being attached. Consequently, the facade panel connections must allow not only for structural

load transfer, creep, shrinkage and thermal movement of the facade elements and the structural frame, but also for field variation in connection location and alignment. The installers of the panels often try to overcome these alignment difficulties by adding supplemental shims or by field welding or bolting supplemental connection plates to make up for poorly matched connectors. However, these field installed connectors do not perform well because the field installers do not fully understand the design and implications of varying the connection. In some instances these failures can be catastrophic as shown in Figure 14.

Figure 14. Failure of facade panel connection

Figure 15. Inadequate bearing of façade panels

The most often observed cause of exterior facade panel failures was the improper construction of panel connections to the building frame. Construction deficiencies included using the originally designed connections for connecting the panel to the building frame even though the frame connections were not located at the intended elevation and location; improper placement of the connection plates, bolts or welds; misalignment of panel; using erection shims and bolts as permanent connection as shown Figure 15; and welding connection plates when they were intended to be bolted to allow for volume changes. Most of these problems could have been easily and inexpensively solved if the erector, after realizing that the connections did not fit the as-built frame, would have alerted the design engineer or had his own engineer modify the connections with due consideration to the forces and the actual dimensions involved.

The design deficiencies in the exterior facade panel connections involve the lack of adjustable connections that may be required for panel erection even for a building frame and facade panels built to the design tolerances, not providing for volume changes in the panels and building frame (welding both ends of a long panel, for example), and not considering the full extent of eccentricities in the connection plates and bolts.

Figure 16. Typical gravity load connection repair

The retrofit design for typical gravity connection included welding fairly thin steel plates on each side of the haunch flange and to the steel bearing plate, as shown in Figure 16. This connection provided transfer of gravity load, restrained out-of-plane movement of the panel, and allowed for the volume change movements along the length of the panel through flexibility of thin steel plates. The interesting aspect of this repair project was that the repairs had to be performed without interruption of the hospital operations. Up to three patient rooms were emptied each day, the inspection openings

were cut in the wall finishes to expose and inspect the as-built connections, the connection repair was designed based on the condition found, and the repair contractor performed required retrofit.

Another example of improperly built connections were the supports for 10 m high facade panels of a four-story police/jail facility. These very heavy panels had the type of the panel built as a haunch which rested on the roof slab of the building. Unfortunately, all the panels were rigidly connected to each other by steel connection plats welded to adjacent supports. This prevented normal volume change movements and caused all panels to crack at and around their bearing supports (Figure 17). The repair included attaching supplemental steel brackets to the panels away from the original supports and transferring through them the weight of the panels to the roof (Figure 18).

Figure 17. Cracking of panel bearing

Figure 18. Connection retrofit with steel bracket

## 6. CONCRETE FACADE

Inadequate concrete cover over the reinforcing bars can result in moisture from the rain penetrating to the steel rebars which will begin to corrode. Expansion forces of the corrosion by-product (rust) will result in cracking and spalling of concrete (Figures 19 and 20). Standard concrete repair techniques include chipping deteriorated concrete around corroded rebar, sandblasting rebar and concrete surfaces, epoxy painting the rebar and pouring or hand applying concrete patches. It is recommended to install stainless steel wire anchored in the concrete substrate to help hold the patch in place if the bond between the substrate and patch weakens (Figure 21). Similar repairs are used for exposed slab edges.

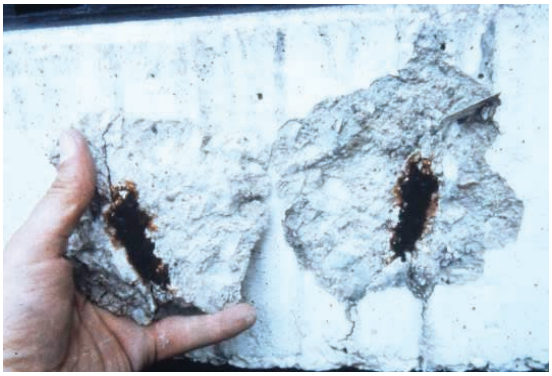

Figure 19. Spalling and cracking of the concrete façade

Figure 20. Spalling and cracking of the concrete façade

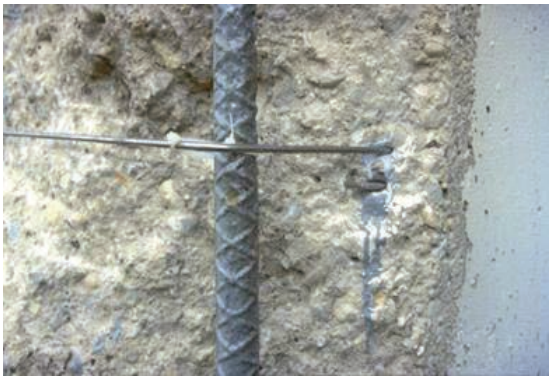

Figure 21. Facade patch repair

Concrete balcony slabs usually have additional deterioration due to the presence of steel railing posts embedded in the concrete slab. The deterioration of balcony slabs is accelerated if there is carpet present on the balcony (Figure 22). Painting the repaired facade with a breathable acrylic paint will improve the appearance and durability of the repair.

Figure 22. Deterioration of concrete balcony

## 7. RECLADDING OF FACADES

In the latter part of the last century, new, thinner facade systems were developed. They included thin composite panel systems where traditional materials like marble or limestone were reinforced to make them thinner. Also, traditional materials were used thinner than before. Using a rational design, the thickness of the non-load bearing wall was reduced down to 4-10 cm.

Currently, in order to reduce the weight and cost of the exterior façade veneer, the thickness of the veneer systems has been reduced down to 2.5 cm. and non-load bearing walls to 4 cm. A reduction of the thickness of the exterior façade cladding resulted in some new problems. In some instances, composite panel systems experienced failures due to unexpected interaction of different materials.

Outward bowing on thin brick bonded to cement board is shown in Figure 23. Connections of thinner marble and granite façade panels to the structural system of the building had smaller safety factors and cracks were occurring in the stone at locations of the connections. Bowing of marble panels is shown in Figure 24.

Figure 23. Outward bowing of thin brick bonded to cement board

Figure 24. Bowing of new marble panels on Finlandia Hall, 2005

As the facades with thin marble panels were beginning to age, bowing of the panels as shown in Figures 25 and 26, and failure of their connections were observed. Loss of flexural strength due to exposure to heat from the weather became evident. While this loss of strength was not affecting the performance of the thicker marble panels in the past, now with thin marble panels it became a major factor in their performance. It also became evident that details and connections for the panels that worked well in the past with thicker panels needed to be revised to accommodate the behavior of the thinner stone panels.

These conditions required removal of the existing cladding, and the design and installation of the completely new different façade system. The new system and connections should be tested with mock-ups before proceeding with the project. Figure 31

shows recladding of the tallest marble clad building in the world.

Figure 25. Recladding of the 82-story tall marble clad Amoco building

Figure 26. Bowing of panels

## 8. CONCLUSIONS AND ACKNOWLEDGEMENTS

The common causes of failures of exterior facades on multi-story buildings are: lack of provisions to prevent water leakage into the interior of the building, lack of proper details to accommodate volumetric changes of the façade materials and to accommodate differential movement between the façade and the building structure. Improperly designed or built connections, and use of thinner than traditional wall systems. Guidance on how to repair these conditions is presented.

The case histories of structures described here were investigated by architects and engineers from the author's firm and their contribution is gratefully acknowledged.

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# SEISMIC RETROFITTING OF STRUCTURES, HISTORIC BUILDINGS AND MONUMENTS - IZIIS’ APPROACH

The impact of present building code requirements for seismic design in new buildings can readily be acknowledged, however applying regulation to existing buildings is an area less well defined. Presently, there is a diverse list of existing code references which could be interpreted to require seismic upgrades of existing structures. Unfortunately, these references do not provide a clear path toward addressing the hazards, evaluation and retrofitting of existing buildings. And when it comes to existing historic buildings and monuments, the topic becomes much more complex and challenging.

The problem of earthquake protection of historic buildings and monuments is radically different from that of other existing structures, due to the priority given to preservation of aesthetic, architectonic and historic values instead of keeping the structure operational. In providing the protection of these structures in a manner that requires the least intervention and the greatest care to preserve authenticity, the experts are permanently challenged by the fast development and the improved performance of new materials and techniques. However, the implementation of retrofitting methodology depends on the extent it has been investigated.

Within the frames of IZIIS ’research activities, in addition to seismic design of modern structures, particularly noteworthy is also the experience gathered in the field of protection of cultural historic structures. Important scientific research projects, involving analytical and unique experimental investigation and extensive research activities resulted on developing of a procedure for seismic retrofitting of valuable historic monuments. This paper present the integrated multidisciplinary approach to seismic protection of structures pertaining to cultural heritage that has been developed by the Institute and implemented in the process of seismic upgrading or reconstruction of important historic buildings and monuments in the country and beyond.

Keywords: seismic retrofitting, historic buildings, monuments, multidisciplinary approach

# 1. INTRODUCTION

## 1.1 HISTOIC BACKGROUND

Protection from earthquakes begins with the appearance of early, now missing, civilization although in these periods of human society development the phenomenon of earthquakes has been considered as God's will or the product of mystical powers. During the 19th and early 20th centuries, interest in earthquakes and their effects increased in Japan, Italy and United States. This was mainly because of major earthquakes such was: in Japan, the 1855 Edo and 1923 Kanto earthquakes, in United States, the 1906 San Francisco and 1925 Santa Barbara and in Italy, the 1908 Messina earthquake. This growing interest results with the establishment of important institutions: the Structural Engineers Association following by Earthquake Engineering Research Institute (EERI) in California, the Earthquake Research Institute within Tokyo Imperial College, as well as Special Committee for investigation of the Messina earthquake in Italy, which report was evaluated as the birth of practical earthquake design of structures since it contains engineering recommendation for earthquake resistant structures by means of equivalent static method, which was gradually used in earthquake countries around the world and was later adopted for building codes too, [1].

However, the beginning of the modern approach to earthquake protection can be considered the First World conference on earthquake engineering, (1st WCEE, Fig.1) held in 1956 to commemorate the 50 anniversary of the San Francisco earthquake, since it was the first proceedings on earthquake engineering. The formation of the International Association for Earthquake Engineering, (IAEE) in 1960 during the Second World Conference in 1960 held in Japan, was a very important for earthquake engineering development especially for the countries that did not have their own national association. Since then World Conferences, (WCEE) held every four years have successfully brought together many researchers, practitioners and public officials and contributed significantly to the process of earthquake knowledge. While the *repair and strengthening* appears as a topic for the first time in 1976 during 6th WCEE, the theme of the next 17WCEE which will be held in Sendai, Japan is "*toward resilient society*" aiming to contribute to the goal of earthquake engineering - to control the seismic risk at socio-economically acceptable levels.

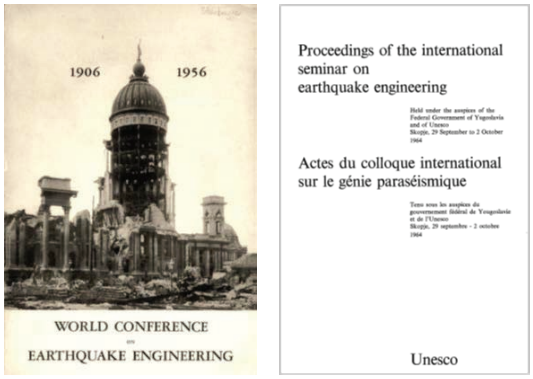



Fig. 1. Proceeding of the 1st WCEE and 1st ECEE

Similarly, the key event for the beginning of earthquake engineering in Europe was the Skopje 1963 earthquake, the most destructive earthquake in the history of ex-Yugoslavia, causing losses of about 15% of the gross national product for the year 1963. This earthquake was unique among the catastrophic earthquakes in Europe and the Mediterranean region, since it triggered a high level of awareness and activity by the government, population, scientists and engineers to the need for organized and sustained efforts to be implemented in earthquake disaster management. Dominated by the activity and financial support by UNESCO and its agencies, seismic zoning map and a network of seismological stations of Yugoslavia have been developed, the Institute of seismology, earthquake engineering and urban planning, (later Institute of earthquake engineering and engineering seismology, IZIIS) has been created in Skopje, code for seismic design and standardization of construction materials and improvement in the quality of construction have been elaborated and it was among the first aseismic codes in Europe.

Following the earthquake in Skopje the members of the UN Consultative Board for the Reconstruction of the City of Skopje signed an agreement, sponsored by UNESCO, for the establishment of a European Commission for Earthquake Engineering, (later European Association for Earthquake Engineering, EAEE). The first symposium was held in Skopje on 1st October 1964, and it was followed by regular conferences (ECEEs) every four years.

The traces of growth, advances and developments of earthquake engineering can be followed by following the milestones of both WCEEs and ECEEs. During these conferences great lessons have been, and will be, learned about the nature of earthquakes, characteristics of ground motion, performance of geotechnical, structural, non-structural and

lifeline systems during earthquakes, and their social and economic aspects, increasingly emphasizing the need for multidisciplinary approach.

Majority of these developments have reached the modern building seismic codes and their impact in new buildings can readily be acknowledged. However, applying regulation to existing buildings is an area less well defined. Presently, there is a diverse list of existing code references which could be interpreted to require seismic upgrades of existing structures. Unfortunately, these references do not provide a clear path toward addressing the hazards, evaluation and retrofitting of existing buildings

## 1.2 SEISMIC UPGRADING OF EXISTING BUILDINGS

The older existing buildings that were constructed with low or no seismic consideration represent the largest risk to most communities. In general, the needs for their structural strengthening are in case of: errors in designing or construction, load increases, damage to structural parts, modification of structural system, improvements in suitability for use or upgrading of standards or national technical provisions. However, the *seismic retrofitting* of existing earthquake damaged or earthquake-vulnerable buildings is needed when the existing structures should be made more resistant to seismic activity, ground motion or soil failure due to earthquakes and it necessarily means modification of existing structural system. The main problems the structural engineers are faced are the lack of standards for risk assessment, evaluation and retrofitting methods as well as the variability of the effectiveness of each retrofitting method depending on type and importance of structures, type and condition of built-in materials and their conditions, amount of damage etc.

Extremely important role in overcoming these problems in the Balkans had UNESCO and UNDP, primarily through actions and financial support for IZIIS' development (1965-1982), and especially for implementation of several subsequent projects (1968 – 1985) on the seismicity, earthquake risk reduction and seismic resistant building construction in the Balkan region. This was for sure initiated by the several deadly earthquakes in the region in that period: in Yugoslavia, 1963 Skopje, 1967 Debar, 1969 Banja Luka and 1979 Montenegro; in Romania, 1977 Bucharest; in Greece, 1978 Thessaloniki earthquake. The final regional project "*Building Construction under Seismic Conditions in the Balkan Region*" UNDP/UNIDO REP/79/015 has been carried out with the participation of the Governments of Bulgaria, Greece, Hungary, Romania, Turkey and Yugoslavia and with UNIDO acting as executing agency for the UNDP. A set of seven Manuals has been produced within this project, reflecting to considerable extent the experience of the participating nations in earthquake resistant design and construction, [2].

## 1.3 METHODOLOGY FOR STRUCTURAL REPAIR AND STRENGTHENING IN BALKAN REGION

The fifth project volume *"Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings*", [2], prepared by the *working* group E convened by Prof. Gavrilovic from IZIIS, was aimed to give qualitative instructions for the step by step repair and strengthening procedure, drawn on the experience gained following recent earthquakes. Without going in details, what follows are the main steps of this post-earthquake repair and strengthening procedure, agreed and adopted by all the Balkan countries:

- *Emergency damage evaluation,* which should be done for each structure by official inspection teams to quickly determine the general level of damage and level of safety for continued occupancy (presented in detail in fourth Manuel, (fig. 2).

Fig. 2. Volumes 4 and 5, UNDP/REP/79/015

- *Preliminary investigation*, as an independent and more thorough evaluation which should be performed by a design engineer. The first phase of this preliminary investigation is to determine in detail the nature, cause and extent of damage and consequently the need for emergency, temporary bracing or shoring. Further phases of investigation involve detailed inspection of the damage so repair and strengthening procedures can be designed and detailed.

- *Definition of seismic parameters* for the region as a prerequisite for successful accomplishment of repair and strengthening of damaged structures. This study should include the expected maximum acceleration of bedrock for different return periods, the amplification factors of soil deposits and to propose adequate time histories and average spectra for design of structures to be strengthened. For important structure or group of typical structures it is necessary to determine the seismic parameters for considered sites and to perform field, soil investigation and geophysical studies to be used as input design parameters.

- *Definition of criteria for repair and strengthening*, as correlation of the seismic force design parameters with the structural characteristic in terms of its strength, deformability, ductility etc. Each country should establish its own criteria based on specific condition related to its seismicity. In our country they are as they were set for ex-Yugoslavia, as follows:

  - *Design Level I:* for slight and moderate earthquakes with return period of 50-100 years, the structure is assumed to perform in linear range up to the yield point, with maximum relative story displacement of h/600 and required displacement ductility D≤1.

  - *Design Level II:* for more severe earthquakes with return period of 200-500 years, the structure could suffer nonlinear deformations but without disturbance of its global stability, with maximum relative story displacement of h/150 and required displacement ductility D ≤ 3-4 for reinforced concrete, D ≤ 1.5-2 for plain and D ≤ 2-2.5 for confined masonry.
  The designer must use the criteria established or to use the existing Code for new buildings of the area as the minimum standard for repair and strengthening projects. For selected projects, like special or important structures, or structures form traditional materials, the designer may have to use additional criteria based on circumstances regarding the project.

- *Additional investigations* for completion of the detailed site inspection to develop knowledge of as-built conditions. First, if documents regarding original construction are available, (drawings, specifications, construction details, material strength data, previous repairs or alterations, code under which original design was prepared), the information gathered should be compared with the actual structure, and if there are deviations they should be recorded. If information is not available, the field measurements and observations must be taken to establish the condition of the original structure, usually with removing the finishes or covers to determine the composition of structural elements. Second, each structural member must be inspected, and the damage or lack of damage should be noted and recorded, as repairs will be based on these data. Third, characteristics and strength of original construction materials must also be estimated, which could be accomplished by various methods, taking samples for laboratory testing and/or using in-situ strength evaluation methods.

- *Damage analysis and evaluation* which should be done by the designer by analyzing the damaged structure, thoroughly understanding why the damage occurred, if the structure suffered due to discontinuities in strength or stiffness, due to torsion moments or improper connection or details, etc. The effects of non-structural elements on the structural performance must be considered. Calculation and analysis must be performed in order to evaluate the existing strength and stiffness capacity of damage structure, following by the decision of the need for its strengthening. If the repaired structure only meets the design criteria, then strengthening will generally not be required. The strengthening will generally be appropriate in addition to repairs of damage if the repaired strength is less than the required one.

- *Selection of a repair and strengthening* based on feasibility evaluation of alternative solutions for repair and strengthening, defined as follows:

  - *Repair* is the reestablishment of the initial strength of damaged structural members and reestablishment of the function of damaged nonstructural elements. Properly repaired structural members will possess approximately the same strength as before they were damaged but will probably have a somewhat reduced stiffness due to very fine cracks which are impossible to restore.

- *Strengthening* is a judicious modification of the strength, stiffness and/or deformability of structural members or the structural system to improve the structure's performance in future earthquakes. It generally includes increasing the strength or ductility of individual elements or introducing new structural elements to increase the structure' lateral force resistance. On occasion, strengthening can also involve making selected structural members weaker to improve the interaction of the members and prevent premature failure of a week adjacent member. Anyhow, strengthening should bridge the gap between the required and existing strength, stiffness and deformability. While repair usually follows the damaging earthquake, strengthening can be done either prior to or following an earthquake.

  A complete listing of all strengthening techniques is beyond the scope of this paper. It becomes a treatise on structural engineering because all materials and systems used in new construction can also be used in retrofit. The selection of the specific type of element is dependent on local cost, availability, and suitability for the structure in question. Calculation for each alternative solution must be performed in enough detail to ensure that the criteria will be satisfied, to evaluate the effects and to provide a reasonable basis for comparison with other proposed schemes. For the selected solution complete analysis must be performed with the appropriate mathematical model and according to the design criteria, with studies for determine the dynamic response of structural system for defined seismic parameters

- *Final design procedures* which include a completion of the detailed calculations of the strengthening solution including dynamic time history analysis for the defined seismic parameters, then preparation of drawings, specifications, detailed procedures and instructions so the work can be accomplished successfully. Regardless of the level of investigation done in the evaluation or design phases, unexpected conditions will be discovered during construction of strengthening. Therefore, it is important that the design engineer is available during the construction phase to provide consistent and acceptable adjustment of details. In certain cases, discovery during construction can be used as a strategy to confirm unknown or vague conditions while minimizing disruption from pre-construction investigation.

Further reading of this manual on *"Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings"* gives complete information about the materials and construction techniques for repair and strengthening, separately for reinforced concrete, stone and brick-masonry buildings, emphasizing that each structure is a unique system and its damage in an earthquake will be different from other structures, requiring custom repair and strengthening solutions and details. The decision of the rehabilitation method and the appropriate construction technique depends on many factors, such as local soil conditions, type and age of structures, type and degree of damage, available time, equipment and staff, architectural requirements, cost and, last bet not the least, the required level of seismic safety.

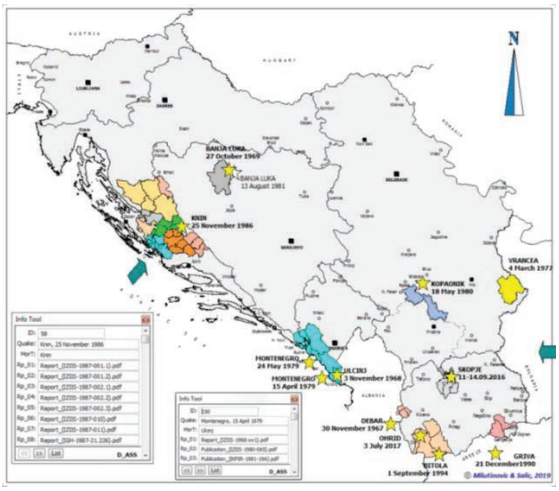



Fig. 3. Damage evaluation, strength assessment and strengthening of buildings after occurrence of major earthquakes in ex-Yugoslavia, [3]

To date, this manual has been regarded in the region as unmatched in defining the necessary steps of strengthening procedure. In keeping with the philosophy of this procedure, IZIIS has implemented a seismic strengthening of a great number of structures, different in their function, main bearing system and built-in materials, prior- or post-earthquake repair and strengthening, (fig. 3). For all of them step by step process of presented strengthening procedure has been thoroughly followed.

## 2. SEISMIC RETROFITTING OF HISTORIC BUILDING AND MONUMENTS

The problem of seismic retrofitting of historic structures is radically different from that of other structures, due to the priority given to preservation of esthetic, architectonic and historic values instead of keeping the structure operational. The specific character of seismic protection of historical buildings and monuments resulting from the variety of structural systems, built-in materials, periods and techniques of construction, stability criteria and contemporary requirements incorporated in the modern principles of conservation and protection, needs systematical and scientific approach to achieving a successful solution, [4]. Although there is a similarity between historical buildings and historical monuments, there also exist differences, for which each of these groups should be considered separately:

• *A monument* is a structure having an important "cultural value" so high that it is necessary to guarantee its preservation, generally with its architectural, typological and material characters;

• *A historical building* is a building of an urban area, which has a "cultural value" as a whole (historical urban area), while a single building is not a monument, meaning that preservation concerns the general character of construction techniques typical in the whole area.

Historic buildings usually present shear wall masonry structures that are basically non-ductile and insufficiently resistant to seismic effects. The problem of interaction between the "old" and the "new" materials and/or elements that arises in their strengthening requires experimental verification of the techniques that have so far been developed, (injection, grouting, jacketing, confining, base isolation). Since monuments are also masonry structures, same basic principles and requirements hold for them also, but are specific. The characteristic structural entity, the variability of the built-in materials, the complex history of successful modifications done in the past, as well as the degree of deterioration, makes each historic monument a case for itself. Therefore, the basic principle of minimum intervention – maximum protection and/or preservation of the monument's identity should be adopted.

Retrofitting of masonry structures is elaborated in numerous books, publications and individual reports. However, from the aspect of conservation and restoration of monuments, historic buildings and sites- 1964 ICCOMOS' Venice Charter could be considered a basic document and a beginning of a systematic approach to general protection of these structures. Since then, the seismic protection of monuments, historic buildings and sites with all their structural and specific characteristics has intensively been developed throughout the last several decades within the frameworks of the scientific discipline of earthquake engineering - typically multi-disciplinary, including other related scientific spheres.

**Materials for Retrofitting:** The key for selecting materials and techniques is classification of retrofitting techniques into two main categories: reversible and irreversible. In selecting materials to be used in reversible interventions, there are usually only a few limitations. However, the materials used in irreversible interventions as are for example the unavoidable injection of cracks, do impose two additional limitations: compatibility of new with old materials and their durability. The best way of assuring compatibility and durability is usage of traditional materials, (stone, bricks, lime mortar and cement), which on the other hand, is not always possible. In selecting injection mixtures, advice should be asked from experts as to preventing separation of the old and new parts. Modern cement mixtures should carefully be applied particularly in the process of jacketing because of the irreversible modification of the surface of existing masonry. Steel, (as externally applied ties or as reinforcement incorporated into the existing masonry is a very frequently applied material in the strengthening processes of both historic buildings and monuments, [4].

**Retrofitting Methods:** The main problem imposed in masonry structures is to provide structural integrity at story level because of enabling of distribution of seismic forces according to the stiffness of individual elements and avoiding of individual vibration of the elements after the occurrence of the first cracks. The most commonly used procedure to achieve structural integrity is to incorporate horizontal steel ties into the existing masonry (at the top of the existing walls in order to be made invisible - churches and mosques), incorporate reinforced concrete belt courses or reinforced concrete slabs into structures where possible - structures in old towns.

To improve the bearing characteristics of the walls and the columns as structural elements sustaining horizontal seismic forces, several techniques are used: injection of masonry,

injection with jacketing, incorporation of vertical reinforced concrete columns, or even incorporation of new reinforced concrete walls (in the structures of the old towns). The injection technique, the material to be used, i.e., the pressure under which the prepared mixture will be injected are selected depending on the size, position and shape of the cracks. To increase the bearing and deformability capacity of the walls, jacketing of the walls with concrete on both their surfaces, i.e., incorporation of reinforced concrete belt courses is anticipated. Vertical reinforced concrete belt courses are used to increase the ductility of the considered element. The techniques of strengthening of the foundation structure mainly consist of extending the proportions of the foundation and their connection to the vertical elements, modifying the foundation structure and consolidating and improving of the characteristics of soil conditions, [4].

## 3. INTEGRATED APPROACH FOR SEISMIC RETORFITTING OF HISTORIC BUILDINGS AND MONUMENTS DEVELPED BY IZIIS

In providing the protection of these structures in a manner that requires the least intervention and the greatest care to preserve authenticity, the experts are permanently challenged by the fast development and the improved performance of new materials and techniques. However, the implementation of particular retrofitting methodology depends on the extent it has been investigated. The delicate problem of proving the effectiveness of the selected consolidation system can be successfully overcome by using the methodology of design assisted by testing.

Within the frames of the IZIIS' research activities, in addition to seismic design of modern structures, particularly noteworthy is also the experience gathered in the field of protection of structures pertaining to the cultural historic heritage. During a period of more than forty years of activities in this field, the Institute has realized important scientific research projects involving analytical research, unique experimental shaking table test and field surveys of historic structures.

Applying the knowledge from repair and seismic strengthening of structures presented in previous chapters and deepening the existing methodology, it can be said that an integrated approach to seismic protection of extraordinarily important cultural historic structures has been adopted by the Institute and used in the process of reconstruction or seismic upgrading of important monuments. This approach that complies with the restoration and conservation requirements as well as procedures and legislative for high category structures, should encompass the following:

- Definition of seismic potential of the site through detailed geophysical surveys for definition of geotechnical and geodynamic models of the site to consider the expected earthquake effect through a probabilistic approach, including also the local soil effects through nonlinear dynamic analysis of a representative geotechnical model;
- Determination of structural characteristics and bearing and deformation capacity of existing structure including investigation of the built-in materials, definition of structural dynamic characteristic through ambient vibration method, developing the corresponding mathematical model and determination of dynamic response for defined seismic parameters;
- Definition of criteria and selection of concept for seismic retrofitting respecting the country regulative as well as guidelines in the ICCOMOS and ISHARCH documents;
- Definition of structural methods, techniques, materials in accordance with defined criteria and positive national and international construction and conservation practice;
- Analysis of dynamic response of retrofitted structure and verification of their seismic stability;
- Definition and documentation of field works, and their execution by constant supervision by professionals from different fields.

Although the above stated seems to be the "normal procedure", it is the only way of providing high quality in protection of cultural heritage. This task is certainly much more than simply listing of what is to be done since it requires a lot of knowledge and efforts.

Further in the paper are presented some of the most characteristic case studies of applying different, previously experimentally verified, seismic retrofitting of historic buildings and monuments that have so far been carried out in IZIIS in accordance with the presented integrated methodology.

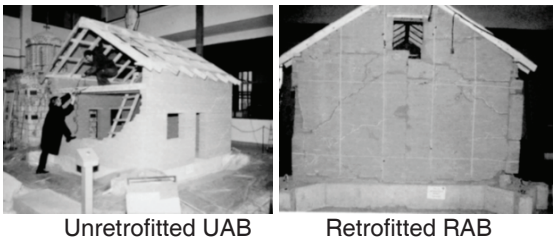
## 4. SEISMIC RETROFITTING OF HISTORIC BUILDINGS - IZIIS' EXPERIENCE

**Old towns along Mediterranean coast:** The old towns of Budva, Kotor, Dubrovnik, etc. were severely damaged due to the April 1979 Montenegro earthquake. Considering the cultural value of most of the buildings in the old towns, extensive investigations for the purpose of searching for the optimum conditions and methods for reconstruction, repair and strengthening of structures were performed, thoroughly applying the methodology presented in chapter 1.3. Firstly, the buildings were classified into structural units and then the methodology for structural retrofitting was determined based on detailed studies of structural characteristics, the damage level, the built-in materials, the foundation conditions as well as local seismicity. Since the injection was the most frequently applied method, for its testing and verification, a 1:2 scaled model of a typical single-story building was constructed of the same original stones and mortar and tested on the seismic shaking table in IZIIS applying the 1979 Petrovac earthquake. After testing and occurring of diagonal cracks, the model was repaired by injection and tested again, in which case the model suffered less damage, (Fig. 4). Using the acquired knowledge on the technique application, the consumption of material and the effect of applied injection, more than three hundred buildings within the old towns along Mediterranean coast have been repaired at the same time, including structural repair, repair of facade walls and the interior of each individual building, [5].

Fig. 4. Repaired Model of a typical building for the old town of Budva, [5]

**Historic Adobe Structures in California:** Since 1990, the Getty Conservation Institute (GCI) carried out a multi-year, multi-disciplinary project, the *Getty Seismic Adobe Project*, (GSAP), including a survey of existing historic adobe buildings in California, dynamic testing of small scale model buildings (scale 1:10) at the Stanford University, and the preparation of an Engineering Guide for designing seismic retrofit measures. As an extension of GSAP, tests were conducted on two large-scale models (scale 1:2) on the seismic shaking table in IZIIS.

The first model was a control model while second was retrofitted with a combination of horizontal and vertical straps, center cores, and partial plywood diaphragms. The applied retrofitting proved very effective in improving stability and preventing collapse, (Fig. 5). Based on all the results and the GCI' Engineering Guidelines a number of historic adobes structures in south California were seismically upgraded applying this experimentally verified methodology, [6].



Unretrofitted UAB Retrofitted RAB

Fig. 5. Gable end wall of UAB and RAB model

**The Parliament building in Skopje:** The Parliament Building of the Republic of North Macedonia was more than 70 years old, (Fig.6). As a historic building, it is protected by the Law on Protection of Cultural Heritage. Throughout its existence, a lot of changes, enlargements, adaptations, damaging during 1963 Skopje earthquake as well as post-earthquake repair and strengthening of this building have been done. Within the project on Enlargement, Building of Another Storey and Adaptation of the Building, the necessity for increasing the seismic safety of main structural system has been defined, [7].

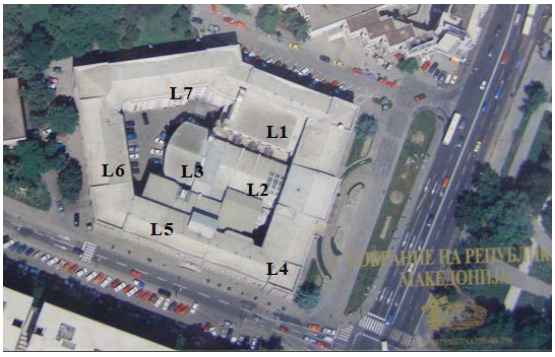
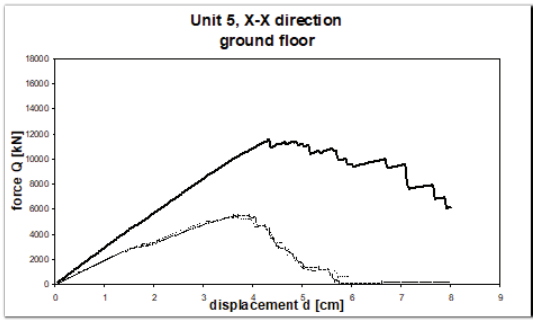



Fig. 6. Individual structural units L1–L7, Parliament building, [7]

Starting with the essential difference between “plain” and “confined” masonry, the most important phase was in-situ technical investigations performed to identify the principal structural system of the pentagonal outline, evaluated as the oldest and most critical part for the planned building of another storey. The analysis of the bearing and deformation capacity of the structure shows that the shear base coefficient for the seven structural units is in the range of 6-18%, much smaller than that required one by the regulations for plain masonry, (30%), while the relative storey displacements do not satisfy the requirements for earthquakes that may occur in this area. Thus, the need for strengthening of the structure becomes even more topical and economically justified.




--- existing state, ▬ strengthened structure

Fig. 7. Comparative Q-d diagrams for unit L5, [7]

The strengthening solution has been defined to improve the integrity and the seismic stability of the structure. Subsequent final analysis of strengthened system has been made up to ultimate states of strength and deformability for each unit taken separately. Comparison for selected unit gives a very clear insight into the effect of the selected strengthening solution, as is shown on Fig. 7, pointing to a considerable increase of both the bearing and deformation capacity of the system. The process of strengthening of the structural system, while continuously functioning of the Parliament, was carried out quite successfully despite a number of limitations in the period 2010-2014.

## 5. SEISMIC RETROFITTING OF MONUMENTS - IZIIS’ EXPERIENCE

**Monuments in Pagan - Burma (1978-1980):** The Pagan plateau in the central part of Burma, is a worldwide known place for the high concentration of magnificent historic monuments like temples and pagodas. Unfortunately, most of these have severely been damaged by natural disasters, including earthquakes. The 1975 earthquake with a magnitude of 6.8 destroyed or heavily damaged many of them.

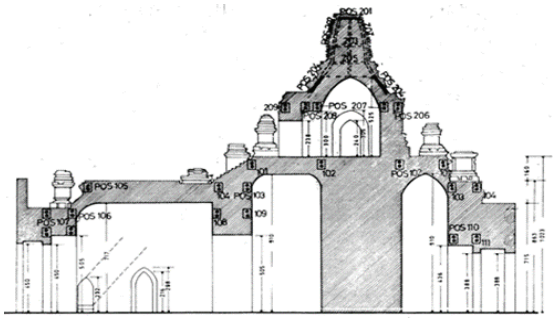
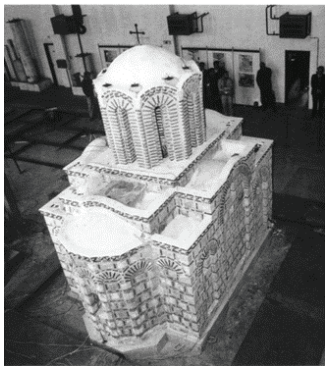


Fig. 8. Strengthening of South Guni Temple, [8]

Based on the defined seismic design criteria, the developed methodology for seismic retrofitting of this type of structures has been applied under the Project – UNSECO/UNDP project Burma 78/023, [8]. Considering the material and structural properties, its dynamic characteristics and the expected possible earthquakes, the method of strengthening and repair has been proposed anticipating injection and inserting of steel bracing or RC belt courses, (Fig. 8). Later fifteen monuments were seismically upgraded according to presented methodology and none of them experienced damage during the recent 6.8 magnitude earthquake in 2016.

**Byzantine Churches:** Within the long-term research project entitled "Study for Seismic Strengthening, Conservation and Restoration of Churches Dating from the Byzantine period in the Republic Macedonia” realized jointly by IZIIS, Skopje, Macedonian National Conservation Center (NKC) and GCI, Los Angeles, [4, 9], ample experimental and analytical investigations were performed to verify an original methodology for the repair and seismic retrofitting of Byzantine churches. For the first time in the world, shaking table testing of 1:2.75 scaled model of St. Nikita church, simulating the existing and the strengthened state, (Fig. 9), was realized at IZIIS to investigate its behaviour in elastic, nonlinear and heavily damaged state. The tests approved that applied methodology, consisting of horizontal and vertical belt courses, increases the bearing capacity and deformability of the structure up to the level of the designed protection.



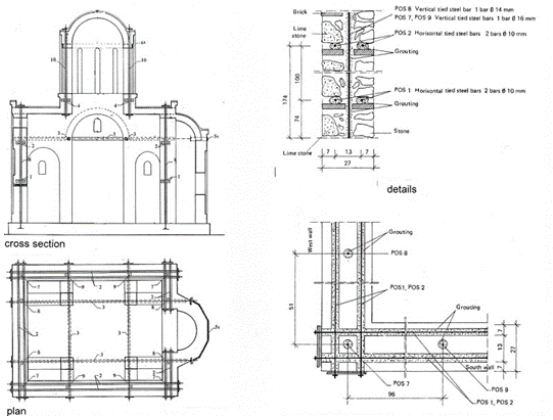



Fig. 9. The 1:2.75 scaled church model and the applied strengthening method, [4, 9]

After the realization of the project on Byzantine churches and gained unique and incomparable knowledge, IZIIS became a partner of the National Conservatory Center, which enabled direct application of the gained knowledge in actual conditions and for specific historic monuments. The most characteristic examples of application of the developed methodology in the process of reconstruction and seismic upgrading of monuments were: (1) reconstruction of blown-up St. Athanasius Church in Leshok, (2) consolidation and reconstruction of the church St. Panteleymon in Ohrid, and (3) reconstruction of the blown-up church of the Holy Trinity in Mostar, [10], (Fig. 10).

St. Athanasius Church in Leshok

St. Panteleymon in Ohrid

The church of the Holy Trinity in Mostar

Fig. 10. Implementation of the developed methodology in the reconstruction of three churches, [10]

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The author wish to extend gratitude to colleagues from IZIIS and especially from Building Structures and Materials Department, for applying thoroughly the developed methodologies for seismic retrofitting of structures, historic buildings and monuments for more than 40 years, as well as to all the partners and participants for the entrusted task and their participation in the realization of abovementioned projects.

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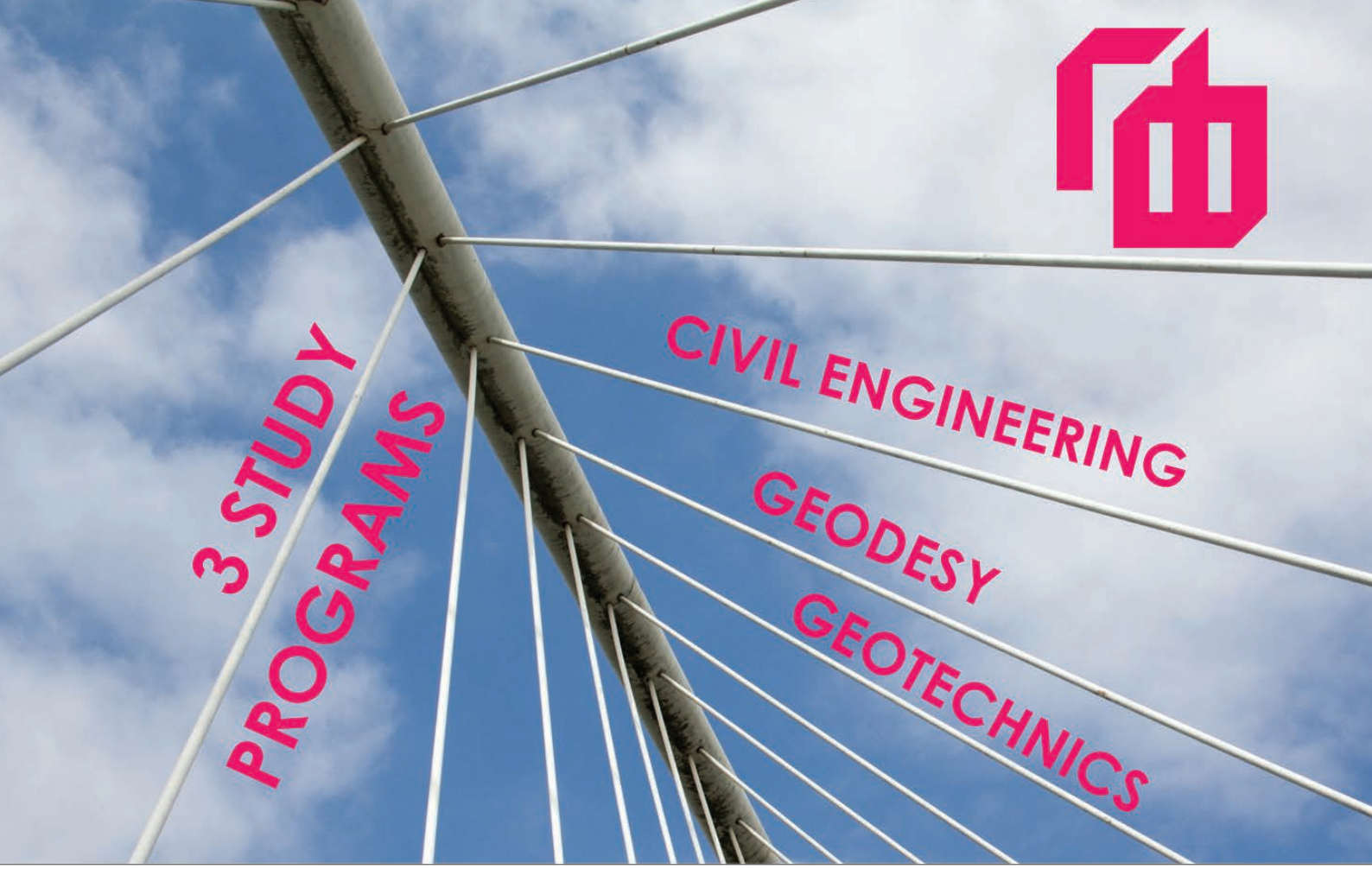


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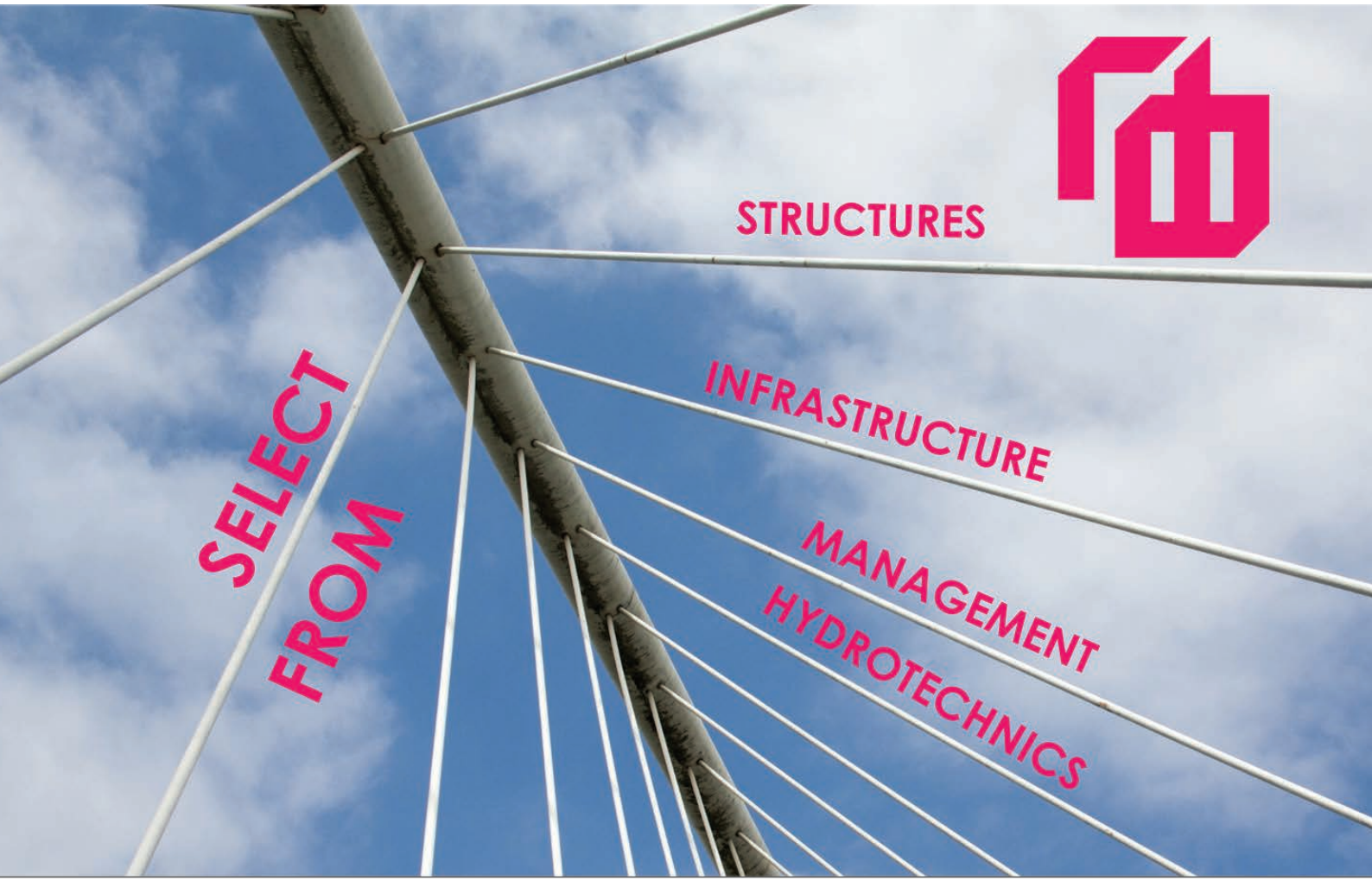


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# REHABILITATION AND RECONSTRUCTION OF SOME HISTORICAL STRUCTURES IN SERBIA

This paper presents rehabilitation and reconstruction design of several structures of historical significance, which are under the protection of The Institute for Protection of Cultural Monuments of the Republic of Serbia.

Repair of walls and towers of the Smederevo fortress is shown, and it includes repair and rehabilitation of towers 14, 15 and 16, as well as the walls between them, which were demolished during the 1941 ammunition explosion. Also, rehabilitation of tower 11, which is 20 m high and tilted by 1.8 m at the top, is shown. Repair of its foundations was performed using a special technology known as "jet grouting". Afterwards, an overview of reconstruction of the medieval keep tower in Bac, which was built in 1329, is given. Reconstruction was largely related to the roof structure. Bridge over the Milesevka River, near the Mileseva monastery is a very interesting project, which was developed based on historical data on similar bridges and on a single footage of the former bridge. The paper also presents rehabilitation of the lighthouses at the mouth of Tamis into Danube, which were built in 1909 and are only twin lighthouses in Europe. Repair works were especially interesting because of an unusual contracting procedure. Due to water presence, while constructing lighthouse base ring, concrete was first installed and then reinforcement was incorporated into the fresh concrete.

All rehabilitation and reconstruction designs described in this paper were developed at Faculty of Civil Engineering in Belgrade.

Keywords: Fortress, Tower, Rehabilitation; Repair, Monument, Bridge, Roof structure

# 1. REHABILITATION OF SMEDEREVO FORTRESS WALLS AND TOWERS

## 1.1 REHABILITATION OF TOWERS 14, 15 AND 16 AND WALLS BETWEEN THEM

The Smederevo fortress was one of the largest medieval fortresses in Europe. The core was the triangular so-called Small town, at the top of the Big, again triangular town. This Big town was built right after the Small town, which was finished in 1430. The construction of Smederevo fortress began in 1428, ordered by Despot Đurađ Brankovic. The Small town had 6, and the Big town had 19 more, giving a total of 25 large towers, square shaped, over 20 meters high. Between the towers, there were walls about 4 m thick and about 15m high (Figure 1).

During the First World War and the crossing of the Austrians over the Danube, the fortress was bombarded several times and severely damaged. The last major destruction came in the Second World War due to an explosion of ammunition on June 5, 1941, when the Varoska gate, Tower 15 and walls towards Towers 14 and 16 were completely destroyed (Figure 2.). Towards the end of the Second World War, the fortress was bombarded by Great Britain forces in the Allied bombing.

Figure 1. Smederevo fortress today

a)

b)

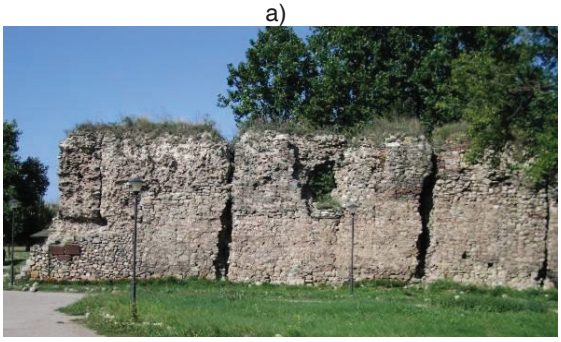
Figure 2. Destroyed parts of Smederevo fortress during ammunition explosion on June 5, 1941 (a and b)

Based on the previous research and the situation in-situ (Figure 3 and 4), the rehabilitation project [1] included the following construction works on the parts of the Fortress within the Varoska Gate sector:
- rehabilitation of Tower 14,
- complete construction of the new Tower 15,
- construction of new parts, that is, rehabilitation of Tower 16,
- construction of a new part and the repair of the wall-rampart between the Towers 14 and 15,
- construction of a new wall-rampart between Towers 15 and 16.

The new parts of the Tower 16, as well as the entire structure of the new Tower 15, are reinforced concrete structures in the form of walls and slabs made out of concrete MB 30. These structural elements have a thickness of 15 cm, with the walls in question having stiffeners in some places in the form of counterforts or shear walls with or without passages. It was planned that new towers and walls, unlike other existing towers and walls within the Smederevo Fortress, which are entirely made of stone and brick with total thicknesses of about 4 m, have a lightened structure – reinforced concrete walls on both sides with exterior cladding of stone and bricks, which are 50 cm thick.

Figure 3. Varoska Gate with walls

a)



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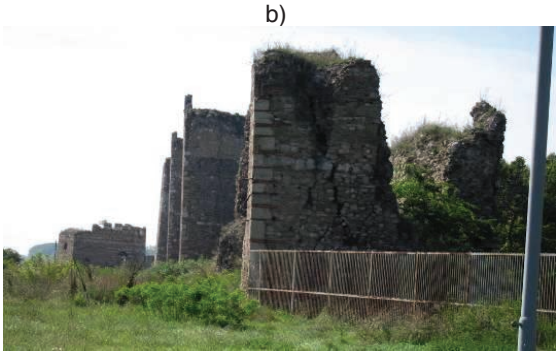


Figure 4. Walls next to Varoska Gate (a) and Towers 16, 17, 18 (b)

Tower 14, shaped as the Cyrillic letter "P" in its base, has two large cracks on the sides that practically go from the lower wall level (elevation 0.00) to the very top of the tower. Observed at the ground level, these cracks completely separate the horizontal parts of the letter "P" shape of the tower from the vertical parts. The repair intervention on this tower involves making the horizontal boreholes of Ø70-80 mm and inserting in them the Ø56 mm tension elements with the corresponding threads (vines) at the ends, along the vertices of the letter "P" at five altitudes in the centre of the walls.

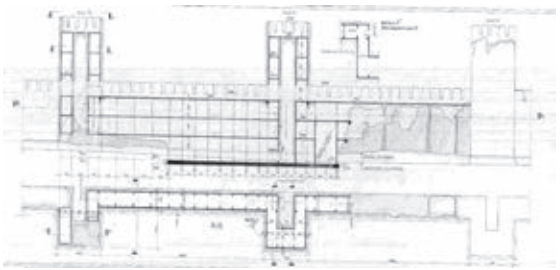


Figure 5. Layout of new walls and towers – ground floor plan and longitudinal section

## 1.2 REHABILITATION OF TOWER 11

As for Tower 11, it is located within the southern walls of the Big town, and it has the shape of the Cyrillic letter "P". The dimensions of the outer base are 10.67 x 7.5 m. The height of the tower is 20.50 m in relation to the current ground level. The thickness of the tower walls goes from 3.75 to 4.07 m. In the foundation zone, the thickness of the tower walls is somewhat higher due to the counterforts. Foundations are at a depth of about 5.5 meters from the current ground level.

The tower itself is in a very poor condition, and this primarily relates to its rotation towards the outside, which is about 1.8 m. What exactly caused this rotation of the tower is unknown. One of the possible causes is the explosion that occurred near the tower on the south side. Rotation (tilting) of the tower led to the separation of the tower walls from the walls of the ramparts, which resulted in a partial collapse of the rampart sidewalls. Apart from the mentioned tower tilting, there are also noticeable vertical cracks, as well as degradation of the walls, that is, damage to the elements from which they were built (stone, brick, mortar). (Figure 6.).

Based on all previous research, two opposite solutions of rehabilitation and stabilization methods have been analysed:

1. Correcting the tilting of Tower 11 and bringing it into a vertical position;

2. Keeping the tower in an inclined position and securing it from further inclination.

Detailed analysis of the possible rehabilitation solutions included all previous activities that were conducted in order to find the optimal solution and consultation with the representatives of The Institute for Protection of Cultural Monuments of the Republic of Serbia. Based on this, it was decided that the rehabilitation and stabilization project [2] be done with Tower 11 kept in an inclined position and secured from further inclination.

This solution involved developing two parts of the project, one relating to the soil stabilization in the tower's foundation area and the other relating to the structural rehabilitation of the tower above the ground level.

For the estimated foundation depth, foundation loading, and the characteristics of the terrain at the site of the tower, it was concluded that it is necessary to repair the soil below the foundation of the tower in order to reduce settlement of the structure. After examining the problem, and in order to increase the load-carrying capacity of the soil, a method for improving soil strength using a technology known as "jet grouting" was adopted (Figure 7).

a)

b)

Figure 6. Rotation (tilting) of Tower 11 (a) and degradation of building elements of Tower 11(b)

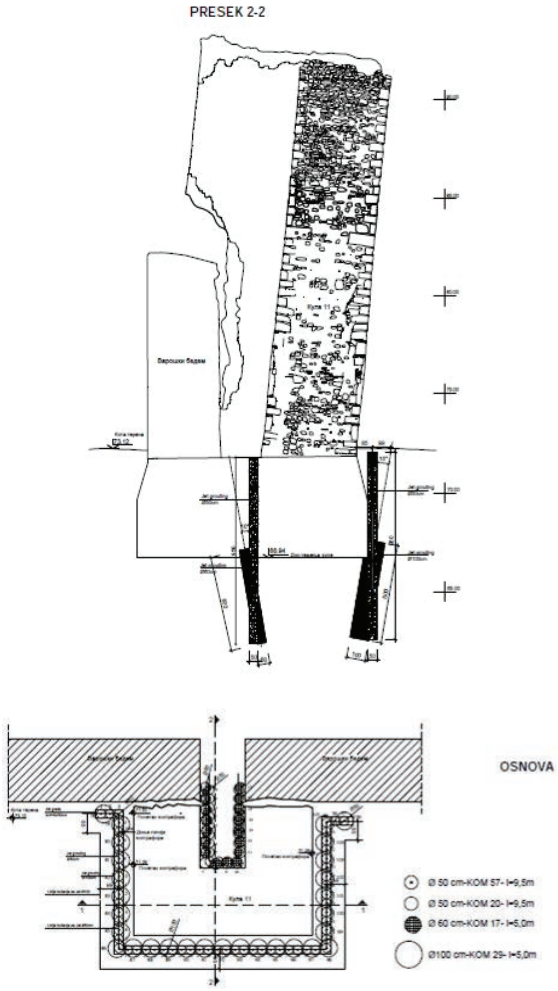



Figure 7. Rehabilitation plan for Tower 11 foundations using "jet grouting"

Within the tower, both on its front and on the sides, there are cracks spreading from the ground and practically to the upper levels of the walls to the left and to the right of the tower. Observed cracks, which are predominantly vertical, but some are also inclined, exist on all sides of the letter "P" - shape of the tower's cross-section.

a)

b)

c)

d)

Figure 8. Repair works (a, b, c and d)

Considering the described state of the cracks on the vertical walls of the tower, the intervention on it also means that within the letter "P", at four height levels on each of its vertical walls, horizontal boreholes Ø80 mm are made and that through these boreholes tension elements Ø56 mm with appropriate threads at the ends are positioned. The ends of the tension elements in are supported on concrete "pillows" with 20 cm thickness (MB 20). The "pillows" are reinforced with reinforcing meshes Q 335 throughout their thickness both on the front and towards the stone walls.

As part of structural rehabilitation and stabilization of Tower 11, and on the proposal of the representatives of the Investor, the steel bridge at the level of walkway on the rampart was also designed. The bridge was based on a similar structure carried out in the towers of the Manasija Monastery, as Investor proposed.

## 2. RECONSTRUCTION OF MEDIEVAL TOWER IN BAC

Medieval fortress Bac with its suburb is located in the southwestern part of Backa, about 14 km from the most important European river Danube. It was built near the town with the same name on the river Mostonoga. The fortress in Bac is a very important multilayer archaeological locality and a historical place, which is why it was placed under the protection of the state Srebia in 1949.

Archaeological research of the fortress in Bac (in 1958-59, 1980-81 and 2003) found that the location where it was built was settled from the Middle Stone Age. According to the written sources from the 11th century, the seat of the county, the bishop and the archbishop were located in Bac.

The fortress was built according to the "water town" principle. The fortress has an irregular tetragon base on which corners are protruding towers. Three corner towers have a circular base, while the northwest tower has a rectangular base (Figure 9 and 10). Initially, the fortress was surrounded by moat filled with water. This moat is still visible today. The fortress was entered through a drawbridge.

Within the ramparts of the fortress, the most dominant and only freestanding structure is the keep tower (the watchtower). This tower is noticeable for its height, which is in accordance with its basic purpose, to observe the surrounding territory from it and to defend the fort.

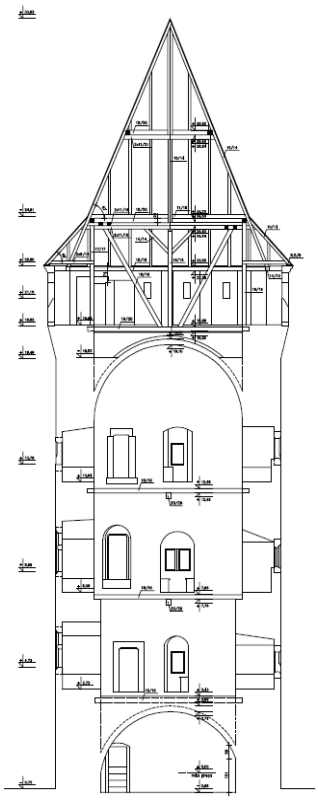
Figure 9. Bac fortress in 1668

Figure 10. Bac fortress before the watchtower roof reconstruction

The tower has almost a square base (10,26 x 10,16 m) and is oriented with diagonals to the sides of the world with a spiral staircase in the northwest facade and a potable water riser on the northeast facade. The tower has a height of 34.33 m, and it has a ground floor (basement), three floors and an attic overhanged on cantilevers (Figure 11).

The watchtower has been repaired and reconstructed on several occasions in its history, but due to lack of maintenance, its decay has occurred. The biggest damages occurred after the fire in 1993. (Figure 12).

a)



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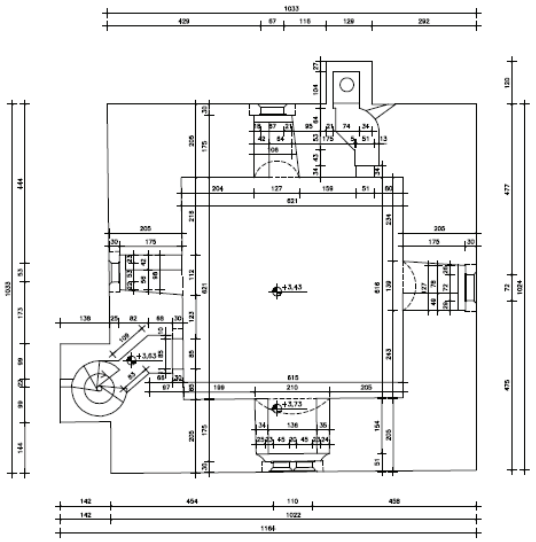


Figure 11. Structural plan (a) and the cross-section of the watchtower (b)

a)

b)

Figure 12. Fire damaged timber floor joists (a) and damages the roof structure (b)

Reconstruction of the tower [3], and above all the roof, was done so that the damaged roof structure was completely replaced by the new

one. The old roof shape was kept, but new members to the roof structure were added to increase its stability. Furthermore, the dimensions of almost all the members were increased, and their surface treatment was carried out by sealing, as it was undoubtedly done more than six centuries ago, when the tower was built (Figure 13). All visible connections of timber members were constructed with wrought iron, which contributed to the authentic appeal of the structure. The roof is covered with old yellow tiles, which were previously protected. With the roof reconstruction, an attractive area with visible roof is provided in the attic, suitable for exhibitions and observation of the surrounding flat Backa landscape.

Figure 13. Final works on roof reconstruction

## 3. RECONSTRUCTION OF THE BRIDGE OVER THE RIVER MILESEVKA NEAR MILESEVA MONASTERY

In the immediate surroundings of monastery complex Milesevo, beside the monastery water mill, there was once an old timber bridge. The exact time of its construction is unknown, but according to some historical data it is assumed that it was constructed in the middle of the 19th century. This bridge was destroyed after enormous floods which struck the Monastery and its surroundings at the end of 19th century.

The design of the bridge reconstruction [4], had to be prepared in a way that completely corresponded to the existing written and drawn historical data and adequate preservation conditions. Since a reconstruction of an unique historical engineering structure was in question the design was based both on knowledge on bridge construction at the end of 19th century and even earlier in this region and on authentic data on stone block paving on the part belonging to the bridge and on the right hand abutment which represented the remainder of the destroyed bridge.

Documentation for design preparation were: an old photograph of the bridge (Figure 14), graphic layouts, plans and cross sections and the river Milesevka regulation.

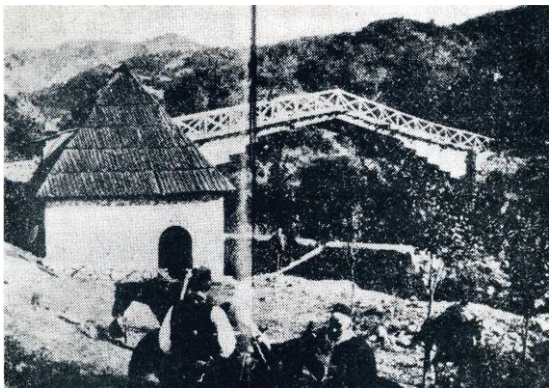


Figure 14. Old photograph of the water mill and the former bridge

Until completion of the new bridge construction, a provisory two-span structure was used (Figure 15). The larger span (10.4 m long) was bridged by a steel structure – frame of a freight vehicle, while the shorter span (5.5 m long) was bridged by timber beams. Sheet timber beams and boards existed partially over these load-bearing elements. The river pier was a concrete block with dimensions 3x3x4 m

Figure 15. Provisory structure used for crossing the river before the new bridge construction

It was known that the structure of the old bridge, which was destroyed by flood flow at the end of 19th century, was a timber one, supported by massive abutments built with ashlar. Only the right-hand abutment remained until today and it gives certain data on the way that destroyed timber structure was supported. The left-hand abutment was probably destroyed together with the bridge itself. By analysing the existing data - old photographs and surveyed situation, the left-hand abutment was fixed and thus the bridge span size of 17.6 m was defined. The adopted size of the bridge span fits favourably into the transverse profile of the river Milesevka at the bridging site.

The size of the bridge span and authentic historical data point out that the bridge superstructure was an unusually shaped structure. During the late medieval times, and even earlier, a specific system of timber structures characterised by "composite beam" girders was often used in the bridge construction. These girders with several simple cross sections are placed one above other, and overhang toward the middle of the bridged profile. Since the authentic data indicated that the destroyed bridge by the mill had such "composition", same as in case of destroyed bridges in Polimlje (over the river Seljansica and over the river Lim), there was no doubt while selecting the "system" of the timber bridge – timber structure with main girders consisting of several simple cross sections (six beams one on top of another) with girders in which certain beams gradually overhang toward the centre of the river (Figure 16). Taking the useful width of the bridge into consideration three main girders spaced at 1.17 m were adopted.

Since the bridge is intended exclusively for pedestrians, useful width of the bridge 2.3 m was adopted.

Figure 16. Bridge in erection stage

Other details – pavement, bridge guard rail and the shape of the levels are identical to the existing autochthonous historical data.

The main girders were fixed to the abutment using steel anchors and concrete blocks.

Newly designed elements of the abutments were built using old ashlar from the existing – old bank protection wall downstream from the mill.

The main girders, as already stated, were designed as composite beams consisting of six cross sections in system of fixed beam with a hinge in the mid-spam. Some of the beams are coupled by timber dowels (by transverse joints and screws) – Figures 17 and 18.

Figure 17. Completed bridge from the downstream side

Figure 18. Completed bridge from the upstream side

Irregular, that is, ununiform arrangement of dowels along the girder resulted from the insight into the old photographs of the destroyed bridge.

The pavement surface was made from timber boards, and a special system for rain dewatering was not planned considering the relatively small length of the bridge and considerable slope of the levels. Three stone steps were made on both approaches to the bridge (Figure 18) in order to prevent access of vehicles onto the bridge.

## 4. REHABILITATION OF THE LIGHTHOUSES AT THE MOUTH OF TAMIS INTO DANUBE

### 4.1 LIGHTHOUSE DESCRIPTION

The lighthouses at the mouth of Tamis into Danube were built in 1909 and are only twin lighthouses in Europe (Figure 19).

Lighthouses 1 and 2 are visually identical, and they are composed of conical base parts - "pins" - made of stone and:

- towers with circular foundations of grouted bricks, built on the platforms - upper bases of tapered cones;
- roofs of the towers, which are erected on metal bars and intended for lights.

a)

b)

Figure 19. The lighthouses at the mouth of Tamis into Danube – then (a) and now (b)

The base platforms are reachable via two identical stone staircases. In both towers, one staircase is on the opposite side of the water, which leads to the tower entrance, while the other staircase is on the opposite side - towards the water.

Towards the water, anchors for anchoring the vessels at both lighthouses were constructed. These are anchored in concrete blocks that are supported by rubbles of broken stone, and, in addition, they are connected to the stone mass in the rear - pedestals of the towers.

## 4.2 DAMAGE DESCRIPTION

By visual-macroscopic examination of both lighthouses, it was concluded that within their "pins" there are practically identical erosion damages, which consist of the following:

- the stone cladding was characterized by a large amount of damage in a form of cracks (Figure 20);
- a very pronounced crack of circular shape also existed at the connection of the lighthouse tower pedestal and the stone cladding;
- due to Danube flow influence, and especially due to waves impact, the stone cladding in the lower "pin" zone had a very large amount of erosion damages (Figure 21);
- due to Danube flow influence and waves impact, the structural assembly of the "pin" - the core made of broken stone - was partially destroyed, in some places there was even a "stripping" of the soil, which was taken away by the Danube together with a part of stone material (Figure 21);
- due to Danube flow influence, and especially due to waves impact, the stone cladding in the lower "pin" zone had a very large amount of erosion damage, which was also significantly contributed by vegetation that led to additional damages to "pin" surfaces (Figure 22).

a)

b)

Figure 20. Cracks in stone cladding (a and b)

Figure 21. Damages to lighthouse basis "pins"

a)

b)

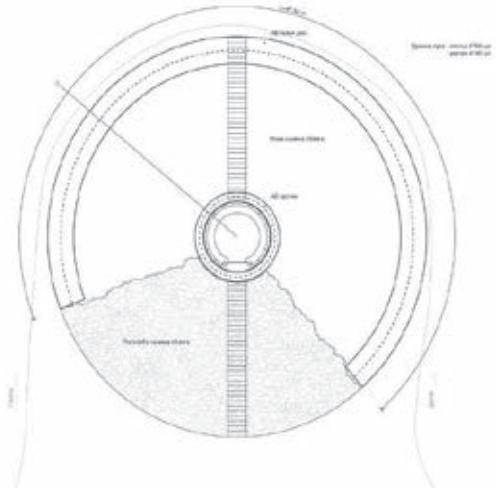
Figure 22. Vegetation in "pins" (a) and staircases (b)

## 4.3 REPAIR INTERVENTIONS

Repair works [5], began with cleaning of the entire "pin" surface from vegetation (roots), as well as a complete removal of the stone cladding from the "pin" part intended for rehabilitation. In addition, all soil parts were removed in order to reach the "healthy" surface of the existing mud-stone embankment.

As part of the initial phase of rehabilitation, a trench was excavated on a part of the lower base of the "pin" approximately 44 m long, 80 cm deep and 150 cm wide. This trench was required for the construction of a reinforced concrete "pin" base intended to support a new "pin" structural assembly, which should have started from that level (Figure 23 and 24).

a)



b)

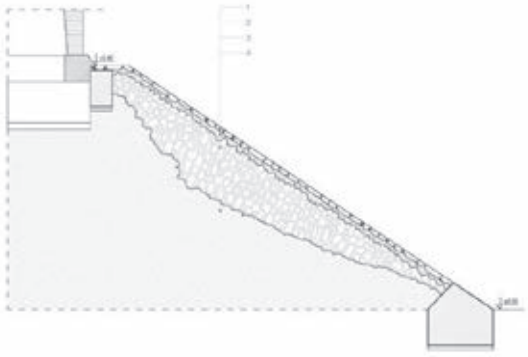


Figure 23. Layout (a) and cross section (b) – new design

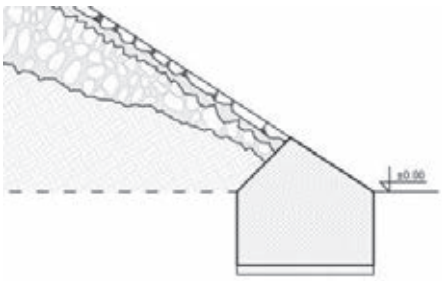


Figure 24 Trench excavation

In addition to the above described reinforced concrete base part within the lower "pin" base, a reinforced concrete ring with a cross section of 50x80 cm within the upper "pin" base was constructed. This ring fully enclosed the existing foundation of the lighthouse tower, and coupled with that foundation contributes to stability of the structure (Figure 23 and 25).

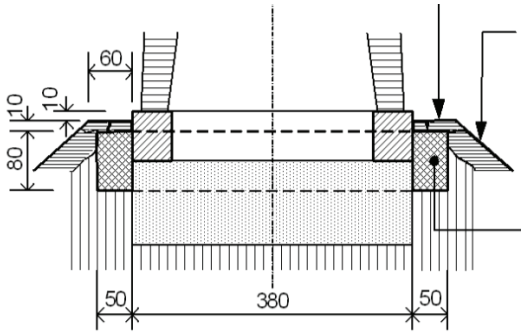



Figure 25. Reinforced concrete ring underneath the tower

In the next phase of construction, a new "pin" structural assembly was constructed. This assembly consists of rubble concrete with pieces of stone 15 to 30 cm thick, over which a new stone cladding about 30 cm thick was placed (Figure 26).

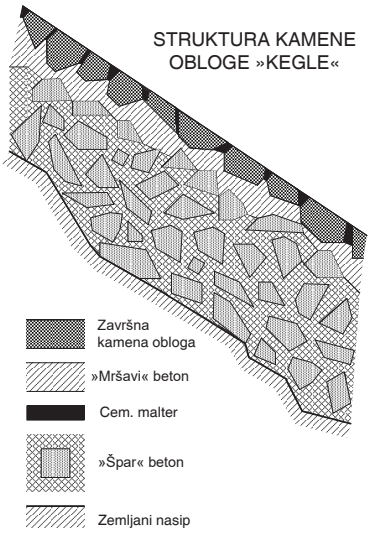



Figure 26. New "pin" structural assembly

Upon completion of the new "pin" stone cladding, the joints between the embedded stones were treated with cement mortar (Figure 27).

As part of the works, new staircases towards the water were also constructed.

Due to unfavourable circumstances, rehabilitation of the lighthouses was carried out in October, November and December of 2016, at very unfavourable weather conditions, i.e. high level of Tamis and Danube, so the contractor had to apply some unusual methods of construction. Due to water presence, while constructing lighthouse base rings, concrete was first installed and afterwards reinforcement was incorporated into the fresh concrete (Figure 28).

a)

b)

Figure 27. New stone cladding construction (a and b)

Figure 28. Concrete works on base ring under water

Figure 29. Rehabilitated lighthouse

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# MONITORING BASED STRUCTURAL IDENTIFICATION OF TWO RAILWAY BRIDGES

Data-driven tools applied on measured structural responses enable extraction of valuable information on the behavior of in-service structures and identification of the actual system in its "natural habitat".

An output-only testing campaign on two railway bridges is in the focus of this study. The specifics of the dynamic behavior of the structures are studied under three different scenarios. More precisely, the measured accelerations at relevant points of the structures, for unloaded and loaded conditions, are herein employed with the following objectives:

I) Structural dynamics identification via estimation of the first natural bending frequency of the structures during ambient unloaded conditions using the AutoRegressive Moving Average (ARMA) method and the time-domain Subspace System Identification (SSI) method

II) Assessment of traffic safety aspects based on the maximal amplitudes of vibrations during loaded operating conditions of the structure, for two separate train induced vibrations (train velocity of 40 km/h and 60 km/h)

III) Evaluation of the dynamic effects through a dynamic amplification factor estimated from the recorded responses during dynamically loaded structure (train velocity of 40 km/h and 60 km/h).

By utilization of the response-based data analysis methods the first natural frequencies of the steel structures are successfully identified and compared with FEM numerical results. The acceleration data is further assessed by preprocessing filtering techniques and extracted features are assessed according to EN 1990:2002 or UIC (International Union of Railways) code 776-2 (2009).

Keywords: Data-driven, railway bridge, structural identification, structural health monitoring, ambient vibration.

## 1. INTRODUCTION

Testing programs can deliver a significant input in the management of important civil infrastructures and reduce the multiple sources of uncertainty and variability typical for assessment of real engineering systems. Outlined by Brownjohn in [1], the cases where civil infrastructure monitoring may be required are: (i) modifications to an existing structure, (ii) monitoring of structures affected by external works, (iii) monitoring during demolition, (iv) structures subject to long-term movement or degradation of materials, (v) feedback loop to improve future design based on experience, (vi) fatigue assessment, (vii) novel systems of construction, (viii) assessment of post-earthquake structural integrity, (ix) decline in construction and growth in maintenance needs, and (x) the move towards performance-based design philosophy.

Since the earliest documented systematic bridge testing in the 1940s, monitoring programs for bridge infrastructure have historically been utilized for the purpose of understanding the load–structure–response chain, which eventually brings improvements and calibration of numerical models [1]. Nowadays, remote control and data retrieving hardware advancements, as well as software solutions for data-driven analysis drive the evolution of traditional bridge monitoring programs towards more holistic strategies based on output-only structural health monitoring (SHM) systems [2].

The focus of this study is to explore data records attained from a short-term output-only testing strategy utilized on two short-span bridge structures. The purpose is to obtain an initial structural identification of the systems' dynamics, data-based estimation of dynamic amplification factors, as well as vibration amplitudes of the bridge decks. [3].

## 2. TESTING AND DATA ANALYSIS

A short-term acceleration measuring system was installed on 2 railway bridges located in Kosovo. A schematic overview of the testing strategy is presented in Fig. 1. Summary on relevant bridge data, overview of the sensor placements and available monitored responses are provided in Figures 2 and 3.

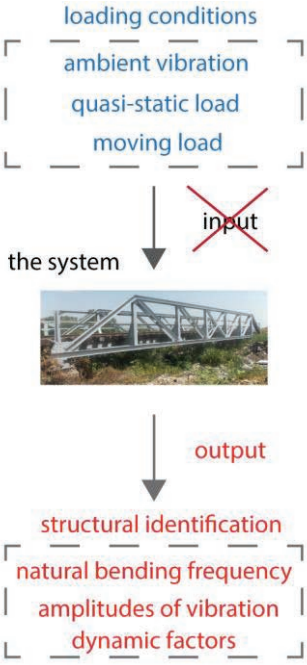



Figure 1. Schematic overview of the short-term SHM strategy.

Acceleration time histories are recorded by three tri-axial accelerometers for ambient vibration (Digitexx D110-T sensor) placed at three measurement points along the span of the structures. The acceleration responses were recorded for the following three scenarios: i) ambient vibration; ii) quasi-static load: a train passing with crawl speed of 5km/h; iii) moving load: a train passing with 40 km/h and 60 km/h.

Figure 2. Photographs of the structures under study.

## 2.1 VERTICAL BENDING FREQUENCIES

An initial step is to validate finite element results for the modal properties of the structures. To this end, first vertical bending frequencies of the systems were identified from the collected acceleration data.

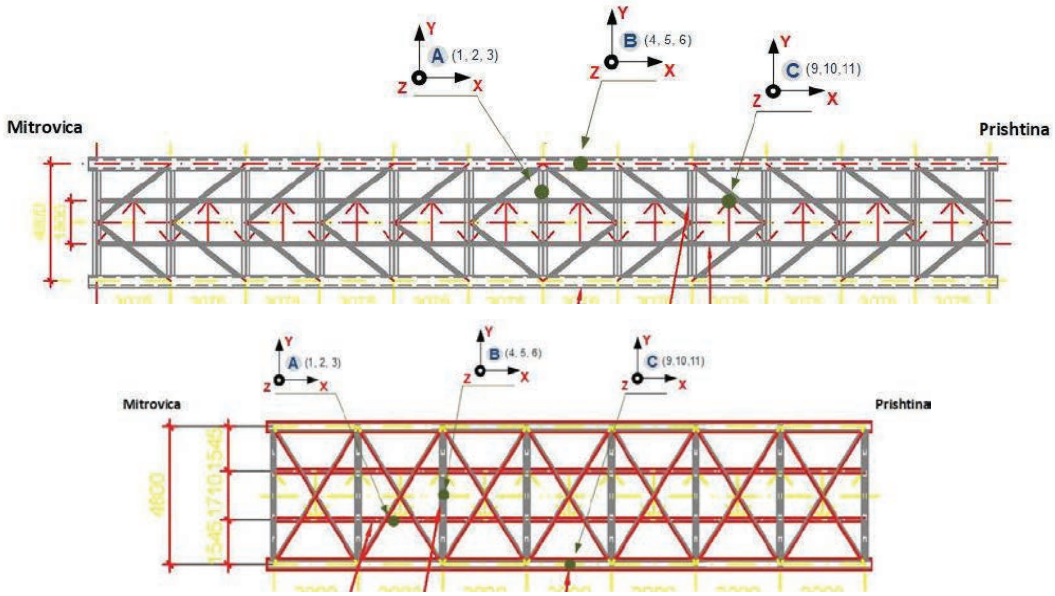
More precisely, the recorded data set lengths, approximately 10 min long and originally sampled with 200 Hz, were primarily preprocessed: detrended, downsampled (from 200 Hz to 50 Hz) and filtered with a lowpass filter (cutoff frequency of 25 Hz). The first vertical bending frequency of the bridges is then identified by employment of the time-domain parametric methods: ARMA method, an approach less sensitive to coupled modes, noise and short data set lengths, and the time-domain SSI method. For further details on the theory behind the methods and their limitations the interested reader is referred to [4].

The stabilization plots and estimated first natural frequencies for the vertical direction are presented in Figures 4 and 5, for the two bridge structures. As an additional criterion for eliminating computational and/or non-structural modes are the estimated modal damping values (accepted values less than 10%). Marked with red arrow at the same plots is an identified harmonic.

## 2.2 VIBRATION AMPLITUDES

Excessive vibrations of a given bridge deck correspond to measured levels of accelerations of the bridge deck. A traffic safety requirement for the prevention of track instability is based on the verification of the maximum acceleration of the bridge for the two loaded cases.

According to EN 1990:2002 or UIC code 776-2 (2009) as far as operating safety is concerned, the deck acceleration should be considered a serviceability limit state. In the case of bridges with slab tracks, the acceleration limit value is set at 0,5g for filtered low pass frequencies below 30 Hz. The measured acceleration responses and maximal amplitudes for the passing train with velocity of 40 km/h and 60 km/h are presented in Figures 6 to 9, for each velocity and bridge structure separately.




| Bridge | Span [m] | 1st FEM bending Frequency [Hz] | Sensors (unloaded case) | Sensors (loaded case) |
|---|---|---|---|---|
| B15 | 36.9 | 8.0 | A,B,C | A,B,C |
| B05 | 21.0 | 9.8 | A,B,C | A,B,C |

Figure 3. Summary on relevant data and overview of the sensor placements.

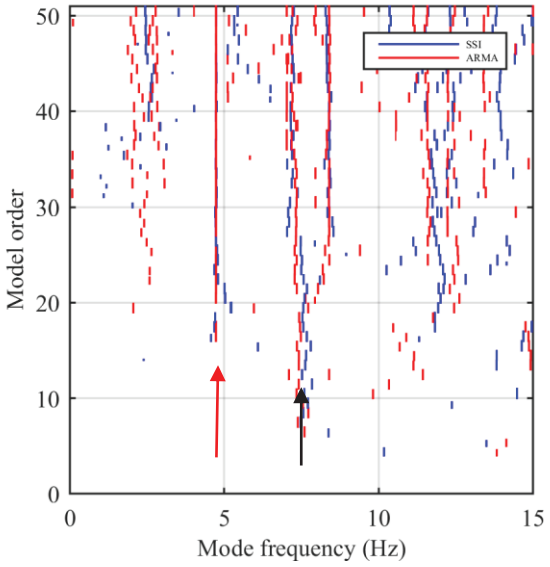



Figure 4. Stabilization plot for ARMA and SSI method for Bridge B15 (red arrow is identified harmonic, black arrow is identified structural frequency)

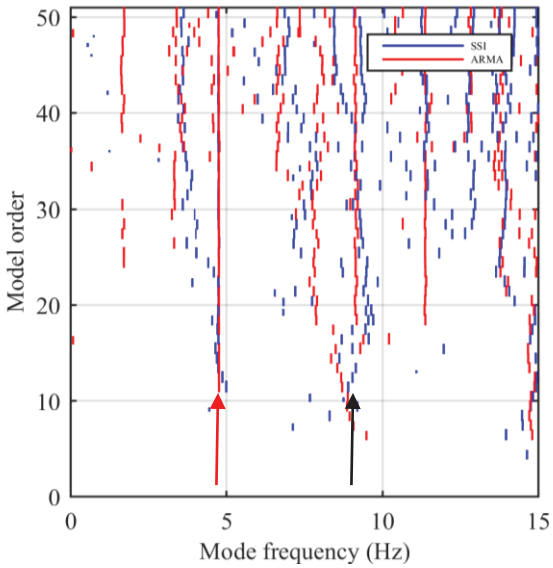



Figure 5. Stabilization plot for ARMA and SSI method for Bridge B05 (red arrow is identified harmonic, black arrow is identified structural frequency)

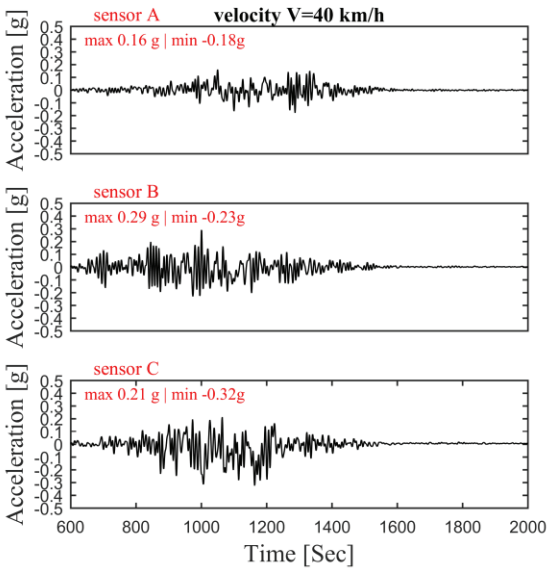



Figure 6. Time history plot of vertical accelerations and maximal amplitudes for velocity 40km/h for Bridge B15

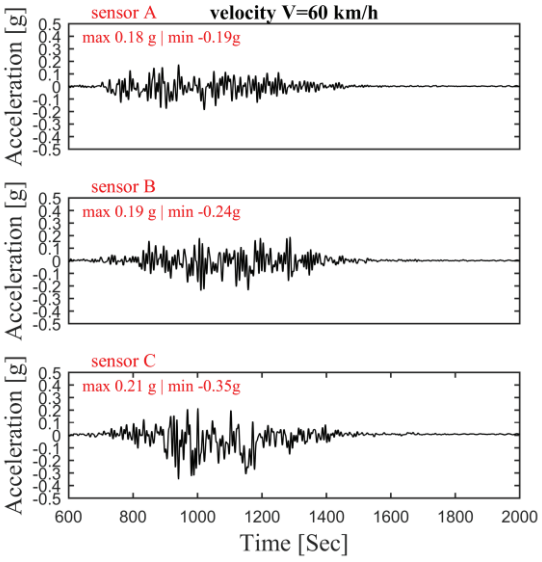



Figure 7. Time history plot of vertical accelerations and maximal amplitudes for velocity 60km/h for Bridge B15

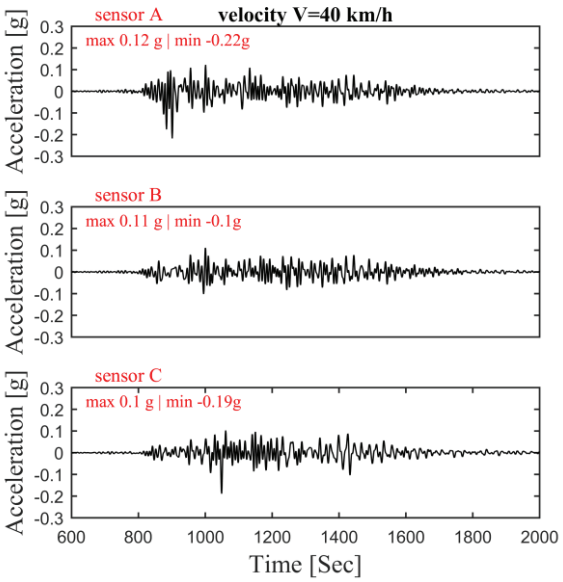



Figure 8. Time history plot of vertical accelerations and maximal amplitudes for velocity 40km/h for Bridge B05

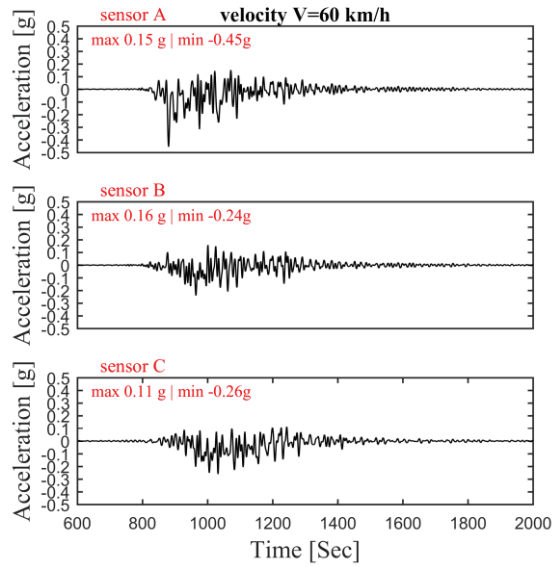



Figure 9. Time history plot of vertical accelerations and maximal amplitudes for velocity 60km/h for Bridge B05

## 2.3 DYNAMIC AMPLIFICATION FACTOR

In order to estimate the Dynamic Amplification Factor (DAF), the recorded vertical acceleration time histories corresponding to the three setups, namely the responses from the testing with crawl speed, train velocity of 40km/h and 60 km/h were preliminary filtered with a low-pass digital filter (cutoff frequency of 25 Hz). Then the DAF is calculated as the ratio of the maximum dynamic response for the corresponding velocity (40km/h and 60 km/h) and the maximum response for the crawl speed testing scenario.

The assessment results for the records of sensors placed at the most affected area of the bridge, namely sensor A for bridge B15, and sensor C for bridge B05 are herein presented for DAF estimation corresponding to velocity 60 km/h, Figures 10 and 11.

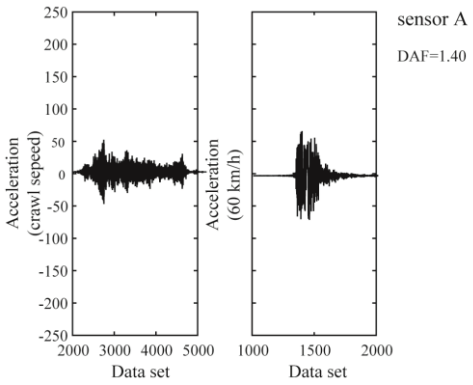



Figure 10. Signals for DAF estimation, velocity 60km/h, Bridge B15

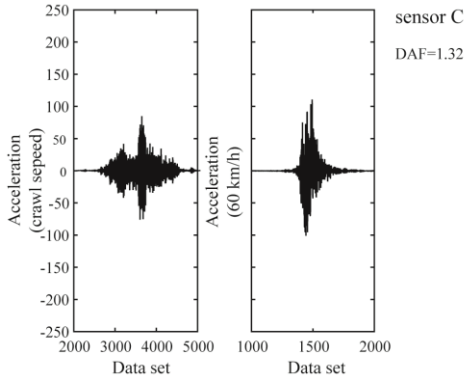



Figure 11. Signals for DAF estimation, velocity 60km/h, Bridge B05

## 3. CONCLUSIONS

The extracted features from recorded acceleration response data for unloaded and loaded operating conditions of the 2 bridges are summarized in Fig. 12 and Tab.2. With the purpose of verification, the FEM calculated natural frequencies (provided by Obermeyer Hellas Ltd.) are presented as well. The experimentally estimated first natural frequencies have a good agreement with the numerical results. The maximum acceleration amplitudes are within the serviceability limit case threshold value (EN 1990:2002) of 0.5g.

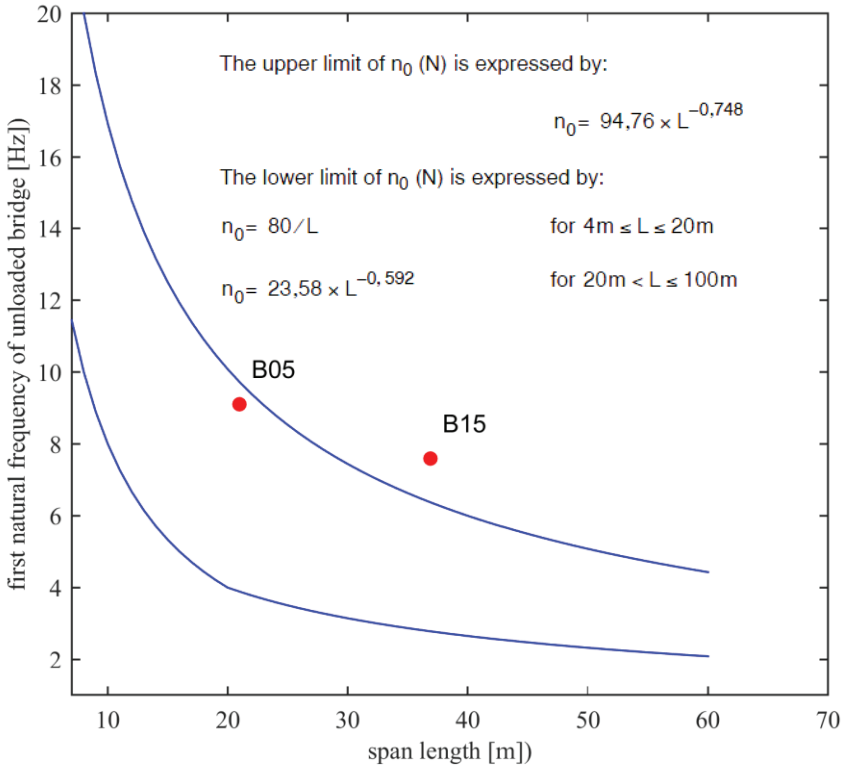



Figure 12. Limits of natural frequencies in relation to span length ( UIC code 776-2, 2009)

Table 2. Summary of estimated parameters from the measured structural responses

| Bridge | Experimental modes [Hz] | Numerical modes [Hz] | DAF 40 km/h | DAF 60 km/h | Max vertical acceleration 40 km/h | Max vertical acceleration 60 km/h |
|---|---|---|---|---|---|---|
| B15 | 7.6 | 8.0 | 1.16 ; 1.07 | 1.40 ; 1.37 | 0.32g | 0.35g |
| B05 | 9.11 | 9.8 | 1.00 | 1.32 | 0.22g | 0.45g |

The necessity for a subsequent dynamic analysis can be further assessed by estimating the domain N (UIC code 776-2, 2009). Dynamic analysis is necessary for speed of lines below 200km/h and first natural frequency above the upper limit of the predefined range N. In Fig. 12 the limits of domain N for the experimentally estimated natural frequencies in [Hz] as a function of the span length [m] are presented. Bridge B15 is above the upper limit of N.

## ACKNOWLEDGEMENTS

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# COMPARISON OF WIND ACTIONS ACCORDING TO EUROCODE AND PREVIOUS REGULATIONS

European norms for civil engineering structures, well known as Eurocodes, are set of highly harmonized codes regarding structural issues. Main rules and recommendations for calculation of wind actions on structures are given in EN 1991-1-4 which is the basic document. Also, some advanced analysis regarding wind actions on towers and masts are given in EN 1993-3-1, with the intention to be transferred in the basic document in the later phase. This paper contains a short presentation of the main rules and recommendations for wind action analysis and their comparison with previous regulations JUS (SRPS). The main focus of the paper is on wind action on buildings and comparative numerical example is also given. Some experiences and conclusions from the process of Serbian National Annex preparation are also presented.

Keywords: wind actions, structures, buildings, Eurocodes, JUS (SRPS), National Annex

## 1. INTRODUCTION

Wind actions on structures are covered by following parts of Eurocode EN 1991-1-4 [2] and EN 1993-3-1 [1]. The basic document for calculation of wind action on structures is [2], but some special problems regarding wind action on slender structures, such as steel towers, masts and chimneys are treated in [1]. EN 1991-1-4 is the main document for wind analysis and it consists of eight chapters (General, Design situations, Modeling of wind actions, Wind velocity and velocity pressure, Wind actions, Structural factor, Pressure and force coefficients, Wind action on bridges) and six informative annexes (A-F). In this paper, some of the most important parts of this code are presented, commented and compared with previous Yugoslavian codes for wind actions [4]-[7]. Special attention is paid on wind velocities and pressure, as well as a structural factor, pressure coefficients and wind analysis for buildings.

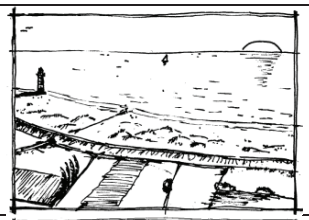
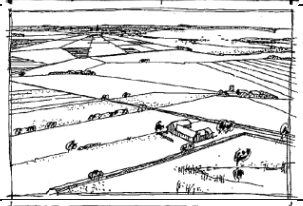
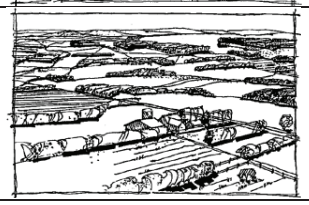
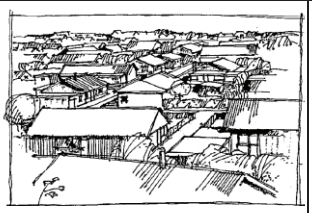
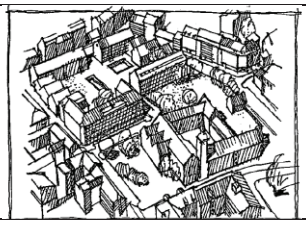
## 2. WIND VELOCITIES

Eurocode [2] deals with three different wind velocities: fundamental basic wind velocity, basic wind velocity and mean wind velocity.

Fundamental basic wind velocity ($v_{b,0}$) is the 10-minute mean wind velocity with an annual risk of being exceeded of $p$ = 0.02 (return period of 50 years), irrespective of wind direction, at a height of 10 m above flat open country terrain (category II - area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights) and accounting for altitude effects. The values of fundamental basic wind velocities, with or without the influence of altitude, should be given in National Annexes of each country, in the form of wind map or/and table. In case those values of wind velocities are without the influence of altitude, that influence should be taken into account multiplying by factor calt. Serbian new wind map from National Annex [8] has been based on 10-minute mean wind velocities with the influence of altitude, so factor $c_{alt}$ should not be used.

Considering that averaging time according to [2] is 10 minutes (600 s), values of fundamental basic wind velocity are generally higher than according to previous codes [4] which were based on one hour (3600 s) mean wind velocities. It is a first and important difference that should be taken into account in wind analysis. It is possible to make the conversion of wind velocities with different averaging time using averaging time factor $k_t$ from [4], but results might be unreliable because of different terrain categories in [2] and [4].

Table 1. Terrain categories – descriptions and main parameters

| Terrain categories | | | $z_0$ [m] | $z_{min}$ [m] | $k_r$ [-] | $c_r(z_{min})$ [-] |
|---|---|---|---|---|---|---|
| 0 |  | Sea, coastal area exposed to the open sea | 0.003 | 1 | 0.156 | 0.906 |
| I |  | Lakes or area with negligible vegetation and without obstacles | 0.01 | 1 | 0.170 | 0.782 |
| II |  | Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights | 0.05 | 2 | 0.190 | 0.701 |
| III |  | Area with regular cover of vegetation or buildings or with isolated obstacles with separations of max. 20 obstacle heights (such as villages, suburban terrain, permanent forest) | 0.3 | 5 | 0.215 | 0.606 |
| IV |  | Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m | 1.0 | 10 | 0.234 | 0.540 |
| $z_0$ and $z_{min}$ are ground terrain roughness parameters, $z_{max}$ = 200 m (maximum considered height of wind profile). | | | | | | |

Basic wind velocity ($v_b$) can take into account the effects of wind directions and season's character of the wind through the values of coefficients $c_{dir}$ and $c_{season}$:

$$v_b = c_{dir} c_{season} v_{b,0} \quad (1)$$

where $c_{dir}$ is the directional factor ($c_{dir} \leq 1.0$) and $c_{season}$ is the season factor ($c_{season} \leq 1.0$).

In the absence of reliable wind records, these values should be taken as 1.0, so in that case, basic wind velocity is equal to fundamental wind velocity ($v_b = v_{b,0}$). In Serbian National Annex [8] values of these factors are adopted as 1.0.

As stated before, fundamental and basic wind velocities are based on return period $T$ = 50 years or annual risk of being exceeded of $p$ = 0.02. In case of different return period $T$ or probability $p$, basic wind velocity should be corrected multiplying by the following factor:

$$c_{prob} = \left( \frac{1 - K \ln(-\ln(1-p))}{1 - K \ln(-\ln(0.98))} \right)^n \quad (2)$$

where $p$ is the required probability of exceeding ($p = 1/T$) and $K$ and $n$ are parameters with following recommended values: $K$=0.2 and $n$=0.5.

With these recommended values of parameters, Eq. 2 gives similar results as return period factor $k_T$ from previous regulation [4] (for example: $c_{prob}$ =1.04 for return period $T$ = 100 years, and $c_{prob}$ = 0.90 for return period of 10 years) [9].

Mean wind velocity $v_m(z)$ takes into account terrain roughness and orography, as well as referent height above ground. In Eurocode, there are five different categories of terrain (see Table 1). Mean wind velocity should be calculated as follows:

$$v_m(z) = c_o(z)\ c_r(z)\ v_b \quad (3)$$

where $c_o(z)$ is the orography factor and $c_r(z)$ is the roughness factor.

Detailed procedures for numerical calculation of the orography factor $c_o(z)$ are given in Chapter A.3 of informative Annex A.

The roughness factor $c_r(z)$ takes into account the variability of the mean wind velocity at the site due to the ground roughness of the terrain upwind of the structure in the considered wind direction and the height ($z$) above ground level. Lowest roughness in upstream wind direction should be adopted. Small isolated zones with different terrain categories can be ignored (see Figure 1), but in the case when the structure is situated near the change of terrain category, transition of different categories should be considered, using one of two alternative procedures given in Chapter A.2 of Annex A. Both alternative procedures are allowed for use in [8].

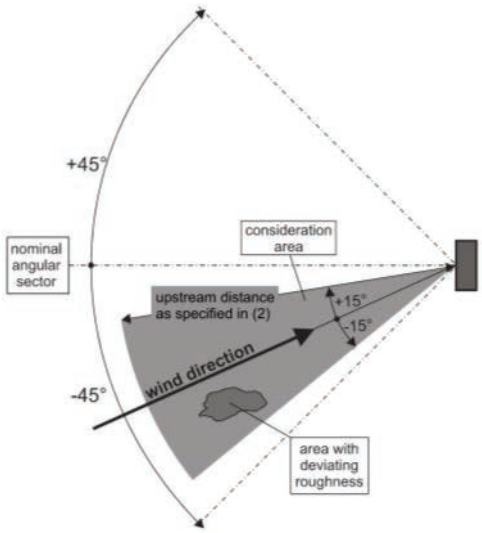



Figure 1. Assessment of terrain roughness [2]

The roughness factor $c_r(z)$ should be determined as follow:

$$c_r(z) = k_r \ln\left(\frac{z}{z_0}\right) \text{ for } z_{min} \leq z \leq z_{max} = 200 \text{ m} \quad (4a)$$

$$c_r(z) = c_r(z_{min}) = k_r \ln\left(\frac{z_{min}}{z_0}\right) \text{ for } z \leq z_{min} \quad (4b)$$

where $k_r$ is a terrain factor which is constant for each terrain category (see Table 1) and given by the following expression:

$$k_r = 0.19 \left(\frac{z_0}{z_{0,II}}\right)^{0.07} = 0.19 \left(\frac{z_0}{0.05}\right)^{0.07} \quad (5)$$

## 3. WIND PRESSURES AND FORCES

Wind pressure is, following Bernoulli equation, proportional to the square of wind velocity, so basic wind pressure (or basic velocity pressure as it stated in Eurocode) should be calculated as follows:

$$q_b = \frac{1}{2} \rho\ v_b^2 \approx \frac{v_b^2 [\text{m/s}]}{1600} \left[\text{kN/m}^2\right] \quad (6)$$

where $v_b$ is basic wind velocity (according to Eq. 1) and $\rho$ is the air density, which depends on

altitude, air temperature and barometric pressure (recommended value is $\rho$=1.25 kg/m$^3$).

Fluctuating component of the wind velocity pressure or dynamic wind action should be taken into account by characteristic peak velocity pressure $q_p(z)$ at referent height $z$ which should be calculated by the following expression:

$$q_p(z) = \left[1 + 7I_v(z)\right] \frac{1}{2}\rho\, v_m^2(z) = c_e(z)\, q_b \quad (7)$$

where $I_v(z)$ is the turbulence intensity and $c_e(z)$ is the exposure factor that depends on turbulence intensity and terrain category.

The turbulence intensity of the wind at referent height $z$ can be obtained as the quotient of the standard deviation of turbulence $\sigma_V$ and mean wind velocity $v_m(z)$ for $z_{min} < z \leq z_{max}$ as follows:

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_r \sigma_V}{v_m(z)} = \frac{k_I}{c_o(z) \cdot \ln(z/z_0)} \quad (8a)$$

and for $z \leq z_{min}$

$$I_v(z) = I_v(z_{min}) \quad (8b)$$

where $k_I$ is the turbulence factor whose value can be defined in National Annex (recommended value is $k_I$ =1.0). Also, it should be noticed that turbulence intensity should be calculated as a unique value for the whole structure at referent height $z_s$.

There is no explicit expression for the calculation of $c_e(z)$ in [2], but the only general expression that is derived from Eq. 7. Using general Eq. 7, with Eq. 3, 4a and 8a, it can be finally obtained in the following shape:

$$c_e(z) = \frac{q_p(z)}{q_b} = \\ = \left[1 + 7\frac{k_I}{c_o(z)\ln(z/z_0)}\right] k_r^2 \ln^2(z/z_0) c_0^2(z) \quad (9)$$

Looking at previous Eq. 9 it can be observed that the exposure factor $c_e(z)$ is a function of referent height $z$ and terrain category and, in some way, define wind velocity pressure profile.

Wind actions on the whole structure, cladding or structural elements, depending on the type of structure and wind analysis, can be analyzed through using wind pressure on surfaces or wind forces.

Wind pressure on surfaces can be external ($w_e$) and internal ($w_i$) and should be taken as positive or negative. The pressure which is directed towards the surface should be taken as positive, but suction, directed away from the surface, should be taken negative (see Figure 2).

Total, net wind pressure is equal to the sum of pressures (with appropriate signs) that acts on opposite sides of considered surfaces such as walls or roofs (see Figure 2).

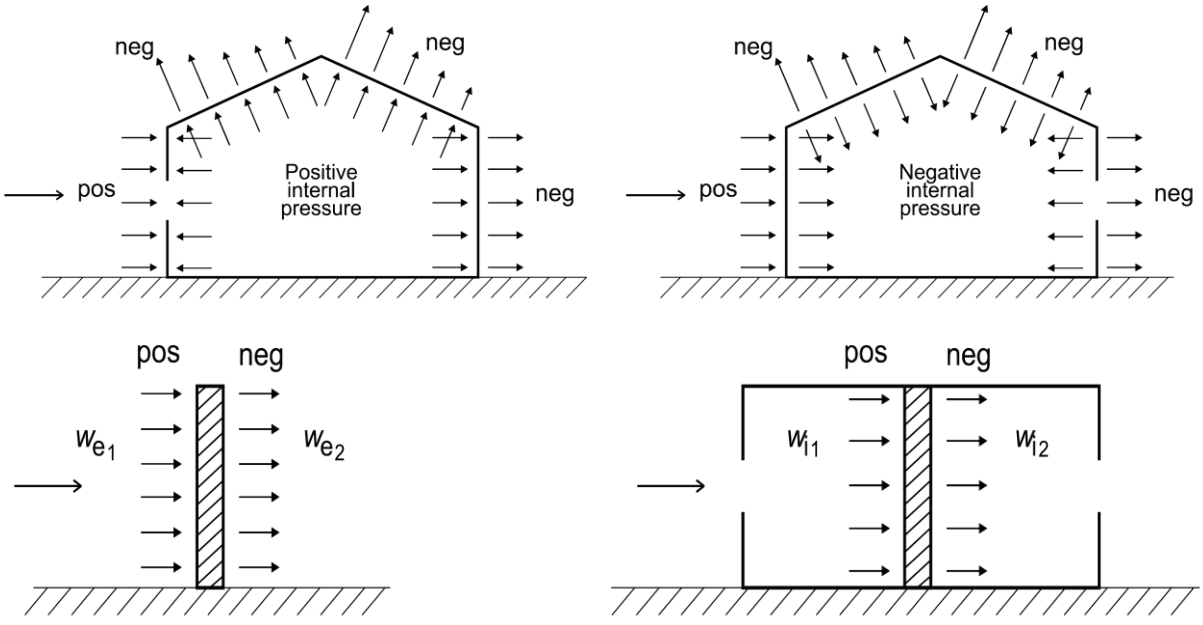



Figure 2. Wind pressures on surfaces – convention for positive and negative pressure

External and internal wind pressures on surfaces for referent height ($z_e$ or $z_i$) should be calculated by the following expressions:

$$w_e(z_e) = q_p(z_e)c_{pe} \quad (10)$$

$$w_i(z_i) = q_p(z_i)c_{pi} \quad (11)$$

where $q_p(z)$ is peak velocity pressure and $c_{pe}$ and $c_{pi}$ are external and internal pressure coefficients, respectively, which are given in Chapter 7 of EN 1991-1-4 for different types of buildings.

According to [2] wind actions on the structure or a structural component should be analyzed using wind forces. Wind forces can be determined by calculation of forces using force coefficient $c_f$ or by calculating wind pressure on surfaces (with pressure coefficients). The first approach, with force coefficient, is usually used for wind action on structural elements. In that case, wind force should be determined by the next expression:

$$F_w(z) = q_p(z)c_s c_d c_f A_{ref} \quad (12)$$

where $q_p(z)$ is the peak velocity pressure, see Eq. 7, $c_s c_d$ is the structural factor, see Eq. 16, $c_f$ is the force coefficient for structure or structural element, $A_{ref}$ is referent area of the structure or structural element.

For wind actions on buildings, using the second approach with wind pressure on surfaces is more convenient. Forces on structure from external wind pressures on surfaces, in that case, should be obtained as follow:

$$F_{w,e}(z_e) = w_{pe}(z_e)c_s c_d A_{ref} \quad (13)$$

and for and internal wind pressures:

$$F_{w,i}(z_i) = w_{pi}(z_i)c_s c_d A_{ref} \quad (14)$$

Replacing $w_p(z_e)$ with Eq. 10 and $q_p(z)$ with Eq. 7, and using Eq. 1 and 3, Eq. 13 can be transformed into the form which is presented with Eq. 14. In this way expression for wind force on structure according to Eurocode [2] is transformed in the form which is similar to one given in previous Yugoslavian codes [4]-[7]. Also, the structural factor $c_s c_d$ should be replaced with Eq. 16 to get the final expression that is completely comparative with previous Yugoslavian codes.

$$F_{w,e}(z) = \frac{1}{2}\rho\left(v_{b,0}c_{dir}c_{season}c_{prob}c_r(z)c_o(z)\right)^2 \cdot\left[1+7I_v(z_s)\right]c_s c_d c_{pe}A_{ref} \quad (15)$$

## 4. STRUCTURAL FACTOR - DYNAMIC RESPONSE OF STRUCTURE

The structural factor $c_s c_d$ takes into account the effect on wind actions from the non-simultaneous occurrence of peak wind pressures on the surface $c_s$ together with the effect of the vibrations of the structure due to turbulence $c_d$. Procedure for calculation of structural factor $c_s c_d$ is given in Section 6 of EN 1991-1-4 and its alternative in Annexes B and C.

As a simplification, the structural factor may be taken as 1 ($c_s c_d = 1$) in the following cases:

a) for buildings with a height less than 15 m;
b) for facade and roof elements having a natural frequency greater than 5 Hz (glazing spans smaller than 3 m usually lead to natural frequencies greater than 5 Hz);
c) for framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth;
d) for chimneys with circular cross-sections whose height is less than 60 m and 6.5 times the diameter.

For civil engineering works (other than bridges, which are considered in Section 8), and chimneys and buildings outside the limitations given in a), c) and d) above, and alternatively for all cases, values of $c_s c_d$ should be determined as follow:

$$c_s c_d = \frac{1+2k_p\ I_v(z_s)\ \sqrt{B^2+R^2}}{1+7I_v(z_s)} \quad (16)$$

where $k_p$ is the peak factor, defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation; $B^2$ is the background factor, allowing for the lack of full correlation of the pressure on the structure surface; $R^2$ is the resonance response factor, allowing for turbulence in resonance with the vibration mode.

## 5. AERODYNAMIC COEFFICIENTS FOR WIND ACTION ON BUILDINGS

Eurocode [2] recognizes different types of aerodynamic coefficients for buildings: external and internal pressure coefficients, net pressure coefficients, friction coefficients and force coefficients.

The external pressure coefficients $c_{pe}$ should be used for calculation of wind action on walls and roof surfaces. Different types of roofs are covered by Eurocode: flat, monopitch, duopitch, hipped, multispan, vaulted roofs and domes. The external pressure coefficients $c_{pe}$ for buildings or parts of buildings depend on the size of the loaded area A.

There are two types of external pressure coefficient for buildings: $c_{pe,1}$ and $c_{pe,10}$. For small loaded areas ($A \leq 1.0$ m$^2$), the external pressure coefficient $c_{pe,1}$ should be used. So, $c_{pe,1}$ is the local coefficient intended for the design of small elements such as cladding elements and roofing elements. Otherwise, values of external pressure coefficient $c_{pe,10}$ should be used for larger loaded areas ($A \leq 10.0$ m$^2$). It is global coefficient that should be used for design of main structure of building.

The values of both coefficients $c_{pe,1}$ and $c_{pe,10}$ are given in appropriate tables for walls and different types of roofs. In case of the loaded area $A$ between 1 and 10 m$^2$ value of coefficient $c_{pe}$ can be obtained by linear logarithmic interpolation, using following recommended expression:

$$c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10}) \log_{10} A \quad (17)$$

Chapter 7 of [2] gives values of both coefficients $c_{pe,1}$ and $c_{pe,10}$ for different zones of vertical walls and roofs with different configurations (flat, monopitch, duopitch, hipped, multispan, vaulted roofs and domes). For cases of roofs (buildings) that are not covered by Eurocode, wind tunnel tests are recommended for determination of aerodynamic coefficients.

The values of internal pressure coefficient $c_{pi}$ depend on the size and distribution of the openings (windows, doors, chimneys) in the building, as well as on background permeability (air leakage around doors). In case of uniformly distributed openings, which is characteristic for regular buildings, the internal coefficient should be obtained using parameter $\mu$ which presents the ratio of the area of openings with negative or zero value of $c_{pe}$ and the total area of all openings. Where it is not possible to reliably estimate the mentioned parameter $\mu$ the value of internal coefficient $c_{pi}$ should be adopted as the more onerous of +0.2 and -0.3. Internal and external pressures shall be considered to act together at the same time, and the worst combination shall be taken.

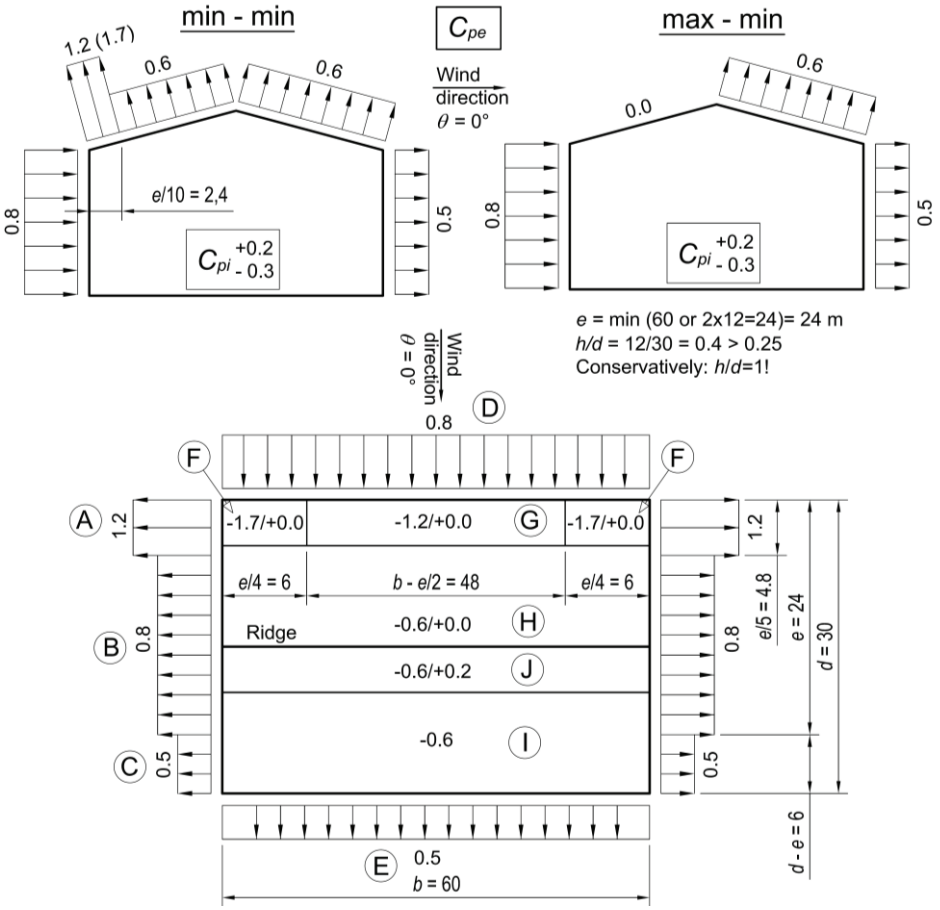



Figure 3. External and internal coefficients for a typical industrial building [10]

In some cases, such as canopies, free-standing walls, parapets etc., unique, total net pressure coefficients $c_{p,net}$ should be used.

Force coefficients $c_f$ are regularly used for: signboards, structural elements with different cross-sections (rectangular, sharp-edged, regular polygonal, circular), lattice structures and scaffoldings and flags.

External and internal aerodynamic coefficients, as well as wind load distribution for typical industrial building ($b/d/h$=30/60/12 m) with duopitch roof (with the slope of 5°) are given in Figure 3.

## 6. COMPARATIVE ANALYSIS - EC VS. JUS (SRPS)

Comparing EC and JUS regarding wind actions, the first important difference is in averaging time for basic wind velocity. Namely, fundamental basic wind velocity $v_{b,0}$ from EC is based on 10-minutes averaging time, but JUS (SRPS) deal with one hour mean wind velocity $v_{m,50,10}$, so EC operates with higher wind velocity (about 8-10%).

Only because of that, considering that wind velocity is main input data for calculation of wind actions that depends on square of wind velocity, the intensity of wind pressures and forces are more than 20% higher. Also, EC gives the possibility for adjustment of wind velocity through direction and season factors, what is not the case in JUS (SRPS). It should be noticed that the return period of 50 years is the same in EC and JUS (SRPS).

Beside of that, there are five different terrain categories in EC (0, I, II, III and IV) instead of three (A, B and C) in JUS (SRPS). So, EC gives more possibilities for the description of site location and more accurate results of wind analysis. Expressions for exposure factor are different: EC uses logarithmic function but JUS (SRPS) uses the exponential function. Comparing the exposure factors, that are shown on Figure 4, it can be observed that, despite different nature of expressions, functions of exposure factors for default terrain categories (II and B) are very similar. The same situation is with turbulence intensity.

Generally, algorithms for the calculation of wind actions (forces or pressures) are similar. Some terms are different, as well as expressions for calculation of some intervalues in different steps of the calculation, but it can be shown that the final expressions are quite similar (see Table 2).

Criteria for dynamically susceptible structures are essentially different in EC and JUS (SRPS). For dynamically susceptible structures (with $c_s c_d \neq 1$), EC proscribes additional calculation of structural factor $c_s c_d$ (see Eq. 16), in accordance with Chapter 6 and Annex B. JUS (SRPS) has a completely different procedure for calculation of dynamic coefficient $G_z$ for structures that are susceptible to resonance (slender structures) and that are not (big rigid structures) for big rigid structures, as follows:

$$G_z = 1 + 2g\, I_z\, B = 1 + 6\, I_z\, B \tag{18}$$

and for slender structures:

$$G_z = 1 + 2g\, I_z\, B \sqrt{1 + (R/B)^2} = 1 + 2g\, I_z \sqrt{B^2 + R^2} \tag{19}$$

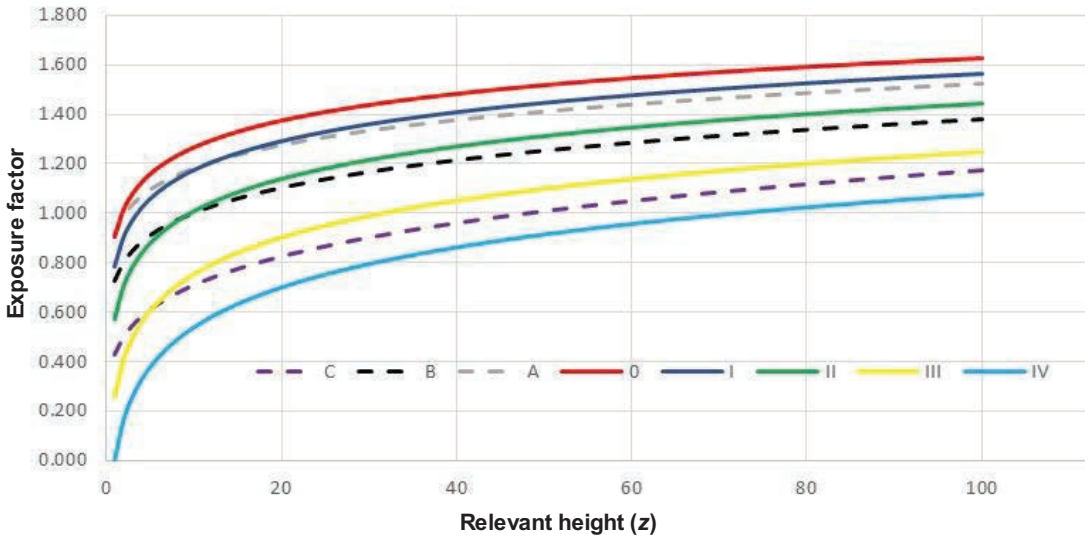



Figure 4. Exposure factor for different terrain categories A, B and C (JUS) and 0, I, II, III and IV (EC)

where $g$ is the gust factor, almost the same as the peak factor $k_p$ from EC:

$$g = \sqrt{2\ln(\nu T)} + \frac{0.577}{\sqrt{2\ln(\nu T)}} \quad (20)$$

The basic expression for the peak velocity pressure $q_p(z)$, according to EC, includes certain dynamic amplification $(1+7I_v(z))$, that is similar to dynamic coefficient $G_z$ for big rigid structures (buildings). On the other hand, if the criteria for a simplified calculation of structural factor are not met, dynamic effects should be taken. In those cases, final expression for wind force can be obtained replacing $c_s c_d$ in Eq. 15 with Eq. 16. Comparison of most important final expressions for wind analysis, such as wind velocity, wind pressure and wind force, according to EC and JUS (SRPS) is given in Table 2.

It should also be noticed that there are some differences in aerodynamic coefficients. Eurocode provides aerodynamic coefficients for almost all usual types of buildings that can be met in engineering practice. In that sense, EC is more comprehensive than JUS (SRPS). Comparison of aerodynamic coefficients for simple industrial building, according to EC [2] and JUS (SRPS) [3] - [7] is shown on Figure 5.

Finally, differences in the calculation of wind action on a typical industrial building according to EC and JUS (SRPS) are shown through a numerical example. Results of calculation of wind actions are given in Table 3. Wind load layouts are shown in Figure 6 and internal forces and moments are presented in Figure 7. Analysing the results of numerical example, it is obvious that the final results of wind actions on structure (internal forces and moments) are significantly higher according to EC than JUS (SRPS).

Table 2. Comparison of main expressions from EC and JUS (SRPS)

| | |
|---|---|
| **Wind velocity** | |
| EC | $v_m(z) = v_{b,0}\ c_{dir}\ c_{season}\ c_{prob}\ c_r(z)\ c_o(z)$ |
| JUS (SRPS) | $v_{m,T,z}(z) = v_{m,50,10}\ k_t\ k_T\ S_z\ K_z(z)$ |
| **External wind pressure on structure (for small rigid buildings - $c_s c_d = 1$)** | |
| EC | $w_e(z) = \frac{1}{2}\rho\ \left(v_{b,0}\ c_{dir}\ c_{season}\ c_{prob}\ c_r(z)\ c_o(z)\right)^2 \left[1+7I_v(z_s)\right]\ c_{pe}$ |
| JUS (SRPS) | $q_w(z) = \frac{1}{2}\rho\ \left(v_{m,50,10}\ k_t\ k_T\ S_z\ K_z(z)\right)^2 (1+6I_z B)\ C$ (for $g$ = 3.0) |
| **Wind force – general case ($c_s c_d \neq 1$)** | |
| EC | $F_w = \frac{1}{2}\rho\ \left(v_{b,0}\ c_{dir}\ c_{season}\ c_{prob}\ c_r(z)\ c_o(z)\right)^2 \left(1+2k_p I_v \sqrt{B^2+R^2}\right)\ c_{pe} A_{ref}$ |
| JUS (SRPS) | $F_w = \frac{1}{2}\rho\ \left(v_{m,50,10}\ k_t\ k_T\ S_z\ K_z(z)\right)^2 \left(1+2gI_z \sqrt{B^2+R^2}\right)\ C\ A$ |

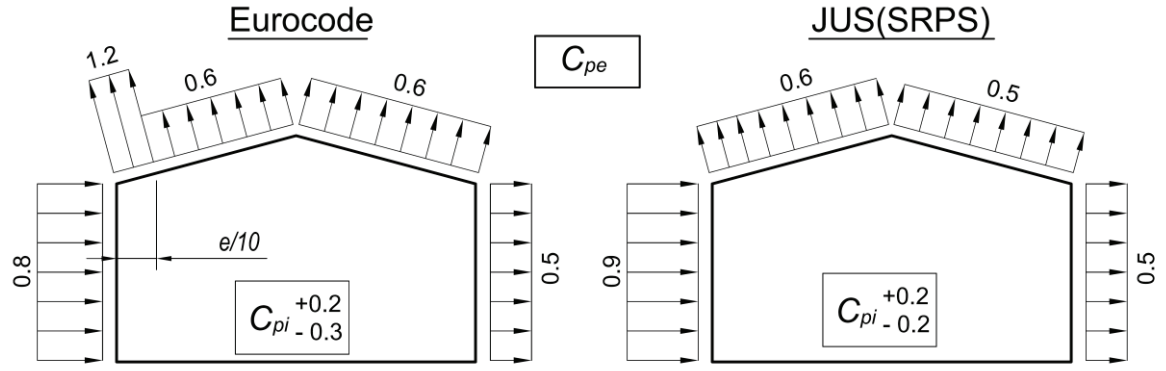



Figure 5. Comparison of aerodynamic coefficients $c_{pe}$ and $c_{pi}$ for typical industrial building

Table 3. Comparative analysis of wind pressure according to EN and JUS (SRPS) – Numerical example for simple industrial building ($h$ = 12 m < 15 m, $c_s c_d$ = 1)

| JUS (SRPS) | EC |
|---|---|
| Terrain category: B | Terrain category: II |
| Basic wind velocity | Fundamental and basic wind velocities |
| $V^B_{m,T,10}$ = **26 m/s** (averaging time 3600 s)<br>$k_t$ = 1.0 $k_T$ = 1.0 (for 50 years return period) | $v_{b,0} = k_t^B v^B_{m,T,10} = 1.6509 \cdot 600^{-0.0645}$ = 28.41 m/s<br>(averaging time 600 s)<br>$v_b = c_{dir} c_{season} v_{b,0} = 1 \cdot 1 \cdot 28.41$ = **28.41 m/s** |
| Orography factor:<br>$S_z$ = 1.0 | Orography factor:<br>$c_o(z)$ = 1.0 |
| Exposure factor<br>(for $z$ = 12 m) | Exposure factor<br>(for $z$ = 12 m) |
| $K_z(12) = \sqrt{b}\ (z/10)^{\alpha} = \sqrt{1} \cdot (12/10)^{0.14}$ = 1.026 | $c_r(12) = k_r \ln(12/z_0) = 0.19 \ln(12/0.05)$ =1.041 |
| Mean wind velocity | Mean wind velocity |
| $v_{m,T,z} = k_t k_T K_z S_z v^B_{m,50,10} = 1 \cdot 1 \cdot 1.026 \cdot 1 \cdot 26$ =<br>**26.67 m/s** | $v_m(z) = c_r(z) c_o(z) v_b = 1.041 \cdot 1 \cdot 28.41$ =<br>**29.59 m/s** |
| Aerodynamic wind pressure | Peak velocity pressure |
| $q_{m,T,z} = \frac{1}{2}\ \rho\ v^2_{m,T,z} = \frac{1}{2} 1.225 \cdot 10^{-3} \cdot 26.67^2$ =<br>0.436 kN/m$^2$<br>$q_{g,T,z} = G_z\ q_{m,T,z} = 2.0 \cdot 0.436$ = **0.872 kN/m$^2$**<br>(for $G_z$ = 2.0) | $I_v(z) = \frac{k_I}{c_o(z) \ln(z/z_0)} = \frac{1}{1 \cdot \ln(12/0.05)}$ = 0.182<br>$q_p(z) = [1 + 7 \cdot 0.182] \frac{1}{2} 1.225 \cdot 10^{-3} \cdot 29.59^2$<br>$q_p(z) = 2.277 \cdot 0.536$ = **1.221 kN/m$^2$** |

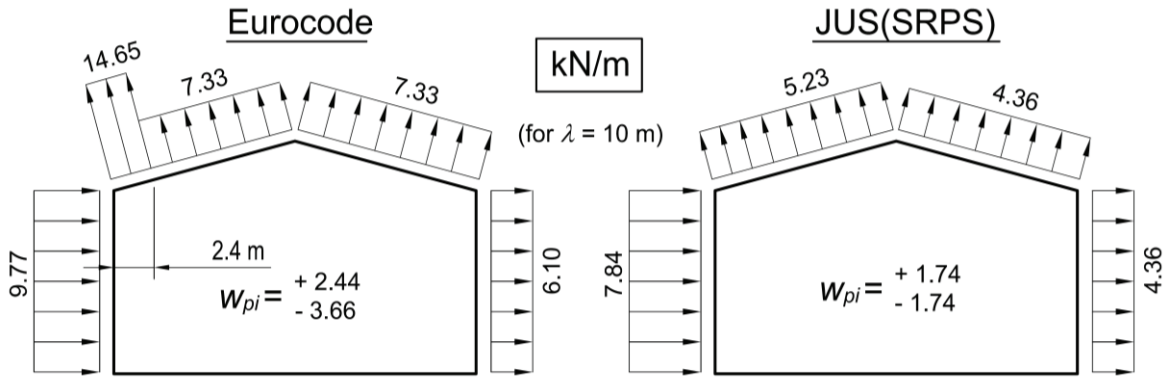



Figure 6. Wind actions on industrial building – EN vs. JUS (SRPS)

Maximal bending moment in beam-column is higher for 34% (503.9/377.3), axial forces in the top and bottom chord of truss girder are higher for 50% (407.2/271.2) and 49 % (393.2/263.9), respectively (see Figure 7). Also, it should be noticed that EC is based on limit state theory and partial safety factors, but JUS (SRPS) deal with allowed stresses for different design cases with unique safety factors. Because of that, it is not easy to precisely compare final results of design according to these different codes. But, USL load combinations with wind action as dominant variable action give more unfavourable effects of wind actions than JUS (SRPS), especially in case of wind suction on roof girders.

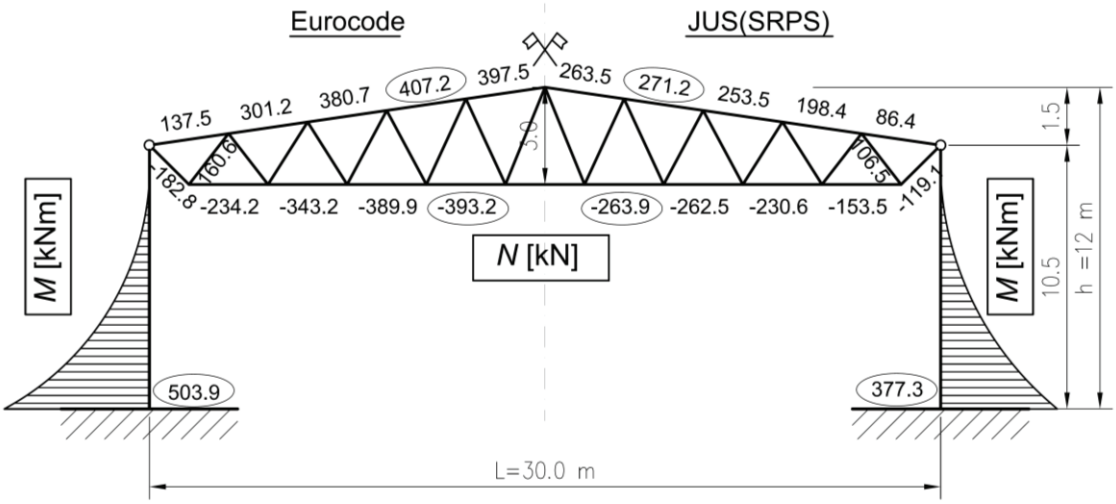



Figure 7. Internal forces and moments due to wind actions on industrial building – EN vs. JUS (SRPS)

## 7. CONCLUSIONS

Eurocode is, without doubt, the most contemporary and the most comprehensive set of codes for civil engineering structures. But, in its part that deals with wind actions, as well as in some other parts, there are some shortcomings and inconsistencies. Speaking about wind actions, one of the biggest problems is the huge number of clauses in which National choice is allowed (even 65). Also, leading European countries (Germany, France and UK) had very different approaches in the process of adoption of Nationally Defined Parameters (NDPs). Beside of that, Annexes of EN 1991-1-4 are quite discussible and even unreliable, so most of them are not recommended for using or have only informative status, without clearly defined alternatives. Rules for design of wind action on slender structures (towers, masts and chimneys) and their dynamic response, as well as criteria for application of multimodal dynamic analysis, are not given in basic standard for wind actions [2], but in part of Eurocode for steel structures EN 1993-3-1 [1]. Observed shortcomings and inconsistencies will be corrected in the new generation of Eurocode.

Regarding previous codes, they were, generally similar to Eurocode but they gave importantly lesser intensity of wind effects on structures than Eurocode. The main reason for that is one-hour averaging time for wind velocity and smaller values of aerodynamic coefficients. If some inconsistencies regarding the choice of aerodynamic coefficients and big changes of wind actions for slender structure with ratio $(R/B)^2$ close to 0.5, as well as some discussible wind velocities are excluded, these codes can be considered quite correct, especially for the time when they have been published (1991).

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# ANALYSIS OF RESISTANCE AND STABILITY OF CASTELLATED COMPOSITE BEAM FOR DIFFERENT NUMBER OF SHEAR CONNECTORS

One castellated and seven composite castellated beams are modeled using the software package SAP2000. The composite beams are modeled using link elements as shear connectors with stiffness of 100 kN/mm. For the composite models, one beam with 52 links, four beams with 26 link and two beams with 15 links are adopted with different positioning: above the openings, above the web-posts and uniformly along the beam. It was determent that the increase of shear connectors leads to increase of normal stresses in the corners of the first opening, where in comparison with castellated beam there is 30 % and 15 % rise for 52 links and 15 links above openings correspondingly for uniform load of 39 kN/m'. There is also a difference of 5 % between the stresses for links above openings and above web-posts, where the favorable position is that across the openings. According to the previous, the most commending beam will be with fewer shear connectors positioned above the openings. Therefor it can be assumed that it is best to design the castellated composite beam with partial shear connection, otherwise it would have to ensure full web near supports or increase the stiffness of the bottom part of the beam – concluding an optimized asymmetric section.

Keywords: castellated beam, composite beam, partial shear connection, local stability

## 1. INTRODUCTION

The stiffness of an I or H steel beam section can be increased by two means: with composite action; and cutting the web along the longitudinal axis in a zig zag or other pattern, and welding the two parts together in a way to produce a higher section where the stiffness can be increased by 40 % (Figure 1.) Depending on the cutting pattern, numerous types of beams can be fabricated: *castellated beam*, cellular beam, beam with sinusoidal and elongated openings. As the most optimal

geometry, according minimal waste material and one cutting procedure in production, for this research castellated beam is used.

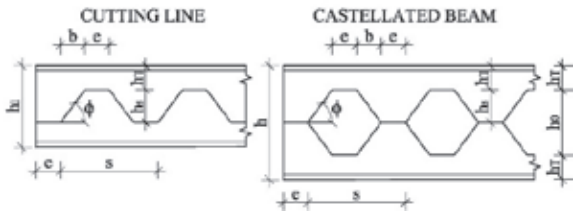


Figure 1. Castellated beam and cutting line.

In some instances, the choice for this type of beam may not be obvious, like in structures where there is substantial concentrated loading (bridges) or for heavy loaded continuous beams where reinforcement is needed around the critical openings, thus increasing the production price. The structural advantage of this type of beam is seen at long span structures where the design is governed by serviceability limit state (SLS). As for functional advantage, the passing of installation through openings and easier installation of secondary structural systems (like ceiling system) provide optimal exploitation of space which can be significant for multistory buildings.

Beside the mentioned advantages, the openings disrupt the stress flow along the web where stress localization occurs at the edges of the openings resulting in additional modes of failure. Experimental and finite element studies on castellated beams have reported six main different modes of failure (Kerdal and Nethercot (1984)): (1) flexural mechanism, (2) 'Vierendeel' or shear mechanism, (3) rupture of welded joints, (4) lateral torsional buckling and web-post buckling due to (5) shear or (6) compression.

Vierendeel mechanism is associated with high shear forces where plastic hinges are formed at the edges of the web openings due to secondary bending moments which are result from the action of shear force in the tee-sections over the horizontal length of the opening ($l_{eff}$ = e). In oppose of non-composite castellated beam, the composite action increases the Vierendeel resistance for $M_{vc}/l_{eff}$ where it depends on the number of shear connectors above the opening, the uplift resistance and length of the opening (Lawson et al. (2013)), figure 2. The composite action also changes the position of the neutral elastic and plastic axis - moving it toward the concrete slab, which results in decrease of the horizontal shear force ($V_{h,Ed}$) and bending moment ($M_{h,Ed}$) in the narrowest part of the web-post. Because the web-post buckles under high horizontal shear force, the redistribution of internal forces is in favor.

In general, the composite action has positive impact on the resistance and stability of the castellated beam and is closely associated with the number, resistance and position of the shear connectors. Despite the numerous studies, there is very little research on the stress distribution around the openings for different number of shear connectors and their configuration above the openings and web-posts. Most of the provided research gives a correlation of the resistance and stability to the opening geometry and concrete strength.

Sheehan et al. (2016) [7] conducted an experiment on two unpropped asymmetric composite cellular beam for determining the minimal shear connection degree. They concluded that the provided minimal shear connection degree requirement according EN1994-1-1 for composite steel beams cannot be used for this type of beams.

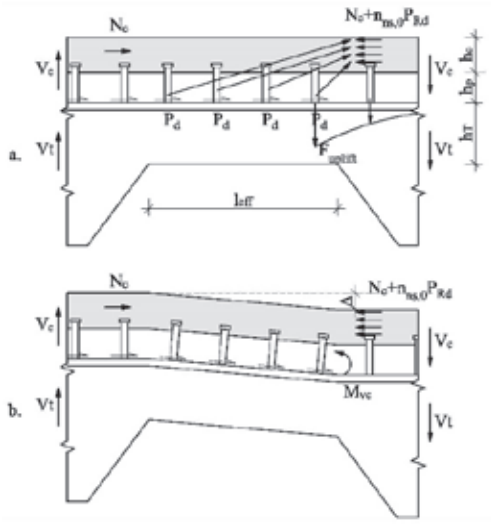


Figure 2. Vierendeel bending and Vierendeel resistance moment due to composite action.

The obtained minimal degree was 36 %, almost half of the calculated minimal degree according the analytical relations in accordance to EN. Aggelopoulos et al. (2018) [1] also conducted research for this specific topic. They provided relations for calculating the minimal shear degree for propped and unpropped composite beams with the conclusion that if the opening diameters for symmetrical and asymmetrical beams exceeds 60 % of the height, the minimal shear degree can be relaxed.

The aim of this research is to define the difference in stress distribution for different

number of shear connectors and configuration along the beam, as for the optimal position and correlation to the different modes of failure. Also, analytical computation is made based on the existing manuals (SCI publication P355, Lawson and Hicks (2011) [6]) and working documents (Lawson et al. (2006), [5]) according to EN 1994-1-1 for comparison with the numerical results

## 2. NUMERICAL ANALYSIS

In the software package SAP2000 seven composite symmetrical castellated beams are modelled using finite elements for the steel and concrete section and link elements for the shear connectors. All the models are geometrically identical, except for the number of shear connectors. The geometric recommendations (empirical relations) for balanced failure between modes that are given in literature are taken in consideration while defining the geometry.

### 2.1 MATHEMATICAL MODEL

IPE550 is adopted for the base steel beam and it's cut in a pattern that a castellated beam with full height of 750 mm are made.

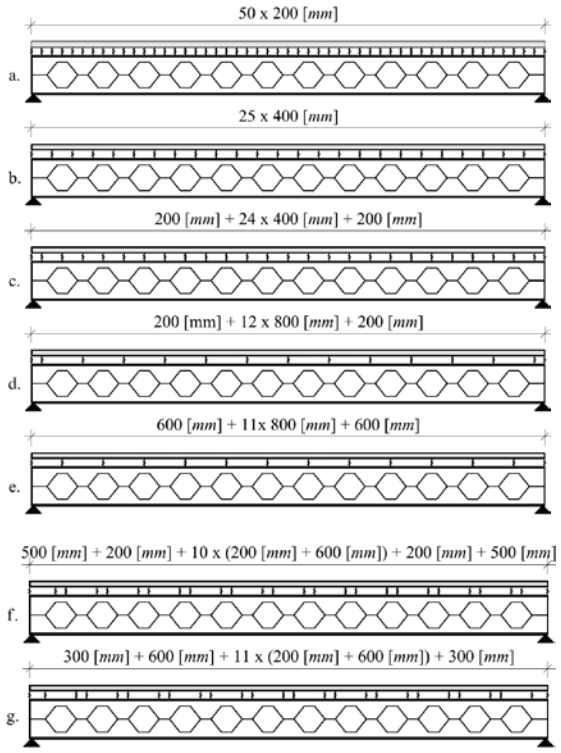


Figure 3. Seven composite castellated beams with different number of shear connections – links (52, 26 and 15 links)

The height ($h_0$) and width of the openings (2xb+e) are taken as 250 mm and 2x200+200 = 600 mm, while the width ($b_{sl}$) and thickness ($h_{sl}$) of the slab are 260 mm and 62 mm. The height of the shear connectors is 86 mm in length and the same are modelled as link elements with stiffness 100 kN/mm, meaning all the models have full composite action. The beams are modeled as simple supported spanning 10 m (Figure 3). For the steel S235 grade is used, while for the concrete slab class C30/37 with σ-ε curve according EN for nonlinear analysis.

The concrete slab and steel beam are modeled with 2D shell elements with discretization of 50x6500 mm and 50x50 mm (Figure 4). The discretization for the steel beam is made denser because the main goal of this research is the effect of composite action on the structural behavior of the steel beam. Because of the high concentration of stress in the corners and the openings in the web, despite the ratio h/L, *thick shell* elements are used for the web, as for the flanges and concrete slab *thin shell* elements. For the shear connectors *linear type link* is used with stiffness 100 kN/mm in U1 and U2 direction and fixed in U3 direction.

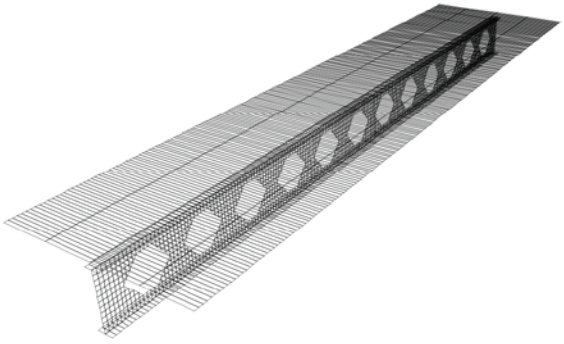

Figure 4. 3D model of composite castellated beam.

To simulate simple support, restrains with free rotation in all directions are assigned to the end joints in line with the beam axis for the non-composite beam, while for the composite beam additional restrains are assigned on the end joints at the support of the slab.

According the analytical calculation, based on the strut model method for web-post buckling analysis, it is expected the castellated beam to first lose its stability under loading of 15 kN/m$^2$, while the other given methods in literature, Aglan and Redwood (1974) [7] and Wang (2016) [9], compute 24 kN/m$^2$ and 23 kN/m$^2$ critical loading. That is why it is decided to load the slab with uniform load from 1.0 kN/m$^2$ to 15 kN/m$^2$.

## 2.2 PROCESSING

Two types of analysis are made: (1) nonlinear static analysis with geometric imperfection (P-δ) – Newton Raphson method; (2) buckling analysis with 6 modes. The fist type of analysis is used because of the high stress concentration in the corners of the openings and the additional generated moment in the link elements from the force transfer while the beam is deformed. With this we simulate the Vierendeel bending (figure 2).

## 2.3 POSTPROCESSING

In table 1. the maximum loading and internal forces are given for web-post buckling failure. The critical web-post is the one closest to the support where the shear forces are largest.

For representing the stress flow along the beam, normal stresses S11 are shown in the following graphical figures along lines 1-1 and 2-2 as given in figure 5. Beside the graphical figures, additional tables are presented, where table 2 correlate the increase and decrease of stresses along the first opening between the non-composite and composite models, given in percentage; and table 3 presents the ratio between the stresses in the beginning and end of the opening for normal and shear stress. In addition to this, a graphical presentation is given in figure 9 and figure 10 for the effect of composite action on the distribution of the maximal $S_{max}$ and minimal $S_{min}$ normal stresses along the beam (half span), as of shear stress.

Table 1. Maximum loading and internal forces during failure from local stability of web-post with double curvature

| | n | q [kN/m'] | [%] | $V_q$ [kN] | $M_q$ [kNm] |
|---|---|---|---|---|---|
| Castellated beam | 0 | 96.26 | 100 | 385.06 | 433.19 |
| Composite a) | 52 | 115.18 | 120 | 460.71 | 518.30 |
| Composite b) | 26 | 106.61 | 111 | 426.45 | 479.75 |
| Composite c) | 26 | 109.04 | 113 | 436.18 | 490.70 |
| Composite d) | 15 | 105.57 | 110 | 422.27 | 475.05 |
| Composite e) | 15 | 105.85 | 110 | 423.40 | 476.33 |
| Composite f) | 26 | 108.56 | 113 | 434.24 | 488.52 |
| Composite g) | 26 | 108.71 | 113 | 434.82 | 489.18 |

n – number of links
q – uniform linear loading

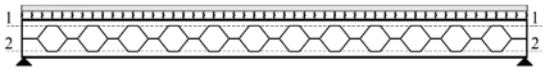


Figure 5. Line 1-1 and 2-2 along the top and bottom of the openings for presenting the stress flow

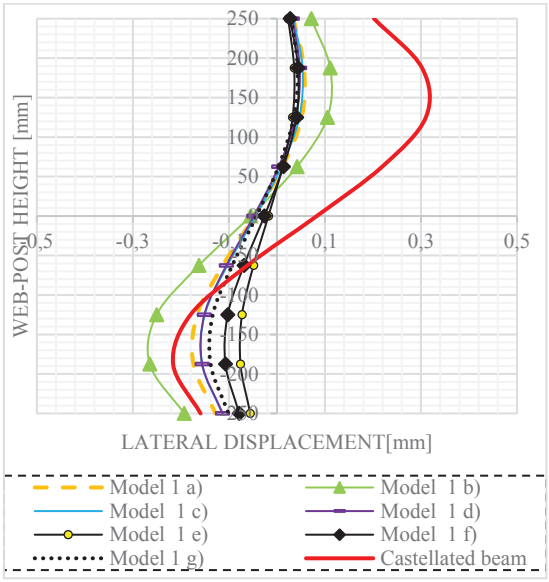


Figure 6. Web-post buckling of first web-post under loading of 15 kN/m$^2$

Table 2. Difference in normal S22 and shear S12 stress between non-composite and composite castellated beam for top tee

| $l_e$, [mm] | 500* | 550 | 600 | 650 | 700* |
|---|---|---|---|---|---|
| | % | S11 | | | |
| a) | -31.10 | -43.01 | -126.9 | 69.89 | 24.56 |
| b) | -30.71 | -42.80 | -121.5 | 57.71 | 15.88 |
| c) | -25.38 | -34.68 | -98.73 | 53.36 | 17.82 |
| d) | -21.64 | -27.58 | -65.17 | 19.62 | -1.28 |
| e) | -14.20 | -20.26 | -60.50 | 33.50 | 11.46 |
| f) | -26.25 | -36.32 | -117.1 | 78.72 | 28.92 |
| g) | -32.26 | -43.30 | -115.8 | 50.87 | 10.49 |
| | % | S12 | | | |
| a) | -18.18 | -12.61 | -15.79 | 1.00 | -7.76 |
| b) | -30.71 | -42.80 | -121.5 | 57.71 | 15.88 |
| c) | -19.29 | -9.57 | -13.92 | 1.81 | -11.1 |
| d) | -11.83 | -14.92 | -13.51 | -12.7 | -15.5 |
| e) | -12.49 | -3.00 | -7.62 | 3.76 | -12.4 |
| f) | -17.64 | -12.44 | -8.96 | 10.49 | 0.17 |
| g) | -16.13 | -18.51 | -15.34 | -13.0 | -14.1 |

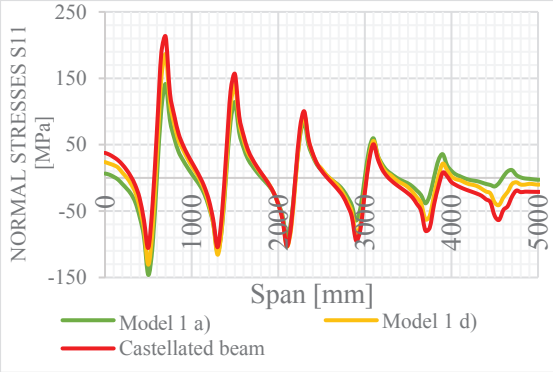


Figure 7. Normal stresses of top tee section along line 1-1 for loading of 15 kN/m$^2$

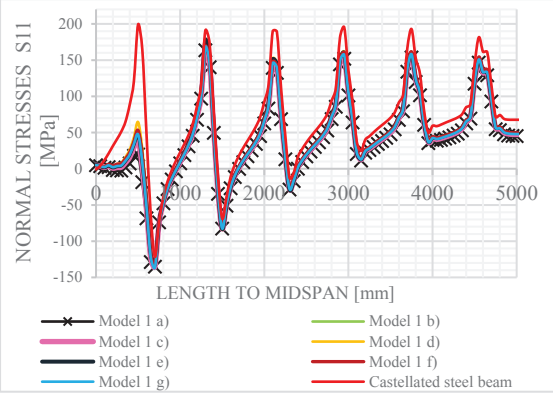


Figure 8. Normal stresses of bottom tee section along line 2-2 for loading of 15 kN/m$^2$

Table 3. Percentage ratio for normal $S11^1/S11^2$ and shear $S12^1/S12^2$ stress between the beginning (1) and end (2) corners of the opening and ratio between the top and bottom corners $S11^{top}/S11^{bot}$

| | $S11^1/S11^2$ | $S12^1/S12^2$ | $S11^{top}/S11^{bot}$ | |
|---|---|---|---|---|
| | | | 1 | 2 |
| | % | | | |
| Castell. | -1.64 | 1.22 | -0.99 | -0.75 |
| a) | -2.85 | 1.33 | -13.24 | -0.67 |
| b) | -2.55 | 1.23 | -6.15 | -0.76 |
| c) | -2.50 | 1.31 | -6.32 | -0.73 |
| d) | -1.97 | 1.18 | -3.74 | -0.91 |
| e) | -2.12 | 1.22 | -4.64 | -0.81 |
| f) | -2.92 | 1.43 | -4.73 | -0.64 |
| g) | -2.43 | 1.24 | -5.54 | -0.79 |

## 3. RESULTS AND DISCUSSION

Based on the previous given figures we can see that the increase of shear connectors increases the normal stresses at the beginning of the first opening for around 30 % for model a) (52 links) and 14 % for model e) with one link over one opening. At the end of the opening, model f) with two links over opening has the greatest reduction of normal stress – 29 %. Based on table 2 we can clearly give a conclusion that for composite castellated beams the best option is to distribute less shear connectors with higher bearing resistance than more shear connectors with lower resistance. Also, the models with shear connectors above the opening give around 5 % smaller normal stress and 1 % higher shear stress in the corners than the models with connectors above web-posts.

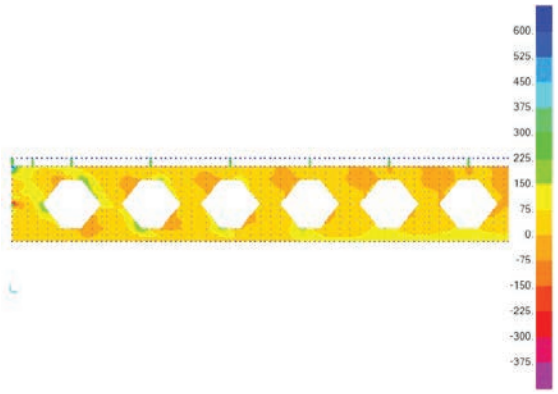


Figure 9. Maximum normal stress $S_{max}$ of composite beam model e) for loading of 15 kN/m$^2$ , MPa

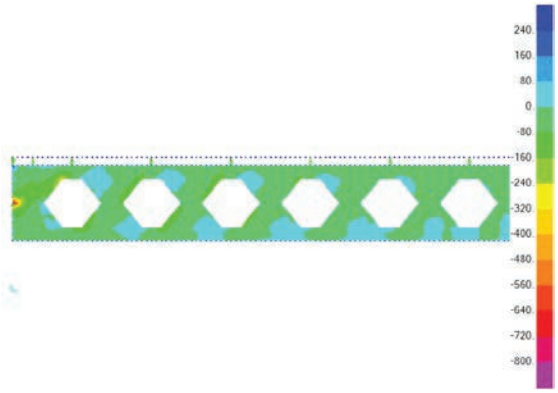


Figure 10. Minimal normal stress $S_{min}$ of composite beam model e) for loading of 15 kN/m$^2$ , MPa

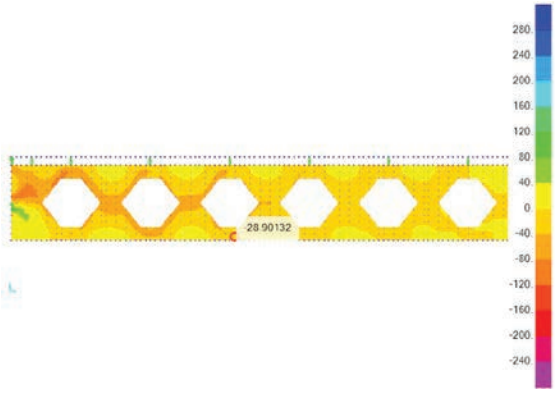


Figure 11. Shear stress $S_{12}$ of composite beam model e) for loading of 15 kN/m$^2$, MPa

Another thing to point is the significant difference between the stresses at the beginning and the end of the opening, Table 3 presents that ratio and we can observe that the difference is less for composite beam with 15 links than those of 52 and 26 links. The same goes for the ratio between the stresses of upper and lower Tee section. We can see that the composite action initiates redistribution of the internal forces, resulting in a more stressed upper Tee section. From this fact we can assume that using asymmetrical section where the lower Tee has higher moment of inertia than the upper Tee section, we can facilitate stress reduction in the upper part.

Beside the given above, the increase of shear connectors improves the stability of the web-post, but according to previous research, this failure mode is less likely to occur if the geometry is properly defined in accordance with suggested empirical relations for balanced failure between modes. For 52 links the stability is increased by 20 %, while for 15 links by 10 %.

## 4. CONCLUSION

Based on the numerical results we can conclude the following:

(1) Using less shear connectors with higher bearing resistance is more optimal rather than more shear connectors with lower resistance for obtaining full composite action, which results in lesser stress accumulated around the first opening;
(2) Increasing the number of shear connectors may improve the stability of the web-post against buckling;
(3) The most favorable position of the shear connectors based on the normal stress is above the opening where there is 5 % increase of resistance against the same number of shear connectors with a position over the web-post;
(4) Based on the analytical relations and finite element analysis, the most probable optimal design of a castellated beam with composite action is an asymmetric castellated beam that can be formed with larger lower Tee section, or with asymmetrical position of the openings (toward the concrete slab) along the axis.

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# PULL-OUT TEST FOR MECHANICAL ANCHORS

In this paper, test procedure and results obtained from experimental research for tension loaded post - installed mechanical anchors are presented. Before conducting the test procedure, additional material for expected type of failures is analyzed according to available literature and standards for calculation of concrete and steel structures, for further analysis of obtained results for the behavior of this type of anchors installed in non-reinforced concrete. Non – standard test of post – installed mechanical anchors loaded with tension force is conducted in non- reinforced concrete, where anchors are embedded according to rules and recommendations given from anchor manufacturer. Results from conducted tests are given in form of load-displacement diagram curves which defines the type of anchor failure. Obtained results are basis for further experimental researches.

Keywords: post-installed anchors, pull–out test, mechanical anchors, load-displacement

## 1. INTRODUCTION

Design of construction and construction elements have to comply with the current norms and standards. In case when the design of structure or detail cannot be in accordance to national norms and standards, then technical approvals should be used. The technical approvals are based on results from qualifications tests conducted by independent laboratories. Modern technology for fastenings is more applied worldwide and every type of anchor is designed for use in special (individual) conditions, such as different type of concrete, load, frequency of load, position of installment etc. If the anchor is not properly installed in conditions which is suitable for, then the safety of the detail is suspected, even the safety of the whole structure. Due to the complexity and diversity of post – installed anchors, all attempts to standardize anchor products have failed.

In the countries in the European Union, calculation of load capacity of anchors is carried out according to Annex C of ETAG001 (Guidelines for European Technical Approval of Metal Anchors for use in concrete) until implementation of CEN/TS 1992-4 (2009) as part of Eurocode 2 (2005) – Part 4. In United

States calculation for load capacity of anchors is regulated from 2002 in Appendix D from ACI318 (2002), revised and supplemented in 2011. Recommendations given in guidelines strictly defines the conditions and the manner in which tests for approval of anchors will be carried out.

Test procedure of anchors can be divided into three types:

- Conformity test for checking the behavior of anchors in extreme conditions
- Test for determination of the load capacity of anchor in exploitation phase
- Test for determination of displacement of the anchor under external loads in exploitation phase

Results from this type of tests represents the behavior of each anchor as well as the type of failure.

In this paper the test procedure and the results obtained from non - standard experimental research of anchors are given. Analyzed anchors are mechanical post – installed anchors which are embedded into non – reinforced concrete blocks and loaded with tension, thus simulating the most unfavorable type of installation and load of anchors.

## 2. EXPECTED BEHAVIOR AND TYPES OF FAILURE FOR ANCHORS LOADED WITH TENSION FORCE

For post – installed anchors loaded with tension force, four types of failure modes are expected and all of the failure modes are characterized with different load – displacement curve in function of different types of factors.

Expected types of failures under external tension load are:

- Concrete cone failure
- Splitting failure
- Pull – out and pull – through failure
- Steel failure

Apart from the type of the anchor, the failure and behavior also depend on installation torque, concrete class, cracked or non- cracked concrete and the way force is applied. Total measured displacement (extraction) of the anchor is compiled of the slip of the anchor, local deformation in concrete zone where the transfer of the friction load occurs, and the deformation of the anchor itself.

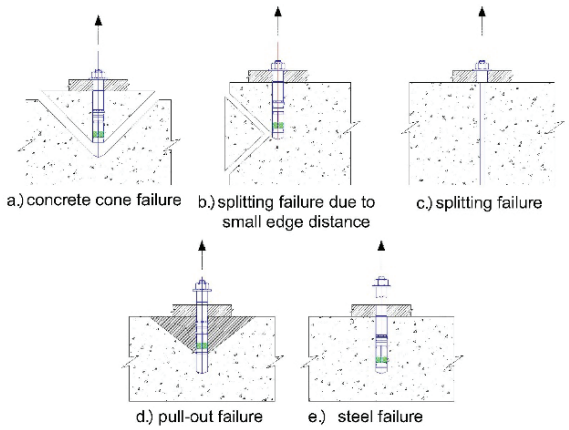


Figure 1. Types of failure due to tension load

Every type of failure occurs after characteristic tension force is reached. Concrete cone failure occurs when full tensile capacity of the concrete is utilised and is characterised by the formation of a “cone” fracture surface in the concrete. This is case when expansion force exceeds or do not fail under force, which is characteristic for pull – through or steel failure. Splitting failure is characteristic failure when anchor is installed near the edge of the concrete embedment or when dimensions concrete cone is limited. When anchor is pulled out of the drilled hole, then pull – out or pull-through failure occurs. In this case anchor may not be damaged and vertical displacement of the anchor is the only parameter which may state to this failure. Pull-through and steel failures are not allowed to happen in some of the approval tests.

## 3. TEST PREPARATION

### 3.1 PREPARATION OF CONCRETE BLOCKS FOR INSTALLATION OF ANCHORS

As mentioned before the pull-out test conducted for mechanical anchors embedded in non-reinforced concrete, presented in this paper, is not standard procedure, which differs from procedures stated in ETAG. Dimensions of concrete blocks for installation of anchors are dictated from minimal conditions prescribed by the manufacturer for each anchor. After analysis of the parameters for each anchor dimensions of concrete blocks are defined. Two types of concrete block with different dimension were made for the experimental research.

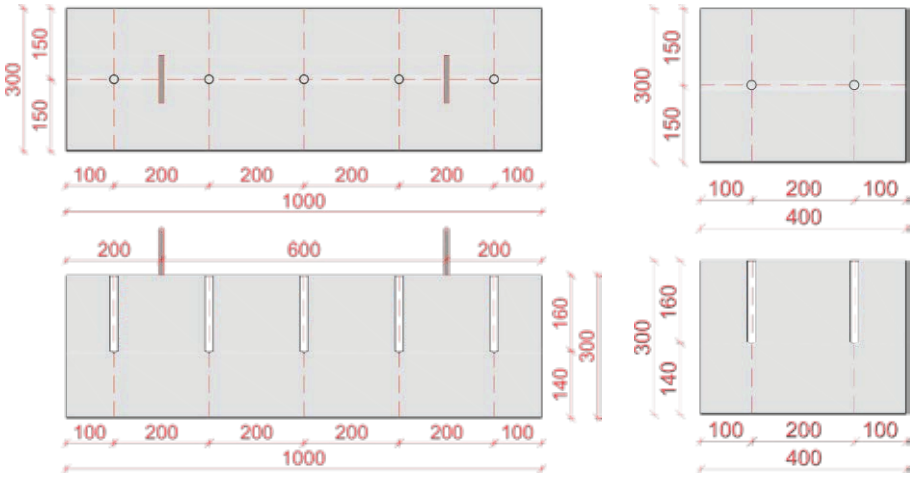


Figure 2. Concrete anchor block type 1 (left) and type 2 (right)

## 3.2 QUALITY CONTROL OF CONCRETE

To control the quality of concrete from which anchor blocks are made, samples from concrete in phase of concreting are taken for further laboratory tests. Samples are taken for cubes with standard dimensions 15/15/15cm in accordance with EN12390-1 (Testing hardened concrete – Part1: Shape, dimensions and other requirements for specimens and molds). According to recommendations, 6 samples are taken in phase of concreting. Samples were stored in controlled laboratory conditions in accordance with EN12930-2 (Testing hardened concrete – Part2: making and curing specimens for strength tests). To determine the class of concrete and compressive strength, 3 cubes were tested after 28 days from concreting the anchor blocks. Obtained results from concrete tests are processed in accordance with EN 13791. From measured results concrete class C25/30 was achieved.

## 3.3 PREPARATION OF CONCRETE BLOCKS FOR DRILLING, INSTALLATION OF ANCHORS AND EQUIPMENT FOR MEASURING TEST RESULTS

In the concrete blocks 8 different types of anchors from 5 different manufacturers were installed. Anchors were divided into group of 5,3 and 2 anchors per block, respecting the installation recommendations.

All parameters for installation of anchors are previously measured and approved. After drilling of the holes, before installation of the anchors, every hole was cleaned from drilling dust.

The electronic equipment is connected to HBM Quantum data acquisition system amplifier, directly connected to computer, with measuring in real time. Results are given in form of vertical deformation of anchor (extraction) in proportion to the applied tension force. Scheme for installation of measuring equipment is shown in Figure 5.

Figure 3. Anchor block type 1

Figure 4. Anchor block type 2

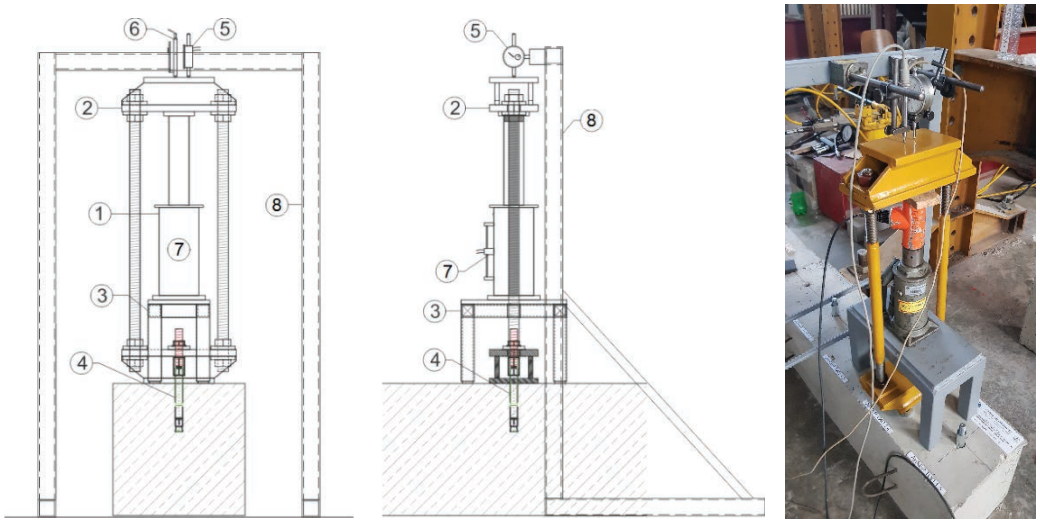


Figure 5. Scheme for installation of the equipment

Where,

1) Hydraulic press
2) Tension load construction
3) Concrete cone support construction
4) Installed mechanical anchor
5) Electronic deformeter (comparator)
6) Inductive deformeter (comparator)
7) Dynamometer
8) Support for deformeters (comparators)

All of the selected anchors were with similar installation procedure, steel quality, equal diameter and same thread M16, but with different load capacity defined by the manufacturer for the same or similar conditions.

Tension force for the anchor was applied through special designed system with hydraulic press positioned on steel construction which does not affect the concrete cone failure. Application of tension load on anchors was continuously monitored through the electronic dynamometer. Installed anchors were loaded in two phases. In the first phase anchors were loaded to maximum force of 50 kN, defined as the maximal load with excluded safety factor. For the second phase, the anchors were loaded until failure occurs. Measurement of vertical displacement of the anchors was carried out by two positioned deformeters (electronic and inductive).

## 4. RESULTS FROM CONDUCTED PULL-OUT TEST FOR MECHANICAL ANCHORS

With the conducted experimental tests of the mechanical anchors installed in non – reinforced concrete blocks, the obtained data was used for the analysis of the behaviour of the anchors. All gathered data was processed in form of load – displacement diagrams. The following are the results of phase 2 for each anchor group from the experimental research. In this phase the anchors were loaded until the failure occurs (maximum force of 160 kN).

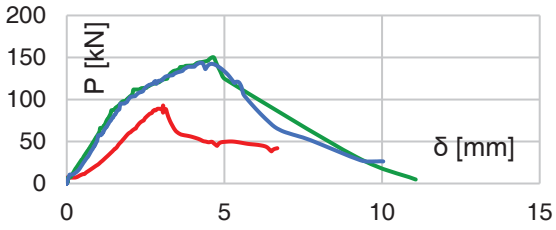


Figure 6. P-δ diagram for anchor type 1

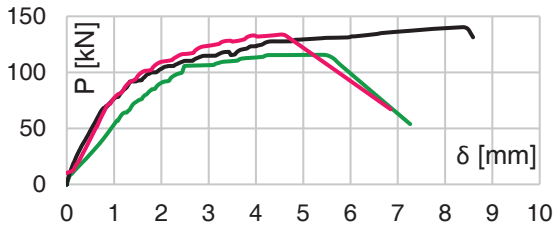


Figure 7. P-δ diagram for anchor type 2

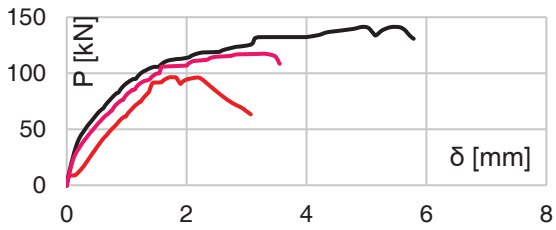


Figure 8. P-δ diagram for anchor type 3

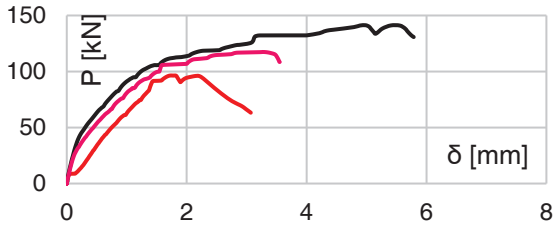


Figure 9. P-δ diagram for anchor type 4

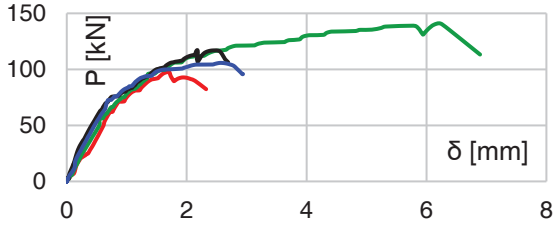


Figure 10. P-δ diagram for anchor type 5

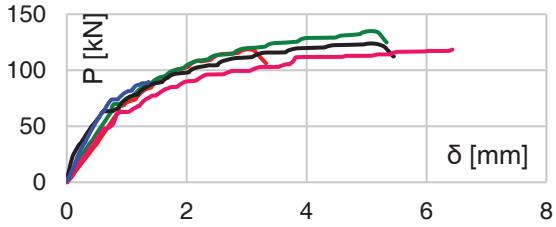


Figure 11. P-δ diagram for anchor type 6

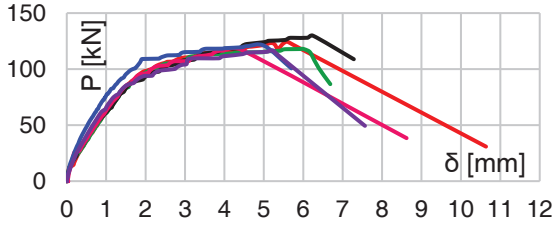


Figure 12. P-δ diagram for anchor type 7

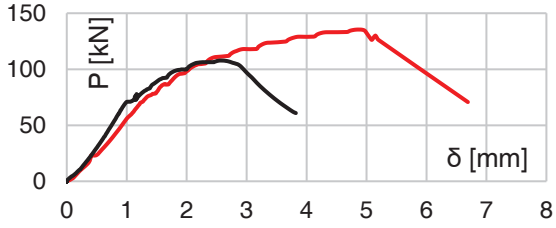


Figure 13. P-δ diagram for anchor type 8

In all cases of the tested anchors, the failure was splitting of the concrete block. The maximum measured slip, for all the types of mechanical anchors, was between 5.5 to 10.6 mm, as can be seen in the diagrams above.

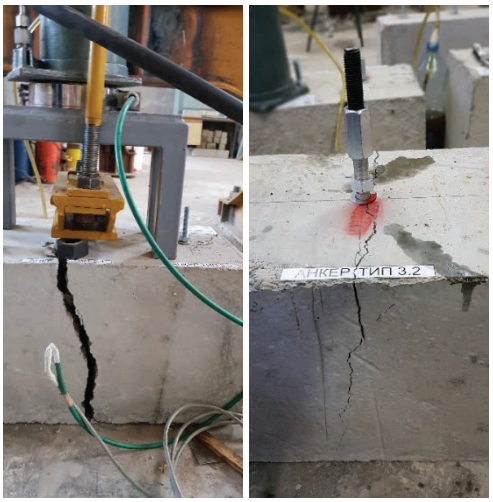


Figure 14. Failure of anchor type 2 and 3

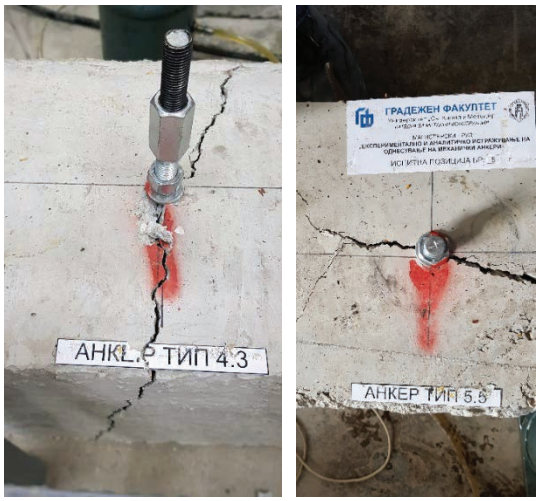


Figure 15. Failure of anchor type 4 and 5

Figure 16. Failure of anchor type 7 and 8

## 5. CONCLUSION

From the occured data for this experimental research can be concluded that the behavior of the mechnanical anchors depends on many different factors, which always vary for each situation.

By obtaining the results and creating the load – displacement diagrams, the behavior of the mechanical anchors as elements for connection gives opportunity for ease of analayzis. Conclusions from conducted pull-out test of mechanical anchors are that:

- For mechanical anchors loaded with tension force and installed in concrete elements with minimal allowed dimensions, the failure is always followed with splitting of concrete. This is a result from progressive spread of cracks in the concrete at the zone of load transfer between concrete and anchors mechanism. In this case spreading of cracks in concrete is not limited with reinforcement.

- With the right detailing of reinforcement around anchoring zone, progressive spread of cracks can be prevented and load capacity of the whole detail with anchoring will be significantly increased. In that case different type of failure is expected.

- According to load – displacement diagrams, ductility of this type of failure is satisfactory and meets the minimum requirements for construction connections.

- With further analysis of the results and comparison with the given design and allowed tension loads from the manufacturers for each type anchor, can be concluded that maximum bearing capacity of the anchors is underestimated. This is a result, as previously said, from many different factors which are not always known and from which depends the behavior of the anchors.

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# AUTOMATIC CLASSIFICATION OF LAND COVER USING MULTISPECTRAL SATELLITE IMAGERY: VALLEY OF RIVER TRESKA, MACEDONIA

The Earth's surface is a fundamental variable that connects many parts of the global environment. Therefore, the existence and distribution of different land cover types must be understood in order to categorize and predict their potential impact on the environment. For that purpose, this paper applies the unsupervised automatic classification and use of the Iterative Self-Organizing DAT Analysis (ISODATA) method to detect land cover types, based on multispectral satellite image from Landsat-8 from 2018 for the valley of river Treska in Republic of Macedonia. The final product is a land cover classification map with four classes: Water bodies, Forests, Shrubland and Herbaceous vegetation.

The general aim of this paper is to understand and emphasize the importance of land cover allocation, which is important for maintaining the daily flow of living, without having to perform terrestrial measurements (which reduces the time and costs required for acquisition), using geospatial tools and remote sensing methods to categorize the land cover classes.

Keywords: remote sensing, geospatial tools, land cover types, unsupervised classification, classification map.

## 1. INTRODUCTION

Land cover classification is a major issue of concern given the fact that it is subject to rapid changes caused by various socio-economic activities and natural phenomena. Rapid growth and expansion of urban centres, rapid population growth, land scarcity, the need for additional arable land, advancing technologies are among the many drivers of the changes in land use in the world today.

Land cover dynamics is accelerating and is primarily driven by human activities, but also

causes changes that affect the humanity. In this regard, land cover classification is key to analysing the environmental processes and problems and maintaining and improving the living conditions and standards.

As early as the 1970s, with the launch of the Landsat program, remote sensing methods have been providing an economical and sustainable approach to the challenges of monitoring land cover in distant, mountainous and inaccessible areas, compared to field measurements. Considering the fact that without spatial data and satellite imagery the monitoring of dynamic changes occurring globally and/or regionally is a challenging task, remote sensed products also provide comprehensive details needed to effectively study the various characteristics of the land's surface.

## 2. STUDY AREA

The scene of interest covers an area of 2.214 km$^2$ and extends along the flow and the entire valley of the river Treska (along with the artificial lake Kozjak) on both sides.

This particular area has been chosen for classification because different land cover types (water bodies, forests, vegetation) can be categorized and a distinction can be made between different types of land cover in mountainous areas, which is the aim of this paper.

Figure 1. Location of the study area (red square)

### 2.1 BAND COMBINATION

In this paper, where different types of land cover are examined and a clear distinction between vegetation areas should be made, the band combination 7,4,2 (infrared, red, blue) has been chosen in which the vegetation appears in shades of red, urban areas are turquoise and the soil varies from dark to light brown. Ice, snow and clouds are white or very light turquoise, and the water surfaces are dark blue. In general, darker shades of red indicate broad leaves and/or healthier vegetation, while lighter red indicates pastures or sparsely vegetated areas (Figure 2).

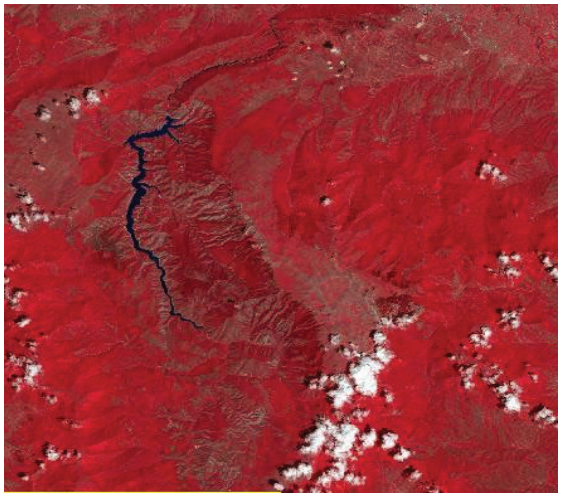

Figure 2.The study area in combination of spectral bands 7,4,2

## 3. RESULTS AND DISCUSSIONS

### 3.1 DESCRIPTION OF LAND COVER CLASSES

In order to classify the different types of land cover for the area of interest, four classes have been differentiated: 1) Water bodies, 2) Forests, 3) Shrubland and 4) Herbaceous vegetation.

Table 1. Description of land cover classes

| Class number | Class name | Class description |
|---|---|---|
| 1 | Water bodies | Areas covered with open waters, rivers and lakes. |
| 2 | Forests | Areas covered with evergreen and deciduous trees (>5 meters high) at an altitude higher than 850m. |
| 3 | Shrubland | Areas with high percentage of shrubby grass cover not higher than 5 meters. |
| 4 | Herbaceous vegetation | Areas dominated by grass (0-02 m) and plants (0.2–2 m), and completely non-vegetative areas or areas with a very small percentage of vegetation cover (bare soils and rocks). |

## 3.2 SPECTRAL DISTINCTIONAMONG THE CLASSES

Spectral distinction is essential to determine whether the classes are different in their spectral signatures and to what extent they can be spectrally differentiated, as their clear distinction is the main determinant in pixel clustering when conducting an automatic classification.

The most effective way to perceive the spectral distinction between the classes is based on a graph. Figure 3 shows three graphs, in all combinations of the three spectral bands. The first graph shows band 1 versus band 2, that is infrared and red light with all four classes displayed as a point cloud whose pixel count corresponds to the number of pixels in the selected polygon for that class, with the corresponding colour. The following visual presentation combines the infrared and blue light bands, while the last graph refers to the red and blue part of the electromagnetic spectrum. The values of the digital number are given on the x and y axes.

In the first and second combination of spectral bands we notice a high spectral distinction of the classes, there is no overlap of the point clouds - they are not even touching. However, there is an overlap of some of the pixels that belong to the water bodies with forests in the third combination, but summed up it seems that all four classes are spectrally different which is important when assigning the pixels in the corresponding class and implementing a quality automatic classification.

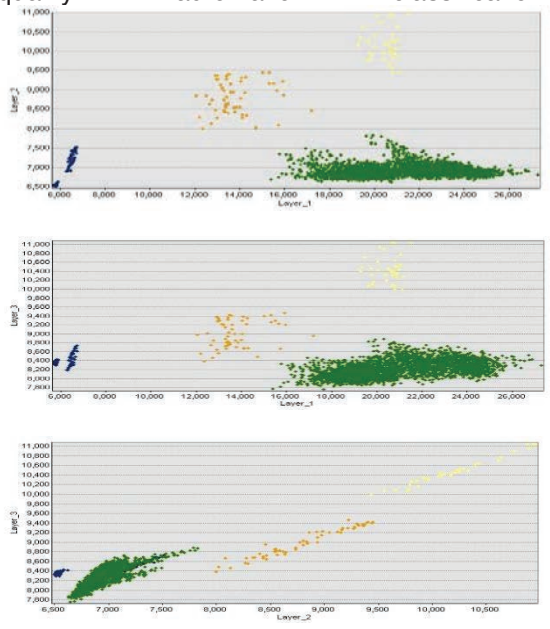

Figure 3. Spectral distinction among the classes

## 3.3 DESCRIPTION AND OUTPUT OF THE UNSUPERVISED CLASSIFICATION

The application of unsupervised classification involves grouping of spectral data, based solely on their statistical structure. Each statistical cluster must be marked, placing it in the appropriate category. The software determines the spectral signature of the pixels based on the statistical structure within each area and uses this information to define the mean value and variance for each of the classes. In this case, the grouping of each pixel of the scene in one of the four classes was performed according to the modified iterative optimization – ISODATA using the Nearest Neighbor algorithm.

But, as it can be seen in Figure 2, condensed masses of water vapor can be identified at the ends of the scene. There is a possibility of their removal from the scene, but still the effects caused by their presence, i.e. the shadows affecting the spectral structure cannot be corrected. In this case, it wasn't approached to their removal, because based on the map of Macedonia from Google Earth and the thematic map of land cover for the country by CORINE Land Cover, it has been determined that the shadows mostly correspond to the class no.3 –shrubland, while clouds generally correspond to herbaceous vegetation.

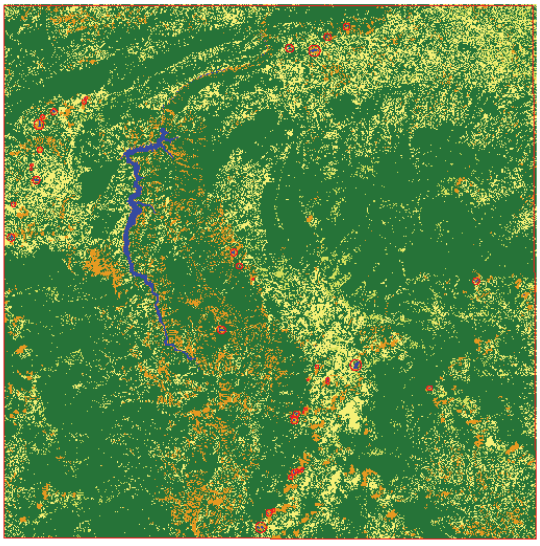

Figure 4. Location and size of incorrectly classified pixels (red circles)

Although this solves the problem of detecting the land cover under the clouds, still a small part of the shadows caused by them are classified in the class - water bodies because they have digital number values that are within

the range of that class. In fact, it will be a matter of incorrectly classified pixels which, instead in class no.1, should belong to class no.3 (Figure 4). However, that is a very small area which participates insignificantly in the area representation of the classes and in the visual presentation and preservation of the recognizability of the details on the scene itself.

Thus, we come to the final product of the unsupervised classification, and that is the thematic map of land cover in a scale of 1: 250 000 (Figure 5).

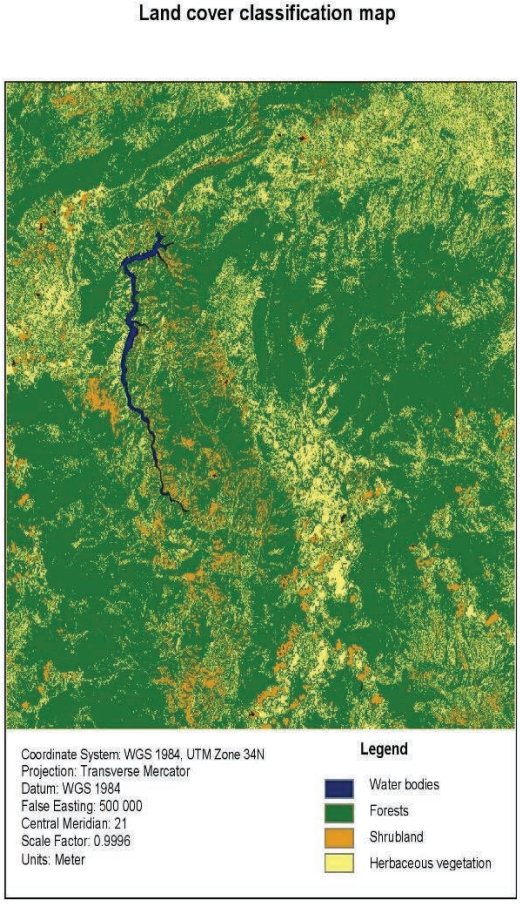


Figure 5. Map of land cover classification

## 3.4 RESULTS OF THE CLASSIFICATION

Based on the number of pixels in each of the classes and having in mind that the spatial resolution of the satellite image is 30 meters, the area and percentage representation of the classes on the scene are calculated (Table 2).

Table 2. Area coverage of the classes by square kilometres and percentage

| No. Class | Class name | Number of pixels | Area ($km^2$) | Per. (%) |
|---|---|---|---|---|
| 1 | Water bodies | 15.032 | 13,5 | 0,6 |
| 2 | Forests | 1.886.832 | 1.698,2 | 76,7 |
| 3 | Shrubland | 143.103 | 128,8 | 5,8 |
| 4 | Herbaceous vegetation | 415.219 | 373,7 | 16,9 |
| | Sum: | 2.317.329 | 2.214,2 | 100 |

From the obtained results it can be concluded that the biggest representation has the class forests that occupies more than two thirds of the scene-76.7%, followed by herbaceous vegetation with almost 17%, then shrubland with slightly less than 6% and the lowest percentage belongs to water bodies - 0.6%, which is to be expected considering that the relief of the whole area of interest is a valley, surrounded on all sides by mountains (Figure 6).

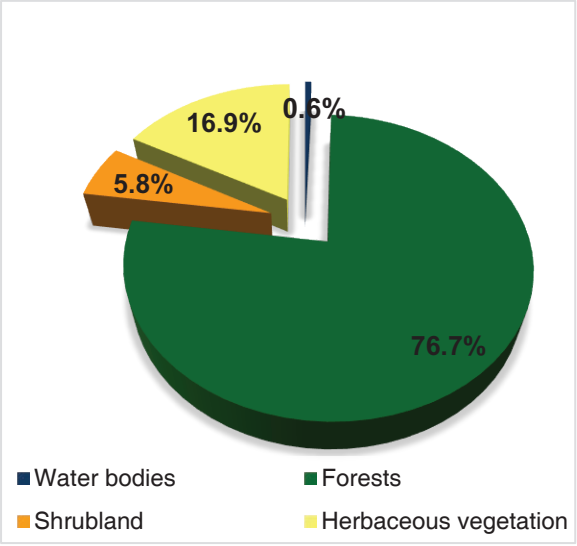


Figure 6. Percentage representation of land cover classes

## 4. CONCLUSION

Considering that Earth's surface is undergoing rapid changes in land use due to various socio-economic activities and natural phenomena, it is extremely important to demonstrate the significance of its spatial distribution. In addition, commercial satellites now provide a large amount of remote sensed data in terms of spatial, spectral, and temporal resolutions that can reveal the natural and artificial characteristics of spatial entities in general.

However, before moving on to the classification itself, analyzes were performed to determine the spectral distinction of the classes. The spectral distinction of the classes is important so that it can be ascertained to what extent classes can be spectrally differentiated. Based on the graphical representations, a high spectral distinction was established, which is essential in the assignment of pixels in the appropriate class and the implementation of a quality automatic classification.

Next, a thematic map of land cover was created and the area and percentage representation for each of the classes were calculated. The largest area of the map is occupied by forests - 77%, followed by herbaceous vegetation with 17%, then shrubland - 6% and the least represented are water bodies with less than 1%.

In summary, the study reveals that the spatial distribution of land cover is the basis for establishing a successful land use strategy in all spatial and temporal dimensions. Updated information on land change with the help of remote detection products are useful for the development of human society.

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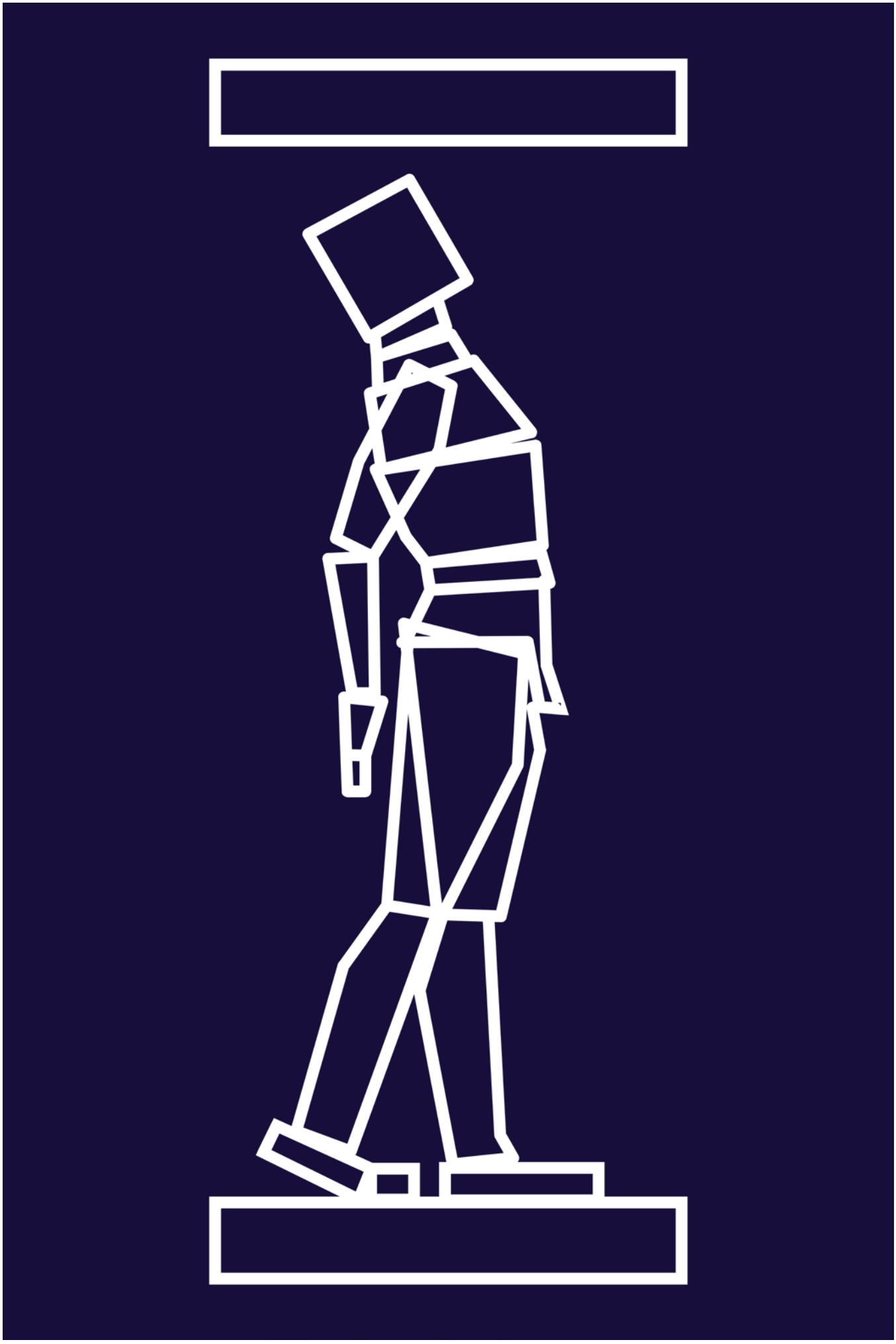

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# DYNAMIC ANALYSIS OF BELL TOWER WITH BELL FORCES

Bell towers are structures dating a few thousand years ago while their structural behavior is discovered in detail only now in the last few decades. They are monuments of culture and civilization, witnessing the time when they were built but also witnessing the history of the region around them. Old bell towers are built as a masonry structures and as that stand the test of time to this day. The new ones are made of reinforced concrete, steel or combination of both materials.

This research paper presents rational procedure for determination of dynamic characteristics of bell tower in order to have an insight into its true structural behavior and to check occurring of resonance during the ringing of the bells. This methodology was conducted on the bell tower of Cathedral Church „St. Clement of Ohrid" in Skopje.

Keywords: bell tower, swinging bell forces, church bells, resonance, natural frequencies

## 1. INTRODUCTION

It is a known fact that first bells originated in the region of today Armenia and were widely used by the great ancient civilizations like Egypt, China, Greece and Rome. Bells had a great role in the people's everyday life because they were used like instrument for signalization that it is time to pray or for holy rituals like weddings, funerals etc. In Christianity, bells are used since 4$^{th}$ century A.D. and from Italy they were successfully spread to Spain, France and Britain where they started to signal the time too. Over the years, cities grew larger and so did the bells. Hence the need to place the bells on a tower that will be high enough so that the bells can be heard as far away as possible and at the same time to accommodate the bells. This tower is called bell tower and first appeared around 11$^{th}$ century. At first it was built as masonry structure and only from the 20$^{th}$ century bell towers are built as reinforced concrete or steel structures. Around the 15$^{th}$ century bells finally got the characteristic form that is known today [1].

Big bell towers usually have big bells with weight over 1000 kg and are placed on the upper levels which present big concentrated

masses. When bells begin to ring, they swing around their axis of rotation and with that they generate horizontal and vertical dynamic forces which are first transmitted to the bearing sub structure and later to the tower itself.

Towers are cantilever like structures and thus concentrated masses at the end subjected to dynamic loading will cause large horizontal displacements. When it comes to dynamic actions, the displacements can be amplified by the effect of resonance. Basically, if one of the natural frequencies of the tower is close enough to the frequencies of the swinging system than the induced forces by the bells motion will be amplified.

Generally, it is known that the maximum vertical force can reach values up to four times the weight of the bell and the maximum horizontal force up to 2 times the weight of the bell [2]. Because of the structural system of the bell tower, it possesses bigger axial stiffness than lateral or torsional stiffness. Therefore, the vertical forces from the swinging bells are not a problem for the structural behavior and are not taken into account. As for the horizontal forces, they present big challenge for the bell tower and can cause big horizontal displacements which can cause structural damage the bell tower or lead to collapse.

Accurate determination of swinging bell forces is an important step when making an overall structural analysis of a bell tower. These forces are generated due to the mass of the bells or in other words due to their inertia of the ringing system and only through dynamic approach these forces can be accurately obtained.

In the past engineers used empirical formulas to assess the horizontal force induced by the swinging bell. Bell towers older than a century are surely designed with these empirical formulas and nowadays, throughout the world, these towers cannot perform normally because the masonry material had deteriorated over time and the tower can no longer accept dynamic loading from the swinging bells.

Bell towers are usually slender structures and may be highly sensitive to the dynamic actions induced by wind, earthquakes and motion of bells. The dynamic forces caused by the motion of the bells are one of the main actions that influence the structural behavior of bell towers every day. Also, the frequencies of these dynamic forces have important interaction with the natural frequencies of the supporting structure. The knowledge of the dynamic action induced by the movement of the bells is of great importance in the structural safety assessment as well as in the restoration of bell towers[3].

Traditionally it has been considered sufficient to keep odd harmonic oscillations of the ringing bell off the fundamental natural frequencies of the tower. The German standard for bell towers, DIN 4178-2005, explicitly suggest that the difference between these two frequencies should be greater than 20% in order to avoid resonance[4].

## 2. CHARACTERISTICS ABOUT CONSIDERED BELL TOWER

This paper deals with the bell tower of Cathedral church of “St. Clement of Ohrid” in Skopje which is a few decades old. It is fully separated from the church, as shown on Figure 1and Figure 2, it is located in the center of the city and has coordinates of 41.998853, 21.426139. The tower is made out of poured reinforced concrete except for the façade panels which are prefabricated.

The structural system can be seen on Figure 3 and is contained of: beams, slabs, shear walls and cantilever slabs at the top levels. It can be noticed that the structural system has no columns which means that all vertical loading is accepted by the four shear walls. Also, the bell tower has a ground floor, Figure 4, plus ten floors.

Figure 1. Position of the bell tower

Figure 2. View of the bell tower

Figure 3. Structural system

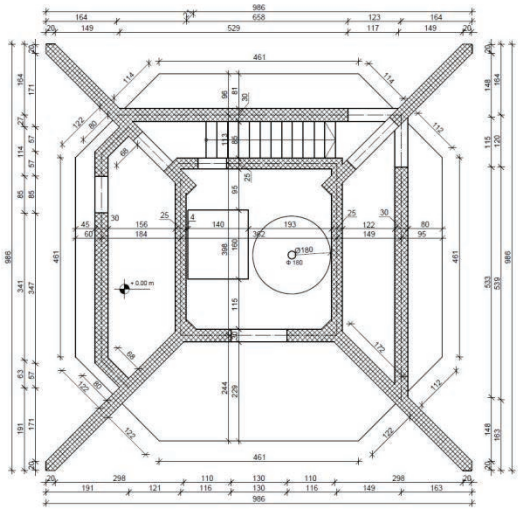

Figure 4. Ground floor layout

Relevant project documentations for the bell tower were not found and therefore it was manually measured by measuring tape and laser. Later, it is planned to conduct measurement with LiDAR method, a geodetic method for measurement that produce high quality point cloud model of the measured object in 3D where all unreachable distances can be measured. Also, with this method the manually measured distances will be additionally confirmed.

This bell tower has three bells located on the top floor and their disposition can be seen on Figure 5. The disposition of the bells has a small plane eccentricity and can cause torsion in the global behavior of the bell tower. These bells are casted in Innsbruck, Austria. In some bell towers where the bells are accommodated at the windows the plane eccentricity is large enough and can cause significant torsion which is not favorable since bell towers have low torsional stiffness. Bell 2 has biggest mass of all other bells and the other two are with smaller masses.

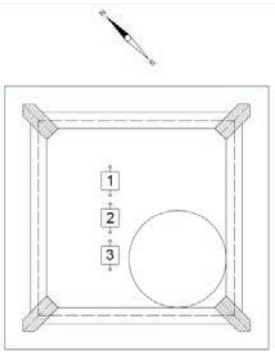


Figure 5. Bells disposition

## 3. MECHANISM OF ROTATING BELLS

Churches can have one or more bells. The bells can be stationary, rung by a mechanical hammer, with swinging clapper or they can rotate around axis. This depends by the size of the bell itself.

The mechanism of rotating bells consists of an electric motor, a chain and a wheel. The electric motor generates torque which is then transmitted on the wheel through the chain. The axis of the bell is fixed on the wheel which by spinning rotates the bell. This system is shown on Figure 6

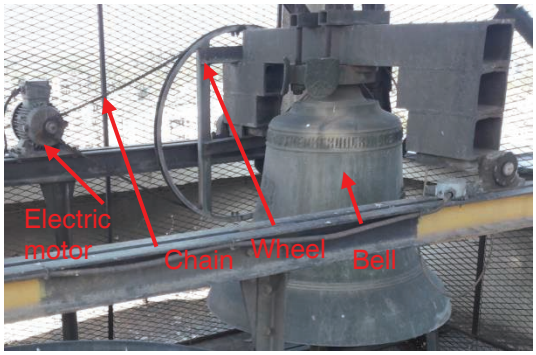


Figure 6. Rotating system

As far as dynamic characteristics are considered, the most important characteristics for the ringing bell are:

- Hits per second
- Max. angle of rotation
- Frequency of the rotating system

- Mass inertial moment(with respect to the axis of rotation)

These characteristics directly affect the dynamic behavior of the ringing bell. If it is needed to change some of these characteristics, one can only adjust the electric motor since it will affect the whole ringing system.

## 3.1 BELL'S PARTS

The geometry of the bell has been changing throughout the years. Around the 15th century, the bell gained its definitive shape that is known today. For better understanding and description of the bell, its parts are named as show on Figure 7(*1.bell yoke 2.headstock 3.canons 4.shoulder 5.waist 6.soundbow 7.lip 8.mouth 9.clapper 10.bead line*) [5].

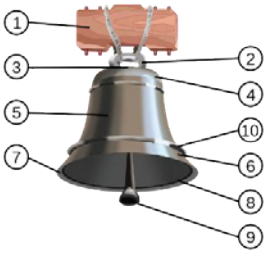


Figure 7. Bell's parts

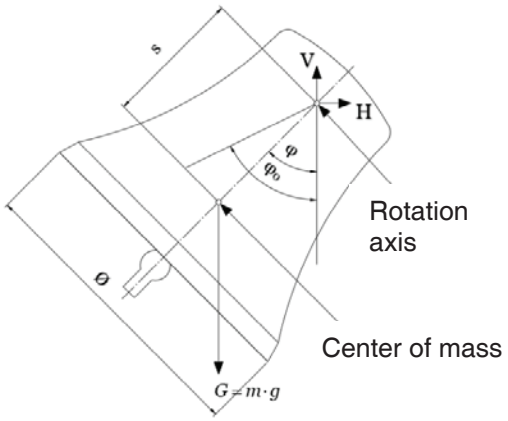


Figure 8. Additional parameters

## 3.2 RINGING BELL CHARACTERISTICS

During the swinging of the bell, it is out of balance and several additional parameters must be observed as shown on Figure 8.

- φ – angle between vertical and bell axis
- $\varphi_0$ – Maximum angle the bell can reach
- s – distance between axis of rotation and the center of mass
- V – vertical force generated from the swinging bell
- H – Horizontal force generated from the swinging bell

While the bell is swinging over time the angle φ changes its values from positive to negative maximum angle. The φ angle directly affect the analytical calculation of the forces H and V and in the meantime this angle depends only on the maximum angle $\varphi_0$. While the bell is swinging the two forces H and V are keeping it in balance while the value of the parameter s shows how well the bell is balanced. Good balanced bells have low value for s which leads to lower mass inertial moment of the bell which finally leads to lower forces generated by the swinging bells.

## 3.3 BELL'S MATERIAL

Throughout the years, bell's material has been changing due to its imperfections. After centuries of experience, innovations and technological advances the bells finally took the shape that is known today. This is due to the fact that a precise mixture of metals for casting church bells has been discovered.

Since the beginning, bells are casted with copper or special alloy of copper, known as bronze. This alloy is precise mixture of 78% copper and 22% tin. Even though the base materials are soft their mixture is pretty solid, little bit elastic, compact, less ductile and resistant to oxidation. The patina layer which forms on the surface of the bell due to oxidation, protects the bell from further oxidation and has green color.

The most important quality of a church bellis to maintain resonance when it is struck and to vibrate and produce the holy om sound. The clapper is usually made of the same material as the bell itself and it can hit the bell with speeds up to 1000 km/h. The specific weight of the bronze is 8900 kg/m$^3$. Knowing this it is easy to determinate the weight of the bell needed for the further calculations.

## 3.4 RINGING SYSTEMS

Nowadays there are three main systems of ringing bells. Local people developed these systems and embedded their culture in these structures.

These systems are:

- Central European system

(Central Europe, USA, Canada, Italy, Eastern Europe and some countries in South America)

- English system

  (Great Brittan, USA, Canada, Australia, New Zeeland, South Africa)

- Spanish system

  (Spain, south France, USA and some countries in South America)

The systems have certain differences between them like: direction of ringing, the maximum angle that the bell can reach and the location of the bell itself. As for the location of the bell, in the Central European system Figure 9 and in the English system Figure 12 the bells are located inside the bell tower at top levels. The bells are supported on separate substructure, traditionally made out of massive wood to dampen the vibrations from the bells, nowadays usually made out of steel, while in the Spanish system Figure 10Figure 11 the bells are hung on a wall or a window of the church.

Figure 9 Bells accommodated inside bell tower (C. European system)

Figure 10 Traditional Spanish bell tower (Spanish system)

Figure 11 Modern bell tower (Spanish system)

Figure 12. English system

The angle between the vertical and bell's axis is constantly changing over time while the bell is rotating. In C. European systems this angle can reach values 30° ÷80°while the bell swings alternately. In the English system the bells make full circle while changing the direction every cycle. The bell begins the cycle turned upside down, makes full circle 360° and returns to the starting position upside down. These two systems are very unbalanced because of the bigger distance from the center of mass of the bell and the axis of rotation. In the Spanish system the bells rotate continuously in one direction with a wooden counter weight placed on top of the bell which balances the system. This means that less force will be needed to rotate the system as opposed to the C. European and English system. Such bell on a bell tower generates smaller horizontal and vertical forces [6], [7].

In Macedonia and the nearby region, church's bells are rung according to C. European system. The bell tower that is considered in this paper has bells that are rotating according to C. European tower.

## 3.5 BELL 2 CHARACTERISTICS

In the considered bell tower only bell 2 is in function and its main characteristics are given in Table 1. According to the conducted measurements the geometry parameters of bell 2 are obtained and with these parameters a 3D model of bell 2 is made in AutoCAD shown in Figure 13 and Figure 14. The bell is made out of bronze.

Figure 13. 3D model of bell 2

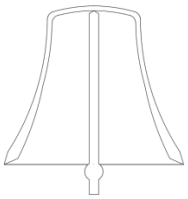

Figure 14. Cross section of bell 2

From the 3D model of the bell it is easy to determinate its center of mass and the position of the rotating axis. From these two parameters the value of "s" is obtained. The value of mass inertial moment is calculated in Eq. (1).

$$I = m \cdot s^2 = 978 \cdot 0.3772^2 = \\ = 139.16\ kg \cdot m^2 \quad (1)$$

The oscillation frequency range of normal bells is in the range of 0.3 to 0.6 Hz. Since the third harmonic oscillation of the bell is predominant in C. European systems, the critical natural frequency of the bell tower is from 0.9 to 1.8 Hz. In slender towers often is the case of the first natural frequencies to be in this range [8].

Table 1. Bell 2 characteristics

| Bell 2 | Symbol | Value | Unit |
|---|---|---|---|
| **Material** | $\gamma_{(bronze)}$ | 8900 | $kg/m^3$ |
| **Volume** | V* | 0.109 | $m^3$ |
| **Mass** | m | 978 | kg |
| **Distance between center of mass and axis of rotation** | s | 0.3772 | m |
| **Mass inertial moment** | I | 139.16 | $kg \cdot m^2$ |
| **Maximum angle** | $\varphi_0$ | 29 | ° |
| **Hits per minute** | A | 55 | 1/min |
| **Diameter** | Ø | 1.13 | m |
| **Height** | h | 1 | m |
| **Thickness** | t | 0.03 | m |

## 4. CALCULATION OF BELL FORCES

There are several research papers that deal with the bell's forces. The work of Bennati et al.(2005) is of particular importance in terms of calculation of bell forces. The authors make complex experimental measurements to obtain real forces over time and then form mathematical model of a pendulum that yields results almost identical to the experimental ones.

Real motion of a bell is complex, forced and damped at the same time. This movement, mathematically, can be described by a moving pendulum with identical geometric and dynamic characteristics Figure 15. Such a system approximates the mass of the bell at one point, the center of mass, which is connected with rigid rod at a certain distance from the axis of rotation. This approximation gives sufficiently accurate results for the calculation of bell forces.

On the one hand, when the bell moves at the axis of rotation two reactive forces appear H and V, which keep the system in balance Figure 16. While on the other hand the movement of the bell's mass generates active forces in tangential direction of the movement and in the direction normal to the movement Figure 17. Putting all these forces in balance, while changing the angle φ gives the answer to the pendulum i.e. the change of the horizontal and vertical forces [9].

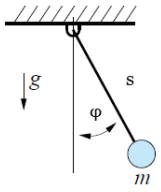


Figure 15. Mathematical model of the pendulum

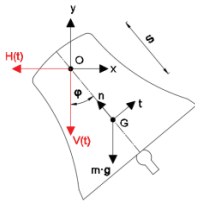


Figure 16. Reactive bell forces

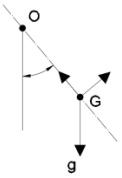


Figure 17. Active bell forces

The dynamic of the bell with the clapper is extremely complex and difficult to analyze due to the nonlinear characteristics, repetitive strokes and complicated excitation. Because of that it is of interest to have a relatively simple model that will produce accurate results.

There are two ways to describe the movement of a bell. The first case is when the bell and the clapper move separately and the second case is when they move together i.e. they have the same angular velocity. In the first case the system has two degrees of freedom while in the second case the system has one degree of freedom. Because the bell has much bigger mass than the clapper case two is adopted.

The impact of the clapper in the bell causes high frequency vibrations that are transmitted to the bearing substructure. Older wooden load bearing structures have filtered out these high frequencies while today's metal structures do not filter these vibrations but instead transfer them to the bell tower.

Considering the bell as a rigid body with a mass m rotating around an axis of rotation O, the horizontal H(t) and the vertical V(t) component of the dynamic loads caused by the ringing of the bell can be represented by Eq. (2) and Eq. (3)

$$H(t) = m \cdot s \cdot [\dot{\varphi}^2 \cdot \sin\varphi(t) - \ddot{\varphi}^2 \cdot \cos\varphi(t)] \quad (2)$$

$$V(t) = -m \cdot g - m \cdot s \cdot \begin{bmatrix} \dot{\varphi}(t)^2 \cdot \cos\varphi(t) + \\ \ddot{\varphi}(t) \cdot \sin\varphi(t) \end{bmatrix} \quad (3)$$

The pendulum with one degree of freedom is one of the most studied and analyzed nonlinear systems. The periodic motion of the pendulum is harmonious only when it comes to small oscillation angles. Once this threshold is exceeded, the equations for motion become nonlinear. Usually this threshold is around 10°. The motion of bell 2 occupies an angle much larger than 10° and therefore trigonometric functions are off the table since this problem is of nonlinear nature. The dynamic equilibrium of a bell pendulum is governed by nonlinear, second-order differential equation Eq. (4).

$$I \cdot \ddot{\varphi}(t) + m \cdot g \cdot s \cdot \sin\varphi(t) = 0 \tag{4}$$

Starting with these few equations the aim is to find a way to calculate the value of φ over time and later its first and second derivatives. With further expansion of Eq. (4) the equation for calculating φ(t) is obtained Eq(5) where sn is Jacobian elliptic function and k is its module. The module k has a certain value from 0 to 1. If k=0 a sine function is obtained for φ(t) and if k=1 a singularity is obtained for the same function. Comparation of sin and sn functions are shown onFigure 18.

$$\varphi(t) = 2 \cdot \arcsin\left[k \cdot sn\left(\sqrt{\frac{g}{l_r}} \cdot t\right)\right] \tag{5}$$

$$k = \sin\frac{\varphi_0}{2} \tag{6}$$

sn – Jacobian elliptic function

k – module of Jacobian elliptic function

g – gravity constant

$l_r$ – reduced length of equivalent pendulum ($l_r$=I/(m·s)

From Eq. (5) and Eq. (6) it can be noticed that the motion of the pendulum depends only on the maximum angle and the reduced length of the pendulum.

Considering the horizontal forces H(t), the results obtained from the analysis are shown onFigure 19. Calculation of the vertical forces is not taken into account since the tower has great axial stiffness.

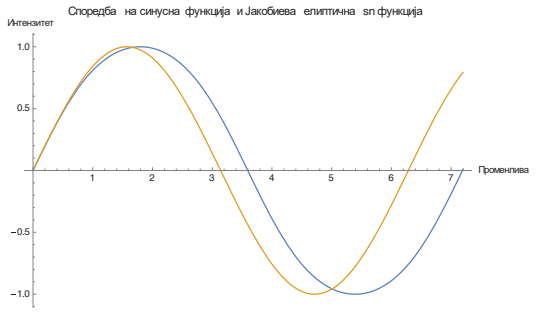


Figure 18. Comparison of sin (orange) and sn (blue) functions

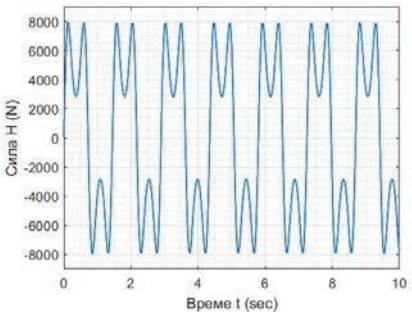


Figure 19. Horizontal force H(t)

From today's literature,[7], it is known that for C. European system of rotation the frequency of third harmonic oscillation from the horizontal force is predominant $f_H$. Due to this fact, care should be taken to ensure that this frequency is far enough from the natural frequencies $f_0$ of the bell tower so that resonance does not occur.

Table 2 Frequencies

| | 1st | 2nd | 3rd | 4th | 5th |
|---|---|---|---|---|---|
| $f_0$ | **1.6** | 1.68 | 3.98 | 4.31 | 5.27 |
| $f_H$ | 0.68 | **1.37** | 2.06 | 2.75 | 3.44 |

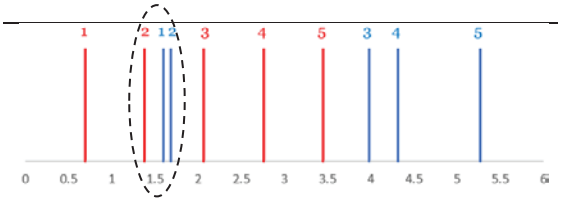

Figure 20. Frequencies comparation (red-harmonic oscillations, blue-natural frequencies)

Table 2and Figure 20 show that low level of resonance is present in the bell tower of Cathedral church "St. Clement of Ohrid" because the difference between first natural frequency and the second harmonic oscillation of the horizontal force is around 15%. The German standard for bell towers DIN 4178-2005 explicitly states that to eliminate resonance the smallest difference between the natural frequencies of bell tower and

frequencies of harmonic oscillations should be at least 20%. When resonance occurs, it is expected during the ringing of the bells the horizontal displacements to be amplified and hence cause greater structural damage.

There are several methods that modify the dynamic characteristics of the bell or the bell tower that eliminate the phenomenon of resonance. One of them is to change completely the system of the ringing bells. Another method is to modify the electric motor that drives the rotation of the bell. Also, there is a method where a steel rods are added to the bell tower to stiffen it in order to move away the problematic natural frequency from the harmonic oscillations of the bells [10].

## 5. CONCLUSION

Considering the stability of a bell tower it can often be severely undermined by the dynamic actions generated by the motion of its bells. In such situations, accurate reconstruction of the dynamic actions transmitted on the tower becomes essential. The procedure presented above for bell forces calculation can be used to accurately predict the motion of bells and their reactions on bell towers.

The aim of this research was to check whether resonance occurs when the bells are ringing. The value of the maximum obtained horizontal force is 82% of the bell's weight. As far as the values of these forces are considered the obtained horizontal forces do not present threat to the structural system. However, if the resonance effects are considered, a low level of resonance is present in the tower. Even though in most bells ringing in the C. European system the frequency of the third harmonic oscillation of the horizontal force presents threat for resonance to occur, in this case the second harmonic oscillation causes the resonance. This frequency pairs with the first natural frequency of the bell tower.

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