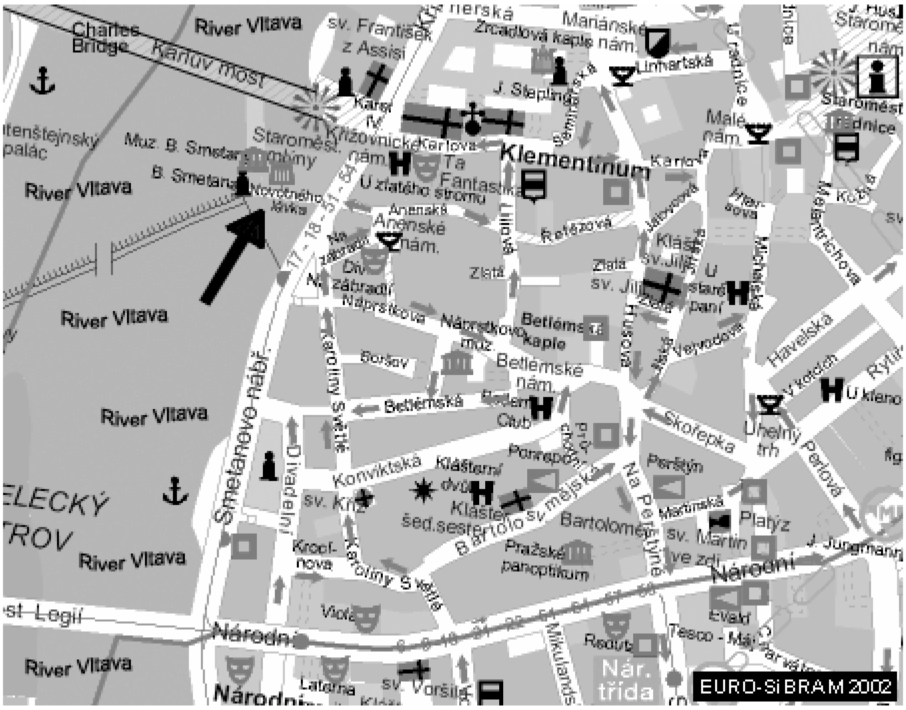
Jaroslav Němec

Euro-SiBRAM'2002

International Colloquium

Volume 1

Prague - Czech Republic
June 24 to 26, 2002



Colloquium EURO-SiBRAM'2002 - Prague

Simulation-**B**ased **R**eliability **A**ssessment **M**ethods
applicable in designer's work and in the new generation of codes

June 24 to June 26, 2002 - Prague, Czech Republic

The Colloquium takes place in the building of
Český svaz vědeckotechnických společností, **Prague 1, Novotného lávka 5**
(100m from the east end of Charles Bridge)

Proceedings - Volume 1

Euro-SiBRAM'2002

International Colloquium

Prague - Czech Republic
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GENERAL INFORMATION

Colloquium EURO-SiBRAM'2002 - Prague

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(100m from the east end of Charles Bridge)

The Colloquium Euro-SiBRAM'2002 Prague is organized by

Institute of Theoretical and Applied Mechanics
Academy of Sciences of the Czech Republic

Sponsors:

- IPS SKANSKA a.s., Prague, Czech Republic
- Czech Constructional Steelwork Association
- Journal "Konstrukce" (Structures), Ostrava, Czech Republic

Under the auspices of:

- Department of Civil Engineering and Engineering Mechanics, University of Arizona, Tucson, Arizona, U.S.A.
- Technical University VŠB TU Ostrava, Czech Republic
- The Center of Advanced Technology for Large Structural Systems (ATLSS), Lehigh University, U.S.A.
- Universidade da Beira Interior, Dept. de Engenharia Civil, Covilha, Portugal
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TECHNICAL PROGRAM

Keynote Speaker

Prof. G. I. Schuëller is the Keynote Speaker for the Colloquium. He will review the state-of-art and outline the problems related to the implementation of the simulation-based structural reliability assessment methods in the design.

The format for the Technical Sessions

The Colloquium consists of nine technical sessions. Each session has a specific technical theme and is conducted under the guidance of two Moderators.
Moderators start a session by making presentations lasting for (about total) ten to twenty minutes. Following the Moderators' presentations, discussions submitted by the participants for a particular session, are briefly presented, subject to the time constraint (5 minutes limit is suggested). Other participants have opportunity to present their thoughts on the subject as well.
The technical content of the session is summarized by a Recorder at the conclusion of the session. The summary of each of the session will be published in the proceedings (Volume 2).
Each session is expected to be tape-recorded.

Role of Moderators

To facilitate the discussion, each moderator was asked to submit two-page Abstract. These Abstracts were posted on the Colloquium web site so that the participants have had enough time to prepare for the discussion segment of a particular session.
All submitted abstracts received from the Moderators and abstracts received from Participants (before the deadline) were edited and are published in the form of a Proceedings - Volume 1 to be distributed at the time of registration.
It is expected that each Moderator will also submit two to six page papers to be included in the Colloquium Proceedings (Volume-2). The papers should be submitted to the Organizing Committee in the final form, following the format specified at the web site (see www.itam.cas.cz/SBRA), before August 1, 2002.

Comments received from Participants

Each participant was encouraged to submit formal comments on any abstract submitted by Moderators posted on the web or their individual thoughts on a particular session.

Instructions to Authors

For format (typesetting) please see www.itam.cas.cz/SBRA. Only properly formatted texts presented at the Colloquium will be accepted for publication.

Colloquium Proceedings

The Proceedings of the Colloquium (Volume - 2) will be published in CD-ROM following the Colloquium. Only the papers presented at the Colloquium will be included in the Proceedings. Each moderator and participant are requested to submit papers between 2 to 6 pages long

following the format specified at the web site (see www.itam.cas.cz/SBRA). Please mention the session number.
Please e-mail your paper(s) to colloquium.2002@itam.cas.cz. or send to the Org. Committee a disk containing your paper(s) plus one hard copy.

It is important that you emphasize in your paper just **one of the topics** covered in the nine sessions outlined in the program. Please feel free to submit papers to more than one session.

The Proceedings (Volume - 2) will also include the summary of all sessions and the conclusions as well as the recommendations resulting from the Final Panel session.

General Comments

(1) The official language of the Colloquium is English.
(2) The time limit for presenting your *discussion* will be set by the moderators.

TECHNICAL SESSIONS

Monday June 24, 2002

8:30 to 9:30 Registration (in front of the Colloquium hall)
9:30 to 10:15 **Opening of the Colloquium**
10:15 to 11:00 **Roadmap of the Colloquium: Topics, Goals, Communication Tools and Limitations** (P. Marek) and Discussion
11:00 to 11:30 Coffee Break
11:30 to 12:30 **Keynote Lecture** Prof. Dr. Ing. G. I. Schuëller, Austria, on
PAST, PRESENT and FUTURE of SIMULATION-BASED STRUCTURAL RELIABILITY ANALYSIS,
chaired by Prof. H. Krawinkler, PhD. (U.S.A)

12:45 to 14:00 Lunch

14:00 to 15:30 **SESSION 1: Basic Simulation Concepts applicable in Codified Design**
Moderators – A. Haldar (U.S.A.) and T. Vaňura (Czech Rep.)
Identify different elements of simulation that will be covered in the subsequent sessions. (For questions to be discussed see corresponding Moderator's Anstract).
15:30 to 16:00 Coffee Break
16:00 to 17:30 **SESSION 2: Representation of the main types of Random Variables considering their application in simulation-based analyses**
Moderators: Y.P. Mack (U.S.A.) and G. Fegan (U.S.A.)
Quantification of randomness including parametric, non-parametric, bounded, unbounded, histogram-based, etc.

18:30 to 20:00 **Icebreaker Reception (same building)**

Tuesday June 25, 2002

8:30 to 9:00 Registration (in front of the Colloquium hall)
9:00 to 10:30 **SESSION 3: Loading and Load Effects Combination**
Moderators: G. Augusti (Italy) and A. Nowak (U.S.A.)
Are the load effect combinations in the current design codes adequate (worldwide review)?
What needs to be done to make them more complete considering the application of simulation techniques?
10:30 to 11:00 Coffee Break
11:00 to 12:30 **SESSION 4: Reference Values (Safety, Serviceability and Durability)**
Moderators: P. Tikalsky (U.S.A.) and J. Menčík (Czech Rep.)
Reference values (RV) are essential in probabilistic designs. How RVs are selected for different failure modes (deflection, fatigue, strength, serviceability, durability, etc.) needs a thorough understanding. Issues related to RVs are the subjects of this session.

12:45 to 14:00 Lunch

14:00 to 15:30 **SESSION 5: Reliability Evaluation of Systems using simulation**
Moderators: Z. Dostál (Czech Rep.) and S. Kmeť (Slovak Rep.)
The reliability evaluation of systems using simulation is in the early stage of its development. Strategies to implement simulation in the reliability evaluation of complicated systems are the subjects of this session. Selection of Reference Values for complicated systems could be very challenging

15:30 to 16:00 Coffee Break

16:00 to 17:30 **SESSION 6: Simulation analysis for time dependent reliability assessment**
Moderators: S. Mahadevan (U.S.A.) and U. Peil (Germany)
The implementation of the simulation reliability assessment techniques to study the time dependent resistance and load effect combinations are the subject of this session.

17:30 to 18:30 **TOUR: Charles the IV Bridge**

Wednesday June 26, 2002

8:30 to 9:00 Registration (in front of the Colloquium hall)

9:00 to 10:30 **SESSION 7: Education of Simulation based reliability analysis to designers and others**
Moderators: Le-Wu Lu (U.S.A.) and S. Wolinski (Poland)
Success of the implementation of the simulation-based design will depend on education of design engineers and students. What are the ways we can educate our current and future engineers?

10:30 to 11:00 Coffee Break

11:00 to 12:30 **SESSION 8:. Codes, databases, software and application of internet**
Moderators: M. Cheung (Canada) and E. Simiu (U.S.A.)
Assess the availability of softwares currently available to implement the simulation-based design in designer's everyday work. Are there other softwares (besides e.g. SBRA) currently available to designers? What will be the future directions in developing such softwares?
How would the new generation of codes, databases and software look like in order to use simulation-based reliability analysis?
How to collect and store data and information in order to create corresponding qualitatively new data-bases and knowledge-bases related to simulation-based reliability analysis?

12:30 to 13:45 Lunch

14:45 to 15:15 **SESSION 9: Selected Applications to simulation-based design**
Moderators: R. Harte (Germany) and G. Alpsten (Sweden)
Case studies on the application of simulation in practical design will be discussed.

15:15 to 15:30 Coffee Break

15:30 to 16:30 **SESSION 10: Panel Discussion**
Panelists: G. I. Schuëller, A. Haldar, H. Krawinkler, N. Cheung, P. Marek
The major conclusions made in the technical sessions will be summarized. The future course of actions or directions in the simulation-based codified design will also be discussed.

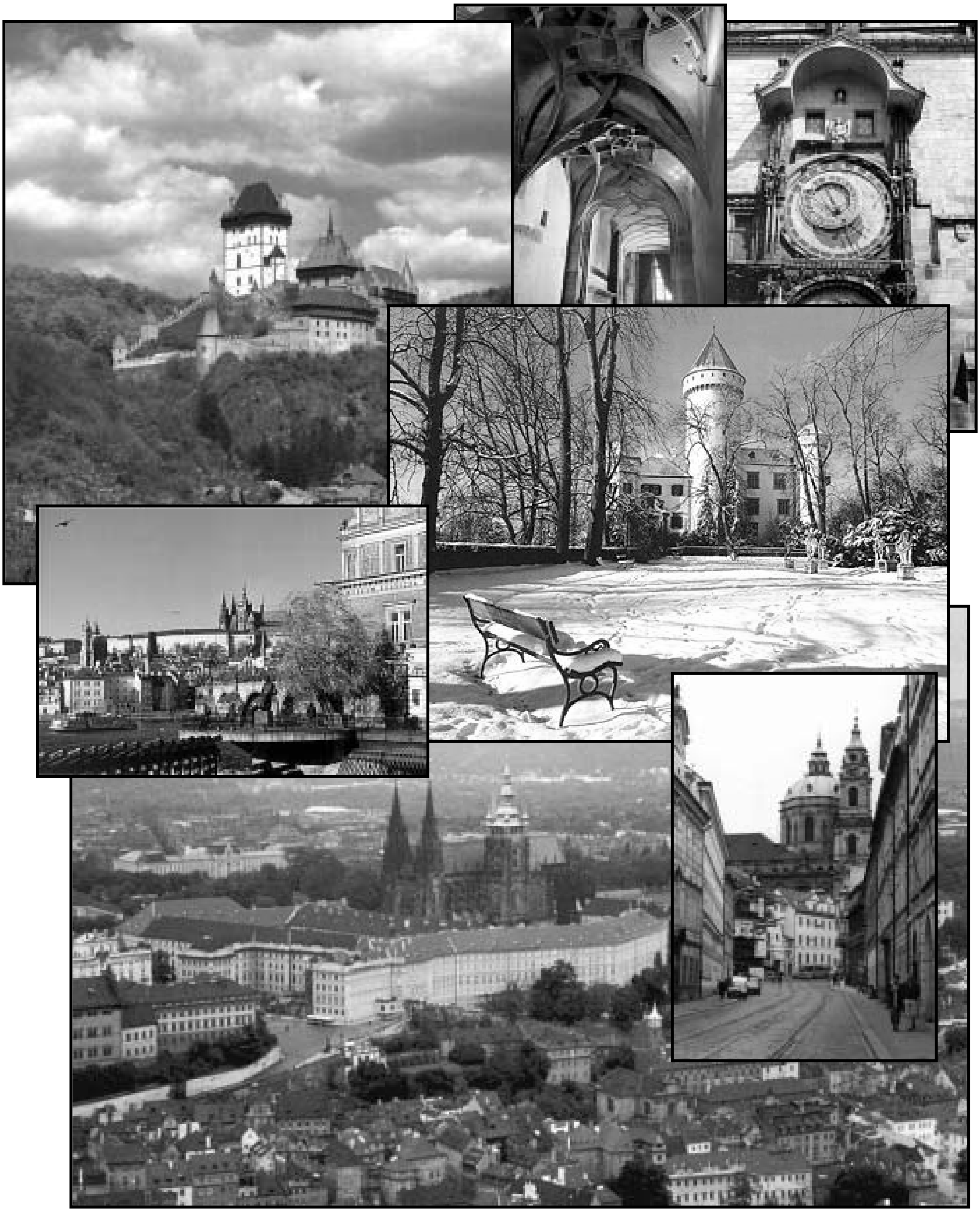
16:30 to 17:00 **CLOSING SESSION. Colloquium adjourned.**

18:30 to 22:00 **Banquet (reception will start at 18:30 and dinner will start at 19:30)**

POST-COLLOQUIUM TOUR

Thursday June 27, 2002

Contact the travel agency ITC Travel&Conference,
Koňevova 41, 13000 Prague 3, +420-2-22581215, itc@login.cz.



REGISTRATION FEE AND REGISTRATION FORM

Participants

The registration fee is EURO 200,-. The registration fee includes proceedings (Volume 1, distributed at registration, and Volume 2, including CD-ROM will be mailed to the participants in Fall 2002), souvenir, three lunches (Monday – Wednesday), morning and afternoon coffee breaks, Monday Ice breaker reception, Tuesday Tour and Wednesday banquet

Students

Student's registration fee is EURO 100,-. It includes three lunches (Monday – Wednesday), morning and afternoon coffee breaks, Monday Icebreaker reception, Tuesday Tour and Wednesday banquet.

Spouses

Spouse's registration fee is EURO 100,-. It includes three lunches (Monday – Wednesday), morning and afternoon coffee breaks, Monday Icebreaker reception, Tuesday Tour and Wednesday banquet.

Registration form

Please submit your registration form (filled and signed) at Colloquium registration (If your registration form was not already mailed to the Organizing Committee).

SESSIONS

Opening Session:
Marek P. Colloquium Roadmap - Topics, Goals, Communication Tools and Limitations

Keynote Lecture:
Schuëller G.I. Past, Present and Future of Simulation-Based Structural Analysis

Session 1:
Moderators:
Haldar A. Basic Simulation Concepts Applicable in Codified Design
Vaňura T. Moderator’s Motions

Session 2:
Moderators:
Mack Y.P. Statistical Issues on Simulation Techniques in Structural Engineering
Fegan G. Issues in Simulation Based Reliability Assessment
Discussion:
Guštar M. Random Variables Representation in Simulation Techniques

Session 3:
Moderators:
Nowak A. Loads and Load Combination Models for Bridges
Discussions:
Marek P. Modeling of Loads
Guštar M. Loadings Representation in SBRA method
Konečný P. Parametric Study of the Safety of a Steel Bar
Hanzlík P. Two-Component Wind Rosette Application
Pustka D. SBRA Method as a Tool for Reliability Analysis of Structural Members...
Václavek L. Probabilistic Modeling of Loading, Combination of APT and MLE Approach
Vaidogas E. Accidental Loads: Codefication of Simulation-Based Procedures for Predicting...
Hudák J. Modeling of More-Component Loads Example – Crane Girder in an Industrial...

Session 4:
Moderators:
Tikalsky P. Reference Values for Durability Based Performance Design Criteria
Menčík J. Reference Values (Safety, Serviceability and Durability)
Discussion:
Rieger M. Reliability Assessment of Composite Steel Concrete Cross Section of Roadway...

Session 5:
Moderators:
Dostál Z. From Simple Structures to Systems
Kmeť S. Application of Modern Simulation Methods in the Time Dependent Probabilistic..
Discussion:
Pustka D. Reliability Analysis of Statically Indeterminate Steel Frame (Pilot Study)
Phoon K. K. Digital Simulation Algorithms for Second-Order Stochastic Processes
Václavek L. Unbraced Frame as Simple Series System

Session 6:
Moderators:
Mahadevan S. Time-Dependent Reliability Analysis Using Simulation Techniques
Tikalsky P. Durability Assessment of Structures
Discussion:
Pustka D. Lifetime Prediction of Steel Component Exposed to Time-Dependent Corrosion...
Korouš J. SBRA Concept in Corrosion Modeling and Inspection Planning
Narmontas D. Peculiarities of Safety Assessment of Steel-to-Timber Joints

Session 7:
Moderators:
Wolinski S. Teaching Reliability Concepts in Civil Engineering Using Simulation Techniques

Session 8:
Moderators:
Simiu E. Codes, Databases, Software, and Applications of Internet
Cheung M. Reliability Assessment in Highway Bridge Design
Discussion:
Pustka D. Reliability Assessment of a Beam According to SBRA Method and Eurocode
Křívý V. Reliability Assessment of a Steel Column According to SBRA Method and CSN...
Konečný P. Safety Assessment of a Frame
Lashoberová The Safety of a Column Exposed to Two-Component Load Effects Combination
Csóková L. Assessment of Serviceability of a Cantilever Beam According to SBRA method...

Session 9:
Moderators:
Harte R. Action and reaction in case of complex building projects – Simulations for...
Alpsten G.
Discussion:
Hlaváček J. Optimization Study of Beam with Sudden Change of its Profile
Lokaj A. Timber Element Reliability Assessment
Pirner M. TV Tower – Serviceability Check Using SBRA Method
Laschoberová Two Component Load Effect Combination

Opening Notes and Information

Roadmap of the Colloquium: Topics, Goals, Communication Tools and Limitations

Professor Pavel Marek, Ph.D., DrSc.
Institute of Theoretical and Applied Mechanics - Academy of Sciences of Czech Republic
Prosecká 76, 190 00 Prague 9, and
Dept. of Civil Engineering, VŠB Technical University Ostrava,
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„A Colloquium is a Conference at which experts discuss a specific topic."
The major objectives of the Colloquium EURO–SiBRAM'2002 (Simulation-Based Reliability Assessment Methods) are to discuss the role of simulation in the codified design of engineered structures. It is expected to be an open discussion among scholars, scientists, practicing engineers and students on how simulation based reliability assessment concept can be implemented in the practical codified design worldwide. Presentations of technical papers containing individual research interests are not the major objective.
The Keynote Speaker for the Colloquium Prof. G. I. Schuëller was invited to review the state-of-art and to outline the problems related to the implementation of the simulation-based structural reliability assessment methods in the design. He is also expected to emphasize the development and improvement of the computer and information technology affecting the reliability assessment.
The Colloquium consists of nine technical sessions. Each of these sessions has a specific technical theme and is conducted under the guidance of two Moderators who will start a session by making joined introductory presentation lasting total ten to twenty minutes (maximum). This presentation should review the substance of the topic covered in the particular session, outline alternatives and list the main items (questions) to be addressed in the discussion. Following the Moderators' presentations, technical questions, proposals, comments, and relevant ideas for a particular session (submitted to Moderators before the session) will be briefly presented by the participants and discussed by the audience. Subject to the time constraint, the time limit for presenting *discussion* will be set by the moderators (a 5 minutes time limit is proposed). The technical content of the session will be summarized by a Recorder at the conclusion of the session.
The subject of the concluding Final Panel Discussion (Session 10) is to summarize and evaluate the achievements obtained at technical sessions and to outline a general outcome and response to the Colloquium assignment.
Each session is expected to be tape-recorded.
The Proceedings of the Colloquium (Volume - 2) will be published later in Fall 2002 (in CD-ROM). Each moderator and participant are invited to submit "full" papers between 2 to 6 pages long following the format specified at the web site (see www.itam.cas.cz/SBRA - please mention the session number) before August 1, 2002. Note: The Proceedings (Volume - 2) will also include the summary of eachl session (Recorder's review) and the conclusions, as well as the recommendations of the Final Panel session
Only the papers (discussions) presented at the Colloquium will be included in the Volume-2.
Please e-mail your properly formatted text(s) to colloquium.2002@itam.cas.cz. or send to the Org. Committee a disk containing your text(s) plus one hard copy before August 1, 2002.

The Colloquium Committee received from the participants since January 2002 numerous questions and suggestions related to the content of individual sessions. The following part of the *Roadmap of the Colloquium* turns attention to some of the fundamental problems suggested by the participants to be discussed at the Colloquium:

Session 1:

Are the simulation-based structural reliability assessment concepts mature enough to be considered as an alternative to the currently available approaches applied in codes and in designers' everyday work?

Session 2:

Should the variables involved in simulation-based structural reliability assessment concepts be expressed by parametric distributions or are the non-parametric (bounded) distributions acceptable representation of variables in the framework of simulation-based concepts applicable in codes?

Session 3:

How should be represented individual loads and load effects in the simulation-based structural reliability assessment concept in order to allow for probabilistic analysis of safety, serviceability and durability of structural elements, components and systems?

Session 4:

While implementing the probabilistic structural reliability assessment concepts based on simulation technique in designers' work (and in corresponding codes), how are actually defined the Reference Values needed for the calculation of the probability of failure?

Session 5:

Can the application of a probabilistic simulation-based structural reliability assessment concept be extended from elements, components and simple structures to complicated systems?

Session 6:

How to express time dependent load combination effects and time dependent reference values in the framework of the durability simulation-based reliability assessment?

Session 7:

What are the most important improvements in the education of students and designers needed in order to change their "way of thinking" from deterministic to probabilistic?

Session 8:

The introduction of simulation based structural reliability assessment concept in codes and in designer's work will require re-engineering of the entire design procedure, codes, databases, software and more. What is to be done first?

Session 9:

Are there already available examples, parametric studies, comparisons and other applications of the simulation-based structural reliability assessment concepts allowing the designers, students, code writing bodies and others to compare the assessment according the partial factors method mainly used in current codes and design practice?

The output from the Final Panel discussion will indicate, no doubt, the high potential of the simulation-based reliability assessment concepts in structural design, especially considering the advances in computer technology.

Keynote Lecture

Past, Present & Future of Simulation-based Structural Analysis

G.I. Schuëller
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The Monte Carlo Simulation Method involves an interesting combination of sampling theory and numerical analysis, and in this context may be viewed as an art of computing. Although it is frequently used in non probabilistic context to solve equations where numerical solutions are not easily obtainable by standard numerical methods. In this lecture, however, it will be used as a device of studying artificial stochastic models of a physical or mathematical processes.

The Monte Carlo Method – which based on the game of chance – was named after the famous Casino of Monte Carlo in Monaco. It was developed in the early fortieth of the last century in context with the Los Alamos Project (development of the Atomic Bomb) for solving the so-called random neutron transport problems, i.e. diffusion in fissile materials. Already at an early stage of these investigations, the direct simulation was refined with certain variance – reducing techniques, in particular "Russian roulette" and "splitting" methods.

About the same time the methods and theories of structural safety and reliability have been developed, and it was until the late sixtieth and early seventieth until Monte Carlo simulation techniques have been – primarily by Shinozuka and co-workers – introduced to this field. First for simulating random variables, and random processes, later on for random fields. It took more than another decade until variance reduction techniques were applied.

It goes without saying, that the efficient use of Monte Carlo Simulation procedures could be advanced only through the availability of digital, high speed computers. (Their development, again fell into the early fortieth).

Currently the Monte Carlo Simulation procedure is applied to various aspects of analysis, design of structures, such as uncertainty, reliability, safety and other types of analyses. It proves to be the most versatile tool in structural analysis when taking into account uncertainties in load, material and geometrical parameters.

Its obvious disadvantage of being very time consuming is counterbalanced by the continuous and rapid advancement of hardware, as well as of software, including methodologies for size (dimension) reduction, parallel computing (either by clusters or by massively parallel computing) and most importantly by advancing variance reduction techniques both for static and dynamic problems respectively.

Current developments of Monte Carlo Simulation techniques also encompass their use within user friendly computer codes for analysis and design, mainly based on a modular basis for maintaining flexibility and reducing efforts for future expansion.

In the future these developments will continue and hence provide the possibility of an efficient analysis of structures of large size with random material and geometric properties under stochastic excitation.

Session 1

Basic Simulation Concepts Applicable in Codified Design

Professor Achintya Haldar, Ph.D., P.E.
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Tucson, AZ 85721, U.S.A.
haldar@u.arizona.edu

In general, engineering design consists of proportioning the elements of a system to satisfy various criteria of performance, safety, serviceability, and durability under various demands. It is accepted that the presence of uncertainty cannot be avoided in every phase of engineering analysis and design, but it is not simple to satisfy design requirements in the presence of uncertainty. As a result of three decades of extensive work in different engineering disciplines, several reliability evaluation procedures of various degrees of complexity are now available. First-generation design guidelines and codes are being developed and promoted worldwide using some of these procedures.

A casual review of these reliability-based design codes indicates that they are very similar to the earlier deterministic codes. The advanced reliability concept used in developing these codes generally remain unknown to designers. Furthermore, it may be difficult to an experienced design engineer to consider the presence of different levels of uncertainty than considered in developing the reliability-based design guidelines in a particular design application. In most cases, these guidelines were developed considering the behavior of elements of complex structural systems satisfying an explicit performance criterion. The evaluation of system reliability using information on element level reliabilities may not be simple. In some cases, the behavior of complex structural systems may need to be estimated based on many simplifying assumptions including the supports and connections conditions, various sources of nonlinearities, etc. The performance functions in general are implicit in nature for complex structural systems. For a given performance function, the reference value to define it properly may not have been developed yet or accepted by the profession.

Due to significant advancement in computer technology and computational power, another alternative to implement the risk-based design concept in practical design is to use simulation. In this approach, a designer will know the characteristics of uncertainty being considered in a particular design, use judgment to quantify randomness beyond what is considered in a typical codified design, evaluate the nature of implicit or explicit performance functions being considered, and will have control of the deterministic algorithm being used to study the realistic structural behavior at the system level.

The method commonly used for this purpose is called the Monte Carlo simulation technique. In the simplest form of the basic simulation, each random variable in a problem is sampled several times to represent the underlying probabilistic characteristics. Solving the problem deterministically for each realization is known as a simulation cycle, trial, or run. Using many simulation cycles, particularly when the number of cycles tends to infinity, will give the probabilistic characteristics of the problem. In essence, simulations using computer is an inexpensive experiments compared to laboratory testing to study the presence of uncertainty in the problem. It also helps to study different design alternatives in the presence of uncertainty, hopefully identifying an optimum solution.

This gathering of distinguished scholars on risk-based design from all over the world is a significant development and is expected help to implement the simulation-based engineering design as an alternative to classical codified approach. However, there remain many challenges that need to be addressed before its implementation. In all fairness, similar challenges may be present in the current codified approach. Thus, the discussions in this colloquium on simulation may help to improve the current codified approach.

The Monte Carlo simulation technique has six essential elements: (1) defining the problem in terms of all the random variables; (2) quantifying the probabilistic characteristics of all the random variables in terms of their probability density functions and the corresponding parameters; (3) generating values of these random variables; (4) evaluating the problem deterministically for each set of realizations of all the random variables, or simply numerical experimentation of the problem; (5) extracting probabilistic information from N such realizations; and (6) determining the accuracy and efficiency of the simulation. The success of implementing the Monte Carlo simulation in design will depend on how accurately each element being addressed. All these elements will be discussed in detail in the following sessions.

Issues related to the efficiency and accuracy of the deterministic algorithm to be used in simulations, the appropriate way to quantify the randomness, information to be used to define the statistical characteristics, defining appropriate performance functions and the selection of reference values, evaluating correlation characteristics of random variables present in complex systems, simulation of random variables versus random field, system reliability, the effect of load combinations, time dependent reliability, available software to implement the simulation-based concept, and several case studies will be extensively discussed in the following sessions. It is expected that this world body of distinguished scholars on reliability-based design will help to formulate the future direction in simulation-based design.

In keeping the spirit of this colloquium, to help guide the discussion, I would like to itemize some broad questions that need our attention during this first session.

1. Is the simulation-based design concept mature enough to be considered as an alternative to the currently available codified approach?
2. At present, should designers have the option to use either the simulation-based approach or codified approach?
3. What would be the mechanism to convey the designers' preference to a governing body responsible for maintaining the overall safety of structures?
4. If simulation-based design is accepted as an alternative approach to satisfy the current requirements, which organization(s) should provide leadership in distributing information on uncertainty in parameters, software, and technical support? Should this support be available at the local, county, state, national, or international level?
5. What is the future of simulation-based design considering the advancement in the computer and information technology areas?

Session 1

Moderator's Motions

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Abstract: This "Moderator's Motions" is a summary of ideas thought to be suggested for expert discussion in frame of Session 1 of SiBRAM'2002 colloquium.

Key Words: Simulation's preferences, optimal trials amount, convergence acceleration, allowable probability of failure

The world surrounding us is stochastic. We have to accept this fact and today's meeting validates our tendency toward it. Mainly designers together with constructors are ought to implement the reliability assessment concept into the file of methods of practical codified computational design, with the final goal to make it governing among computational design methods at all. Recently it is not nearly realized in building codes of Czech Republic. The steel structures standard ČSN 73 1401 only allows to apply the probabilistic reliability assessment concept on simulation technique as a designer's tool.

Why especially the simulation technique? That is the question we have to discuss during this EURO – SiBRAM 2002 colloquium.

Let us start our considerations from the point of view that the designed structure is not an isolated object but a part of a clasped system: Structure – Load – Environment (S-L-E). There is no doubt that all elements of this chain are highly stochastic.

To do justice to such mutually tied system using accurate methods of mathematical analysis succeeded in few simple cases only. On the contrary simulation technique offers unlimited chance to study the behavior of the S-L-E system exploiting applied mathematics commonly used by engineers.

Certainly for this must be paid by a large amount of computational operations what. However, this represents an unessential problem with regard to the powerful development of recent computational systems.
In spite of this the size of n (n stands for number of trials independently drawn from the population) remains the general problem of structural safety simulation. The increasing n effects favorably on precision of simulation results at all, e.g. the approximations of the mean, the standard deviation… It reduces spans of so-called Confidence Intervals etc. etc. On the other hand it is necessary to keep it within rational limits.

It follows that the settlement of optimal value of n is an important business connected with simulation technique. It would be very useful to anatomize different points of view on this problem at this session.

No less attention should be paid to the connected problem of acceleration of the convergence of simulation process. With other words the matter is to make the best use of given number of trials n.

Passing away the *crude (simple, classic) Monte Carlo method* there were developed improved simulation concepts directed to the saving of computational time which profit by a certain kind of knowledge about the solution of the simulated problem. The method called *Importance Sampling method* gives better chance of being drawn to the region closed to the expected solution. The other called *Stratified Sampling method* works by dividing the sampling space into subspaces and choosing one point randomly into each of them. Again some knowledge about the solution enables to improve the convergence and thereby to diminish the number of trials n.

Here very often the *Latin hyper cube* technique is implemented for sampling from very high-dimensional space. This scheme can be used much more efficient than simple random sampling scheme. Simple random sampling is for high-dimensional space inefficient because it typically gives higher probability to the middle of a distribution than to its tails especially in above noticed high dimensions. Latin hyper cube efficient scheme would sample the tails quickly. This can be accomplished by stratifying the support of the distribution. Latin hyper cube gives a hope that all possible combinations were handled in a balanced way.

However, one should take into account that the classic Monte Carlo method preserves much more certainty against fault and is independent upon subjective strains. Thus, one must weigh up the decision considering the common usefulness of chosen method.

Expert discussion about perfections and imperfections of all of those above mentioned methods and of their susceptibility for structural reliability simulations would be very convenient, too.

Al last there are to discuss several problems of common interest for both analytical reliability theory as for reliability simulation technique like the worth of allowable probability of failure, i.e. the target probability of P_d which must not be gone beyond during the entire service life of structure. One meets two special concepts to be reflected: intolerable damage connected with common design situation and tolerable damage for extraordinary design situation (e.g. earthquake). A large scope of conceptions should be brought to agreement.

Session 2

Statistical Issues on Simulation Techniques in Structural Engineering

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The use of computer simulation techniques represents a major advancement in reliability assessment in structural engineering. While Monte Carlo methods have gradually come to be recognized as an important tool for many applied disciplines, its widespread acceptance has only occurred in recent years due to advancements in computer technology in terms of memory, speed, and cost. The excellent text by Marek, Gustar and Anagnos (1996) put forward a road map as well as numerous examples to guide the reader who is interested in simulation-based assessment of reliability of structural designs subject to various stress loading schemes. Not only are the Monte Carlo methods material-saving and time-saving for actual experimentation, they also provide useful teaching tools for the engineers-in-training in university curriculum.

One outstanding feature mentioned in the book by Marek, Gustar and Anagnos is the generation of random samples based on bounded histograms corresponding to different loading characteristics. Many well-known parametric models such as the normal and gamma distributions have unbounded supports. In order to employ these distributions to model characteristics in structural engineering, truncation and normalization become necessary to produce bounded histograms. Moreover, in situations where parametric modelling is unsatisfactory, the program M-Star mentioned in the text of MGA actually generates random data (for example, the wind loading effect may be presented by the WIND1 histogram) which may not correspond to any common parametric distributions. (There are many such histograms deployed in MGA.) This feature, the simulation of data from a bounded histogram, not necessary of the parametric types, is at the very heart of the improved Monte Carlo techniques in reliability assessment.

In the same time period when there was significant advancement in computer technology, the subfield in statistics known as "nonparametric function estimation" has seen important developments as well. Various techniques in estimation such as the kernel method, splines, local polynomials, and wavelets have now become mainstream statistical tools. In particular, the theory and implementation of kernel density estimation, including the setting when the underlying distribution has bounded support, are by now well-understood.

Here, we propose to improve on the "bounded histograms" technique mentioned earlier by the kernel method. A nonparametric method in the context of reliability assessment is especially relevant since the data sets are typically of sizes where asymptotics become effective. We demonstrate by examples the effect of generating random samples from a kernel density estimate (the smoothed histogram) on subsequent assessment of reliability.

Session 2

Issues in Simulation Based Reliability Assessment

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The SBRA approach to reliability assessment is characterized by its efficiency and intuitive methodology in the evaluation of a reliability model. The use of histograms with finite support is fundamental to the process. The SBRA book by Marek, Gustar, and Anagnos[1] covers the concepts and procedures for unrelated and stand alone problems.

As the SBRA methodology is applied to systems and larger problems, there are issues that deserve some attention. On the short list of issues would be a careful selection of the random number generator, a general methodology for simulation of correlated data, and Bayesian updating.

The cycle length of the random number generator becomes an important aspect in the simulation of large-scale systems. Even for complicated stand-alone problems, cycle length can be an issue. In past literature there is mention of generators with cycle lengths of 10^9 as being sufficient to carry out Monte Carlo simulations. There has been much criticism of system simulations done with insufficient cycle period. Today the class of Subtract-with-Borrow generators has a cycle period of $4.1*10^{354}$. Math Works, Inc., regards this issue so important that it has used a form of this type of generator in all its releases starting with MATLAB 5.0.

In a system application, the existence of correlated data can be expected. Holder and Mahadevan[2] have discussed the need for handling the generation of random variates in even stand-alone problems. There is a need for an efficient and speedy implementation. The algorithm must also be general enough to handle a large class of variables. I propose that a variation of the well-known Karhunen-Loeve transform be considered as a possible algorithm. Its generality has been established. An exploration of whether the algorithm can be developed to meet the speed requirement should be made.

The last issue is the need for keeping a database for system simulation or complicated stand-alone problems. As results evolve in a study, probabilistic information should be updated in the database. The Baysian updating process should become the key methodology. For an excellent discussion of the updating process one may consult the discussion by P. Popela and M. Vik.[3]

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Session 2

Random variables representation in simulation techniques

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When we take samples of some variable, we observe somewhat different outcomes of observations. If we are not able to find all causes of such variation, the variable is called *random*. The most of variables in simulation methods have more or less random character. Therefore the proper representation of random variables can significantly affect the precision, effectivity and speed of simulation.
The distributions describing the majority of ”real-life” random variables are not known. The types and statistical parameters of the their distributions are estimated from data obtained from the results of measurements, tests and experiments. From these data a histogram or other non-parametrical distribution is usually created and this empirical distribution is approximated by a selected theoretical distribution. In the cases when correct approximation by the theoretical distribution is impossible, non-parametrical distribution can be used directly as the approximation of the original distribution. If the histogram is used, the distribution of a variable within individual histogram bins can be assumed to be uniform over the bin. Then the resulting histogram represents a **piecewise uniform distribution**. If the all values inside individual bins are represented by only one value (for example, at the centre of the bin), the resulting histogram represents a **general discrete distribution**. For the relationship between the original distribution, piecewise uniform distribution and general discrete distribution see Figure 1.

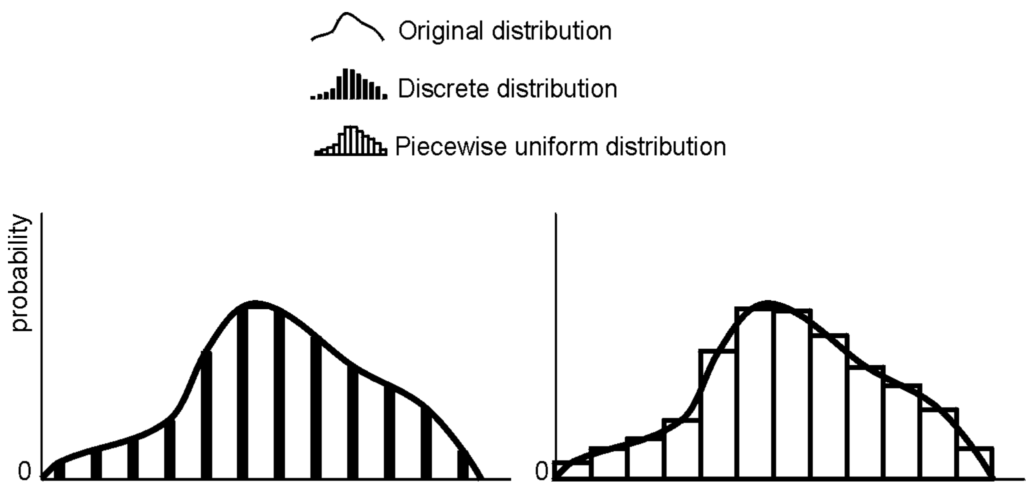

Fig. 1: Discrete and continuous distribution

Since the range of the most random variables is limited to the interval [a, b] a histogram defined on the limited interval can be used for the approximation. General discrete and piecewise uniform distributions defined on the limited interval can be very efficiently generated using the modified inverse distribution function method with tabulated values of the quantile function.

Session 3

Loads and Load Combination Models for Bridges

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The development of rational design and evaluation criteria for bridges requires the development of efficient load models. Load parameters are random variables, depending on uncertainties in loads, load distribution factors, prediction of future loads, and so on. The actual measurements indicate that loads are site-specific and component-specific. The statistical load models are based on the test data, measurements and simulations. The paper is focused on dead load, live load, dynamic load, extreme load events and their combinations.

Live load covers a range of forces produced by vehicles moving on the bridge. It includes the static and dynamic components. The static live load is considered first. The effect of live load depends on many parameters including the span length, truck weight, axle loads, axle configuration, position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), girder spacing, and stiffness of structural members (slab and girders). The effect of these parameters is considered separately. The development of live load model is essential for a rational bridge design and/or evaluation code. The live load model presented in this paper is based on the results of truck survey. The uncertainties involved in the analysis are due to limitations and biases in the survey. It was observed that truck loads and traffic volume are strongly site-specific. The survey results were extrapolated to obtain statistical parameters of the load effects for various periods of time. Multiple presence was considered using Monte Carlo simulations.

Dynamic load effect is considered as an equivalent static load effect added to the live load. It has been observed that the dynamic load, as a fraction of live load, decreases for heavier trucks. The dynamic load model is a function of three major parameters: road surface roughness, bridge dynamics (frequency of vibration) and vehicle dynamics (suspension system). An analytical procedure was developed for simulation of the dynamic load on girder bridges. It was observed that dynamic deflection is almost constant and it does not depend on truck weight. Therefore, the dynamic load, as a fraction of live load, decreases for heavier trucks. For the maximum 75 year values, the corresponding dynamic load does not exceed 0.15 of live load for a single truck and 0.10 of live load for two trucks side-by-side. The coefficient of variation of dynamic load is about 0.80. The results of the simulations indicate that dynamic load factor values are almost equally dependent on road surface roughness, bridge dynamics and vehicle dynamics. The actual contribution of these three parameters varies from site to site and it is very difficult to predict.

Extreme events include scour, earthquake and collision loads. In the past, each extreme event was considered separately, and combinations were considered as a conservative simultaneous occurrence of these events. There is a need for the development of rational design criteria for combination of scour, vessel collision and earthquake. These extreme events often govern the design. Simultaneous occurrence of scour and ship collision may result in a dominating load combination. Calibration of the new bridge design codes covered mostly the basic design combinations with dead load and live load. Extreme loads and their combinations could not be considered in the calibration because the statistical data and methodology were not available.

Scour is not considered a load, however, it can have a considerable effect on bridge performance because of load re-distribution. In addition, from a statistical point of view, occurrence of scour is similar to other extreme. Scour is the major cause of bridge failures in the United States. There are three types of scour: (1) long-term channel degradation (or aggradation), (2) contraction scour, and (3) local scour. Occurrence and magnitude of scour can be affected by a flood event. The following definitions of scour will be used: local scour refers to severe erosion around the abutment and piers, contraction scour refers to scour caused by the constriction to the stream caused by bridge approach embankments, local and contraction scour occur under the bridge and are usually refilled after the flooding event, and therefore can be referred to as short-term, temporary, or transient scour, long-term channel degradation refers to scour across the entire waterway breadth (this is a long-term scour degradation problem that increases in depth with time, and it occurs regardless of the bridge). A long-term scour event is a gradual channel deterioration of base support. Its duration can be measured in years. A short-term scour event is a sudden loss of base support, usually during a storm (or flood). The duration of a short-term event can vary depending on the water-bed material (from hours to days, sometime longer). Often, a short-term scour reaches an extreme level only for a short period of time (the bed scours and fills again).

A vessel collision can be statistically represented by a time varying product of three variables representing variation in the vessel collision force, variation due to uncertainty in transition from load (vessel collision) to load effect in a component (moment, shear forces, axial force), and variation due to approximate method of structural analysis. The major parameters which directly affect the vessel impact force are: type (barge ship), displacement tonnage, and speed. However, the actual collision force is also dependent on site-specific parameters that include: waterway characteristics and geometry; vessel and/or barge configurations; and bridge type and geometry. Expected collision forces can vary widely, however, typical values are 250-750 kN for a drifting empty barge; 15,000 kN for a barge under power; and 250,000 kN for a ship under power.

Extreme load events are specified for different return periods. There is a need for a common denominator. The combination of extreme event loads will require evaluation of the return periods for combined effect and component events. The actual risk of an extreme event is strongly site-specific. The local characteristics can help in a more accurate evaluation of risk and development of rational control measures. There is a need to quantify the risk associated with the current practice. The rational safety level can be selected on the basis of reliability and economical analysis. Then, the design provisions can be developed accordingly.

The basic load combination for highway bridges is a simultaneous occurrence of dead load, live load and dynamic load. Therefore, these three load components are considered first. It is assumed that the economic life-time for newly designed bridges is 75 years. The extreme values of load are extrapolated from the available data base. The combination of extreme event loads requires the evaluation of the return periods for combined effect and component events. In addition, current bridge design methods and analysis models may be inadequate for extreme design. In the past, designers were aware of the need to design for these extreme events, but comprehensive codes and rational design models were not available. As a result, the design of bridges for extreme events can vary broadly throughout the United States. As an intermediate solution, it is recommended that the load models be calibrated for both the available data and the site specific parameters associated with each extreme event.

Session 3

Modelling of Loads and Analysis of Load Effects Combination in Reliability Assessment Concepts based on Simulation

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The transition from current structural reliability assessment methods applied in current codes (such as Load and Resistance Factor Design) to probabilistic methods based on application of simulation technique would require re-engineering of the entire reliability assessment procedure including modeling of loads and introduction of qualitatively new approaches to the load effects combination analysis. In the framework of the Colloquium Euro-SiBRAM'2002 attention should be therefore given to such load representation which will correspond to the development of methods which are using simulation technique as a powerful tool in designer's hands.

In current codes the loads are expressed by characteristic ("nominal") values and load factors. The analysis of load effects combination is defined in codes (e.g., the multi-step and multi-conditional approach is applied). The differences in "combination" rules in individual codes may lead to significant differences in the resulting probabilities of failure.

It can be suggested to discuss, e.g., following questions:

(A) Representation of Loads:

a - How to express the individual loads within the framework of a simulation-based reliability assessment method?

b - Should there be considered only one-, or also two- and more-component load representation?

c – What is the difference between "static" and "dynamic" ("elastic" and "elasto-plastic", or "1st order" and "2nd order", etc.) loads?

(B) Load Effects and Analysis of Load Effects Combination

a - How should be expressed the load effects in the framework of of a simulation-based reliability assessment method?

b - What kind of load-effect combination analysis should be used in the framework of methods applying simulation technique?

c - Why there are significant differences in single-component combination analyses according to LRFD, EC, DIN, Canadian Code, CSN etc.?

d - Current codes remain silent regarding the combination of two- and more-component load effects. How to approach this type of load-effects combination analysis?

e – Why are the load effects combination rules according to LRFD and EC leading to different probabilities of failure?

Session 3

Loadings representation in SBRA method

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Random variables can be described in various forms. A continuous **Probability Density Function** and a discrete **Probability Mass Function** $f_X(x)$ give a natural, easily comprehensible description of the variable distribution. The integral of the probability density function or the summation of the probability masses over all discrete values over the interval $(-\infty, x_0]$ give the **Cumulative Distribution Function** $F_X(x_0) = Pr[X \leq x_0]$. The cumulative distribution function can be used for the assessment that the probability does not exceed a certain value. The **Inverse Distribution Function** also called the **Quantile Function** $Q(p) = F_X^{-1}(x)$ is another form of random variable representation. Using the quantile function, for a selected probability of non-exceedence p, the corresponding value of the random variable can be assessed. The Quantile function can be constructed only for continuous cumulative distribution function, since for non-continuous cumulative distribution function the inverse function does not exist.
Simple distributions can be described using functions in closed form with few parameters. In this case the representation is called **parametrical**. For more complicated or irregular distributions non-parametrical representation can be used, for example in the form of histogram.
Empirical quantile functions can be obtained by sorting, in ascending order, a representative sample over the time a random variable is observed. In Simulation Based Reliability Assessment (SBRA), the Quantile function of a load distribution is called the **Load Duration Curve - LDC**, since it represents the duration of individual load values. Non-parametric form is used since most of the loads can not be characterised using simple parametric distributions. The Figure 1 shows relationship between described forms of random variables' representation.

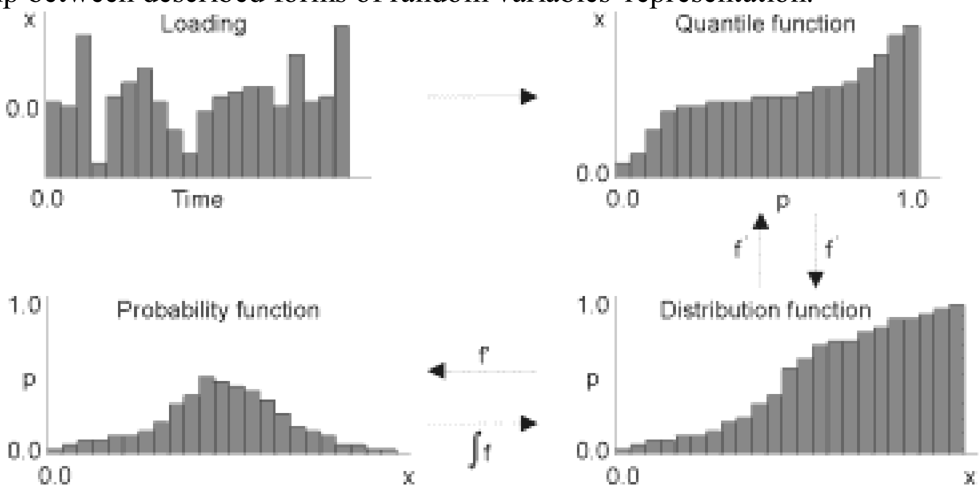

Fig. 1: Random variables' representation

All described forms of random variables' representation are equivalent and the distribution of random variables is fully characterised by each representation.

Session 3

Parametric study of the safety of steel a steel bar

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The subject of the study is the safety assessment of the 26 steel tension members designed according to EC [3] and LRFD [1]. The bars are exposed to an axial tensile force corresponding to the combination of one to seven mutually uncorrelated loads. The safety is analyzed by SBRA [4]. The reliability is expressed by probability of failure P_f. The steel bars are designed for comparison also by SBRA [4]. The probabilities of failure P_f are compared with similar study of the steel bars designed according to LRFD (U.S.A.) published in the Probabilistic engineering mechanics [1]. Study of the bars designed according to LRFD [1] shows significant variations in the probabilities of failure P_f .

Simple steel bars are made of S235 steel grade (material of the bars in the [1] is A36 steel grade) exposed up to seven mutually uncorrelated loads. Characteristic values of the load effects are given in the study [1] (see Fig. 1). Load effects combination is considered according to /1/. The partial factors γ and Ψ used in the load effects combination are summarized in Table 1. The maximum (extreme) load effects according to SBRA correspond to the factored load effects according to EC [3]. The bounded histograms applied in SBRA are those specified in Table 1.

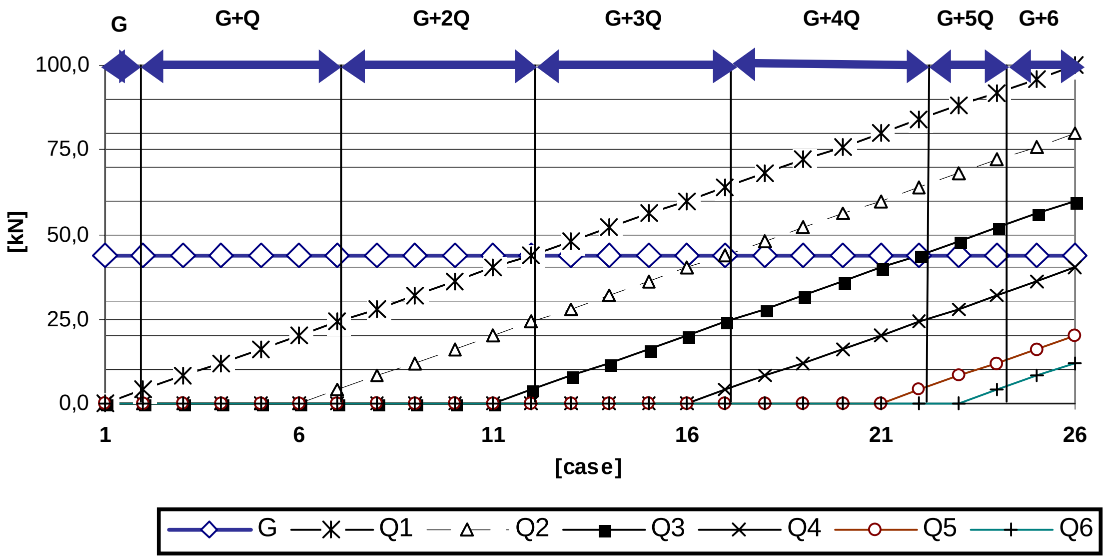
Loadings

Symbol	Load effects	γ_G	γ_Q	Ψ_0	Histogram
G	Dead load	1,35	-	-	Dead1
Q_1	Long-lasting load	-	1,5	-	Long1
Q_2	Snow	-	1,5	0,6	Snow1
Q_3	Wind	-	1,5	0,6	Wind1
Q_4	Short-lasting load	-	1,5	0,7	Short1
Q_5	Short-lasting load	-	1,5	0,7	Short2
Q_6	Short-lasting load	-	1,5	0,7	Short2

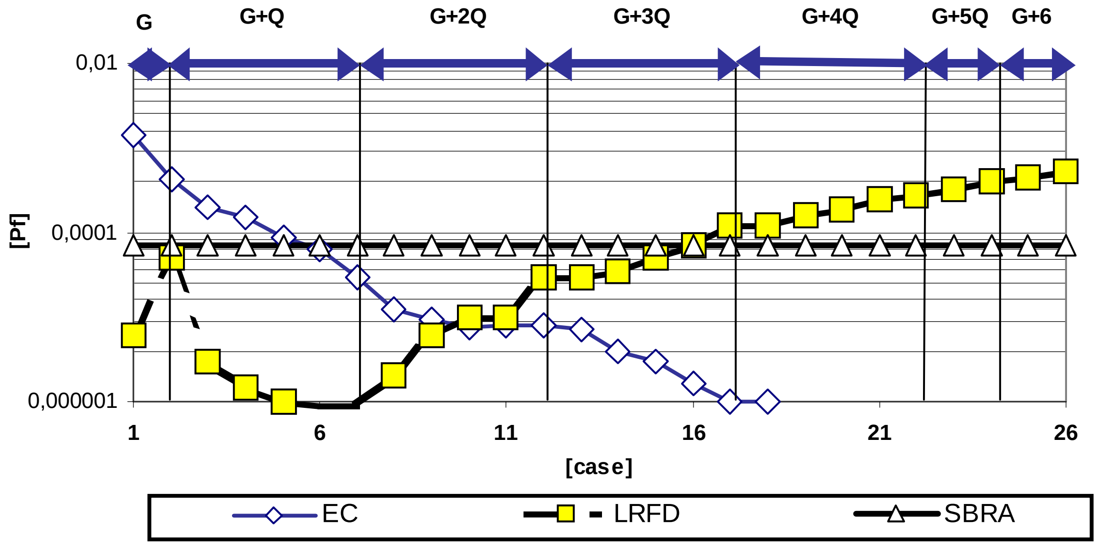
$$S_{EC} = \sum_j \gamma_{G,j} \times G_{k,j} + \gamma_{Q,1} \times Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \times \Psi_{0,i} \times Q_{k.i} \qquad [kN] \qquad /1/$$

Cross-sectional area A_{EC}, as determined by EC specifications [3] for simple bar in tension, can be expressed as $A_{EC} = S_{EC} \times 1{,}1/f_y$ where 1,1 is a safety factor and f_y = 235 Mpa is the nominal yield strength of S235 steel grade. The probability of failure P_f of the bars designed according to EC [3] is determined by program Anthill for Windows ™, which uses bounded histograms for representation of random variables.

Fig. 2 shows probabilities of failure P_f for 26 bars designed according to EC [3] and LRFD [1]. Differences between the P_f determined according EC and LRFD are more visible in cases with more loads. Significant differences in the probabilities of failure P_f between the bars designed according to LRFD [1] and EC [3] can be observed.



Characteristic load effects combination [kN]



Probability of failure Pf [..]

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Session 3

Two-component wind rosette application

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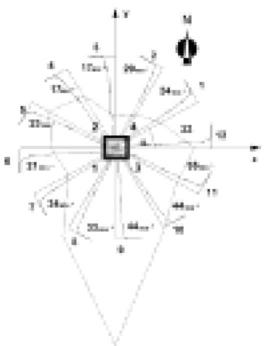
When designing building structure exposed to horizontal wind load, we usually keep this procedure: design it for one direction and then for the opposite direction. But as we know, the wind blows in all directions with different intensity and duration.



In this example I was engaged in the new wind load approach. This time the wind load is expressed by the two-component wind rosette [1] (see Fig.1), the first component (expressed by a histogram) refers to the duration of the wind in twelve directions and the second component consists of twelve load duration curves (expressed by histograms) corresponding to those directions. Meteorologic institution monitor these factors, so it enables us to use it practically.

Fig.1 - Two-component wind rosette

Imagine a square column footing of high construction (e.g. reservoir) exposed to a combination of axial vertical loads and horizontal wind load. The construction resistance is supposed to be the same in all directions. The aim was to determine the pressure under four corners of the column footing and to compare the results.

The result is the pressure under the four corners of column footing (see Table 1). On the target probability level P=0,999 the result pressure differs from more than 12%. This is already a significant difference. That is why this method could be profitably used in cases, where the wind load is very important and critical.

probability	0,999	0,9999
pressure under 1st corner [kPa]	200,8	237,9
pressure under 2nd corner [kPa]	211,8	341,3
pressure under 3rd corner [kPa]	222,2	461,8
pressure under 4th corner [kPa]	197,6	214,8

Table 1 - Results

[1] Marek, P., Brozzetti, J., Guštar, M.: Probabilistic Assessment of Structures using Monte Carlo Simulation. Background, Exercises and Software. Institut of Theoretical and Applied Mechanics, Academy of Sciences of the Czech Republic, Praha, 2001.

Session 3

SBRA method as a tool for reliability analysis of structural members exposed to multi-component load effects

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Current codes for design and assessment of building structures are mainly based on semi-probabilistic prescriptive interpretation of limit state method, so-called "partial factor method" (used for example in Eurocode and LRFD). These standards define the rules for determination of load effects using combination formulas. In case of multi-component load effects determination of the critical load effects combination according to these codes can be extremely difficult because of insufficient load description based on characteristic values and load factors only.

Enormous development of computer technology simultaneously with intensive research on the field of reliability allows for considering the transition to fully probabilistic concepts of design and assessment of building structures. Among these qualitatively new concepts can be also included SBRA (Simulation-Based Reliability Assessment) method, documented in [1], [2] and [3].

The simply and efficient way of determination of multi-component load effects using SBRA method can be demonstrated on an example of column exposed to several random variable loadings acting in various directions (see Fig. 1) [4].

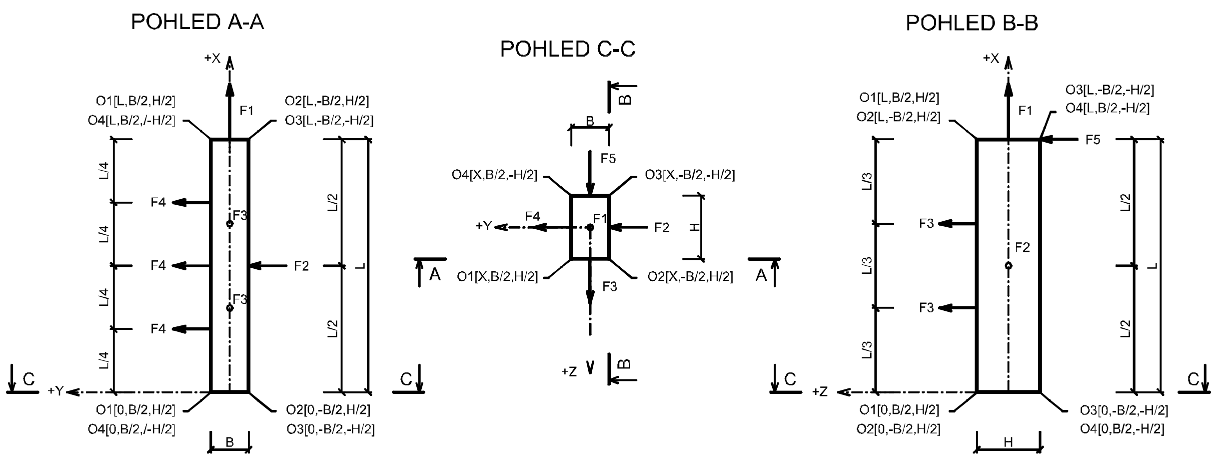

Fig. 1. Vertical column fixed in foot exposed to several random variable loadings

Resulting load effects of several random variable loadings (see Fig. 1) in particular point of structure can be expressed by normal stress σ_x according to formula (shear forces are neglected):

$$\sigma_x = \frac{N_x}{A} + \frac{M_y}{I_y} z - \frac{M_z}{I_z} y \,, \qquad (1)$$

where N_x, M_y and M_z are three components of internal forces. Mutual relation of these internal forces can be visualized by means of three-dimensional "AntHill" (see Fig. 2). To selected probability P and particular point of the investigated cross section of structural member corresponds a "plane" (each point of this plane represents one combination of three-component load effects, i.e., there exist infinite number of combinations corresponding to selected probability and to particular investigated location of the cross section.

The "plane" can be constructed using equation:

$$a \times N_x + b \times M_y - c \times M_z - d = 0 \,, \qquad (2)$$

where a = 1/A, b = z/I_y and c=y/I_z. The quantity d represents normal stress σ_x corresponding selected probability P. Using AntHill computer program the parameters of the plane can be obtained.

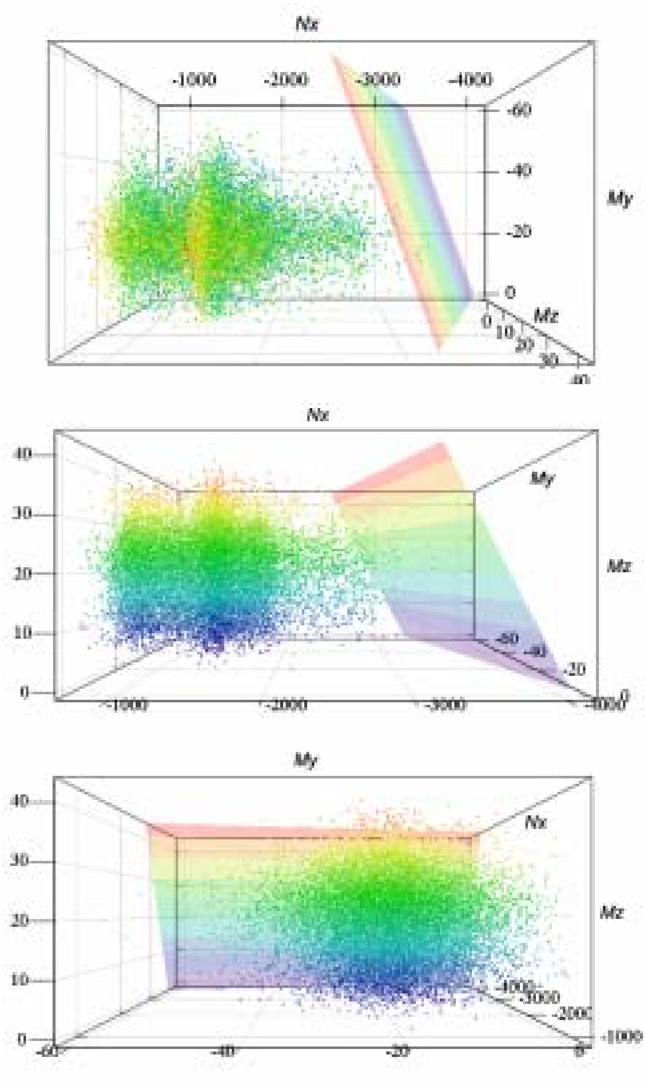

Fig. 2. Three dimensional „AntHill“ and „plane“ which expresses infinity number of combination of three-component load effects

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Session 3

Probabilistic modeling of loading, Combination of APT and MLE approach

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Loads acting on structures can be classified (especially according to these time duration) into three categories (for more detailes see [1], [2]).

Type I. Dead loads. The time dependance of these loads is not considered and it can be assumed, that it is possible to describe all of these components by corresponding distribution (most frequently normal distribution is used).

Type II. Live loads (random long-term and short-term). Load data in this category are obtained by measurements at determinate time interval. This type includes wind and snow loads, weight of people, vehicles, technological equipments, etc.

Type III. Extraordinary, extreme loads. These loads occur during extreme events, such as earthquakes, explosions, tornadoes. Their load effects last very short time.

The analysis of simultaneous load effects corresponding to aforesaid three types is possible by different approaches. Two conceptions are given bellow (a) APT (Arbitrary Point in Time), i.e. individual loads are expressed by Load Duration Curves – LDC, see [2], and for chosen "points in time" the combination of all load effects is defined. Histogram for combination of load effects is obtained by many times repeated random choise of loads, see SBRA method and examples in book [3]. It is appropriate to remind that this approache respects not only frequency of the various magnitudes of load during the life of structure, but also time intervals when load is zero. (b) Agreeably to conception MLE (Maximum Load Effect) the combination of effects of all loads is analyzed only in relatively short time interval which relates to the acting of extraordinary load, e.g. earthquake, while all other (conventional) loads are combined in accordance with conception APT.

Evaluation of the probability of failure P_f according to APT approach corresponds to probability of appearance of unfavourable events (i.e. excess of the reference value *RV* – that is failure) during the all live of structure. Conception MLE conduce to probability failure assessment in the spell when extraordinary load comes into being. It is necessary to emphasize that considerable different target probabilities match to conception APT and MLE, see e.g. [4].

In the current SBRA method all loads are considered to be time independent random variables expressed by bounded histograms and appropriate LDC which correspond to loads during the service of structure. Evaluation of the reliability of structure at arbitrary time point in the current SBRA correspond with APT approach.

Using APT conception in case of type III. extraordinary great intensity loading (e.g. earthquake) do not give, with respect to the substance of APT approach within the framework of simulation technique, quite suitable results (time duration of such loading is inconsiderable with respect to the return period of the earthquake) and consistent calculation would require implementation of outsize histograms and great number of simulation steps.

A new SBRA approach based on MLE is proposed in research report [5].The transition from conception APT to MLE constitute LDC and corresponding histograms, three of them see Fig.1, suggested for the seismic load effects and used in the course of probabilistic reliability assessment of unbraced steel frame, see [6]. The difference among histograms in Fig.1 is related to the actual ratio of the time interval the structure is exposed to the earthquake and the assumed total service life of the structure. When histogram Earth51 is used, this ratio is least, approx. 7.6% and for histogram Earth65 is this ratio greatest, approx. 97.5%. Range of histograms is ±1, when zero value is eliminated, most frequent values are around ±0.75.

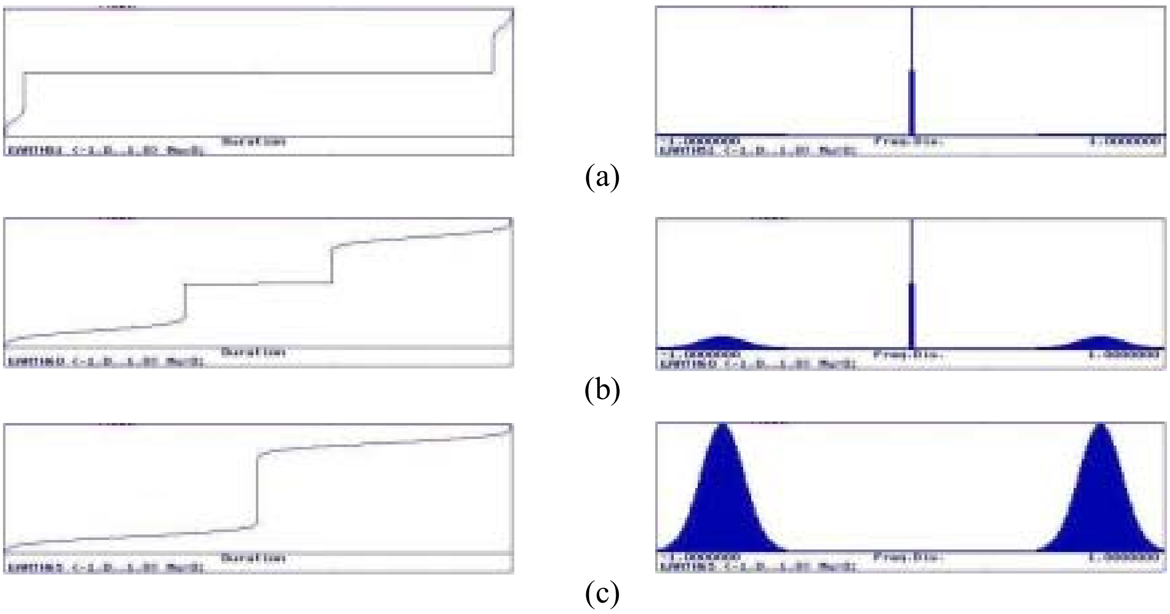

Fig. 1. Earthquake loading – Load Duration Curves and histograms
(a) Earth51, (b) Earth60, (c) Earth65

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Session 3

Accidental loads: codification of simulation-based procedures for predicting load characteristics

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The paper is devoted to predicting characteristics of accidental loads (actions). The attention is focussed on the problem of coping with scarcity of statistical information on occurrences of the loads and need to introduce in a formal way engineering judgement in the predicting. It is analysed how the predicting based on the scarce statistical information combined with the expert judgement could be arranged by applying codified procedures. Two simulation-based algorithms suitable to propagation of uncertainty related to the accidental loads are suggested as candidates to the codification.

The central idea followed in the paper is that the codification related to the accidental loads should regulate the quantification of uncertainty related to the accidental loads rather than cover a direct pre-setting the characteristics of the loads from available and, as a rule, scarce statistical data. The codification could be realised in the way that presets a coordinated applying procedures making use of the scarce statistical information and allowing eliciting expert opinions on the physical phenomena preceding imposition of the loads. What is more, the codification must regulate how to update the expert opinions when new statistical data becomes available.

The procedures suggested to the codification serve in essence to uncertainty propagation via simulation-based mathematical modelling the physical phenomena sequences of which lead to an imposition of an accidental load. The modelling allows to get simulated samples of characteristics of the accidental load and so to fit probabilistic models to the samples. Expert opinions are used in the procedures in the form of probability distributions expressing state-of-knowledge uncertainty in the physical phenomena, more precisely, mathematical models describing the phenomena. Mathematical framework for the elicitation and use of expert opinions is based on the Bayesian approaches to quantitative risks assessment.

It is stated that a practical application of the procedures can require to employ experts from different areas of engineering and a codification is needed to gather information from the experts and to propagate the information up to the characteristics of the accidental loads. The Monte Carlo method, more precisely, stochastic simulation is suggested in the paper as the best mathematical tool for the propagation.

Session 3

Modelling of More-Component Loads
Example – Crane Girder in an Industrial Building

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Crane girders in industrial buildings, as shown in Figure 1, must be designed and assessed with respect to safety and serviceability. In the safety assessment two main criteria apply: safety regarding strength and safety regarding fatigue. The load (crane, trolley and payload) can be expressed by three component representation (see the three histograms shown in Figure 1). Such representation, using simulation-based approach, allows for assessment the safety related to the carrying capacity as well as the determination of safety related to the remaining fatigue life.

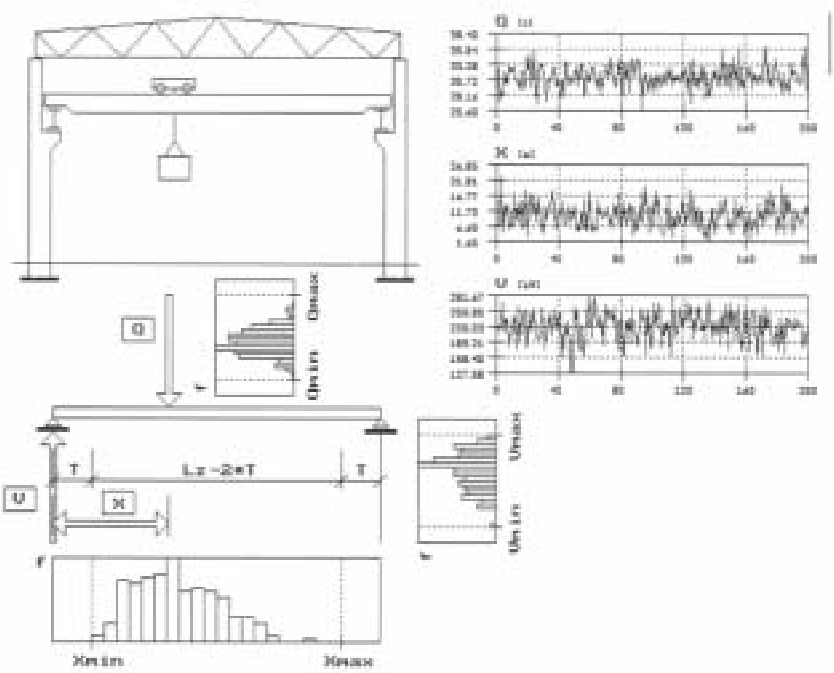

Figure 1 Three-component loading of a crane girder.

For more details see Hudak J. - Chapter "11.3 - Expert Opinion" in textbook Simulation-based Reliability Assessment for Structural Engineers (Marek P., Guštar M., and Anagnos T.), CRC Press, Inc., Boca Raton Florida, 1995.

Session 4

Reference Values for Durability Based Performance Design Criteria

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Defining the reference value (RV) for most durability parameters is a difficult task that requires detailed knowledge of the environment of the structural element, material science, and deterioration mechanisms. Determining the probability that a specific structural element will not perform the design function because of a single durability parameter is also difficult, if not improper, when determining the failure of a structural element or system. The multitude of deterioration mechanisms that contribute to potential safety or serviceability related failure is a compounding factor in the definition of reference values for durability based performance design.

By its very nature, most durability-based deterioration is time dependent. Since durability related distress is often a function of exposure, time, material properties and acceptable performance, it is necessary to understand the nature of each and its interrelationship with other parameters. Exposure can be thought of as a load effect, and specific material properties can be considered as resistance effects. However, exposure is limited or elevated by material properties, and material properties can change over time due to exposure. Therefore the load, i.e. exposure, can be function of resistance, and the resistance can change with time. For example, corrosion of steel in a structure is a function of access to moisture, air, salts, and galvanic current. This is further complicated in reinforce concrete by the passivating effect of high pH pore water solutions.

In addition to defining reference values for any particular deterioration mechanism, there is a need to understand that one deterioration mechanism can accelerate another mechanism, or several combined factors may increase exposure and the probability of failure. For example, the magnesium sulfates in seawater may cause sulfate attack in concrete structures. These exposed structural elements become cracked and the chemistry of the pore water changes from its exposure to seawater, as which time the ingress of chlorides enter the concrete to further decrease the passivity of the pore water, as well as strengthen the electrolytic solution, by which corrosion is accelerated. The safety related distress is the loss of reinforcing due to corrosion, but it would not have been happened in the given time period had not the sulfate attack accelerated the process.

With this type of interaction in mind, the technical and design community must consider the various forms for durability related distress and determine the acceptable levels of deterioration. The acceptable level of deterioration may be different for different structural elements and applications. If may also vary with the desired time in service. Therefore the probability of failure must be considered as part of a life-cycle design philosophy. The designer will have a make a realistic estimate on service life. Short service life buildings (electronics manufacturing facilities, suburban retail space, etc.) may be considered in a different class than long-life structures (highway bridges and government buildings, etc). A third class of structures for ultra-long life might be considered for special structures (nuclear waste repositories, monuments, etc.).

A more comprehensive set of criteria is needed to define the desired durability properties for structural elements. For reinforced concrete structures with would include: freeze-thaw resistance, scaling resistance, permeability, abrasion resistance, resistance to alkali aggregate reaction, shrinkage, corrosion potential, autogenous and external thermal properties, and sulfate resistance. Bacterial and fungal resistances have to be added for wood structural elements. Coating quality and thickness have to be added for steel structures.

Finally, material properties and laboratory testing must be adequately tied to field performance and deterioration for reference values to have real meaning. While the building codes have spent years perfecting the reference values for strength related issues. Serviceability and durability reference values are much more difficult to define and will require considerable effort to incorporate into probabilistic design.

An acceptable model and methodology is needed to define the probability of failure for durability related distress. The models must be based on material science and verified by field performance. The combined materials science and field performance will lead to reliable predictors of high level of performance. Only after these steps are completed can design engineers be expected to complete true life-cycle designs based on probabilistic reality.

Session 4

Reference Values (Safety, Serviceability and Durability)

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Modern methods for reliability assessment use the concept of limit states, which form the boundary between the ability and inability of a component or structure to fulfil particular function. Two principal groups are those of serviceability and of load carrying capacity.

Exceeding of load carrying capacity (ultimate limit state) means the collapse of a structure. Thus, this limit state is related to safety. The main kinds are: fracture due to exceeding the strength of an important cross section, brittle fracture due to exceeding the fracture toughness, fatigue fracture, plastic collapse, loss of the shape stability, loss of static equilibrium.

Exceeding of serviceability limit states usually means significant worsening or impossibility of performing some functions. These states are related to the following kinds of failures: unacceptably large deformations, unacceptable dynamic response, unacceptable degree of damage (e.g. due to corrosion or fatigue or permanent deformations after overloading).

Limit states can be associated with reference quantities and reference values. While limit state characterizes the possible failure in a qualitative way (for example plastic collapse), reference quantity characterizes the resistance R of the structure in a measurable way (e.g. bending moment), and the reference value RV is the value of R, whose exceeding means a failure.

Reference values are usually associated with the structure. For serviceability limit states, it can be the value of a deformation, which makes some function impossible or strongly limited. In ultimate limit states, the reference value can be based on the characteristic stress, force, moment, stress intensity factor, or energy, whose exceeding could lead to a collapse.

Reference values can be elementary (e.g. yield strength) or compound, based on several variables (e.g. bending resistance moment, depending on the yield strength and the shape and size of the cross section). They can be single, defined by one number (e.g. allowable deflection of a beam) or blurred, characterized by probability distribution (e.g. strength).

Many quantities, which have influence on the performance of a structure, have random character. It is therefore reasonable to use probabilistic simulation methods for reliability assessment (e.g. Monte Carlo), and to consider the reference values as those, which may be exceeded with some small probability. Actually, **in reliability assessment the probability of failure is not necessarily identical with the probability of actual failure or collapse; it just corresponds to the probability of exceeding the reference value.**

Reference values can be divided into three groups according to the situation, at which they can be reached or exceeded, and to the consequences of their exceeding:

1) Reference values corresponding to the common use of the structure. They should not be exceeded, but small exceeding does not cause big problems or permanent damage.
2) Reference values corresponding to extreme situations, such as overloading due to a hurricane, earthquake or another accident. Here, permanent damage arises, and a small exceeding of RV can mean significant increase of damage or even the collapse.

3) Values corresponding to some degree of gradually increasing damage, reasonable for repair. In this case, the damage (e.g. wear or fatigue) usually changes slowly with time in the considered interval, and small exceeding of RV does not cause big problem.

Usually, there are several limit states for one structure, with various reference values and allowable probabilities. The general condition for reliability check is $P_{f,d,i} < P_{f,a,i}$, and must be fulfilled for each failure mode. $P_{f,d,i}$ means the calculated design value of failure probability for the *i*-th mode, and $P_{f,a,i}$ denotes the allowable or target value of P_f . The lowest allowable probability of exceeding a reference value corresponds to the case 2. Higher probabilities pertain to the RV for common use (case 1) and to the case 3.

In design, therefore, all possible failure modes and their consequences should be found first. Then, reference values can be assigned to each failure mode. It is reasonable to know generally the behavior of the structure in the relevant range of input parameters. This makes possible to relate the reference values to the probability of their exceeding. In some cases, criteria based on minimum life cycle costs (including the failure related costs) can be used.

According to the common LRFD codes, a structure is assumed safe if the design value of a quantity characterizing the load effects is lower than the design value of the corresponding quantity for the resistance of the structure. Nothing is said about the actual degree of safety. However, the current codes for steel structures use not strength, but yield stress as the limiting material parameter. For bending, the codes allow creation of plastic hinges, but only as many as the structure is statically indeterminate. For a statically determinate structure, the codes demand that the highest stress must not exceed the design value of the yield strength. Thus, there is some - not exactly specified - safety margin in the codes.

Simulation-based reliability assessment respects the random character of load, material and geometrical properties, as well as of the influence of environment, and can describe the actual safety in a quantitative way, by means of failure probability. Nevertheless, also with this approach it would not be wise to consider the design as suitable just if the calculated probability of total collapse is negligibly low. The reasons are the following uncertainties: 1) simplifications of the computational models and errors in the calculations, 2) limited knowledge about the actual course of the collapse, 3) some quantities are not exactly known or can change during the life of the structure, for example the traffic load.

With this in mind, it seems reasonable to use also in SBRA some reserve between the actual ultimate state and the ultimate state assumed in our calculations. The less information we have about the load, environment, and material or component behavior, the more distant should the reference value be from the value corresponding to the (assumed) actual collapse or failure.

Thus, there are several questions, which could be discussed at this Colloquium:

1. **What do we know about actual conditions for collapse (e.g. plastic one) of structures or components?**
2. **What values should be considered as reference values (RV) for ultimate limit states in reliability assessment? How to choose a safety margin between the state of actual collapse (often not well known), and the "ultimate" limit state used in safety check?**
3. **Is the term "ultimate limit state of load carrying capacity" appropriate in reliability and safety assessment? What about using the term "limit state of safety" instead?**

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Session 4

Reliability assessment of composite steel concrete cross section of roadway bridge

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In accordance with today's national bridge standardes and corresponding Eurocodes the design bending resistance may by determined by plastic theory where the effective composite section is in Class 1 or Class 2. It allows for the benefit from existing plastic reserves. Then the cross section should be verified separately at the ultimate limit states – safety assessment (effects of temperature, reological concrete changes and assembling may be neglected) and at the serviseability limit states – limitation of stresses (effects of temperature, reological changes and assembling shell be taken into account, including elastic behaviour of steel section).

On the optimized real bridge composite cross-section, see Fig. 1 and 2, representing roadway composite beam of the span 34,7m, comparison will be made between mentioned conventional method and fully probabilistic SBRA method (Simulation Based Reliability Assessment) based on the application of Monte Carlo simulation technique [1]. The marginal fibres of steel section will be observed.

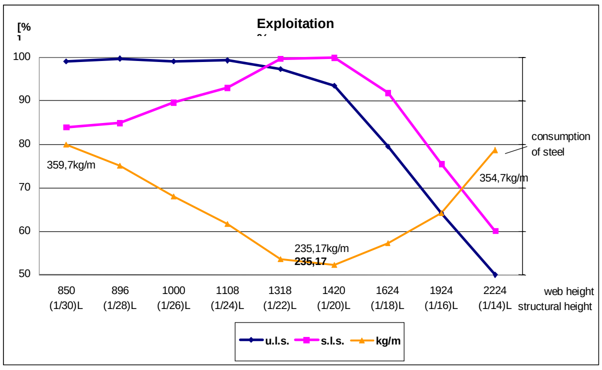

Fig.1: Exploitation of cross section

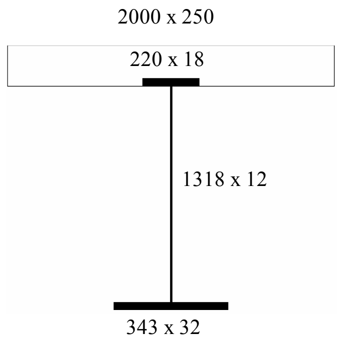

Fig.2: Verified composite cross section

The analysis based on fully probabilistic concept is performed using computer program M-Star™ [2] and realized for both elastic and plastic theory of behaviour. Output of this program is the probability of failure P_f, safety function is expressed by histogram, see Fig. 3 and 4. Obtained probabilities of failure P_f are then compared with target probability of failure P_d according to ČSN 73 1401 [3].

Fig. 3: Probabilities of failure according to elastic theory

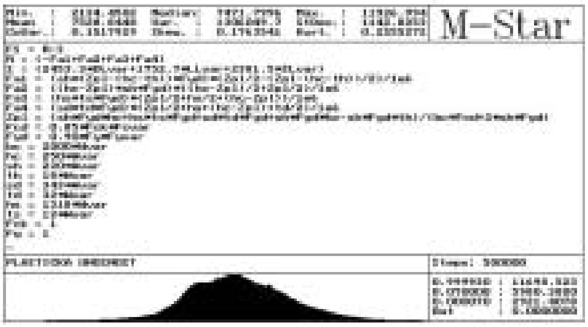

Fig. 4: Probability of failure according to plastic theory

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Session 5

From Simple Structures to Systems

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The application of SBRA method has been so far limited mostly to structural elements, components, members and simple systems. The subject of this session is to discuss the main challenges related to expansion of SBRA to complex, possibly higly nonlinear structural systems, such as

- What is the specific structure of mathematical problems that may be exploited in implementation of the Monte Carlo methods?
- How to exploit the specific structure of mathematical problems (such as solution of meny similar problems) that must be solved in implementation of the Monte Carlo methods?
- What is the impact of recently developped algorithms (such as scalable solvers for solution of some problems of computational mechanics) for Monte Carlo simulation?
- How to exploit effectively modern computational facilities, in particular fast parallel computers?
- What is the outlook for Monte Carlo simulation of complex dynamic non-linear structures?
- How to implement efficiently the Monte Carlo simulation (variance reducing schemes atc.)
- How to implement stochastic analysis of continuum?
- What has been done in these lines?

Session 5

APPLICATION OF MODERN SIMULATION METHODS IN THE TIME DEPENDENT PROBABILISTIC RELIABILITY ANALYSIS OF THE GEOMETRIC AND PARAMETRIC NONLINEAR LARGE SPAN CABLE AND COMBINED STRUCTURES

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1 Present state of the art

At present the stochastic approaches in the reliability assessment of structures are developed. The conduct of building structures in course of operation is described as depending on the factors of random nature. Construction materials possess properties of statistic variability in a certain time period. Loads on structures represent random processes unfolding during a certain period of time.

Modern codes of practice and design of building structures take into account probabilistic nature of loads and of carrying capacity of constructions only on the level of initial data treatment. Limit state design method used in codes of design is semiprobabilistic and structure reliability is ensured by partial coefficients – coefficients of reliability for different loads and materials, model coefficients, coefficients of liability, whose values have no sufficient theoretical and experimental basis. Building structures design reflecting their real conduct in course of operation should be totally based on the theory of reliability taking into account probabilistic methods which give a more unbiased evaluation of the structure and of its fittingness to the expected exploitation. The development of computer technics allows to change the approach in probabilistic design and to abandon traditional statistic methods of structural mechanics. Modern quick computers give the possibility to automate complex calculations, which allows to use in course of reliability calculation the modern numerical statistic simulation methods. Their most valuable characteristics are simplicity, absence of any limitations concerning the nature of initial statistic information, as well as the possibility of using real data (experimentally obtained physical, geometrical and load parameters of the structure etc.) without making preliminary theoretical probabilistic models of stochastic factors taken into account in calculations. This opens large opportunities to use probabilistic methods of reliability based design.

In the time dependent reliability analysis of innovative large – span combined structures with cable elements are the input parameters random time variables, thus we speak about the random processes, which are statistically dependent. The time dependent random processes are creep strain increments from longterm load (prestress of structure) and corresponding decreases of a cable modulus of elasticity. Explicit determination of random variability mentioned relevant quantities is not simple, because they are possess statistic uncertainities.

A response of the cable and combined structures in an exploitation regime can be characterised by finite number of independent, and/or dependent random variable parameters, including load, material and geometrical characteristics, imperfections, etc.

At present the probabilistic approaches in the reliability assessment of elements and structures are developed, together with their implantation into FEM. The progress in the use for simpler structures and elements reached the Monte Carlo simulation method. For complicate innovative large – span cable and combined structures, with parametrical and geometrical nonlinear rheologic properties, the analysis needs repeating solution of large number of equations. In these cases is the application of Monte Carlo method not effective, according to the large number of necessary simulations. One of the ways how to reduce number of simulations is the application of effective numerical simulation methods, which are based on stratifying of the input random variable quantities hyperspace, which are characterised by the distribution functions.

Statistic assessment of high - strength one - strand cables random variable rheologic quantities were not sufficiently realised till now.

Considerable attention has recently been devoted to the study of the order in chaos of variable response of dynamic systems, by application of the chaos theory.

The investigation of the rheologic characteristics of steel cables under variable – increasing and decreasing impulses has the practical importance in the case of the modelling of stochastic loading effects and their influences on the response parameters. The load history is transformed on the stress history, and/or strain history and a nonlinear creep or relaxation theory is applied.

2 Particular contribution expect

The application of modern simulation methods in the time dependent probabilistic reliability analysis of the geometrical and parametrical nonlinear large span cable and combined structures with stochastic rheological properties

By application of the modern simulation methods and the discrete transformation approach the model for time dependent probability reliability analysis of the geometrically and parametrically nonlinear large – span cable and combined structures with the stochastic rheologic properties will be derived.

The approach for probabilistic time dependent serviceability analysis (analysis of the required stiffness in a needed time period) of cable and combined structures with deterministic limitation will be formulated.

The application of the Monte Carlo method in reliability investigation of cable structures with response characterised by transformation models in the close analytical forms

By application of the Monte Carlo simulation method the simple practical approaches for probabilistic reliability evaluation of the cable structures, with response characterised by the transformation analytical models in close form will be derived.

The random variable rheologic quantities – statistic evaluation

The statistic evaluation of the random variables of rheologic material high strength single strands cables quantities (cables used in civil engineering) will be performed and their suitable distributions will be formulated. The random variable load and geometric characteristics (the initial imperfections and their time development) of the investigated large – span structures will be analysed.

3 Proposal of the ways to reach the goals

The application of modern simulation methods in the time dependent probabilistic reliability analysis of the geometrical and parametrical nonlinear large span cable and combined structures with stochastic rheological properties

By application of the modern simulation methods, like the methods LHS (Latin Hypercube Sampling), ULHS (Updated Latin Hypercube Sampling) and MCSS (Monte Carlo Stratified Method), which are based on the selection space of distribution functions divided into layers and on the application of the discrete transformation computational approach, based on the FEM and their implementation by object oriented programming into software LANSTAT, the model for a time dependent probability reliability analysis of the geometrically and parametrically nonlinear large – span cable and combined structures with the stochastic rheologic properties will be derived. Because the software LANSTAT, created during the last solution of the scientific project supported by Scientific Grant Agency, is the open – flexible dynamical system, it is possible its gradual expansion in the form of the implementation of the files with the simulation methods suitable for a time dependent probability reliability analysis of the geometrically and parametrically nonlinear large – span cable and combined structures with the stochastic rheologic properties.

The application of the Monte Carlo method in reliability investigation of cable structures with response characterised by transformation models in the close analytical forms

By application of the Monte Carlo simulation method the simple practical approaches for probabilistic reliability evaluation of the suspended one - chord and double - chords prestressed biconvex and biconcave cable structures, with response characterised by the transformation analytical models in the close forms (the analytical computational models were derived during the solution of the scientific project supported by Scientific Grant Agency in the past periods), will be derived.

The random variable rheologic quantities – statistic evaluation

The statistic evaluation of the random variables of rheologic material high strength single strands cables quantities (cables used in civil engineering) is based on the large files of measuring – the creep tests of the investigated cables during the successful solution of the scientific projects supported by Scientific Grant Agency in the past periods.

From the point of a probability approach in the reliability analysis of large – span structures the estimations of statistic rheologic characteristics of the cables structural elements, like the forms of histograms with corresponding density distribution curves theoretical approximation functions with equivalent distribution functions, fractils etc. for the relevant random variables, which are • creep strain increments as the stress and time functions under one step and multi step load levels, by means of the application of a nonlinear creep theory, • decreasing of the modulus of elasticity values in a dependence of current stress, strain and time level by means of the application of a nonlinear creep theory, • the influences of a corrosion on a cable cross sectional reduction in the dependence of the degree of an environmental aggression and time, will be necessary to investigate. The random variable load and geometric characteristics (the initial imperfections and their time development) of the investigated large – span structures will be analysed.

Session 5

Reliability analysis of statically indeterminate steel frame (pilot study)

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Extensive development of computer and information technology together with advances achieved in the field of structural reliability assessment allow to consider the transition from current methods applied in codes to fully probabilistic concepts based on simulation technique.

One of such method, SBRA (Simulation-Based Reliability Assessment method [1], [2], [3]) is based on application of the direct Monte Carlo technique and limit states philosophy. Following pilot study indicates the application of SBRA in case of a frame.

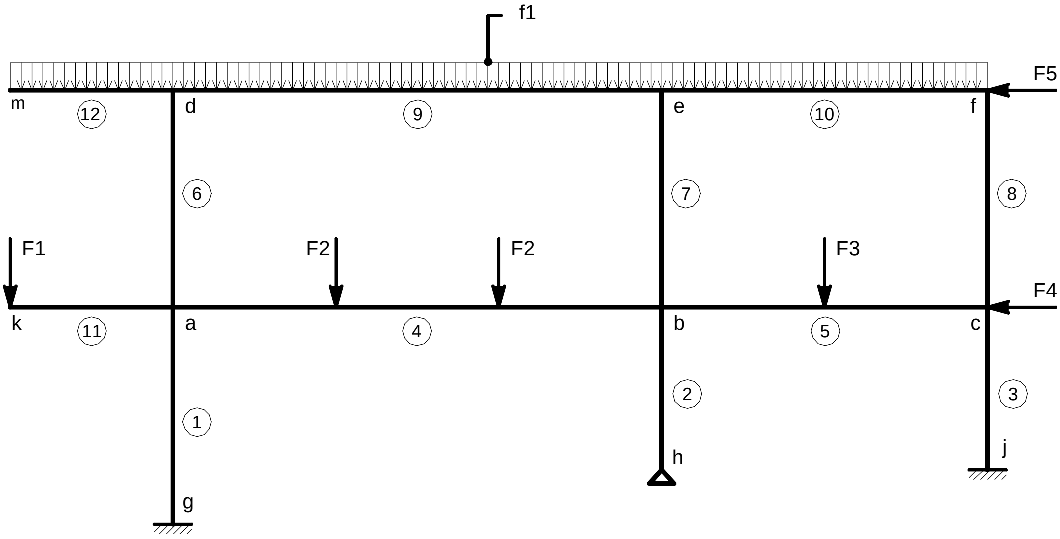

Fig. 1 Scheme of a planar steel frame

The potential of this method can be demonstrated on an example of statically indeterminate planar steel frame exposed to several random variable loadings (see Fig. 1) [4].

The loads are represented by histograms corresponding to Load Duration Curves.. The resulting response of the frame to the load combinations obtained according to the SBRA method is in Figure 2 expressed by scatter of dots ("Anthills") corresponding to bending moments, shear forces and normal force. Such representation of load effects combination allows for calculating the probability of failure related to critical cross sections.

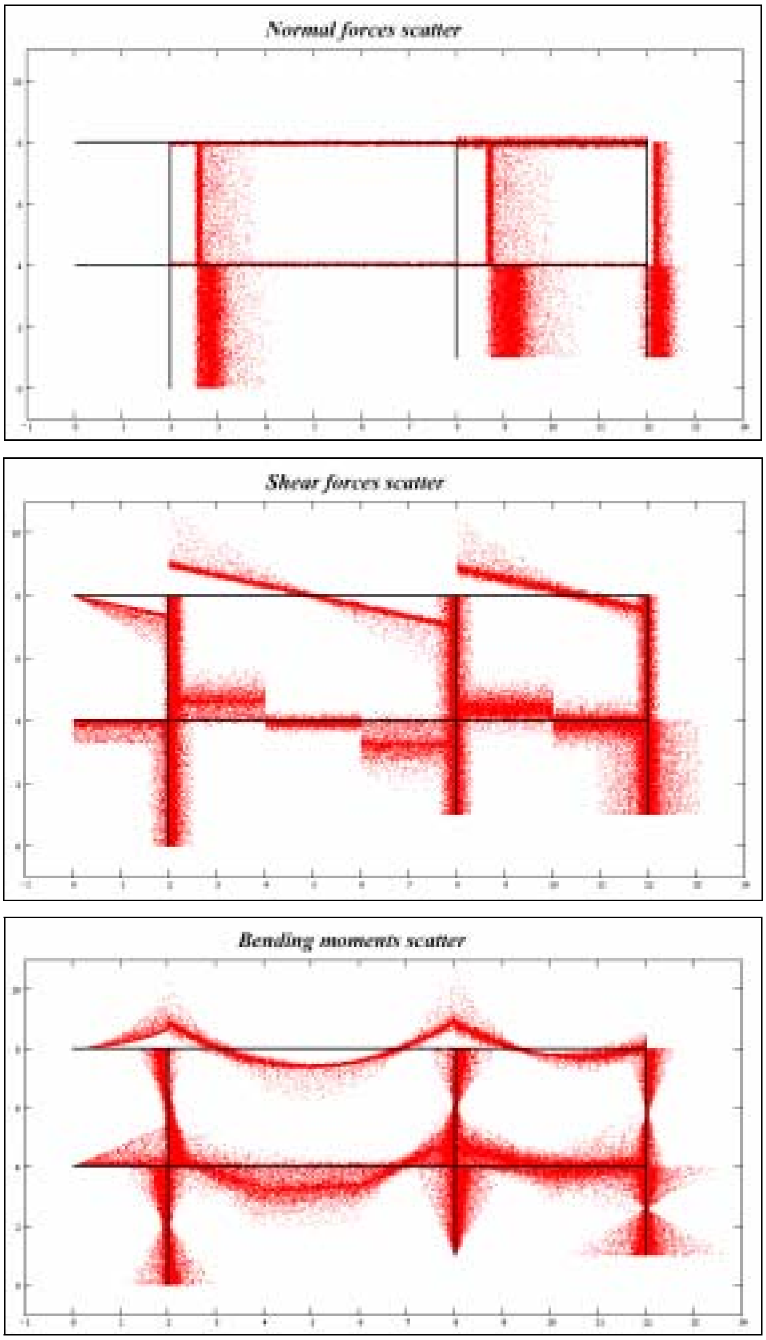

Fig. 2. Scatter of normal forces, shear forces and bending moments

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Session 5

Digital simulation algorithms for second-order stochastic processes

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Time dependent reliability assessment is expected to grow in importance with the gradual evolution in design philosophy towards performance-based design (PBD). Existing implementations of simplified reliability-based design (RBD) procedures in the form of multiple-factor formats do provide a familiar "look and feel" but become increasingly unwieldy to apply as the degree of realism in analysis and design increases. Simulation-based reliability assessment is most likely to form the core component in the next stage of design code development for two reasons. First, it is capable of addressing complex nonlinear time-dependent design issues in PBD. Second, it benefits directly from Moore's Law which states that computer speed will double every 18 months. Desktop computers with gigahertz processors, gigabyte of memory, and hundreds of gigabytes (verging on terabyte) of disk are already used on a routine basis by practicing engineers for deterministic analyses.

For time dependent reliability assessment using simulation, one of the main challenges is to develop efficient computer algorithms that can generate realistic sample functions on a modest computing platform. This task is much more difficult than the simulation of random variables because the sample functions must produce the prescribed covariance function besides producing the target marginal distribution. To ensure sufficient generality for practical applications, the simulation procedure should be capable of handling: (a) stationary or non-stationary covariance functions, (b) Gaussian or non-Gaussian marginal distributions, and (c) short or long processes, in a uniform way. Aside from theoretical consistency and generality, computational efficiency is a key practical concern because the typical practicing engineers will not have access to computational resources beyond a desktop PC.

This paper presents a suite of algorithms that are capable of generating realistic sample functions in a theoretically consistent and computationally efficient way for applications involving time dependent reliability assessment. The theoretical basis of the procedure is Karhunen-Loeve (K-L) expansion that provides a second-moment characterization of random processes in eigen-space. This expansion has been commonly thought to be of theoretical interest only because eigensolutions are difficult to compute numerically for many covariance functions. A simple numerical algorithm will be described to demonstrate that many eigensolutions can in fact be obtained cheaply and accurately using the discrete wavelet transform. When applied in conjunction with the K-L expansion, this simple algorithm is capable of simulating stationary and non-stationary Gaussian processes over any finite interval. The Gaussian assumption can be relaxed by a second algorithm involving interative mapping.

This paper casts a new perspective on the potentialities of the KL expansion for practical simulation, which has been previously perceived as being useful only in stochastic finite element applications.

Session 5

Unbraced frame as simple series system

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The unbraced planar frame, see Fig. 1, consists of one fixed (cantilevered) column supporting three leaning columns. Upper ends of columns are joint by three crossbars. Steel structure is loaded by vertical and horizontal forces. Horizontal forces are carried only by fixed column *1*. If fixed column *1* fails, collaps of all structure occure, structure behaves as series system [1].

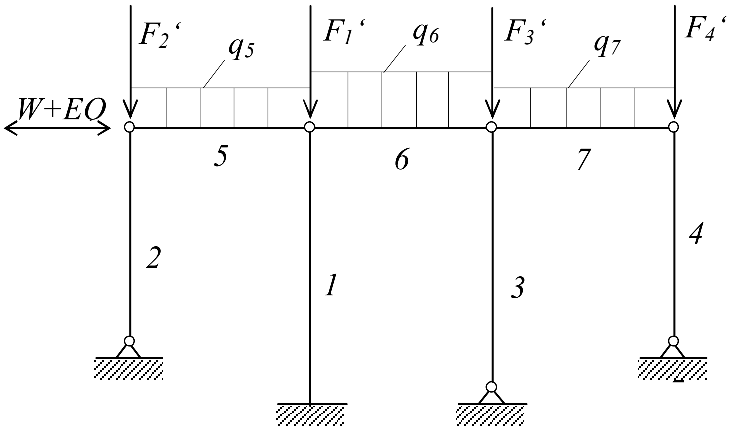

Fig. 1 Unbraced frame with leaning columns

But failure of elements *2,3,…,7* need not necessary cause failure of the whole system.
Scheme of the deformed loaded structure which is shown in Fig. 2 serves to the calculation

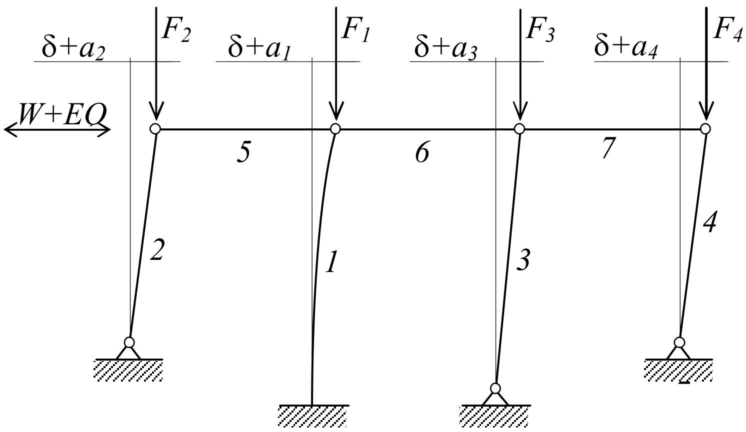

Fig. 2 Scheme of deformed loaded structure

of the stress and strain response of fixed column *1* to external loading of the structure. All vertical forces (F_1, F_2, F_3, F_4 or F_1',F_2',F_3',F_4', q_5, q_6, q_7), horizontal forces *W* (wind), *EQ* (earthquake) and initial geometrical imperfections a_1, a_2, a_3, a_4, influence the stress in the critical cross section of fixed column *1*, as well as the horizontal displacement δ of its upper end.

Each of vertical forces consist of three parts (dead, long-term, short-term). Force *EQ* depends on the magnitude of vertical loads in the instant of earthquake. All aforesaid input values (as well as for example cross-section characteristics) are random variables, statistically independent (except *EQ*). The calculation of the probability of failure of column *1* (element of system consisting of seven elements), in case of such number of mutually uncorrelated random input variables, is possible to execute for example by using SBRA method, see [2],[3]. Input variables are in SBRA concept expressed by means of bounded nonparametric histograms and their combinations simulated using direct Monte Carlo method.

When the limit of the resistance of the fixed column *1* is defined by the onset of yielding in the critical (fixed) cross-section, it is not necessary to make FEM model of the structure. On the bases of the second order theory can be derived analytical transformation model, see for example [4]. The histogram of loads effects combinations could be set by multiple repeated using of the transformation model (up to millions simulations) with the aid of SBRA. The reference value, yield stress of the material of the fixed column, enters into calculation as the random variable too.

As horizontal force, and especially extraordinary force *EQ* – earthquake, is carried by fixed column *1*, it seems to be suitable to make the probabilistic reliability assesment of this column by means of MLE (Maximum Load Effect) approach, see [5],[6].

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Session 6

Time-Dependent Reliability Analysis Using Simulation Techniques

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Time-dependent reliability analysis problems, especially system-level reliability analysis problems, have been difficult to solve using analytical FORM/SORM type methods. Therefore, Monte Carlo simulation-based methods have been pursued for various time-dependent reliability problems. Several methods have been investigated during the past decade to improve the efficiency of Monte Carlo simulation, especially for events with very low probability. Adaptive importance sampling has been effective in this regard. Both uni-modal and multi-modal sampling techniques have been pursued. The latter have been found to be particularly useful and flexible to handle a number of time-variant and time-invariant system reliability problems, including problems with multiple failure sequences.

The adaptive simulation technique has been applied to several large redundant structural systems with both ductile and brittle behavior, for both time-dependent and time-independent problems. These include building frames, transmission towers, tethered satellite space trusses, automotive structures, and aircraft fuselage panels with multiple site damage under corrosion fatigue. Various strength, buckling, and fatigue limit states are considered for system reliability analysis. The convergence properties of the adaptive sampling technique show that the method has significant promise for practical applications.

Session 6

Durability Assessment of Structures

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Reliability assessment requires the use of limit states to define the "loading" for a individual or system of elements; and the "resistance" of the material to the particular type or combination of loadings. In the case of durability-based reliability assessment the "loading" is not necessarily a mass or stress, but rather an exposure or a strain. These exposures and/or environmental strains must be defined by the design engineer in terms of chemical concentrations, temperature differentials, humidity, biological exposures, etc. Each of these exposures is a function of time.

Exceeding of "exposure" capacity (ultimate limit state) means both the static exposure condition and time of exposure has been exceeded. This limit state is directly related to serviceability, but often affects the load carrying capacity. When durability-based resistance is reduced, it is typically accompanied by a reduction in the physical properties that are considered in a strength related reliability assessment. The exposure may reduce fracture toughness, cross section, fatigue resistance, stability, or loss of static equilibrium. In the case of durability, exceeding a serviceability limit state may trigger a repair, restoration or rehabilitation action. This limit state may be defined in terms of the limit beyond which time alone will deteriorate structure or present the impossibility of performing some functions. The difference in ultimate limit state and serviceability limit states in durability assessment is not a clear line.

Limit states exist in series with resistance characteristics. Different levels of exposure have different consequences with different materials. For example, soils or ground water with a sulfate concentration of 500 ppm can deteriorate a normal portland cement concrete foundation, but have little effect on a low tricalcium aluminate cement, or one containing modest amounts of fly ash. The assessment of durability can lead a designer to limit exposure or improve resistance. The reference values for acceptable concentrations change with the choice of materials or the choice of materials may change with exposure concentrations. In either case, the designer has substantial latitude to improve the probability of acceptable performance with multiple solutions. The nature of this complex problem creates a rich application for simulation based solutions and design.

The more difficult task is to generate the variations in "loadings" or "exposure" as a function of both concentration and time. A static concentration variable creates a manageable variation in exposure that is only a function of time with a constant coefficient of exposure. However, in many systems, the concentration changes with time as well as the exposure effects with time.

The time-dependent effects have yet another consideration, the life-cycle of the structure or system. While we may be able to compute the expected exposure from the prediction of the change in concentration with time, C(t), and the effect of exposure with time, E(t), there is a terminal time T_t, by which the calculation is moot. This is the case where the design of the structure is durable throughout its expected life. The prediction and variation of T_t is a desirable target for simulation based design. The reference value may be the desired life of the building.

For each type of durability variable, C(t), E(t), and the variations in T_t must be computed and the compounding effects of each must be considered. Simulation-based reliability assessment provides for the random character of concentration, exposure, material properties and potential interactions in the environment, and can describe the actual durability in a quantitative way, by means of failure probability.

The questions that need to be discussed are:

1. How should interactions between different durability effects be addressed? What type of data is needed to develop reasonable models?
2. How can designers choose at terminal time, T_t, in a developing real estate or infrastructure environment that is designing for both obsolescence and sustainability?
3. Do sufficient time-dependent models exist, that could be brought into simulation based design programs, to assist designers in making life-cycle design decisions?

Session 6

Lifetime prediction of steel component exposed to time-dependent corrosion effects

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Extensive development of computer and information technology allows for considering introduction of qualitatively new structural reliability assessment methods in the designer's work and in codes. Among these new methods can be included SBRA (Simulation-Based Reliability Assessment method [1], [2], [3]) based on direct Monte Carlo simulation technique and limit states philosophy. Application of SBRA is subject of the following discussion focused on lifetime prediction of a steel components exposed to time dependent corrosion effects. The planning of inspections and application of the evaluated inspection data can lead to more precise estimate of the actual lifetime of the structure..

This approach can be demonstrated on example of simply supported steel beam exposed to time-dependent corrosion effects (see Fig. 1) [4]. The beam is exposed to the random variable loadings with time-dependent nominal values. Due to the time-dependent load effects and time-dependent corrosion effects the resulting probability of failure is time-dependent as well. The reliability of the structure according to the SBRA method can be performed using time-dependent safety function expressed by equation:

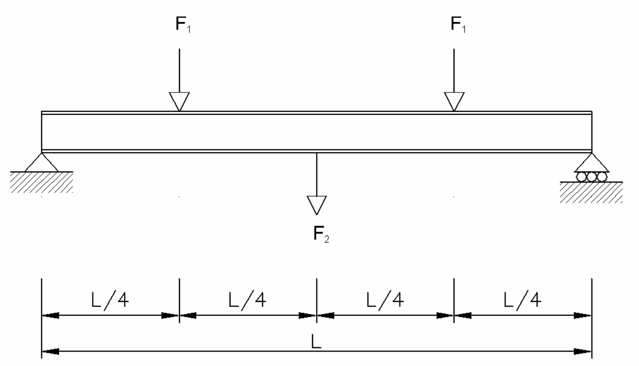

Fig. 1 Scheme of simply supported steel beam

$$SF(t) = R(t) - S(t)\,, \qquad (1)$$

where R(t) is time-dependent resistance of structure and S(t) is time dependent load effects. The interaction of R(t) and S(t) can be illustrated by using "Anthill" (see Fig. 2). The figure indicates the decrease of the resistance due to corrosion. In the example the load effects combination is also time dependent. Analysis of the safety function (1) leads to probability of failure in selected points in time. The reliability assessment is expressed by the criterion (2)

$$P_f[SF(t)] < P_d\,, \qquad (2)$$

where P_f is probability of failure and P_d is target probability given in codes.

The Fig. 2 express a-priori estimate of resistance and load effects. By considering inspection data, the corrosion model can be corrected and the reliability analysis improved in order to obtain more precise estimates of the lifetime (see Fig. 3).

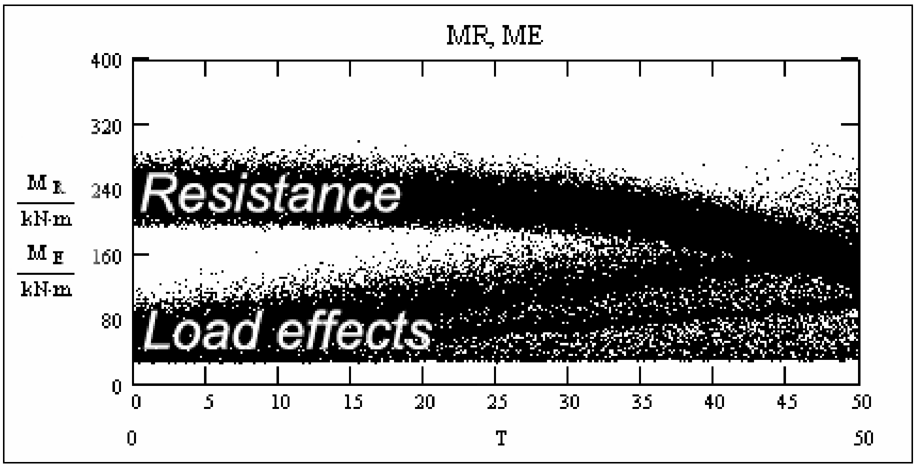

Fig. 2 The interaction of time dependent beam resistance R(t) and load effects S(t)

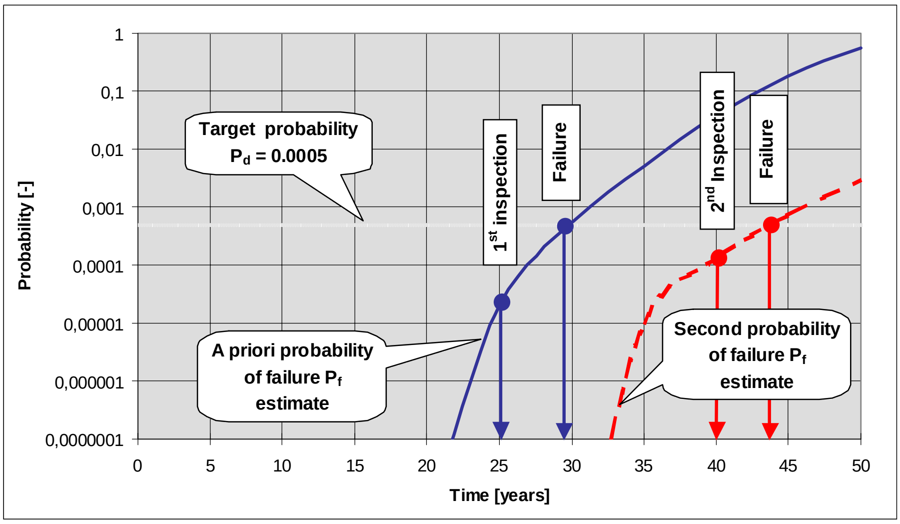

Fig. 3 Lifetime of the beam

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Session 6

SBRA Concept in Corrosion Modelling and Inspection Planning

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The paper turns attention to the application of the Simulation Based Reliability Assessment Method (SBRA) in case of a structural member exposed to tension and to corrosion. The probability of failure, as a function of time, is calculated using direct Monte Carlo simulation.

A simple structural member is chosen for the analysis – a steel bar is exposed to variable tension. The cross-section area represents the geometrical characteristic of the bar. The initial nominal value is A_{nom} = 450 mm^2 and the variation of the cross-section due to the rolling process in mill is assumed to be within the range ±10%.

The applied load consists of three components – dead load (*DL*), long lasting load (*LL*) and short lasting load (*SL*). Each load component is described by the extreme value and variable coefficient which represents the scatter.

The product of the yield stress F_y and area A expresses the resistance R of the bar:

$R = F * A * C(t)/1000$ [kN]

The yield stress F_y and area A_v are stochastic variables. Function $C(t)$ expresses the time dependent degradation due to corrosion. A simple model is taken into account. The function $C(t)$ is assumed in form:

$C(t) = 1 – K * (t/t_{tot})^n$

The coefficient K is considered to be variable. The exponent n is assumed as deterministic. Fig. 1 shows time dependence of resistance R(t) and load effect combination S(t).

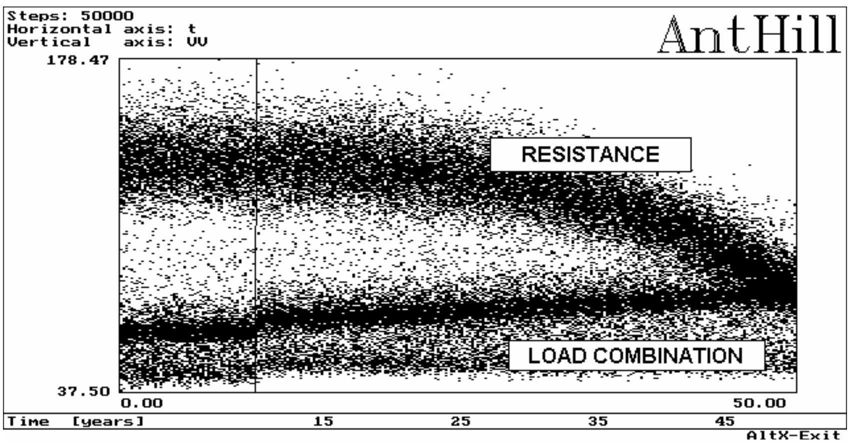

Fig. 1 Time dependence of load effect combination and resistance of the bar

The probability of failure is determined using a safety function $SF(t)$ defined as:

$SF(t) = R(t) - S(t)$

Functions $R(t)$ and $S(t)$ are resistance and load effects combination, respectively. The function *SF(t)* is analysed by program M-Star using the simulation technique. A distribution of the safety function $SF(t)$ is calculated for selected points in time.

The inspection intervals can be determined using the estimate of the probability of failure. If the value of *SF(t)* reaches certain value, the inspection would be performed. The value of *SF(t)* could be given for example by operator etc.

If the inspection provides information on rate of the corrosion process, the corrosion model could be corrected and new probability of failure *SF(t)* calculated. Following inspection of the structure can be planned in the same manner.

Fig. 2 shows time dependence of probability of failure. It is assumed, that the initial estimation of corrosion behaviour in this example is conservative.

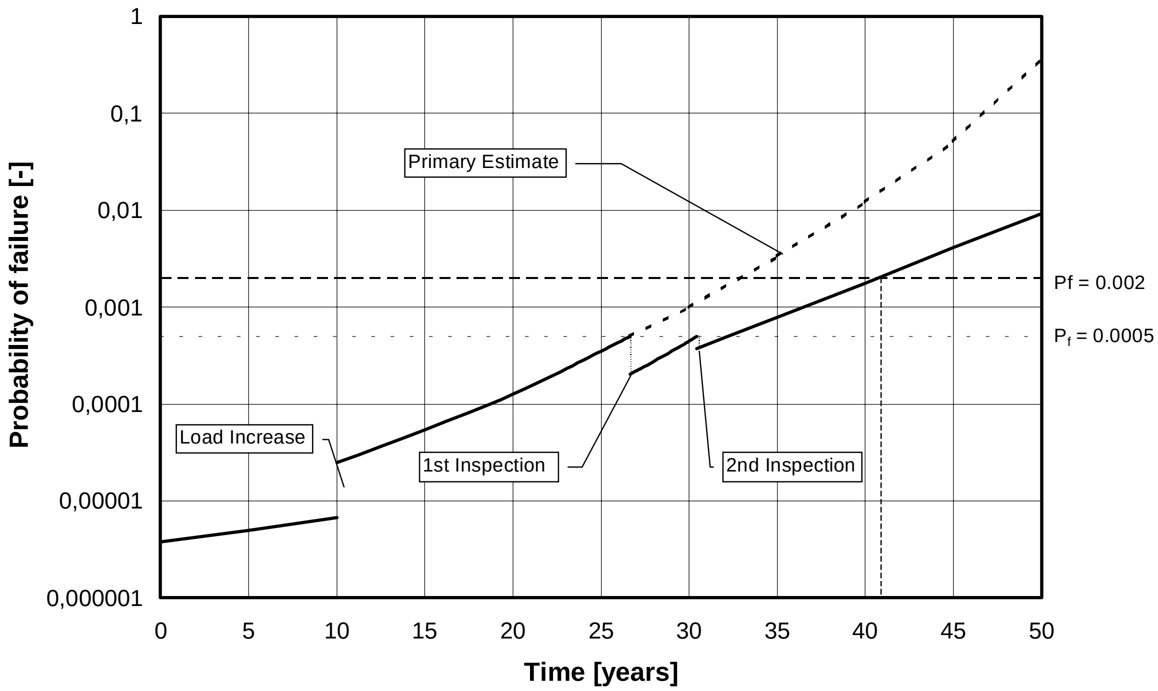

Fig. 2 Time dependence of probability of failure with corrections after inspections.

In the discussed example, only two inspections are considered. However, this procedure indicates a better understanding of the lifetime assessment.

Proposed procedure is strongly dependent on corrosion model. The sensitivity analysis should be performed in order to study different models. In the discussed example, a conservative inspection model related to the corrosion effects is selected. If, according to the inspection, the corrosion effect would be more intensive than it was expected, the application of several models may give a better understanding of the corrosion impact on the reliability and, for example, replacement of the structural member may be proposed.

Session 6

Peculiarities of safety assessment of steel-to-timber joints

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Using of wood structures in construction practice is limited by unsufficient load- carrying capacity and increased deformations of structural joints. Therefore, progressive timber flat-chords and triangular steel tube web of open-web truss roof girders are used in commercial, office and sport buildings. Usually, for steel-to-timber joints single-sided toothed steel plate connectors and bolts are used (Fig.1).

Fig.1. Steel-to-timber joint with toothed steel plate connectors (1) and bolts (2)

The resistance of steel-to-timber joints may be written in the forms:

$$R_h = A_h \cdot d_c^{1,5} + R_{b,h} \tag{1}$$

$$R_{h-M} = A_{h-M} \cdot d_c^{1,5} + R_{b,h-M} \tag{2}$$

$$R_M = A_M \cdot d_c^{1,5} + R_{b,M} \tag{3}$$

Here A_h; A_{h-M}; A_M and $R_{b,h}$; $R_{b,h-M}$; $R_{b,M}$ are cumulative parameters of joint performances and bolt resistances, respectively, if joint failure is caused by embedding stresses, embedding stresses and bolt yield moment or only by bolt yield moment; d_c is the steel connector diameter. Thus, the joint resistance presents the multi-criterion failure vector.

The components of axial compressive and tensile forces of truss members N_g, N_q and N_s are caused by permanent g, long-term variable q and reiterated transient s random actions. Histograms and probability distribution curves of the joint resistances R by (1)…(3) and the action effect components N_g, N_q are identical. Therefore, it is expedient to use the conventional resistance of girder joints

$$R_{conv} = R - N_g - N_q \tag{4}$$

Probability distribution of the action-effect component N_s obeys the extreme distribution low of the Type 1. Thus, the time-invariant reliability problem exists in steel-to-timber girder and its joint safety analysis.

In general, safety indexes of girder members may be computed by Monte-Carlo simulation or numerical integration methods. The long-term safety index $\mathbf{P} = \mathbf{P}\{T \geq t_r\}$ of girder joints may be expressed by the formula:

$$\mathbf{P} = 1 - \mathbf{P}_f \{\cup \cap g_{ij}(\mathbf{X}, t); t \in [0, t_r]\} \tag{5}$$

where g_{ij} are the failure histograms or functions in the space of the basic variables [2]. If this index is computed by numerical integration method, the formula (5) may by written in the form:

$$\mathbf{P} = \int_0^{+\infty} g_{R_{conv}}(R) \cdot G_{N_s}(R)\, dR \tag{6}$$

or

$$\mathbf{P} = \int_{-\infty}^{+\infty} g_{R_{conv}}(R) \cdot G_{N_s}(R)\, dR \tag{7}$$

where $g_{R_{conv}}(R)$ and $G_{N_s}(R)$ are the probability density and distribution functions of the joint conventional resistance and reiterated transient action-effect, respectively.

Joint reliability computation must be carried out using three resistance values by (1), (2) and (3). Then, minimum value of safety indexes characterizes structural safety of design steel-to-timber joints. Structural safety index help to terminate rationale parameters of joint components.

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Session 9

Teaching reliability concepts in civil engineering using simulation techniques

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Most present day building codes are based on the limit states approach and the semi-probabilistic Partial Factors Method. The Eurocode system of design codes introduces the concept of reliability management freely exert the knowledge and expertise of the designer to select the appropriate models and procedures for probability based design and assessment of a structure. Although the fully probabilistic design methods may be considered as more rational and consistent than the partial factors design, usual way to achieve probabilistic solutions by means of analytical and numerical methods are too sophisticated and difficult for effective practical applications. Simulation techniques enable to solve, at least approximately, complex problems for which closed form solutions are either not possible or very difficult.

Success of the implementation of the probabilistic design methods into engineering practice depends to a large extent on education of students and design engineers. They should be taught well in mathematics, structural mechanics and engineering. On the other hand, traditional methods of learning and the structure of academic courses of the structural reliability bring about them a reputation of being difficult, boring and useless. Majority of students and designers are using codes without the meaning of safety measures used in codes. These state the thesis that there is a need for significant changes in the teaching of the probability theory and statistics as well as the structural reliability. Simulation techniques and computers can assist teaching and implementation of probabilistic concepts in design and evaluation of many practical problems in civil engineering.

The paper attempts to begin the discussion about the following questions:

a) How to bring the understanding of the structural reliability concepts to designers and students? All components of the building process including planning, design, construction, use and demolition, involve various uncertainties. Because of these uncertainties, majority of the state variables which are needed to formulate a performance function for the considered mode of failure of a structure, are random variables. This fact should be accepted and practically applied in the educational process.
b) How to introduce simulation techniques into civil engineering education and practice? Simulation techniques are powerful and efficient tools that can be used to solve complex problems for which closed-form solutions are either not possible or very difficult and to check the results of other solution techniques. Applications of simulation techniques are not limited to structural reliability problems. They can be applied to basic problems of structural mechanics, risk management and other engineering problems.
c) Which innovations in teaching and learning in civil engineering education can be used for efficient implementation of the simulation-based design? The nature of engineering places a particularly high requirements on its practitioners to undertaken continuing professional development, continuing engineering education or life long learning. The

cooperation between faculties or departments of civil engineering and professional milieus in the civil engineering domain (engineering institutions, councils, chambers and/or associations) is essential for the implementation and dissemination of the probabilistic simulation-based design and assessment of structures. Self-directed learning in an interactive computer environment which integrates textual material, animation, qualitative and quantitative simulations, and video clips, as well as use of Internet technologies can foster self-learning abilities as an essential prerequisite for life long learning.

The Monte Carlo method is very powerful and useful technique for performing probabilistic as well as deterministic analyses. However, in some instances, the time needed to evaluate the problem for a single trial may be very long and/or the total number of random variables which have to be taken into consideration is very large (e.g. when the finite elements model of a structure is considered). The calculation of the reliability of such a structure with the standard Monte Carlo method may be very time consuming. The importance sampling, the directional sampling and the stratified sampling methods are examples of techniques for reducing the number of simulation needed to obtain reasonable results.

A set of computer programs for generation of random variables of the most common distributions used in structural reliability analysis and for the statistical analysis of variable expressed by mathematical expressions as well as for evaluation and display of multi-dimensional random variables should be made available to students. More specialized simulation-based programs which allow for analysis of the load effects, for determining the carrying capacity and probability of failure or decommissioning of structural elements and systems should be used to provide students and designers with a visual and numerical representation of the failure surface and failure probability.

The problem of the qualitative improvement of the structural reliability assessment concepts connected with the training of students and designers in this respect, was the reason why the international project on "Teaching reliability concepts in civil engineering using simulation techniques" (TERECO Project) – sponsored through the Leonardo da Vinci Agency in Brussels was introduced into practice in 1999 – 2001. The purpose of the project was to propose an innovation methods in education and to develop teaching tools leading to understanding the approaches applied in probabilistic design and reliability assessment of building structures, including the application of simulation techniques. The final product of the project consists of classes, courses and seminars for students and professional engineers, articles, papers and the textbook: Marek P., Brozzetti J. and Gustar M., editors, "Probabilistic Assessment of Structures using Monte Carlo Simulation", ITAM, Praha, 2001. The application of the simulation-based reliability assessment and probabilistic design method is there explain using 150 solved examples worked out by 31 authors from eight European countries and from U.S. On the attached CD-ROM it can be found the input files and computational tool enabling the duplication of the examples on a PC, and presentations of examples in Microsoft Power Point. A series of courses for designers and graduate student have been organized in Portugal, Poland and other countries, using the lecture notes and computational tools developed within the confines of the TERECO Project.

Section 8

Codes, Databases, Software, and Applications of Internet

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In these notes I will refer specifically to applications pertaining to wind effects on structures. The purpose in developing standards and design procedures is to achieve, to the maximum possible extent, economic structures that perform safely. The quest to achieve this purpose goes back to Irminger, whose 1894 aerodynamic tests the statement: "It will be due to him that we surely in the future shall save tons of material in our roofs" [1]. Flachsbart's 1932 boundary layer wind tunnel experiments, and University of Western Ontario (UWO) tests sponsored by the Metal Buildings Manufacturers Association (MBMA) in the 1970's further advanced the state of the art.

The technological gap is far wider in wind engineering between 2002 and the 1970's than between the 1970's and 1894. In the 1970's wind pressure measurement, data storage, and data processing capabilities were still relatively primitive. For example, 1970's UWO tests were conducted predominantly for wind directions in increments of 45E. In contrast, UWO conducted in 1997 and 2001 similar test in increments of 5E, an order of magnitude improvement. Similar improvements were achieved with respect to numbers of pressure taps per unit area. The low resolution in the 1970's UWO tests is just one reason why the ASCE Standard contains inadequate aerodynamic information. More importantly, the use of the results of those tests to develop, "by eyeball," drastically simplified aerodynamic tables and plots designed for slide-rule era calculations have been shown to lead to errors in the estimation of wind effects that can exceed 60 %.

More than one century after Irminger's experiments and a quarter of a century after the UWO/MBMA tests, major advances in measurement, data storage, and computational capabilities warrant revision of the methods by which buildings are commonly designed for wind. Improved methods that make use of those capabilities can save large amounts of material, meaning that construction costs and the energy embodied in new construction can be reduced. These methods can also achieve designs resulting in significantly reduced losses from extreme winds, and help to identify weaknesses of existing construction in need of retrofitting.

A modernization of the methods for estimating wind effects is in progress. This will bring computations of wind effects in line with the now routine computer-intensive calculations of internal forces induced by specified loads. Thanks to cooperative efforts by the National Institute of Standards and Technology (NIST), UWO, Purdue University, Texas Tech University (TTU), CECO Building Systems, and MBMA, a pilot project initiated by NIST to develop a computer-intensive, user-friendly design procedure for the calculation of wind effects is now close to completion. The procedure, referred to as database-assisted design (DAD), is complemented by reliability-based modules, and uses the following input:

- *Aerodynamic information*. This is supplied by an aerodynamic database containing simultaneous records of time histories of pressure coefficients for as many as 37 wind directions at 500 to 1,000 ports on the exterior and in some cases interior surfaces of the building, for a sufficiently large number of building types and geometries. In this context "sufficient" means larger than the number of building types and geometries used to develop current ASCE 7 Standard provisions. For low-rise industrial buildings with gable roofs, constant eave height, and a rectangular shape in plan, about 15 distinct geometries were tested at UWO in both open terrain and built-up terrain conditions. In view of the improved resolution of the measurements and the absence of "eyeball" summarizing of test results, a new aerodynamic database that would cover only 15 distinct geometries would still lead to improved wind effects calculations. However, this number can easily be exceeded in a future aerodynamic database. For the time being the database – which is being augmented – will cover about 25 geometries, a number sufficient for developmental purposes.

- *Climatological information*. To estimate wind effects the procedure can use databases consisting of the extreme wind rosettes (i.e., plots of maximum directional wind speeds) for each of a sufficient number of storms. Currently the climatological database includes rosettes for 999 hurricanes at about 50 equidistant mileposts on the Gulf and Atlantic coastlines. To our knowledge this is the only open, publicly available hurricane wind speed database. Databases could be developed and/or made available to the public that would cover the entire area of the United States affected by hurricane winds, rather than just the coastline, and would include rosettes associated with more than 999 simulated hurricanes. Where directional wind speed data, simulated or recorded, are not available, the procedure uses wind speeds with specified mean recurrence intervals estimated without regard for wind directionality.
- *Estimates of knowledge uncertainties*. This includes estimates based on sample sizes of extreme climatological data, the length and resolution of wind tunnel records, and other information pertaining to the definition of the wind environment.
- *Structural information*. For an industrial metal building with portal frames this consists of the distance between frames, the locations, types of support, sizes of purlins and girts, and the cross sections of the frames or the influence lines for the frames' bending moments, shear forces, and axial forces.

Software for using the aerodynamic, climatological, and structural information to obtain internal forces was developed by the National Institute of Standards and Technology, Purdue University, and the the University oof Perugia. At this time the software, named WiLDE, covers low-rise buildings that do not exhibit dynamic amplification effects. An extension to flexible buildings is in progress. Added to the software will be an automated procedure for interpolation of pressure time histories for buildings with dimensions different from those covered in the database. The procedure is based on simulations of the pressure spatio-temporal fields on the buildings from which the interpolations are to be effected. WiLDE has been expanded to include probabilistic modules that provide information on the estimated probability distribution of the peak internal forces for a specified wind speed. A further expansion is in progress, aimed at using this information, as well as statistics of extreme wind speeds and estimates of knowledge uncertainties, to estimate the probability distribution of the wind effects being sought. The knowledge uncertainties pertain to the wind speed or other observations on the basis of which wind speeds may be inferred, terrain roughness, ratios between wind speeds in different roughness regimes, extreme wind speed distribution parameters, ratios between 3-s gusts and the corresponding hourly mean speeds, length of wind tunnel records, wind tunnel performance characteristics, extent to which the wind tunnel reproduces correctly the full-scale aerodynamics, and so forth. The requisite estimates are obtained principally by Monte Carlo simulations. The software has been expanded to account for the effects of aerodynamic and climatological wind directionality. The inclusion of comprehensive aerodynamic, climatological, structural, and reliability information and models makes it possible to account in a faithful manner for the complexity of the wind loading, and the stochasticity and knowledge uncertainties inherent in the estimation of wind effects. As the state of the art evolves, model improvements achieved through research can easily be incorporated in DAD for codification, design, retrofitting, and loss estimation purposes. In collaboration with NIST, Lehigh University has developed a procedure for estimating ultimate capacities of frames and purlins subjected to wind loads defined by aerodynamic databases as described earlier.

The work aimed at developing database-assisted, reliability-based design methods and standard provisions for wind loads is proceeding in parallel with the development of quality control methods for wind tunnel testing so that inter-laboratory comparisons of test results and wind tunnel certifications can be conducted effectively and reliably. It is anticipated that in the future databases obtained from wind tunnel tests may be replaceable by pressure data obtained by Computational Fluid Dynamics methods. Issues pertaining to the development of protocols for aerodynamic database formatting and of methods for effecting systematic corrections of the aerodynamic databases based on full-scale test results are also being debated. Finally, optimal methods for accessing aerodynamic databases via CD's or Internet sites are also being sought.

Session 8

Reliability assessment in highway bridge design

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Currently, the highway bridge design in Canada follows the approach of the limit state design, which is also called as the load and resistance factor design in the United states, or the partial factor design in some other countries. Theoretically speaking, by means of the properly calibrated load and resistance factors listed in the applicable design code, the procedure of the limit state design leads to a consistent and uniform safety level for all designed highway bridges with safety index equal to its target value, say 3.5 for the ultimate limit state under normal load conditions. From this point of view, the limit state design has achieved a significant progress in comparison with the earlier working stress design.

However, the design code and its calibration procedure must cove a wide range of bridge structures. Consideration for efficiency in the calibration process, and for convenience in implementing the design code often require some simplifications and approximations. For instance, the same value of impact factor is used for all the bridges, only the span length and the girder spacing are used to determine the distribution of live load moment among girders, the same set of probabilistic parameters are used for the moment capacity of the entire family of composite steel girders, the same resistance factor is used for all grades of concrete, the values of all load and resistance factors were rounded off to the nearest 0.05 etc. In fact, each bridge has its own static, dynamic and material properties, as well as the corresponding probabilistic characteristics, which may be more or less different with those used in calibration of a design code. Therefore, a bridge, even design exactly following the design code, may still have a safety index more or less deviate from its target value, or in other words, a certain extent of underdesign or overdesign may still occur. If a reliability assessment can be carried out for each designed bridge with its individual characteristics being considered as accurate as possible, and the subsequent adjustment are taken accordingly, a significant economical saving may be achieved either by reducing the initial cost or by avoiding potential failures in the service.

As an effort in this direction, a method for reliability assessment of girder bridges is proposed based on the simulation technique. Using this method, each reliability assessment takes only a few minutes on an ordinary desk computer.

Design examples are presented to show the necessity of the recommended reliability assessment, and the capability of the proposed method.

Session 8

Reliability assessment of a beam according to SBRA method and Eurocode

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Today's national as well as international codes for design and assessment of building structures are mainly based on semi-probabilistic (prescriptive) interpretation of limit state philosophy, on so called "partial factor method" (applied e.g. in Eurocodes and LRFD). Extensive development of computer and information technology leads to a qualitative improvement of reliability assessment methods. Numerous published papers indicate the transition from current deterministic and semi-deterministic concepts applied in current codes to fully probabilistic reliability assessment concepts. Some of the new methods use simulation technique. Application of one of such methods, SBRA (Simulation-Based Reliability Assessment method, see [1], [2], [3]) is demonstrated next. The differences between SBRA and Partial Factors Method are emphasized.

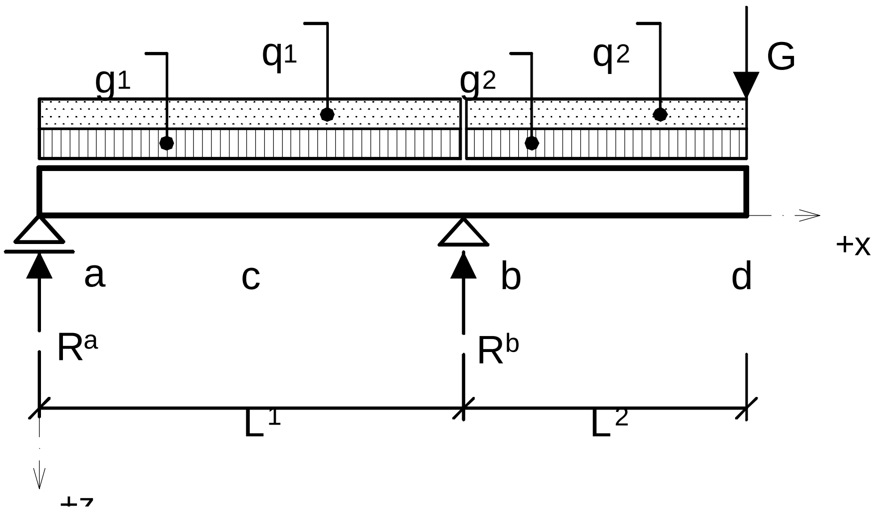

Fig. 1 Scheme of simply supported steel beam with overhanging end

The Figure 1 indicates the differences in the approaches to the calculation of the response of a simply supported steel beam with overhang to the combination of several loads. The load effects combination is expressed by bending moments and shear forces, see bending moment and shear force diagrams vs. "anthill" representation of the response in Figure 1.

For the assessment of the beam according to current codes and SBRA and corresponding details see [4].

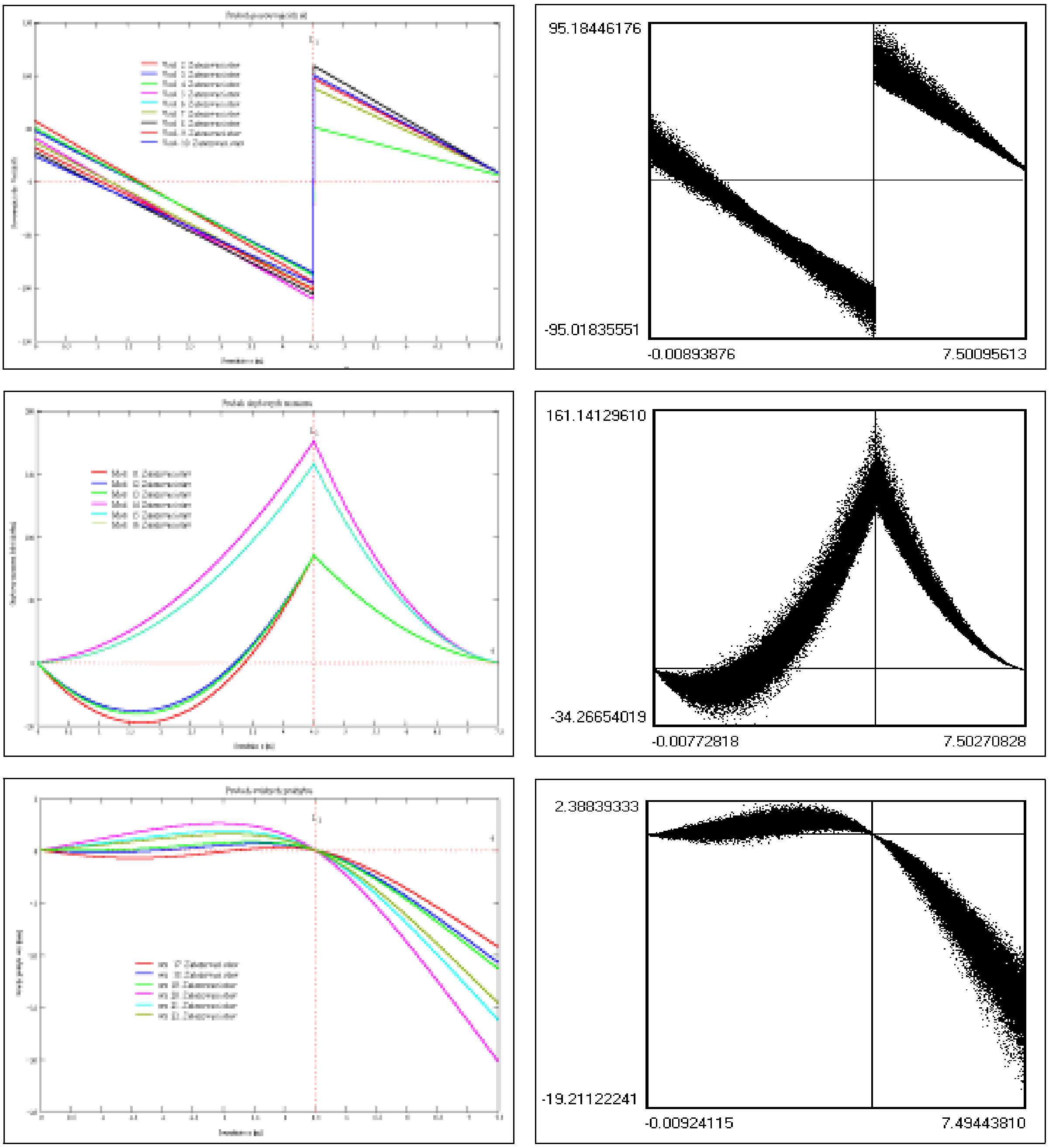

Fig. 2 Resulting load effects calculated according to Eurocode (left) and SBRA method (right)

[1] Marek, P., Guštar, M., Anagnos, T.: Simulation-Based Reliability Assessment for Structural Engineers. CRC Press Inc., Boca Raton, Florida, 1995.

[2] Marek, P., Guštar, M. a Bathon, L.: Tragwerksbemessung. Von deterministischen zu probabilistischen Verfahren. Academia, Praha 1998.

[3] Marek, P., Brozzetti, J., Guštar, M.: Probabilistic Assessment of Structures using Monte Carlo Simulation. Background, Exercises and Software. Institut of Theoretical and Applied Mechanics, Academy of Sciences of the Czech Republic, Praha, 2001.

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Session 8

Reliability assessment of a steel column according to SBRA method and ČSN 73 1401

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The subject of the paper is the application of the SBRA method in case of the design and reliability assessment of steel industrial building. Special attention is given to the assessment of a steel column exposed to several mutually uncorrelated loadings such as wind, snow, crane and dead load.
The difference in the results and the procedures obtained by the design according to the method SBRA (see ČSN 73 1401 1998 – Appendix A) and according to the code ČSN 73 1401- 1998 is evaluated and discussed.

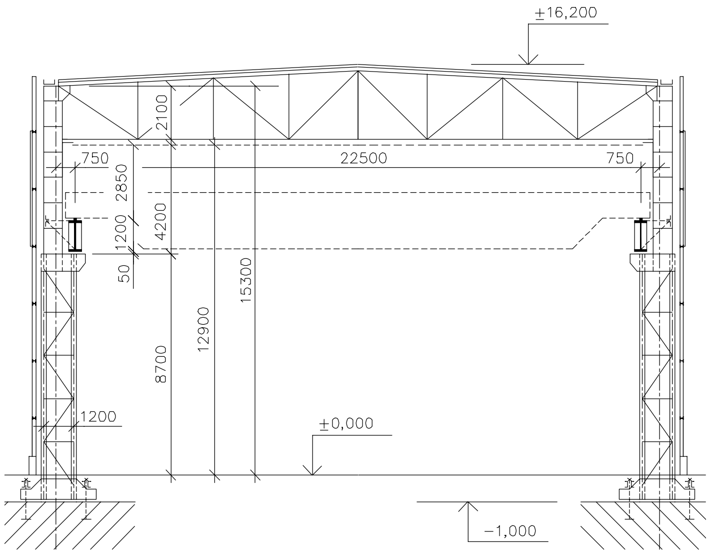

Fig. 1: Scheme of the steel hall

Results:

Referring to the investigated cross sections of the structure the application of SBRA method leads to significant material savings. The required cross-sectional areas according to SBRA are approximately by 11% smaller compared to cross sections designed and assessed according to the Partial Factors method applied in the current code CSN 73 1401.
Note: The minimum required cross cross-sectional areas were determinated according to the criteria applied in the SBRA method and in the Partial Factors method.

Conclusions:

(a) By comparing both applied methods, it can be concluded that SBRA method leads in investigated situations to non negligable material savings mainly due to more consistent load effects combination analysis.

(b) The resulting response of the structure to the loadings and the resistance considered in the SBRA approach allow for better understanding of the actual behavior and reliability of the structure.
(c) The SBRA method makes the load effects combination analysis simple, does not require determination of extreme „design“ values of internal forces (as it is the case in Partial Factors design) and allows for direct calculation of the probability of failure.

References.

[1] Gere, J.M., Timoshenko, S.P. Mechanics of Materials, 3rd Ed., solutions manual. PWS-KENT Publishing Company, Boston, 1990.
[2] Marek, P., Guštar, M. a Bathon, L.: Tragwerksbemessung. Von deterministischen zu probabilistischen Verfahren. Academia, Praha 1998.
[3] Marek, P., Brozzetti, J., Guštar, M.: Probabilistic Assessment of Structures using Monte Carlo Simulation. Background, Exercises and Software. Institut of Theoretical and Applied Mechanics, Academy of Sciences of the Czech Republic, Praha, 2001.

Session 8

Safety Assessment of a Frame

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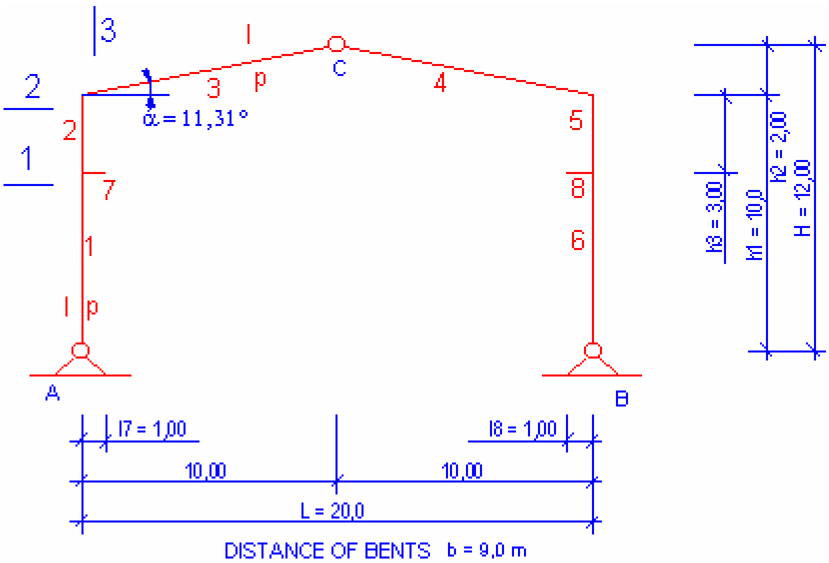


Geometric chart of the frame

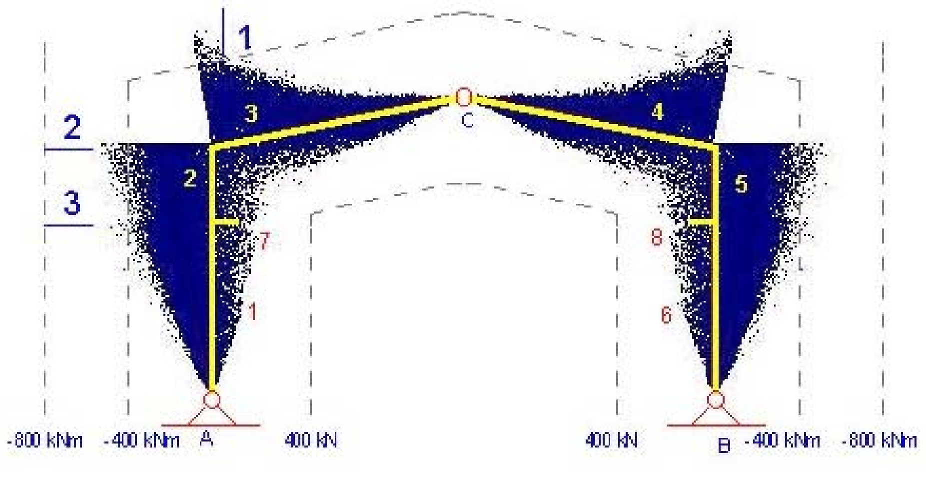
Subject of this study is safety assessment of members of the steel frame (see Fig. 1) by fully probabilistic SBRA method [3]. Study outline possibilities of the SBRA in the assessment of systems with multi component load effects combination. Assessment by the SBRA is compared with another one performed in accordance with the European ČSN P ENV 1993-1 [2] (shortly denominated as EC).

There are differences between both methods:

(a) SBRA [3] characterize random variables (e.g. loading) by their extreme and bounded histograms. The maximum (extreme) load effects according to SBRA [3] correspond to the factored load effects according to EC. Carrying capacity of the shape is defined by onset of yielding. Safety of the structure is expressed by probability of failure $P_f < P_d$.

(b) EC [2], which uses load and resistance factors, looks more likely deterministic and reference value of safety is defined by plastic carrying capacity of the shape.

Statically determinate plane steel frame (see Fig. 1) is a part of bent of industrial building. Loadings considered in this study are wind, snow, crane girder and dead load. The second order effects, the buckling and torsion-buckling, are not considered in this study. Safety of the selected cross-sections (see Fig. 1) is assessed. The effect of shear forces is neglected.



Bending moment M [kN]

Comparison of designs

	Designed rolled shapes	
	SBRA	EC
1-1	HEB 400	HEB 500
2-2	HEB 400	HEB 450
3-3	IPE 600	HEB 450

Assessment of the planar frame according to EC [2] has 144 cases of the load effects combinations. Combination of the load effects for a three-dimensional frame could be more difficult.

Study denotes possibilities of the SBRA [3] in analyzing of the multi component load effects combination and determining of the steel frame reliability.

In this example are shapes designed according to EC [2] more conservative than designs according to SBRA providing considered conditions (see Table 1).

References

[1] ČSN P ENV 1991-1, *Basis of design and actions on structures, Part 1: Basis of design Eurocode 1*, ČNI, Prague, 1994.

[2] ČSN P ENV 1993-1-1, *Design of steel structures, Part 1-1: General rules and rules for building, Eurocode 3*, ČNI, Prague, 1994

[3] MAREK, P., BROZETTI, J., GUŠTAR, M. (editors), *Probabilistic Assessment of Structures using Monte Carlo Simulation,* ITAM CAS, Prague, 2001.

Session 8

The safety of a column exposed to two-component load effects combination

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The subject of the following discussion is the application of the SBRA method in case of the safety assessment of a column exposed to a combination of variable vertical and horizontal forces. The 1^{st} and the 2^{nd} order analyses are considered.

Assignment:
A steel column is exposed to dead, snow and wind loads. The two component load effect at the critical section of the column should be analysed and the differences in results obtained according to the 1^{st} and 2^{nd} order theory should be discussed.

Assigned input values (SBRA method):
extreme values of the loads are:
dead load G = 290.5 kN, snow load S = 165 kN , wind load W = 10.5 kN
nominal length of column L = 6.0 m
eccentricity of vertical load e = 0.30 m
modulus of elasticity E = 210 x 10^6 kPa

Variables:
Variation of input quantities is expressed by histograms:
Loads – dead, short lasting and wind loads: Dlvar, Slvar, Wlvar
Length - and eccentricity: histograms Lvar and evar
Steel - yield strength: histogram fyvar.

Load combination and load effect:
The two component load effect (corresponding to the axial force N and bending moment M at the critical section) is expressed by stress in extreme fibres at the critical cross section.

Reference value
In the probabilistic safety assessment according to the SBRA method (using the Anthill computer program) the Reference Value, RV, is defined by the yield strength of the steel column (represented by a histogram).

The results of the probabilistic safety assessment of the column (as obtained according to the SBRA method) are reviewed and the difference between the results corresponding to the 1^{st} and the 2^{nd} order analysis are discussed.

Session 8

Assessment of serviceability of a cantilever beam according to SBRA method and Eurocode

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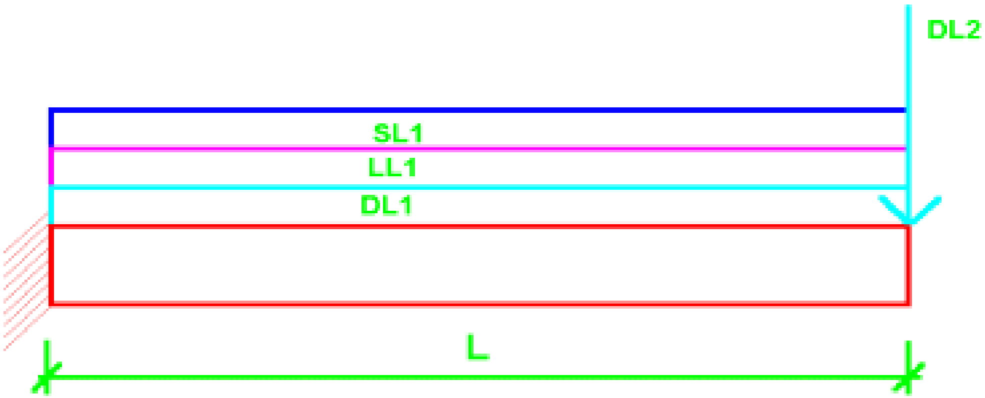
Abstract

The subject of the paper is the assessment of serviceability of a cantilever beam according to the Eurocode and to the SBRA. Steel beam is exposed to the combination of three loads.

Assignment

Check the serviceability of a cantilever beam exposed combination of three loads (see fig.1). The target probability $P_d = 0{,}07$.



Assessment according to EC

Deflection of the steel shape IPE 220 is assessed according to EC. Load effects combination is based on the equation (1). EC uses characteristic values of the load effects combination in the assessment of serviceability.

$$\sum_{j\geq 1} G_{kj} \oplus P_k \oplus Q_{k1} \oplus \sum_{i>1} \psi_{Oi} Q_{ki} \qquad (1)$$

Assessment according to SBRA

A fully probabilistic approach, SBRA, is used to determine the deflection of a rolled steel beam IPE 200. Load effects are combined according to (2). Loads are characterized by extreme values and bounded histogram. The extreme value corresponds, according to EC3, to design value (i.e. the product of the characteristic value and load factor).

$$p = DL1nom * DL1\,\mathrm{var} + LL1nom * LL1\,\mathrm{var} + SL1nom * SL1\,\mathrm{var}$$
$$F = DL2nom * DL2\,\mathrm{var} \qquad (2)$$

Conclusion

Load effects are considered by its characteristic values in the assessment according to EC. SBRA method characterizes load effects by extreme value and bounded histogram. The extreme values correspond to design values (factored characteristic values) according to EC.

Note

Application of a fully probabilistic serviceability assessment concept (such as SBRA) can be considered. In such qualitatively new approach, the serviceability check is based on a prove that the probability of exceeding the serviceability limiting values will be less than the target probability given in codes (see, e.g. [2]).

Literature

[1] Marek, P., Brozzetti, J., Guštar, M.: Probabilistic Assessment of Structures using Monte Carlo Simulation. Background, Exercises and Software. Institut of Theoretical and Applied Mechanics, Academy of Sciences of the Czech Republic, Praha, 2001.
[2] ČSN 731401 : Navrhování ocelových konstrukcí

Session 9

Action and Reaction of Complex Building Projects - simulations for glass-roof and railway bridges of Lehrter Bahnhof Berlin

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The reliability of buildings is not only dependent on the variation of actions and reactions, respectively loads and material characteristics, but especially on the interaction of the stiffness properties of all structural components. This is of great importance, if structures do not have fixed, but flexible boundary conditions, for example in the case of complex building-blocks consisting of different structural members or even different buildings.

One significant example for this is the new Lehrter Bahnhof in Berlin, which is not one building, but consists of several buildings interacting with each other: underground tunnels, railway station, parking garage, railway bridges, glass roof, office buildings. Changing stiffness-properties of the lower situated buildings will influence the load-bearing behaviour of the onstanding buildings, especially the railway bridges. This is of great importance, as the numerous load-cases according to the demands of the German railway bridge standard DS 804 imply adequate numerical tools for a safe and reliable design and proof of all structural components. As well the stability of the glass roof is influenced significantly (Fig. 1).

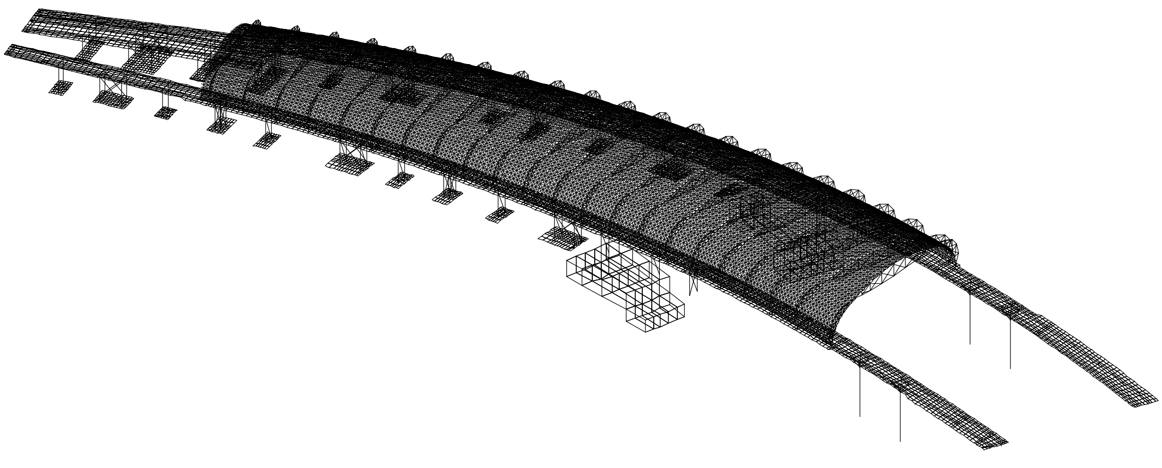

Fig. 1: Glass-roof and railway bridges of Lehrter Bahnhof Berlin

Second example is the proof of pre-stressed concrete towers for wind energy converters, because enhanced flexibility due to degradation of soil-properties, cracking of concrete or other deteriorations will reduce eigenfrequencies and thus may drastically enhance the dynamic action on the structure during operation. These degradations might be caused by thermal or hygric constraints, by fatigue due to high-cycle dynamic wind-action or by chemical deteriorations in case of offshore wind energy plants. Though the design of wind energy towers in general demands that the operational frequencies should keep away from the virgin structure's eigenfrequencies with reliable distance, resonant action might be possible in case of reduced stiffness and thus reduced eigenfrequencies.

The moderator will report on both projects in order to attract for this interaction problem. The participants in the conference are invited to present building projects, where similar problems have occurred in practice and where the application of simulation-based concepts may help to achieve a proper and safe design of engineering structures.

Session 9

Optimization Study of Beam with Sudden Profile Change

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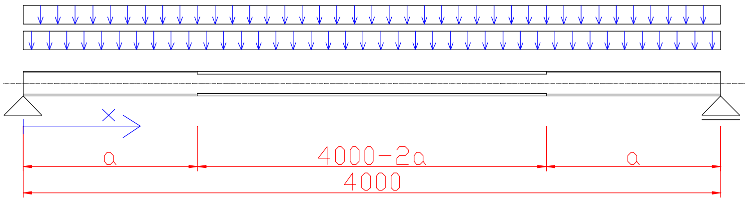
This paper deals with the application of the *SBRA* (Simulation Based Reliability Assessment) method for safety assessment of a welded steel beam with a sudden profile variation. Close attention is paid especially to the determination of the position where the profile changes.

Assignment

The beam is simply supported. Lateral-torsion buckling of the beam is prevented. The span is L = 4 m, the material is steel S235. The beam is exposed to two continuous loads. The design values are: dead load g = 10 kN/m, long lasting live load q = 8 kN/m. Assess I-profiles of this steel welded beam (see pic. 3) regarding the bending moment. Determine the position where the profile is to be changed with respect to material savings. The design probability of failure is $P_{f,lim}$ <= 0.00007.



Pic.1 Beam scheme

Solution

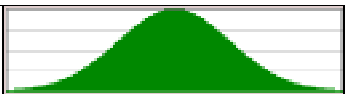
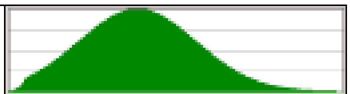
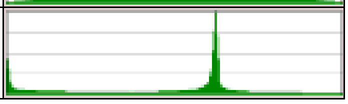
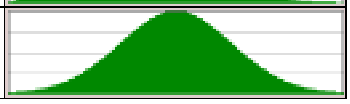
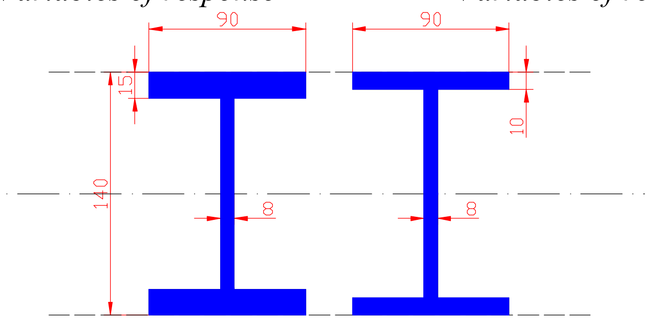
dead load g_{var}		steel yield stress f_{var}	
live load q_{var}		moduluses W_{var}	

Chart 1 Variables of response *Variables of resistance*

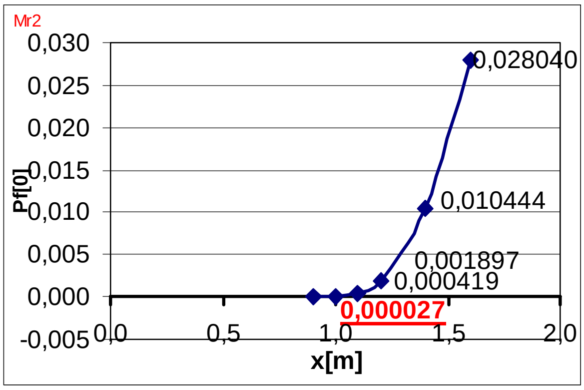


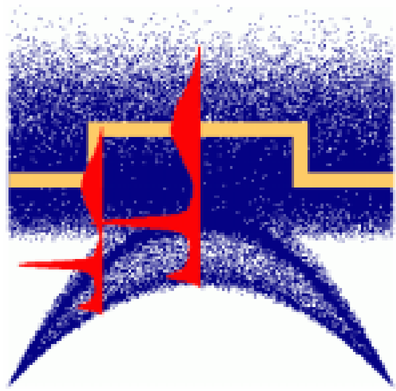
Pic.2 Profile 1 (x=2m), Profile 2(x=?m)

The nominal values of the cross-sections properties are $W_1 = 1.64E^{-4}$ m^3, $W_2 = 1.25E^{-4}$ m^3. The Critical cross-sections, considering the bending moment, are for Profile 1 the center of the beam (x = 2m), for Profile 2 the place of its sudden profile variation (x = ?).

Profile 2, x = ? m

With assistance of the *AntHill* [1] software, the probability of failures is found in individual positions near the estimated location of the sudden profile change.





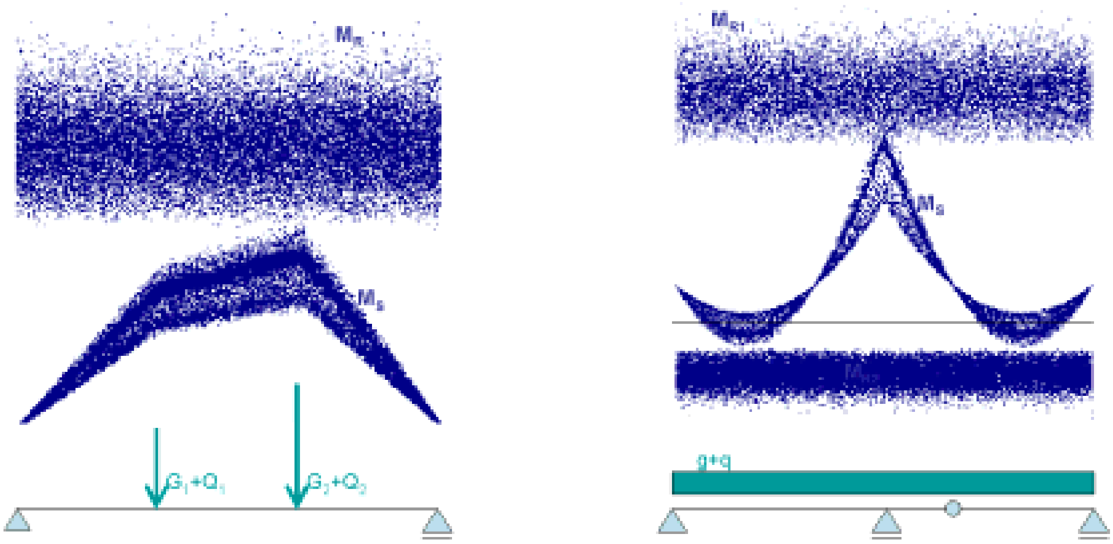
Pic.3 Probability of failure calculating P_f near the sudden change location
Pic.4 Response and resistance, bending moments M_x, M_{r1}, M_{r2}

The profile should be changed in position **x = 1 m** from the support. The failure probability in this location $P_f = 0.000027$ (see chart 3) < $P_{f,lim} = 0.00007$. Profile 2 satisfies the requirements. Calculated by the *AntHill* [1] software.

Pic.5 Cross-section of pic.4 in the x =1 m position, P_f has been determined by the intersection of the graphs of resistance and response

The assessment of profile 1 in its key-position (x = 2 m) is described in the full text of this article. See volume 2 of the Euro-SiBRAM'2002 proceedings.

Additional Studies



Pic.6 AntHill: response M_s and resistance M_r

Literature

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Session 9

Timber element reliability assessment

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The timber element reliability assessment is based on the accumulation of "partial damages principle" and on SBRA (Simulation - Based Reliability Assessment) method.

Compared to current codes, there is a different approach applied for verification of the safety in case of timber components. This alternative approach is based on (a) the load effect combination analysis, (b) the application of the Madison curves, (c) the accumulation of damage procedure, and (d) the simulation technique.

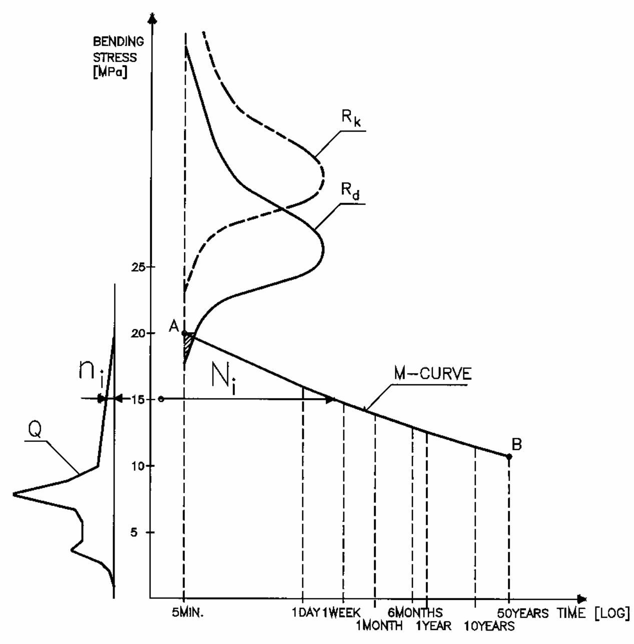

Fig. 1 expresses the main idea of such approach. A procedure of load effect combination analysis leads to the resulting load effect combination histogram ***Q***. Evaluation of this histogram allows the determination of: (a) the extreme load effect combination corresponding to a selected percentile, and (b) the magnitudes n_i which express the load effect (stress) duration N_i corresponding to individual discrete stress levels.

The accumulation of damage assessment procedure is based on the summation of "partial damages" corresponding to individual discrete stress levels. For each stress level i, corresponding number n_i of "dots" is used to express the frequency of occurrence of that stress level.

Fig.1 The accumulation of damage – scheme

The sum of n_i for all levels is equal to total number of Monte Carlo simulations. As shown in Fig. 1, the Madison curve ("M-curve") defines the time to failure N_i for each stress level i. The M-curve is defined by points A and B and the shape. Using the Palmgren-Miner rule

$$\sum n_i/N_i \leq 1 \quad (1)$$

if the criterion (1) is satisfied, the timber component is safe.

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TV Tower - Serviceability Check Using SBRA Method

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a) In the year 1968 the TV tower Middle Bohemia was reconstructed by the replacement of its lattice part by a steel tube. The requirement of angular deflection of the top of tower, limited to 0.55^0 was given during its reconstruction. Such an angular deflection corresponds to the displacement of the top 0.90 m. The designer was not capable to fulfill this condition without substantial changes in tower construction, and it was therefore agreed that this limit should be guaranteed within 99 % of time only; it can be exceeded with the probability of $P_d = 0.01$ (i.e for 0.01 x 24 x 60 = 15 minutes per day). To solve this problem, the designer did not use the design wind velocities and loads according the Code of that time, but he derived the histogram of mean wind velocities from meteorological measurements, performed during previous 5 years. This happened in 1969, and it can be considered as a primeval example of the probabilistic approach.

The histogram of the mean wind velocity (for a period of 5 years) was determined on the basis of longterm measurements and the dynamic displacements of the reconstructed tower in the air flow direction were determined by wind tunnel tests (Pirner 1972).

Further it was agreed with the owner requesting the mentioned afore limit that the displacement would be inferior to the required limit during 99% of the time. The wind velocity with 1% occurrence is $V_{SK,10}$ = 48.5 km/h = 13.47 m/s. On this basis were determined static and dynamic responses.

The overall angular deviation $0.4 < 0.55^0$ with 1% occurrence was determined.

b) The authors came back to this problem, as the European prestandard ENV 1991-2-4 (1995) introduces an other method for the calculus of dynamic component of wind loading, taking also into account the transversal vibrations of the structures, designed as the loss of aerodynamic stability (or Kármán vortex trail or Strouhal effect).

Static and dynamic response to wind action – solution according to ENV :

The response of the TV tower to wind loading can be (according to pr ENV, 1995) separated to two components, namely:

- static in wind direction $x_{w,stat,}$ corresponding to the sum of static loadings on tower-surface
- dynamic in wind direction (see prENV, formula B16 and following)
- dynamic transversal to wind direction

For the resulting amplitude of the tower top it holds

$$v_{\max} = \sqrt{\left[x_{w,stat} + \max \quad x(z)\right]^2 + \left(\max \quad y_F\right)^2} \qquad (1)$$

Supposing that the tower deflection curve corresponds to its fundamentals natural mode (with $f_{(1)} = 0.267$ Hz), it holds for the inclination of the tower top φ

$$\varphi = \frac{16.3 v_{\max}}{v_{ref}} \tag{2}$$

This relation was found on the aeroelastic model in wind tunnel.

c) Probability of achieving the serviceability condition – (SBRA) :

As the first step required for SBRA solution, the random values are to be expressed by adequate truncated histograms, then the reliability function *FS* is to be defined, namely

$$FS = R - S \tag{3}$$

Where R means the allowed displacement of the tower-top (0.90 m)

S - response of the tower to winds loading – equ. (1).

This reliability function was analyzed by 200000 evaluations of the equ. (1); the random input parameters are being taken with the use of Monte Carlo method from corresponding sets according to their histograms. This is controlled by the M-Star program (see Marek et al., 1995, 2001). The condition of $(v \geq 0.9m, or FS \leq O)$ gives the probability that the real displacement is larger than the allowed one. The result leads to probability

$P_f = 0.00804 = 0.8\%$, i.e. smaller than the tolerable 1%,

Which means that the serviceability criterion is satisfied in this case.

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Two component load effect combination

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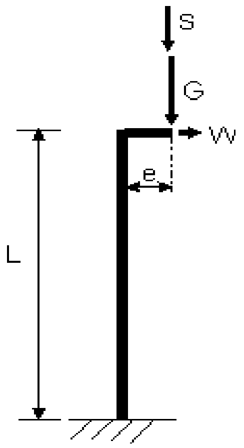
The subject of the paper is the calculation of the effect combination in the critical section of the steel column exposed to dead, snow and wind loads (see Figure 1.1). Similar problem was already published (see [1]) where authors did not consider the 2nd order analysis. The results of the 1st order analysis and the 2nd order analysis were compared. The differences are specified and discussed.



1.1Steel column. Load arrangement

Bending moment : a) 2nd order analysis: $M = W * L + (S+G) * (e + \delta)$
where δ is the maximal deflection

b) 1st order analysis: $M = W * L + (S + G) * e$

As reference value was used the yield stress of steel: fy

Load combination in the critical section of the column: $\sigma = N/A \pm M/W$,

Safety function can be written as: $SF = fy - \sigma$

The probability of failure is calculated using M-Star and AntHill computer programs.