ADVANCES AND TRENDS IN STRUCTURAL ENGINEERING, MECHANICS AND COMPUTATION

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Advances and Trends in Structural Engineering, Mechanics and Computation

Editor

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Preface

The Fourth International Conference on Structural Engineering, Mechanics and Computation (SEMC 2010) was held in Cape Town (South Africa) from 6 to 8 September 2010. First held in 2001, the SEMC conferences are aimed at "bringing together from around the world academics, researchers and practitioners in the broad fields of structural mechanics, associated computation and structural engineering, to review recent achievements in the advancement of knowledge and understanding in these areas, share the latest developments, and address the challenges that the present and the future pose". Right from the beginning, the SEMC conferences have attracted strong participation from all over the world, and the latest conference has been no exception, despite the damping effects of the global economic recession. Close to 400 participants from 58 countries worldwide participated at SEMC 2010.

Although participation at SEMC conferences continues to be dominated by academics and researchers from the international community, the last two conferences have seen an increasing level of participation from industry and engineering practitioners, a trend which is set to continue. Over the past 5 years, South Africa embarked on a massive expansion of its transport and sporting infrastructure as part of its preparations for hosting the 2010 FIFA World Soccer Cup, and the demand for innovative design solutions and the latest construction technology has never been greater than during this period. The SEMC conferences have provided the ideal opportunity for the exchange of ideas between local engineers and world-renowned experts from other parts of the world.

These Proceedings contain the more than 300 papers that were presented at the Fourth International Conference on Structural Engineering, Mechanics and Computation. The Proceedings are divided into 21 sections, and a quick overview of these sections is given below. The Proceedings have been published in the form of a printed book of extended abstracts, and a CD-ROM containing the full papers. All papers submitted for the Conference were reviewed by members of the International Scientific Advisory Board and other identified experts acting independently, and only those papers that were accepted have been included in the Proceedings. The editor would like to thank all reviewers for their invaluable input in this process.

The first section of the Proceedings contains the six keynote papers that were presented at the Conference. The diverse themes of these papers reflect the broad scope of SEMC conferences. The opening contribution by Bathe (USA) highlights the fascinating advances made within the domain of computational structural mechanics, and points to the major problems that still need to be solved. In a world where earthquakes and storms continue to cause much devastation on human communities, structural vibration and seismic control have become major areas of research in many countries, and the paper by Li & Huo (China) outlines the latest developments in China in these areas. Mention has already been made of the infrastructural expansion in South Africa in the past 5 years, and in the paper by Goeppert (Germany), we see how advances in structural mechanics, materials and construction technology have been harnessed to create some of the most spectacular structures in the world. The paper by Frangopol & Okasha (USA) deals with the whole issue of life-cycle performance and maintenance of infrastructure, a topic of increasing importance globally, given the huge amounts of money that are annually invested in infrastructure, and the increasing pressure to "get the most" out of these long-term investments. Those in the world of steel-framed construction will know the importance of joints in the delivery of effective and economical designs, and the paper by Bijlaard (the Netherlands) gives an authoritative perspective of the provisions of Eurocode 3 in this regard. The last of the keynote papers is a contribution from Bradford and his co-researchers (Australia), and this treats the rather specialist problem of the long-term performance of concrete domes.

The twenty sections that follow contain the rest of the papers presented at the Conference, including 36 invited papers contributed by identified experts on particular topics. Catastrophic events such as the recent Haiti and Chinese earthquakes vividly remind us from time to time of the importance of a proper understanding of structural dynamics and vibration response, and of the need for effective structural vibration control and sound procedures for seismic design. All these topics continue to generate massive interest among researchers worldwide; the 45 papers in Sections 2 and 3 are testimony to this. Following these sections, the cluster of 35 papers making up Sections 4 and 5 is concerned with various aspects of the numerical modelling of materials, structures and phenomena. Issues of loading, full-scale testing and practical design of structures (including code provisions) are covered in Section 6, which also contains papers reporting on completed structural-engineering projects in the construction industry.

The papers in Section 7 concern the mechanics, analysis and design of plates, shells and related structures, while papers pertaining to thin-walled metal construction and steel structures (including connections and composite construction) appear in Section 8. As would be expected, there is considerable overlap in the subject matter of the various sections. Additional papers dealing with steel structures, but with a stronger emphasis on particular aspects of design (such as fire resistance or optimization), will be found elsewhere in the Proceedings. The performance of structures in fire is a subject receiving considerable attention worldwide and, indeed, modern design standards such as the Eurocodes now make specific design provisions for fire resistance. The 16 papers in Section 9 cover various aspects of the behaviour of structures in fire, and the design of structures to resist the effects of fire.

In many countries, concrete is the most utilised material in the building and civil engineering construction industry. A huge amount of research is going on around the world on the behaviour of concrete (as a material), and on the analysis, design and performance of concrete structures. The 45 papers appearing in Sections 10 to 13 report on the most recent findings on these topics, as well as on advances in the use of related construction materials, and innovative applications such as fibre-reinforced concrete, high-strength concrete and high-performance concrete. The papers that deal with timber, masonry and glass as construction materials have been placed in Section 14.

The principles of structural safety and reliability form the basis of modern design. Section 15 covers both theoretical and design aspects of these issues, and includes the equally important consideration of risk analysis. Optimization is now an important consideration not only in aircraft design, motor-vehicle design and naval architecture, but also in many civil engineering applications, particularly steel construction. The 16 papers on structural optimization (Section 16) cut across a range of engineering disciplines, and report on new methods and techniques for achieving this.

In recent years, the topics of structural health monitoring and damage detection have received a lot of attention. For large and expensive structures such as long-span bridges, tall buildings and concrete arch dams, it makes sense to detect signs of damage and deterioration as early as possible before these get to a stage where costly repairs become necessary, or before the structure fails altogether. The 32 papers in Sections 17 and 18 describe innovative techniques in structural health monitoring and damage detection, and new strategies for the repair, strengthening and retrofitting of existing structures. Some interesting cases of recent structural failures are also reported. The papers in Section 19 tackle the important issues of sustainable construction, preservation of historic buildings and reconstruction, while those in Section 20 deal with soil-structure interaction and its application to the design of foundations, tunnels and other underground infrastructure. The last section features two papers on the teaching of structural engineering.

Needless to say, without the Authors who contributed the papers, these Proceedings would not exist. The editor would like to thank all Authors for their efforts in putting together papers of high quality and delivering them on time. It is hoped that these Proceedings will serve as a useful source of the latest information on the various topics covered, and also serve to stimulate further progress in these areas.

Finally, the editor would like to gratefully acknowledge the financial support of the following organisations, who were the official sponsors of the SEMC 2010 International Conference:

- The Joint Structural Division of the South African Institution of Civil Engineers and the Institution of Structural Engineers (UK)
- The Southern African Institute of Steel Construction
- The Cement and Concrete Institute of South Africa
- The National Research Foundation of South Africa
- The South African National Roads Agency
- Anglo American Corporation
- Murray & Roberts
- ADINA (USA)
- Goba Consulting Engineers and Project Managers
- PD Naidoo & Associates
- Schlaich Bergermann & Partners (Germany)

Last but not least, special thanks go to colleagues and students at the University of Cape Town, as well as family members, for their patience and understanding in giving the editor the space and time required to accomplish this work.

A. Zingoni
Editor

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1. Keynotes

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Challenges and advances in the analysis of structures

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ABSTRACT

The analysis of structures is largely performed using finite element procedures. These are now widely used in engineering and the sciences and we can expect a continued growth in the use of these methods (Bathe (ed.) 2009, Zienkiewicz & Taylor 2005, Bathe 1996, Bathe 2009a).

Considering the analysis and design of civil and mechanical engineering structures, we can categorize the analyses into two broad groups. In the first group, it is quite possible to perform physical tests and thus compare the analysis results with laboratory test data. The analysis of motor cars falls into that group. In the second group, physical tests can only be performed to a very limited extent. The analysis of long-span bridges under earthquake loadings falls into that group. For this second group of analyses it is most important to use reliable finite element methods in order to have the highest possible confidence in the computed results. In addition, of course, the finite element procedures should be efficient. While in the first group, confidence in the safety of a design can be reached by evaluating the analysis results against physical test data, of course, both these requirements – reliability and efficiency – are very important in all analyses, even when physical test data can be obtained – if only to reduce the number of tests to be performed.

When assessing the current use of finite element methods for the analyses of structures, we naturally find that there is also a great interest in solving structural problems that heretofore could not be tackled. Hence, clearly, novel finite element procedures need to be researched, established, and eventually offered widely in computer programs.

The objective in this paper is to briefly discuss major challenges in the development of methods for the analysis of structures, considering the above two categories, and briefly focus on our recent developments to advance the state of the art. In our research, we have continuously focused on the efficiency and reliability of the methods. Of course, any simulation starts with the selection of a mathematical model, and this model must be chosen judiciously (Bathe 1996). However, once an appropriate mathematical model has been selected, for the questions asked, the finite element solution of that model needs to be obtained reliably, effectively, and ideally to a controlled accuracy.

The general challenges are – to solve problems more reliably, accurately and efficiently, and to solve problems that so far cannot be analyzed. These challenges have also been summarized in the Preface of Bathe (ed.) (2005). We present our recent developments for the finite element analysis of shells, the solution of wave propagation problems, the time integration in long-time large deformation analyses, the analysis of large deformations of beam structures, and the simulation of fluid flow-structure interactions including various physical phenomena. This paper is an updated version of Bathe (2009b).

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Recent developments in structural control research and applications in China

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ABSTRACT

Civil engineering structures located in environments where earthquakes or large wind forces are common will be subjected to serious structural vibrations during their lifetime. These vibrations can range from harmless to severe with the later resulting in serious structural damage and potential structural failure. Even though engineers can not design a building which is damage proof during earthquakes and strong winds, the structural control is promising in reducing the vibration of structures. Different from the traditional anti-seismic method, the structural control technique suppresses the structural vibration by installing some devices, mechanisms, sub-structures in the structure to change or adjust the dynamic performance. The structural control system is commonly classified by its device type resulting in four general control types: passive, active, hybrid and semi-active control. An active control system is one in which an external source power control actuators that apply forces to the structure in a prescribed manner including active tendon system (ATS) and active mass damper (AMD). A passive control system does not require an external power source, such as base isolation method, energy dissipation devices, tuned mass damper (TMD), tuned liquid damper (TLD), and so on. The hybrid control implies the combined use of active and passive control systems. Semi-active control systems are a class of active systems in which only small magnitude of external energy is needed to change the parameters of control system, such as active variable stiffness (AVS) system and active variable damper (AVD) system. In recent years, much attention has been paid to the research and development of structural control techniques with particular emphasis on alleviation of wind and seismic response of buildings and bridges in China. Structural control in civil engineering has been developed from the concept into a workable technology and applied into practical engineering structures. The aim of this paper is to review a state of the art of researches and application of structural control in civil engineering in China. It includes the passive control, active control, hybrid control and semi-active control. Finally, the possible future directions of structural control in civil engineering in China are presented.

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South African Stadium Projects: Johannesburg, Durban, Port Elizabeth and Cape Town

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ABSTRACT

South Africa's successful application for hosting the World Cup has resulted in the construction of several new stadia. The construction or modernization of these new arenas coincides with novel ideas for the design. In all cases the roofs play the major role when it comes to the question of unique design and easy recognition.

The roof is the most important element to create the stadium look. Since the cable net structures for the Olympic Stadium roof in Munich, we use all our knowledge and creativity to make the roof of a stadium the star of the event.

This can easily be observed in South Africa. The stadium roofs in Johannesburg (Soccer City), Durban, Port Elizabeth and Cape Town were designed by structural consulting engineers **schlaich bergermann und partner** under the lead of Knut Göppert, are already widely known for their special designs.

Roof structures for enormous spans had to be developed, often with a depth of more than 50 m. Spatial load transfer structures are state of the art when it comes to efficient and sustainable engineering solutions. Fabrication and erection aspects influence the early design ideas and lead to unique installation processes.

1 SOCCER CITY STADIUM JOHANNESBURG

Capacity: 94.000 seats
Roof area: Upper membrane: 23.000 m^2 PTFE/Glass, Lower mesh membrane: 25.000 m^2 PTFE/Glass, Vertical mesh membrane: 2.000 PES/PVC, Glazing: 12.000 m^2 Polycarbonate, t = 12 mm
Facade area: 35.000 m^2 fibre reinforced concrete, t = 13 mm

2 MOSES MABHIDA STADIUM, DURBAN

Capacity: 70.000 World Cup mode, 54.000 legacy temporary seats: 85.000 Olympic mode
Roof area: 39.000 m^2 (vertical projection), 46.000 m^2 (membrane surface)

3 NELSON MANDELA BAY STADIUM, PORT ELISABETH

Capacity 48600 seats, thereof 45 940 permanent
Covered area 30000 m^2, Steel Structure 2300 t, Membrane 22000 m^2, Glass-PTFE membrane
Aluminium Cladding: 22500 m^2

4 THE NEW CAPE TOWN STADIUM

Capacity: 68.000 (WC), 55.000 afterwards (temporary seats will be substituted by lounges)
Roof area: Glazing ~37.000 m^2, ~9000 single panes (2 × 8 mm TVG)
Membrane: 35.000 m^2 mesh fabric (PES/PVC) Facade area: ~27.000 m^2 mesh fabric (Glass/PTFE)

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Life-cycle performance, reliability, redundancy and optimal management of infrastructure systems: Applications to bridges and naval ships

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ABSTRACT

Throughout the life-cycle of an infrastructure system, gradual deterioration of its performance and sudden occurrence of hazards inflict risks that may seriously compromise the reliability of this system and make its use no longer safe. Optimally managed infrastructure systems are those with reliable predicted performance and planned maintenance interventions so that their life-cycle performance, reliability and redundancy are sustained, with a minimum expected cost, over their intended life-cycle (Frangopol 2010). The evaluation and prediction of the life-cycle performance, reliability and redundancy of infrastructure systems is crucial in ensuring their durability and the safety of their users (Okasha and Frangopol 2010a,b). In order to keep track of these structural qualities, indicators are developed and used as the main tool in deciding the optimization of life-cycle maintenance. The uncertainties in the available information for conducting life-cycle performance analyses impose a great challenge in the process. However, with the use of proper probabilistic tools for handling these uncertainties, more accurate outcomes do become achievable.

Considering multiple objectives in the optimization provides a set of optimum solutions, which give a more rational and flexible decision support (Okasha and Frangopol 2009, Frangopol and Liu 2007). The infrastructure system may also be monitored in order to improve the knowledge regarding the real loading demands on the structure, and reduces uncertainties in the input information. The proper management of a structure is best conducted according to an integrated and well coordinated life-cycle framework as shown in Fig. 1. A life-cycle framework is an interdisciplinary process that is implemented over the lifespan of a structure in order to upkeep its reliability and durability against deterioration due to aging processes and abnormal loads.

In this paper, the life-cycle performance, reliability, and redundancy of infrastructure systems and their role and integration in the optimal management of these systems are discussed. The targets of this study are highway bridges and ship structures.

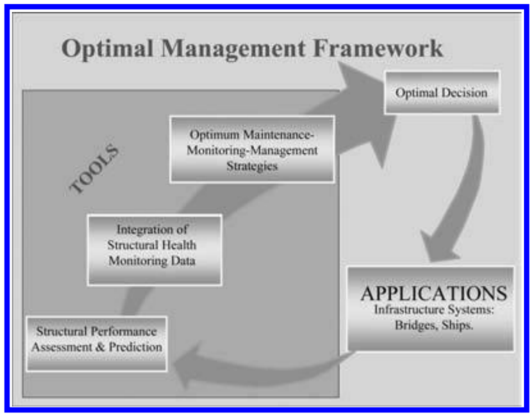

Figure 1. Life-cycle integrated management framework under uncertainty (adapted from Frangopol 2010).

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The use of Eurocode 3 "Design of Steel Structures" in the design and verification of joints

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ABSTRACT

Eurocode 3 "Design of Steel Structures" is available for use in practice. Joints are the most important components in steel structures and by the design of joints about 40% to 50% of the costs of steel structures are determined. So it is vital for practitioners to use the Eurocode 3 in an efficient way in order that steel structures remain competitive compared to other materials. Therefore it is important that there is sufficient explanation available how to use Eurocode 3 in general and the part 1–8 "Design of Joints" in particular. This paper focuses on the global design of frame structures, more in particular how joints play an important role in this process. Furthermore attention is paid on how to verify the moment resistance (strength), the rotation rigidity (stiffness) and rotation capacity (deformation capacity) of joints in frames and trusses. The first reaction about the Eurocode 3 from practitioners is that the Eurocodes are very complete but complex to use. As a reaction to that, the paper shows that indeed the Eurocode 3 part 1–8 "Design of Joints" is rather complete but the question remains how to deal with joints which are not easily covered by the Eurocode and that the Eurocode is not complex but indeed it is a lot of work using it. The paper gives guidance how to use the computer in the design and verification activities that the practitioners have to carry out designing steel frame structures taking the slogan "Simple Tools sell Steel".

In order to keep a competitive position in the market, the costs of steel structures, in particular steel frames, need to be reduced as much as possible. As the costs of steel frame structures are determined for about 40% to 50% by its joints, the need to design modern joints preferably without unnecessary stiffeners is of increasing economic importance. In this way the costs fabrication and of on-site construction work together with safety measures can be reduced significantly. Although the Eurocode 3 "Design of Steel Structures" Part 1–8 "Design of Joints is still based on traditional joints with bolts and welds, in many cases these design rules can be used for the design and verification of modern joints in which the basic components can be recognized. This is because the design rules for joints are related to these basic components in which almost all joints can be sub-divided and because the requirements for stiffness, strength and rotation capacity of joints are given in so-called performance based requirements and are irrespective of the type of the joint. However, where non-traditional joints, like so-called "Plug and Play" joints with components like clamps and hooks are used, experiments have to be carried out and the results have to be evaluated statistically, in order to obtain reliable design values for stiffness, strength and rotation capacity to be used in the design of the structure. Last but not least there is a need for reliable software for verifying the structural safety of steel structures including the joints to make the work for the designers much more efficient.

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Design life of thin-walled concrete domes

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ABSTRACT

Thin-walled concrete domes and shells are subjected to indirect straining during their design life, as a result of shrinkage and thermal straining. In particular, shrinkage and creep strains lead to time-dependent deflections and cracking, and these effects may cause creep buckling of the thin-walled concrete shell. Although the effect is usually associated with serviceability limit states, when it causes buckling the effect must be associated with strength limit states and so governs the design life of the dome.

The paper investigates the short and long-term buckling response of thin-walled axisymmetric shallow concrete domes of the type shown in Figure 1 with creep and shrinkage deformations, both theoretically and experimentally.

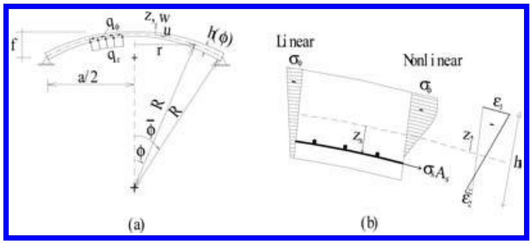

Figure 1. Axisymmetric shallow thin-walled concrete dome.

Figure 2. Shallow dome tested to failure.

The theoretical model accounts for the non-linear behaviour of the materials under failure levels of load, the creep and shrinkage of the concrete material, and the buckling of the dome. The field differential equations for the short-term and long-term analyses, and the challenges associated with the modelling and non-linear analysis of concrete domes, are addressed in the first part of the paper. In particular, it is shown that the non-linear material behaviour of the concrete material under biaxial loading, its interaction with other stability issues, and the aging of the concrete require the use of detailed modelling and analysis.

For the experimental study, the testing of two moderate-scale shallow domes of the type shown in Figure 2 under short-term and long-term loading are reported. A series of tests for the characterisation of the mechanical properties of the concrete material are also discussed. The experimental results show that thin-walled shallow concrete domes are characterised by buckling failures under short-term loading, and that creep significantly reduces the buckling resistance of these structures.

The comparison between the theoretical and the experimental results reported in the paper clarifies some of the aspects of the structural response, and provides a level of validation of the theoretical model, with some aspects that need to be further elucidated. These include the consideration of the non-axisymmetric deformations of the dome, the effects of geometric and material imperfections, different thickness profiles to be considered in the analysis due to uncertainties with regard to the actual profile, local damage, and others. The paper concludes that the modelling and the theoretical and experimental results shed light on the failure behaviour of shallow concrete domes, and it contributes to the establishment of theoretical knowledge required for their analysis, effective design, and safe use.

2. Dynamic analysis, vibration analysis, vibration control

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Structural dynamics research and applications

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ABSTRACT

Research in structural dynamics with real world applications has received considerable attention in recent times. This has happened due to two reasons. On the one hand research was needed to address the issues pertaining to (i) vibration problems in slender structures which have emerged as a consequence of new materials technology and aesthetic requirements, (ii) increased vulnerability of structures to random and unpredictable loads such as seismic, impact and blast loads and (iii) safety concerns of aging infrastructure. On the other hand the research has been made feasible by the availability of enhanced instrumentation and sophisticated computer simulation techniques. Motivated by the need to address some of these issues and the knowledge gaps therein, a number of research projects have been undertaken at the Queensland University of Technology using computer simulation techniques supported by experimental testing. The research focused on three areas: (i) disaster mitigation of structures under impact, blast and seismic loads, (ii) vibration in slender structures and (iii) structural health monitoring of bridge and building structures.

This paper presents the research carried out in four projects: (i) dynamic characteristics of a composite floor, (ii) dynamics of a cable supported foot-bridge, (iii) seismic mitigation of a shear wall frame structure using passive dampers and (iv) damage localization in a bridge girder. It discusses the main findings and their applications. Some of the findings of these projects are summarized as follows.

Higher and multi-modal vibration can be induced in composite floors under human induced pattern loads (De Silva and Thambiratnam, 2009). In addition, these higher modes have an impact on adjacent panels not exposed to direct activity. Cable supported footbridge structures often exhibit coupled vibration which can be excessive under certain load patterns (Huang et al, 2005). For comprehensive dynamic evaluation of these slender structures it is necessary to consider the types of analytical techniques used in the studies referred to herein.

Strategically located passive dampers can provide seismic mitigation of shear-wall frame structures. The best results are obtained when visco-elastic dampers are placed at the lower levels and/or when friction dampers are placed in regions of high inter-storey drifts (Marko et al, 2006).

Accurate damage localisation in bridge girders is possible by considering the changes in the two vibration based parameters of modal flexibility and modal strain energy (Shih et al, 2009). This technique can be extended for damage localisation in the main load bearing elements of slab on girder and truss bridges, under single and multiple damage scenarios and hence will contribute towards maintaining the structural health of aging bridge structures.

The findings of this paper will enhance our understanding in some of the important areas of structural dynamics and will find applications in the real world.

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Dynamic response of large primary structures to wind loads: Experimental tests in wind tunnel

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ABSTRACT

The evaluation of the response of an arbitrary structure to wind induced loads can be extremely difficult, due both the air flow – structure interaction, and to the structural behaviour.

Moreover for very complex structures or in case of complex orography it is necessary to proceed to wind tunnel tests in order to correctly estimate the actual pressure distribution or load resultant over the structure. Hence it is often necessary to develop the following steps:

- Perform the wind tunnel test campaign in order to recover the actual pressure distribution or load coefficients on the considered structure;
- Generate time histories of random processes with the same statistical characteristics of the wind induced load on the given structure;
- Perform a full time domain analysis, including any non-linearities.

In the present work some results on wind tunnel tests at CRIACIV laboratory and on structural dynamic response will be presented with reference to roofing structures and bridge sections. In particular for it that concerns some roofing structure a reference to the following works will be made: Olympic stadium in Piraeus (Figure 1), High speed train station and Highway tollgate roofs in Reggio Emilia.

Figure 1. Model of the Piraeus stadium.

Figure 2. Model of the Piraeus stadium.

Moreover some results about the test campaign on the section model of the Messina bridge (Figure 2) will be shown.

For it that concerns the proposal for the Messina Strait Bridge (Figure 2), wind-tunnel tests have also been performed on several section models at a scale of about 1:100. The purpose of the static tests was the determination of the aerodynamic load and its sensitivity to the presence and degree of porosity of grids and other nonstructural details.

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On space-time isogeometric analysis of panel flutter

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ABSTRACT

This contribution discusses the modelling of panel flutter due to high-frequency movement-induced excitations, which is of interest in aerospace and civil engineering, since it may lead to fatigue failure due to vibrations even if the amplitudes are rather small, i.e. of order of the panel thickness [1, 2].

Flow around and inside civil engineering structures can be the source of harmless motions as well as dangerous and destructive structural vibrations. Interaction effects between fluid flow and structures have become important with the utilization of new materials for lighter constructions and crucial temporary states during erection. The resulting designs are often more sensitive to dynamic excitation while ad-hoc prediction of their response behaviour to flow-induced excitation is not available to the design engineer.

In civil engineering, the majority of structural designs lead to bluff objects subjected to fluid flow like wind and water, which are seldom aerodynamically optimized.

The lack of a universal straightforward model for fluid-structure interaction established the field of experimental and numerical investigation of flow-induced vibrations. Nowadays, numerical simulation tools gain acceptance in wind, coastal and hydraulic engineering as they provide important local and global information of e.g. the stress and deformation state and therefore enable further insight to physical mechanisms of coupled problems.

A concept of isogeometric analysis [3] based on Non-Uniformly Rational B-Splines (NURBS), which are popular in computer graphics and computer-aided design communities for representing surfaces, is adopted for numerical analysis and extended to be used within a concept of space-time discretization [4, 5, 6]. A concept of the Space-Time Isogeometric Analysis (STIGA) is proposed in this contribution.

One of the nice features of the isogeometric analysis, compared to FEM, is that the former performs very well throughout the whole frequency spectrum, whereas the latter results in a so-called acoustical-optical branch effect, i.e. the accuracy of the FEM drastically decreases in the high-frequency range, if the order of the basis function is increased (p-refinement). The advantages of the NURBS-based approximation are presented and the conditions for onset and vibration control [7] of flutter as well as different flutter modes are investigated.

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Computation method for the prediction of gust-induced vibrations for permeable membrane cladding elements

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ABSTRACT

Due to the current tendency towards permeable membrane elements as lightweight and translucent silhouettes for buildings, dynamical wind aspects become more relevant during the design phase.

The structural characteristics of pretensioned membrane elements are dominated of significant nonlinearities. Although with the numerical time step method there principally exists a precise procedure for the determination of the structural answers, the associated numerical effort makes an alternative spectral method desirable. Therefore, in this paper a nonlinear corrected spectral (NCS) approach for MDOF systems is presented, which allows for the determination of stochastic responses under usage of a correction algorithm which considers the geometrical nonlinearities. Different dynamic discretisation levels have been considered (1-DOF, 3-DOF and 9-DOF) to take into account the coherence influence of the generated wind fields and a more realistic load distribution over the system's length.

To model the spectral loading for the frequency domain approach, the power spectral densities $S_{VV}(f)$ of the wind velocities according to current wind standards, are directly usable. Furthermore, they base for the generation of artificial wind fields to perform calculations in the time domain (to obtain reference results) by means of a *Shinozuka-II* wind field generation approach. Due to the consideration of MDOF systems, the correlation structure of multivariate stochastic processes therefor has been taken into account.

For the determination of dynamic responses, an alternative procedure is described which takes place in the frequency-domain: the NCS (i.e. nonlinear corrected spectral) method. As generally, for nonlinear structures the superposition principle is not valid, for the approximate consideration of the nonlinear influence, a correction algorithm is formulated which devides the spectral density of the linear response SAA in n-bands for a subsequent frequency dependent correction (Kemper et al. 2010).

To summarise the performed numerical simulations (time-domain and NCS), the structural deflections r, the associated deflection shapes and the inner tensile forces N are compared. For the computed structural systems with $\delta \leq 80$ (Structure S03), the maximum deviations for the structural deflections and the tensile inner force are smaller than 13%. Even the highly nonlinear structure S04 has been approximated with a variation of 16% for the maximum deflection.

It has been shown, that the decisive structural results (deflections and inner forces) could be determined with a suitable approximation by the NCSmethod. The presented approach is applicable for the structural design of nonlinear MDOF systems and advantegeously for the performance of parameter studies. The obtained results are verifiable good approximated compared to transient methods.

Table 1. Parameters of the structural systems S00-S04.

	S00	S01	S02	S03	S04
A [mm^2]	0.001	0.01	0.1	1.0	10.0
ε_0 [−]	1.0	0.1	0.01	0.001	0.0001
δ [1/m^2]	8.0E−2	8.0E−1	8.0E+0	8.0E+1	8.0E+2

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Aerodynamic force coefficients for rain-wind induced vibrations from full-scale measurements

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ABSTRACT

Numerical and experimental investigations indicate that rain-wind induced vibrations (RWIV) can considerably affected by the wind turbulence, so that, at high turbulence intensities, an excitation is not expected. However, due to the scale factor between the rivulet on the cable and the integral length scale of turbulence in wind tunnel tests, the experimental results can only be interpreted as a trend. To avoid these scale problems, static and aeroelastic experiments on a full-scale section model in the natural wind are arranged, to analyse the quasi-steady aerodynamic force coefficients and the excitation mechanism in turbulent flow, respectively. The aerodynamic forces of a cylindrical section model with an artificial rivulet are measured with load cells, while a 3D ultrasonic anemometer records the wind velocities (Fig. 1).

Figure 1. Test rig for measuring aerodynamic forces on a cylindrical section model with artificial rivulet.

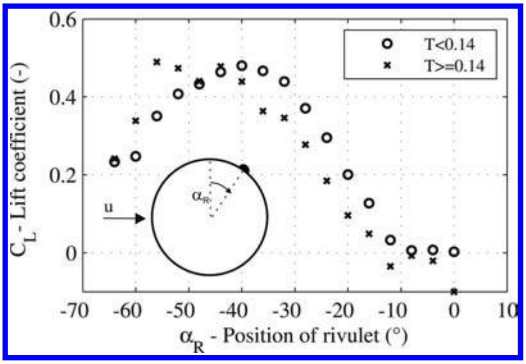

Figure 2. Turbulent aerodynamic lift coefficients of a cylinder with artificial rivulet.

Due to the non-stationarity of the natural wind, a special analysis technique is proposed that yields quasi-steady aerodynamic force coefficients: Extracting trendless 10-min-events from the recorded wind data, calculating Taylor's parameter $T = I_u(D/L_{ux})^{0.2}$, low pass filtering the data to eliminate the influence of the small scale turbulence, classifying momentary aerodynamic forces and computing the least square solution for each class.

For rivulet positions $-40° < \alpha_R < -10°$, the results indicate the typical unstable region (Fig. 2), which is assumed to be responsible for the excitation mechanism of RWIV (Hikami & Shiraishi 1988). In order to verify this assumption, aeroelastic tests using artificial rain are pending.

Due to the small changes of the Taylor parameter, no distinct trend can be identified for the influence of turbulence. More test series under different flow conditions are necessary.

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Shape modifications of bridge cables for aerodynamic vibration control

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ABSTRACT

In this paper, the viability of modifying cable shape and surface for the purpose of controlling wind-induced vibrations is examined. To this end, an extensive wind-tunnel test campaign was carried out on various cable sections in the critical Reynolds number region under both smooth and turbulent flow conditions.

Cable shapes were chosen to passively modify flow in a particular manner. Shape modifications of a plain cylinder included waviness, faceting and shrouding (see Fig. 1). The cable models were tested in a static inclined test rig, allowing them to be installed at varying cable-wind angles.

The aerodynamic damping of each section is evaluated by applying 1- and 2-DOF quasi-steady aerodynamic models (Macdonald and Larose, 2006, 2008). This allow for the prediction of regions of aerodynamic instability, as a function of flow angle and Reynolds number.

The evaluation of aerodynamic instability shows that the plain cylinder could be prone to both “drag crisis” and dry inclined galloping at specific skew winds.

The wavy cylinder have the properties similar to a typical rough circular cylinder where no significant drag increase is found, indicating some effectiveness of the waviness. The steep dip in drag results in the prediction of a “drag crisis” instability. Outside the critical Reynolds number region, it is more stable than the plain cylinder which could be a result of the reduced axial flow.

The hexagonal faceted cylinder had too large a drag coefficient (1.5–2.0) and depending on the angle of attack and the angular variation of the geometry, Den Hartog galloping is predicted.

The shrouded cylinder is found to have a very low dependency on the Reynolds number and a drag coefficient slightly above 1.0 based on the inner diameter. The shroud is found to stabilize the cylinder against any type of dry state instability and it also significantly reduces the vortex-induced oscillating lateral forces.

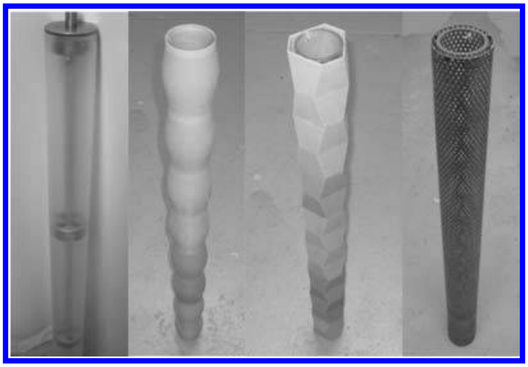

Figure 1. Test models (from left to right) plain cylinder, wavy cylinder, faceted cylinder and shrouded cylinder.

Finally, turbulent flow is shown to introduce a significant amount of aerodynamic damping by proving a more stable lift force over tested Reynolds numbers. This could be a result of the free-stream turbulence suppressing the formation of single separation bubbles.

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Pedestrian induced vertical vibrations: Response to running using the Response Spectrum Method

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ABSTRACT

Most international codes of practice do not provide an adequate or accurate methodology for the response prediction of a footbridge to the loading due to a crowd. Moreover the loading due to other kinds of motion, like running or jumping are not treated at all.

A codifiable and accurate methodology for the evaluation of footbridge response to running is here presented, based on the already validated Response Spectrum Method due to Georgakis & Ingólfsson (2008) for walking crowds. The pedestrian crowd loading is generated through Monte Carlo Simulations and the footbridge response is given in terms of peak acceleration as a function of the return period of the loading. Since the Response Spectrum is developed for reference bridge and crowd morphology, correction factors are introduced to accommodate for other structural as well as crowd configurations.

A pedestrian's behavior when crossing a footbridge is assumed random and, therefore the main parameters influencing the loading function, i.e. pacing frequency as well as running speed, are modeled according to statistical distributions.

The vertical component of the physical foot force whilst running is reproduced by the model of jumping on a spot, which is mathematically described by a half-sinus wave during the contact time, (Bachmann and Amman, 1987).

The footbridge response is evaluated for a reference footbridge configuration assuming that the structure is linear, viscously damped, and with well separated the mode shapes.

The reference response spectrum, defined as the peak acceleration associated with a certain return period, has been created based on Monte Carlo simulations. For each simulation, lasting a total time of 8 hours, the footbridge acceleration response is divided into $n = 96$ non-overlapping time windows, each of duration $T_w = 300$ s; from each time window the peak acceleration is then extracted and a General Extreme Value (GEV) distribution is fitted to the peaks.

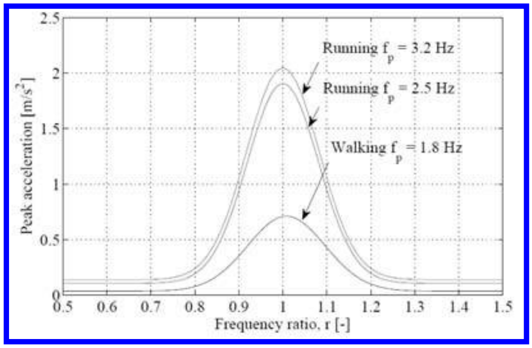

Figure 1. Reference response spectra for 10 minutes peak acceleration for different pedestrian populations.

In Figure 1 are plotted the reference 10 minutes response spectra for two running populations with characteristic mean pacing frequencies of 2.5 Hz (slow running) and 3.2 Hz (fast running), together with the spectrum developed by Ingólfsson et al. (2008) for a walking pedestrian populations characterized by a mean pacing frequency of 1.80 Hz.

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Human induced vibrations of lightweight floor systems supported by cold-formed steel joists

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ABSTRACT

The intention of this research is the development of dynamically optimized floor structures made of cold-formed steel C-shaped joists with a deck made of cementitions or wood based plates. The perception of vibrations or the definition of comfort and discomfort of floor vibrations is very subjective. Therefore it is very difficult to define suitable acceptance limits of floor vibrations. Limits differ between various publications. Many design criteria use a limitation of midspan deflection under permanent load. But this is only essential to define a minimum fundamental frequency and so a protection against resonance. This is no limitation of accelerations.

Tests were performed to assess the dynamic reaction close to the reality, the natural frequencies, the mid span deflection, the damping ratio and to find all necessary parameters for a numerical simulation. A three dimensional finite element model of the test floor was used to simulate the dynamic behavior. After calibration the results of the numerical simulation displayed a good correlation to the measured results. So we received a well suited model for parametric studies by variation of mass, stiffness and damping.

But the results of these studies are unsatisfactory. Typical recommendations are increasing of stiffness, a better load sharing or reduction of span if possible. Increasing of mass is counterproductive to light floor system and so not an adequate solution. All these approaches upgrade the vibration behavior but not in the necessary quality.

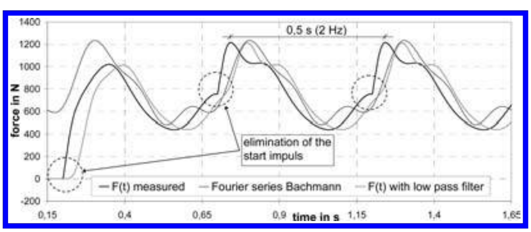

Figure 1. Comparison of excitations.

Another approach is energy absorption directly at the excitation. The load-time process of walking can be idealized by a constant part and a series of sinus curves. The frequencies (Ω_i) and the amplitude (F_i) can be calculated by a Fourier analysis of the measured load time process. Based on the fact that by using the modal analysis every single eigenmode ψ_i is isolatable and so a multi degree of freedom system (MDOF) can be numerical reduced to a number of idealized single degree of freedom systems (SDOF). The results of every sinus curve can be linearly superposed under consideration of the phase shift. Equations (1 & 2) show the solution of the differential equation of a single curve $P_i^*(t)$. Every time the crux of the lightweight floor systems is searched by the low mass. But the partial solution (1) shows that the dynamic response of a sinus curve is only depending on the force and the stiffness. There is no mass in the equation. The maximum amplitude is delivered by the homogeneous part of the solution (2). And this is only depending on the induced start impulse. At our example it is almost a linear increase of the force.

$$\ddot{x}_p = -\Omega^2 \cdot A \cdot \sin(\Omega t - \phi) \cdot v_f = -\frac{\Omega^2 \cdot F_0}{k} \cdot \sin(..) \cdot v_f \quad (1)$$

$$\ddot{x}_h = -\frac{\Omega \cdot F_0}{\sqrt{k \cdot m}} \cdot e^{-D \cdot \omega t} \sin(\omega \cdot t - \varphi) \quad (2)$$

Floors with a high construction weight show other relationships between the homogenous and partial solution. But for the design of lightweight floors it is required to regard this effect. In figure 1 the force time processes look almost identical. The only difference is the reduction of the inducing impulse and so the dynamic response depends on the stiffness.

Trying to optimize the lightweight floor systems for the dynamic load of walking, it is essential to increase the stiffness by activating non structural elements. More important is the reduction of the start impulse of the load of every single foot.

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A new method for evaluation and assessment of floor vibration

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ABSTRACT

To achieve flexibility concerning space-arrangements and kind of occupancy multi-storey buildings require large span floor structures with a minimum of internal columns and walls. Modern materials and of construction methods as slim floors, composite floor systems or pre-stressed flat concrete floors with high strength materials are becoming more and more suitable to fulfill these requirements. It is common for slender floor structures that their design is usually not only controlled by the ultimate limit states but by serviceability criteria as deflections and vibrations. However, the calculation and assessment of floor vibration is still connected with a number of uncertainties that are related to a suitable design model including all aspects of dynamic response, the choice of boundary conditions for the model, the form and magnitude of excitation and the judgment of floor response in the light of the use and acceptance of the users.

The paper provides a procedure for the determination and assessment of floor responses to walking of pedestrians that takes on one side account of the mechanical vibration problem, but on the other side leads – by appropriate working up – to easy to use design charts, Figure 1.

The procedure was developed in different European research projects which finally ended up in a guideline and a background documentation which can be accessed for free from the "HIVOSS" project web page in different languages. This guideline contains design charts and gives advice how to apply them. With about 11000 downloads since March 2009 the guidelines found huge interest in practice and research.

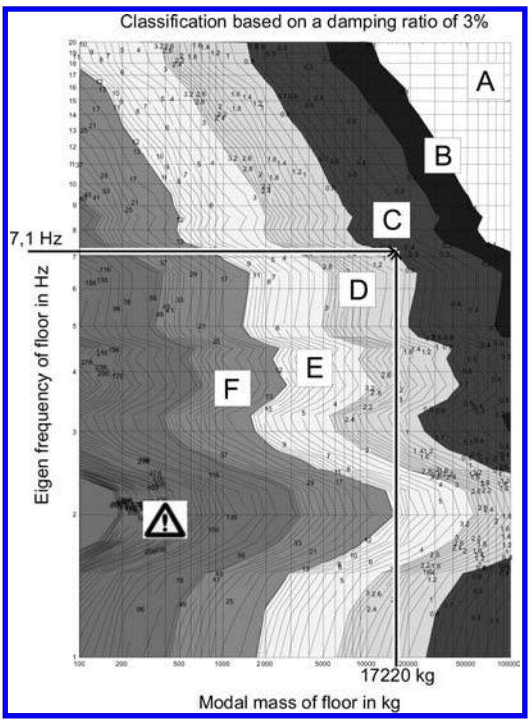

Figure 1. Example for a design diagram – here 3% damping ratio of the floor an its finishing.

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Dynamic testing of bracing patterns of a demountable grandstand

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ABSTRACT

Several investigators have considered bracing systems for temporary grandstands and other structures in general and recommended guidelines for effective bracing systems based on fundamental structural principles and performance of laboratory models (Ji et al. 1997). However, there is very little measured data on the actual performance of bracing patterns in real life structures. In addition, the potential nonlinear behaviour due the use of diverse materials and complex connector technologies common in modern scaffold systems is not accounted for.

In this study, a series of vibration tests were conducted to determine the horizontal dynamic characteristics of a novel modular demountable grandstand system for a range of bracing-patterns and load cases. The tested grandstand module (Fig. 1) is used as a repeating building block of a large-scale system. Fifteen bracing patterns were investigated and are illustrated in the main article. The results were plotted against a frequency limit of 4 Hz which is the recommended requirement for temporary grandstands (IStructE 2007).

The main results found are depicted in Figure 2. The horizontal axis represents the bracing patterns tested which were assigned numbers 1 to15. The letters FB, SS and T represent front-to-back, side-to-side and torsional vibration modes. The latter were observed more often because of the towering nature of the tested module, representing a more extreme case. It is evident that the stand met the 4Hz requirement for a majority of bracing patterns tested. However the dynamic response of the stand was also characterized by several aspects of nonlinear behavior which are discussed in the main article.

It is clear from these results that the bracing pattern has a primary effect on the dynamic characteristics of the stand; however the effect of nonlinear structural behavior on the dynamic response is also significant. The frequencies reported here, which were found at modular level, will be used to predict the stiffness of a large-scale system by taking into account the nonlinear effects observed. Complying with the frequency based design approach which is applicable only to linear structural behavior is also less justifiable in the case of nonlinear behavior, making a nonlinear analysis approach necessary.

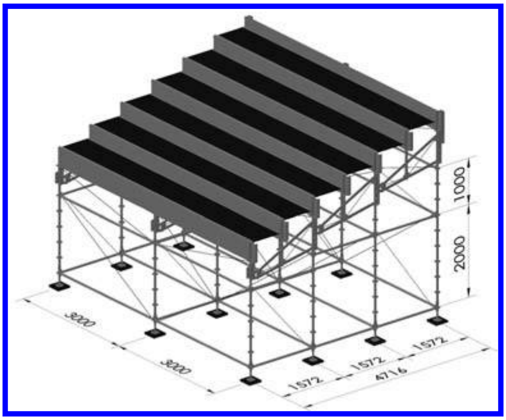

Figure 1. ClearView™ grandstand system.

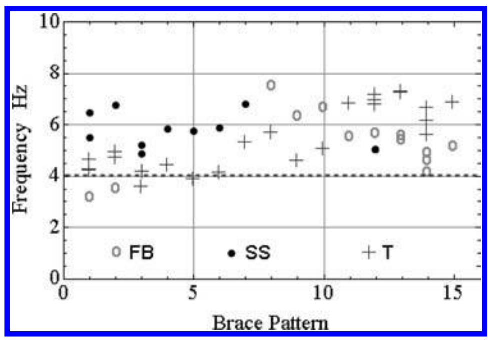

Figure 2. Effect of bracing pattern on stand frequencies.

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Asymptotic results for highly anisotropic spinning disks

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ABSTRACT

Imperfect mechanical structures are used frequently for investigating stability issues. One of the attractive features of this approach is intimately linked to the fact that it avoids solving complicated eigenvalue problems. The critical instability thresholds are identified as those values of the loading parameter for which a displacement (or some other mechanically relevant field) becomes unbounded. The first chapter of the classic text (Timoshenko 1961) contains a representative selection of problems in this direction.

From a theoretical point of view, linear imperfect problems are related to generalised eigenvalue problems associated to inhomogeneous differential equations, such as

$$\mathcal{M}[\boldsymbol{w}] - \lambda\mathcal{N}[\boldsymbol{w}] = \boldsymbol{f}, \tag{1}$$

where $\mathcal{M}$ and $\mathcal{N}$ are differential operators, λ plays the role of a loading parameter, and the *arbitrary* function $\boldsymbol{f}$ characterises the degree of imperfection. The unknown $\boldsymbol{w}$ and the right-hand side $\boldsymbol{f}$ are possibly multi-dimensional vector fields. Equation 1 allows us to formally write $\boldsymbol{w} = (\mathcal{M} - \lambda\mathcal{N})^{-1}[\boldsymbol{f}]$, and thus it seems reasonable to expect that $\boldsymbol{w} \to \infty$, as λ approaches one of the eigenvalues of the homogeneous generalised eigenproblem (corresponding to $\boldsymbol{f} \equiv 0$).

The work reported here is related to precisely such a general framework; full details are available in (Coman 2010). In particular, we are interested in the potential instabilities experienced by a highly anisotropic spinning disk. From the point of view of numerical analysis the problem was discussed in (Portnov et al. 2003). On the practical side the motivation comes from recent technological advances in the manufacturing of hoop-wound composite flywheels having elastomeric resin and carbon fibres. Characterised by strengths comparable to their isotropic counterparts, these structural components are significantly lighter and allow much higher speeds of rotation. For instance, in the case of composite disks based on carbon fibres in a flexible polyurethane resin, the ratio between the Young's moduli in the azimuthal and radial directions is as large as 1.7×10^3. This observation calls for the use of asymptotic methods in a natural manner.

By considering the plane-stress systemof equations of linear elasticity for the steady rotation of an annular disk with polar orthotropy, a careful rescaling reveals that the corresponding homogeneous problem depends on two key parameters,

$$\mu := \frac{E_\theta}{E_r}, \quad \beta := \frac{E_r}{G_{r\theta}},$$

where E_r, E_θ, and $G_{r\theta}$ have the usual connotations. The linearity of the problem suggests the use of Fourier series in the azimuthal direction, leaving us with a set of differential equations with variable coefficients which depend on the above two parameters, as well as on the Fourier mode number $n \in \mathbb{N}$. The asymptotic regime investigated corresponds to $\mu \gg 1$ and β, $n = \mathcal{O}(1)$. Our main result is an approximation of the critical angular speeds – in non-dimensional form λ – via the following compact formula

$$\lambda = \lambda_0 + \lambda_1\mu^{-1/2} + \mathcal{O}(\mu^{-1}),$$

where $\lambda_j = \lambda_j(\beta, n)$ are found through a series of manipulations with Bessel functions.

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Modelling and verifying of an anthropomorphic passenger dummy model using multibody dynamics

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ABSTRACT

Current Guardrail-systems, which are mounted on European streets, have to fulfil several standards regarding their containment ability and their deformations under some impact of different standardised vehicle types. Besides the guardrail's behaviour itself, the protection of the vehicle's occupants on the basis of the evaluation of load signals during impact is a major aspect of these standards in terms of threshold values (CEN 2007).

These threshold values are currently obtained by extrapolating the accelerations of the vehicle measured in its centre of gravity. To achieve more realistic physiological stresses of passengers, numerous simulations of impact tests have been carried out within the frame of the research project P 717 of the German research foundation FOSTA (Feldmann. et al. 2010). To reduce the enormous costs of real impacts, a passenger dummy model was embedded in an existing multibody dynamics vehicle in order to allow for computational parametric studies.

The method of multibody dynamics can be applied to simulate the dynamic behaviour of structures with large nonlinear movements. To model a mechanical system as a multibody-system (MBS), it is necessary to assign properties such as elasticity, viscosity, friction, damping, inertia and force to discrete rigid body elements, springs and dampers, friction, impact and contact elements, which then are combined in a global model. The rigid bodies react according to external, internal, inertia and constraint forces.

The object-oriented multibody-system program MEPHISTO (**M**ultibody systems with **E**lastic **P**lastic **Hi**nges and changeable **ST**ructure **O**rganisation) was specially developed for use in civil engineering applications at RWTH Aachen, Institue for Steel Structures (Neuenhaus 1993). The program contains all modules necessary to model the dynamic performance of members, connections and any mechanisms including contact or failure.

Based on the new software solution, the relationship between the vehicle's accelerations and the loads acting on occupants could be analysed in detail. This paper focuses on the modelling and verifying of an anthropomorphic passenger dummy model using the method of multibody dynamics.

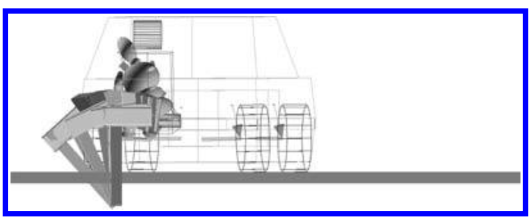

Figure 1. Passenger dummy model during impact.

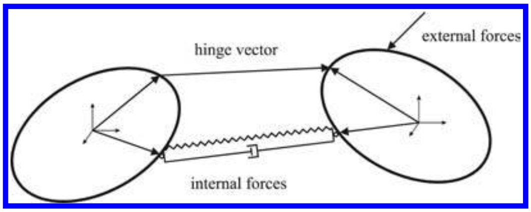

Figure 2. Rigid body model.

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Transmissibility of whole-body vibration experienced by off-road vehicle operators based on ISO 2631-1 and ISO 2631-5

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ABSTRACT

The current study evaluated the tri-axial acceleration transmissibility from the floor to the operator seat of off-road vehicles using two methodologies on rough terrain where transient shocks are dominant. Acceleration transmissibility refers to say, the ratio of acceleration at the floor of a vehicle to the acceleration at the seat of the vehicle. There have been several transmissibility studies. Most whole-body vibration investigations carried out in the past measured either vertical or horizontal transmissibility from floor to seats separately. However, it has been shown that horizontal vibration as well vertical vibration at the floor does contribute to vibration in either axis at the seat. Experimental data was collected in accordance with the measuring procedure of the ISO 2631-1 (1997) standard and the measurement period lasted for over one hour using tri-axial accelerometers and SVAN 958 Human Vibration Meter. Subsequently, analysis was carried out using MATLAB based software complying with the ISO 2631-1 (1997) and ISO 2631-5 (2004) standards. This study therefore evaluated transmissibility from the three axes at the floor to the three axes at the seat. The study was done using the established ISO 2631-1 (1997) and ISO 2631-5 (2004) for transient shocks and the differences explored.

For vibration without transient shocks ISO 2631-1 (1997) is sufficient for evaluating the exposure levels. However, the ISO 2631-1 (1997) does not handles shocks sufficiently (Marjanen (2005)). Therefore, for high amplitude shocks experienced in off-road vehicles, a separate approach needs to be taken. Hence, the ISO 2631-1 (1997) and the new ISO 2631-5 (2004) which handles shock much better would be used in the computation of transmissibility and compared (ISO 2631-5 (2004)).

To the best of our knowledge there is no transmissibility study that has been carried out on ISO 2631-5 (2004). This study computes off-road vehicle transmissibility using ISO 2631-1 (1997) and ISO 2631-5 (2004) standards. The objective of this paper is therefore to evaluate the whole-body vibration (WBV) results obtained on off-road vehicle using ISO 2631-1 (1997) and ISO 2631-5 (2004) standards.

Table 1. Comparison of transmissibilities.

Machines	T_r Based on VDV	T_r Based on S_{ed}	Percentage Difference
Case 1	1.02	1.30	21.54
Case 2	1.00	0.58	72.41
Case 3	1.08	1.59	32.08
Case 4	1.00	0.80	25.00

From Table 1 it can be seen that there is a significant difference between the transmissibilities based on VDV of ISO 2631-1 (1997) and the S_{ed} based on ISO 2631-5 (2004). The minimum difference between the transmissibilities is 21.54% and the maximum difference as high as 72.41%. This difference is as a result of the different frequency weightings based on the two standards. The new standard handles shock better than the old standard.

Transmissibilites of off-road vehicle have been computed and evaluated based on the ISO 2631-1 (1997) and ISO 2631-5 (2004) standard methodologies. From the computed transmissibilities it was found that the percentage difference between the ISO 2631-5 (2004) and the ISO 2631-1 (1997) standard in the presence of shocks was significant. It is recommended that the new ISO 2631-5 (2004) be used where transient shocks are dominant. It is equally advisable that more transmissibility studies be carried out using the new methodology.

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Dynamic behavior of soil-steel bridges

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ABSTRACT

Loads moving across the bridge at high speeds cause vibration of the bridge structure, impact as a result of rough road surface and unequal deflections of the vehicle springs, etc. Dynamic reaction stress exerted on elements of the bridge structure as well as the strains caused by mobile rolling stock become bigger than similar static loads; i.e. slow placement of immobile rolling stock of the same or similar weights. The exact calculation of the bridge dynamics considering all the above mentioned factors, the free vibration of the structure and also possibilities of resonance of the bridge require exceptionally complicated calculations which do not yet yield an absolute satisfactory solution.

Generally, the dynamic coefficient value is related to the so-called critical speed of the truck and the value of the largest vibration amplitude that occurs. This velocity can be calculated using many tests (movement of the same load at different velocities across the same bridge – Figure 1). The critical speed is defined as a speed during which the value of dynamic coefficient is maximum.

For the dynamic tests, two inertial inductive sensors were fixed at the edge of roadway (or sidewalk) and reinforced concrete collar and strain gages for strains measurements in the transversal and longitudinal directions of the bridge respectively, located on top of the corrugated plates within the effective span.

Figure 1. Front view on the *Scania* vehicle passing with speed of 40 km/h by the threshold during dynamic tests.

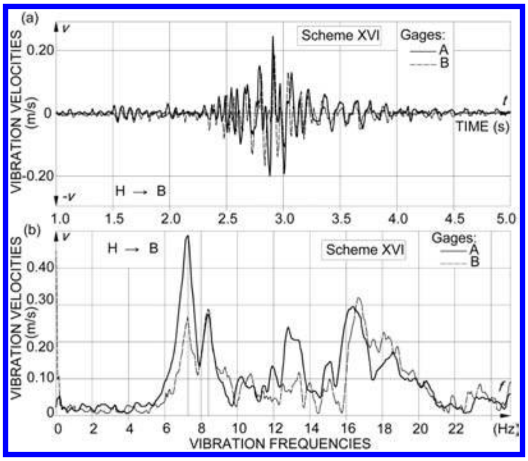

Figure 2. Courses: (a) vibrations velocity in time, and (b) corresponding to the vibrations frequencies of the structure in the selected points (A & B) during passing a vehicle with a speed of 70 km/h (Gimån Bridge).

Some selected graphs of the dynamic results of the soil-steel bridges is given in Figure 2.

The obtained dynamic amplification factor (DAF) values based on the research were compared to values given from three bridge design codes that were calculated as fallows:

1) The Polish Standard of PN-85/S-10030 given formula (1):

$$\varphi_{\text{PN-85}} = 1.35 - 0.005 l_\varphi \le 1.325. \tag{1}$$

2) The Eurocode 1 (2002) suggests the usage of Eq. 2 for the dynamic coefficient (DAF) for the well maintained truck:

$$\varphi_{\text{Eurocode}} = \frac{1.44}{\sqrt{l_\varphi} - 0.2} + 0.82 \text{ with } 1.0 \le \varphi \le 1.67 \cdot \tag{2}$$

3) According to the Swedish Bridge Design Code BV Bro (2004), the DAF is calculated as in Eq. 3:

$$\varphi_{\text{BV Bro}} = 1 + \frac{4}{8 + l_\varphi} \cdot \tag{3}$$

where: h_c is the cover height including the ballast, l_φ is effective span of shell in meters.

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The stick-slip joint effect on dynamics behaviour of nonlinear beam

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ABSTRACT

This work focus on modeling the frictional joints, such as bolted or riveted joints.

It is found that the frictional joints are extensively used in a variety of application since the antiquity, particularly recently in aerospace industries.

To date they dominate jointing mechanism to join not only secondary but also primary structural components.

In spite of the huge development in numerical simulation, the area of this sort of modeling will be a hot research for years.

The mechanism of frictional joints mainly depends on creating a tangential frictional force by a normal clamping force to prevent any possibly relative motion between the contacted parts. However, it was found that the relative motion is inevitable. The fretting corrosion is a concrete prove of the presence of motions.

It is also found that the relative motion has a beneficiate effects as a major role in energy dissipation in structures thereby that improve the damping properties in the modern machine.

Most of the simulation process of such joints is either oversimplified or unacceptable detailed models. The oversimplified models adopt the classical boundary conditions. While the other type of modeling rely on 2D or 3D detailed finite element simulation. However, the detailed simulations are useful to understand the local behaviour of such joints, but it has drawbacks mainly the high computational costs, the lack of coupling with real built-up structures difficulties in implementation.

In this paper an attempt to work out a compromise solutions between the two schemes. Therefore, the joints are modeled using four mechanical elements namely: Linear horizontal, linear vertical, rotational springs and friction element using Coulomb's law (see Fig. 1).

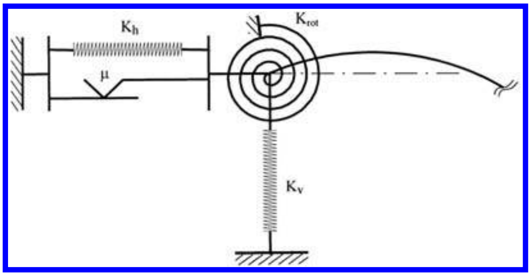

Figure 1. Idealized riveted/bolted joint.

The suggested model has been analyzed simultaneously with geometrical nonlinear beam model instead of shell model for sake of simplicity. The finite elements method has been used to achieve the spatial discretization. The incremental/iterative scheme is used for integration in the time domain. Matlab routines are coded. Initially, for verification purposes, simple case of clamped-clamped beam has been analyzed using the same code to show good agreements with other methods. Later, the full run has been done to simulate jointed beam. The joint parameter has been tuned to capture the stick-slip phenomena.

The results show several conclusions: mainly the important role of the friction as a means to prevent peaks (high values) in the membrane force this is in addition to the known vital roll as a source of energy dissipation.

Also the results show the capability of the model of capturing the local beahviour of the joints such as stick-slip phenomena and the feedback of the joints on the overall behaviour of the structure. Obviously, this is done within a relatively low computational cost. That assures the portability and implementation in full-scale run in built-up structures.

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Algorithms for time-domain transformations to treat wave propagation in soil

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1 INTRODUCTION

During a demolition process, the soil is dynamically loaded by falling rubble. The wave propagation extents to infinity and is known as radiation damping. Therefore, the scaled boundary finite element method (Wolf 2003; Wolf 1996) in order to describe local nonlinear effects and strong gradients. For the transient load, the model has to be transformed into the time-domain; this is shown here by using two diffrent algorithms. Ruge et al. (Ruge 2001) proposed transformation of frequency-dependent dynamic stiffness relationships to the time-domain using the 'mixed variables technique'. The derivation of the acceleration unit-impulse (Wolf 1996) response is based on the interaction force-acceleration relationship, which is formulated in the time-domain with the inverse Fourier transformation. For the comparison of in-situ measurements and numerical results, the impact caused by the falling chimney, which is calculated with the angular velocity.

2 RESULTS

The velocity curves achieved by two different algorithms are extremely similar Figure 1. The acceleration unit-impulse response does not always have the same amplitudes as the rational approximation. A possible reason for the small deviation is the number of DOFs and the degree of the polynomials of the rational approximation.

When comparing in-situ measurements and numerical results, one should keep in mind that nature is three-dimensional and the numerical model is two-dimensional.

3 CONCLUSIONS

The two different algorithms provide extremely compareable numerical results. The advantage of the acceleration unit-impulse is that it generally finds a solution in time domain if the storage capacity is sufficient. The rational frequency- to time-domain transformation is senitive with respect to the soil parameters and the geometry of the elements, which influence the numerical process and can cause to instable eigenvalues. The work required in terms of run time and storage capacity is less than for the unitacceleration impulse response. The numerical results are suitable for the somulation of wave propagation.

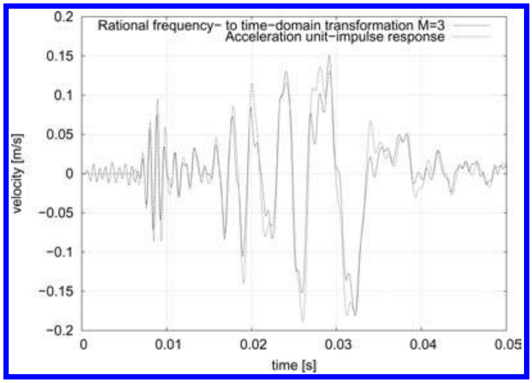

Figure 1. Velocity caused by the first impact.

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Effect of variable geometric piezoelectric patches on vibration of FG plate under constant electric charge

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ABSTRACT

Piezoelectric materials are used as actuators and sensors because of their specific characteristics like lightweight, relatively low cost, small size and good frequency response. Several investigators conducted experimental researches in a number of fields such as vibration control in plates, shape control in composite plates and vibration and buckling control.

In this paper, the vibration control of a simply supported functionally graded rectangular plate with bonded piezoelectric sensor and actuator patches in different geometrical shapes on the top and/or the bottom surface(s) of the plate are presented. At first, the strain–displacement relations for PZT actuator and sensor patches in trapezoidal shape and rectangular simply supported FG plate are extracted, according to the classical laminated composite plate theory (CLPT). FGM properties such as density ρ, elasticity modulus E, and shearing modulus G, are functions of volumetric ratio and Poisson's ratio υ, is considered to be constant.

The stress and strain components for the attached piezoelectric patches and the constitutive stress–strain relations for the FG laminate are obtained. Then the Navier solution for the simply supported FG rectangular plate with actuator and sensor patch incorporated can be obtained by expanding the displacement in a Fourier series and the natural frequency of smart plate is extend. The effect of feedback gain and shape of PZT patches in vibration control is studied. An active vibration control approach is used to control the vibration of the smart plate. The effect of feedback gain and FGM volume fraction on the plate frequency and displacement for trapezoidal, triangular and rectangular shape patches of the sensor and the actuator are studied. Increasing the feedback gain leads to the reduction of the frequency and the displacement and therefore a better control of the plate's vibration. For different geometrical shape of actuator and sensor patches, the comparison indicates that the trapezoidal patches have the behavior between rectangular and triangular patches in the vibration behavior of the smart plate. The rectangular shape patch is more effective than the other patches' shapes.

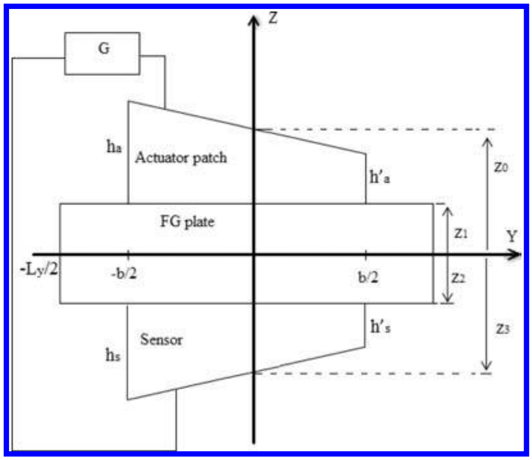

Figure 1. FG plate with piezoelectric trapezoidal patch.

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Effects of rotation and thermoelastic/pyroelectric couplings on the dispersion of acoustic surface waves in a piezothermoelastic half-space

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ABSTRACT

Few research of new surface acoustic waves (SAW) devices, under the effect of the increasing demands for largevalue data transmission in mobile communication and development of the components high frequency have contributed to the design and the optimization of filters, resonators and lines of delay to transverse surface waves and Rayleigh waves. The integration in adaptive or intelligent structures of thermopiezoelectric sensors and actuators becomes common. In spite of the technological interest of these new innovating components, making call in their design with piezoelectric crystals and ceramics, few theoretical results related to their high frequency behaviour are available due to the complexity of the analytical and numerical procedures (periodic stacks or not, material properties, interfaces and boundary conditions, . . .). The governing equations of a thermopiezoelectric material where the mechanical, electrical, and thermal fields are coupled have been studied in (3). General theorems of thermopiezoelectricity are derived in (4). General models for piezothermoviscoelectric materials can be found in (2). A review of the recent developments in thermopiezoelectricity with relevance to smart structures was presented in (5). The analyze of the effects of Coriolis force and centrifugal force on acoustic waves propagating along the surface of a piezoelectric half-space has been presented in (7). The rotation perturbation of the surface acoustic waves propagating in piezoelectric crystals are discussed in (6) and rotation sensitivity of these waves on rotating piezoelectric plates can be found in (1). Thermal effects greatly influence the performance of piezoelectric actuators and sensors, especially when they are required to operate in severe temperature environments.

In this paper, the study of the effects of rotation and thermoelatic and pyroelectric couplings on the dispersion of the *Rayleigh* waves being propagated on the surface of a thermopiezoelastic half-space, is presented. The equations of motion including *Coriolis* and centrifugal accelerations, the equations constitutive of the linear thermopiezoelectricity, and the equation of the temperature are formulated. The surface of the half space is considered free in mechanical loads, electrically electroded or free of charges, thermically isolated, and in rotation around the three symmetrical axes of material. The half space is cut according to a normal plan with x_2-axis. The secular equations of dispersion and the boundary conditions relations in the cases of the absence and the presence of rotation and in both cases of electric conditions, are explicitly derived. By using an iterative method based on the technique, we solve the systems of the nonlinear equations resulting from the secular equations and those of the boundary conditions. The curves of dispersion according to rotation/frequency rate and the surrounding temperature are traced and commented. We present a selection of numerical results concerning the propagation of Rayleigh wave on the surface of a PZT-6B thermopiezoelastic half space rotating around its symmetric axes and analyzing the effects of the centrifugal forces and thermoelastic and pyroelectric effects on the dispersion of theses waves.

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Thermoelastic and pyroelectric couplings effects on dynamics and active control of smart structures using finite element method with localized thermopiezoelectric elements

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ABSTRACT

With the consolidation of composite materials for the construction of aerospace structures, enormous effort is currently being focused on the development of smart or intelligent structures. Under external loadings, conventional structures deform in a passive manner with no control over the configuration of the deformed state. Whereas in smart structures, the structural deformation is continuously monitored and controlled (using distributed sensors and actuators) to achieve the desired configuration of the deformed state of the structure. In principle, distributed sensing and actuation can be implemented by embedding piezoelectric material in the structure. Piezoelectricity represents the interaction between electrical and mechanical characteristics of a material. Though piezoelectric material was discovered by Curie brothers in 1880's, the potential for use in deformation control, health and usage monitoring of flexible structures is of recent origin. Thermal effects greatly influence the performance of piezoelectric actuators and sensors, especially when they are required to operate in severe temperature environments.

This work investigates the influence of thermal and pyroelectric coupling on the dynamic behavior of the flexible composite structures and on the control procedure by application of a thermal gradient between the faces of the two structures. Linear constitutive equations of a thermopiezoelastic medium involving mechanical, electrical and thermal fields are presented with the aid of a thermodynamic potential. Finite element equations for the thermopiezoelastic media are obtained by using the linear constitutive equations in Hamilton's principle together with the finite element approximations. Two structures are studied in this work. The first consists of a modeling of cantilevered piezolaminated *Timoshenko* beam with embedded PZT thermopiezoelectric elements between two Aluminium layers and placed at different positions. The second is a *Kirchchoff* cantilever piezocomposite plate on which pairs of thermopiezoelectric actuator/sensor in PZT are symmetrically bonded to the opposite plate surfaces on dynamic behavior of the composite structure. The two structures are modelled analytically then numerically by a finite element approach based on Timoshenko's beam and Kirchhoff's plate models, and the results of simulations are presented in order to visualize the states of their dynamics and the state of control. For each structure, the formulation of the finite element method, the space state model and the sensor and the actuator equations are established. The optimal control LQG accompanied by the Kalman filter (like regulator and observer) is applied. We study here the effects of thermoelastic and pyroelectric couplings on the dynamics of the two structures and on the control procedure. An important motivation of this work is to present a finite element modeling of a plate with localized thermopiezoelectric sensors and actuators using a formal and clear methodology.

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3. Seismic response, seismic design

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Towards earthquake resistant design of steel buildings for uniform ductility demands

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EXTENDED ABSTRACT

Recent research on the inelastic earthquake response of non-symmetric, multistory buildings designed in accordance to modern codes, has indicated that ductility demands under the action of design-level earthquakes are not distributed evenly throughout the structure, as it would be desirable for a well balanced design. In fact, it has been found that frames at the so called "flexible" edges of the buildings exhibit higher ductility demands, compared to the frames at the opposite sides, the so called "stiff" edges of the building. This has been observed in reinforced concrete buildings designed according to the European and US earthquake resistant design codes, EC8 and IBC, respectively. In some cases the differences in the computed ductility factors between elements at the two opposite edges exceeded 100%. This is undesirable because it may lead to premature failure of members at the "flexible" edges. In the present paper this phenomenon is verified for a number of eccentric steel braced frame buildings, also designed in accordance with the EC8 code. One, three and five story buildings have been used, with original mass eccentricities $\varepsilon_m = 0$ (symmetric), $\varepsilon_m = 0.10$ and $\varepsilon_m = 0.20$, but due to space limitations results are presented only for the 3 and 5 story buildings. The verification is made through non linear, time history, inelastic dynamic analyses of detailed building models of the plastic hinge type. Groups of ten, two component, artificial motion pairs, generated from historical records to closely match the design spectrum, are used as input. Mean values of ductility factors, based on axial deformations for brace members and on plastic hinge rotations for beams, are the indices compared for the stiff and flexible edges of each building along the x and y directions. Subsequently, a design modification is introduced to alleviate the problem. This modification was to increase the bracing sections at the flexible edges and decrease them at the stiff edges by factors based on the top floor edge design displacements. The results from the modified design are quite satisfactory and meet the objective of more uniform distribution of ductility demands. The improvement can be seen by comparing ductility demands in Figures 1 and 2 (original vs. modified designs) for braces and beams of the 5-story eccentric buildings with $\varepsilon_m = 0.10$ and $\varepsilon_m = 0.20$

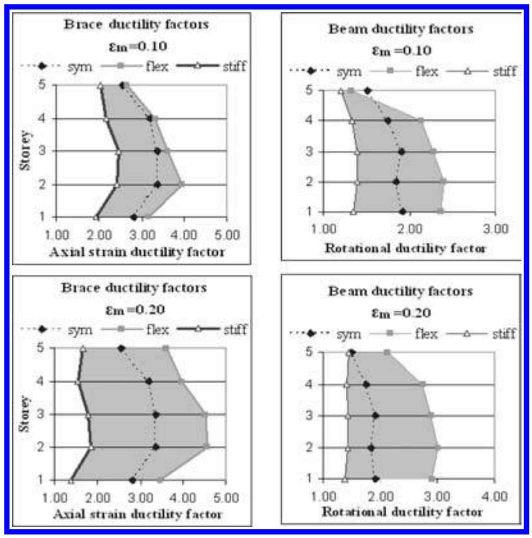

Figure 1. Ductility demands for braces and beams of the 5-story frames in the Y- direction.

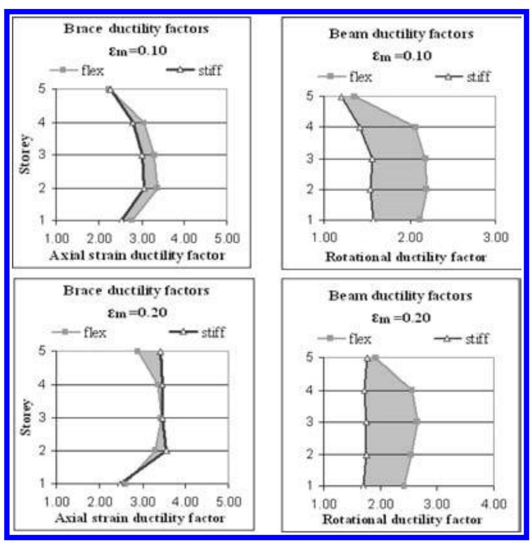

Figure 2. Ductility demands for braces and beams of the modified 5-story frames in the Y- direction.

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Early observations from the magnitude M_w 7.0 January 12, 2010 Haiti earthquake

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ABSTRACT

The MAE Center team traveled to Haiti February 27th through March 6th as part of an Earthquake Engineering Research Institute (EERI) field reconnaissance team. The paper reports early observations from the M_w 7.0 Haiti earthquake of January 12th, 2010. Considering the economic impact of this event, the Haiti earthquake may be the most destructive modern natural disaster ever.

The paper first reviews the seismo-tectonic setting. Haiti is situated on a tectonic microplate between the Caribbean and North American plates. Because of this setting, Haiti is located between two major, subparallel faults, the Septentronial fault to the North and the Enriquillo Plantain-Garden fault to the South. The fault-plane solution and shaking intensity maps are also shown.

The seismological aspects are followed by assessments of selected engineered and nonengineered buildings, to highlight some of the observed design and construction shortcomings. Nonengineered buildings shown include a one-story residence that suffered moderate damage, a two-story residence that suffered severe damage, and a small chapel near the National Cathedral that remained unharmed. Both residences were constructed of unreinforced masonry walls with reinforced concrete columns. The chapel, a rather unique structure in the area, performed very well during the earthquake with no visible cracks. The good performance of the chapel, along with other observations from the team, indicates that poor construction quality is likely the main cause of devastation and destruction in Haiti.

Engineered buildings highlighted in the paper include the Presidential Palace, a religious radio station office, and the European Union embassy building. The Presidential Palace, primarily composed of masonry, collapsed tragically during the earthquake. The heavy weight of the domes was a major cause of the collapse. The radio station office, located next to the previously discussed chapel and the National Cathedral, underwent a soft story failure. The European Union embassy building performed well during the earthquake. The RC structure was not damaged.

After examining key structures representative of engineered and non-engineered buildings, social and economic impacts are briefly touched on. The economic impact is estimated to be up to $14 billion, approximately 120% of Haiti's GDP. The social impact is perhaps even greater with over 200,000 people killed, 1.3 million living in tents, and an additional 500,000 displaced.

By reviewing buildings that performed poorly as well as those that performed well, the influence of construction quality and materials becomes clear. Because of the adequate performance of most modern, well engineered buildings, it is concluded that poor construction quality and materials are major factors in the damage and collapse of many buildings. The intensity of shaking is inconsistent with the exceptionally-high level of damage sustained by all types of construction.

The paper concludes with recommendations for the development of visually-based guidance for construction of buildings that identifies and avoids common pitfalls and recommends desirable features for reducing the impact of future natural disasters. Other suggestions are also offered for a realistic, short term plan for rebuilding.

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Vulnerability of buildings due to far-field effects of earthquakes

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ABSTRACT

The vulnerability of buildings subjected to far-field effects of earthquakes in Sumatra is evaluated by comparing the seismic demand and the capacity of the structure in A-D (acceleration- displacement) format. A macro-scope model for capacity evaluation of the shear wall-frame structure is presented and compared with the updated demand incorporating the two recent large earthquakes in Sumatra (Lam et al. 2009). An earthquake with moment magnitude of 9.5, 600 km away from Singapore is identified as the worst scenario. Three sites are selected, viz. MP with site period of 0.76s, KAP with site period of 1.6s and KAT with site period of 1.85s. The surface acceleration response spectra for these three sites are shown in Figure 1.

A 25 story, 64.8 m tall shear wall-frame building shown in Figure 2 is chosen as a case study. The plan dimensions of the building is 24.4 m by 19.8 m. Concrete cube strength of 20 MPa and 30 MPa are considered. Also two loading cases of the vertical loads are considered in the study, viz, (1) ultimate loading case: 1.2 dead load+1.2 live load; and (2) common loading case: 1.0 dead load +0.4 live load. The structure is modeled using 3D RUAUMOKO. The program is first validated by calibrating the experimental studies of scaled models, before pushover analysis on the chosen building is carried out. Four different cases as indicated in Figure 1 are considered. The first failure occurred due to shear at the 1st story I shape web wall, when base shear to weight ratio reaches 3.8%. This value is considered as the capacity of the structure for no damage. The capacity curves converted to A-D format are superimposed with the demand curves in Figure 1 for ultimate and common load cases with cube strength of 20 MPa and 30 MPa. It is seen that the capacity curves do not intersect the demand curves of KAT site, implying that for certain loading cases, the high rise building considered in this study may suffer some damage due to excessive shear caused by the chosen worst earthquake.

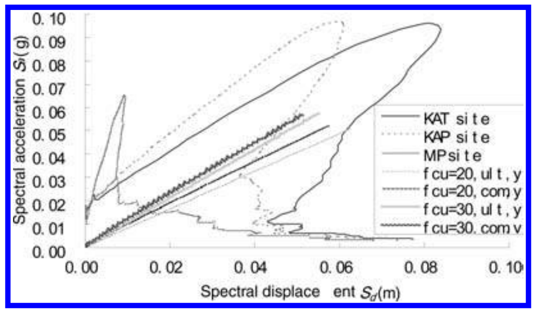

Figure 1. Capacity and demand curves for cube strength of 20 MPa and 30 MPa under ultimate and common load cases.

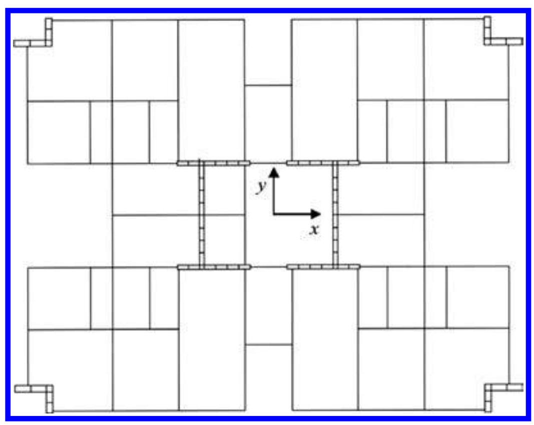

Figure 2. Plan view of the 25 storey building.

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Time delay effects of earthquake excitation on responses in high-rise buildings

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ABSTRACT

The force transmitting path in a structure due to earthquake excitation has been elaborated, and it is clear that upper floor responses are initialized upwards floor by floor like a chain. Lower floor material begins first to be deformed when the ground motion arrives at the basement. Afterwards, the shear wave travels upwards until to the top of a building, i.e., a phase difference exists between adjacent floor responses. In other words, the top floor begins to respond to the earthquake excitation only after the basement has moved back and forth for a while, usually in several seconds. Under such circumstance, the top floor displacement would result in big errors if all the floor displacements were still regarded and computed as being simultaneous as conventionally.

The nature of time delay effects of structural responses is then treated in the paper from the perspective of wave propagation, and then the time delay is used to modify governing equation of motion in structural dynamics which takes the assumption of implicit simultaneity of structural responses at various heights.

The external 'earthquake force' term considering time delay effects not only becomes complex, but the initial conditions under certain floor, i.e., the displacement and acceleration after certain time instant, have to be considered to compute actual structural responses. This brings many inconveniences for the analysis of structural responses for actual tall buildings. To tackle the problem from another perspective, the time delay in the equivalent 'earthquake force' is considered in structural responses. The time delay effects in the 'earthquake force' input are transformed in structural responses by adding corresponding time delays for various upper floors. With the new reformulation, structural responses can be computed easily and much faster than with conventional means.

For an existing building, actual wave velocity can be measured by inputting an impulse in the basement and measuring arriving time delay in upper floors. However, actual wave propagating velocities in many structures is unknown since they are still in the design phase. Snieder and Şafak (2006) estimate a 322 m/s shear velocity for the Millikan Library in Pasadena, California. Kohler etc. (2007) demonstrated that the shear wave velocity is about 160 m/s for the Factor Building at University of California, Los Angeles. For a general high-rise building, average design wave velocity of 100–350 m/s is appropriate (Todorovska and Trifunac, 2008).

However, it should be noted that certain row in the modified governing equation is not strictly satisfied. As certain time instant, the shear wave propagates to this floor and all the floors beneath began to move accordingly. However, the upper floors still remain rest due to initial effects at this time instant. Consequently, the major bearing structural components, such as columns and beams in this floor, are under nonuniform deformation, and the elastic restoring force are in the phase of transformation from partially to completely deformation. As a result, the modified governing equation is only approximately satisfied at the time instant. Further investigations are suggested to deal with this intriguing and delicate issue.

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The L'Aquila earthquake of April 6th 2009: Seismic response of the hospital facilities

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ABSTRACT

An earthquake hit the Abruzzo region in Italy on April 6th 2009 at 3:32am local time. The earthquake had a moment magnitude of 6.2 MW and a shallow focal depth of about 8–9 km. The epicentre was located 10 km West of the city of L'Aquila, which with its population of more than 68,000 inhabitants was devastated by the earthquake. The earthquake intensity reached IX EMS-98 close to L'Aquila. Though not of very high magnitude compared to some worldwide events, this is a significant magnitude for a European densely populated country.

A very discussed case was the performance of the S. Salvatore Hospital facilities, which were completely evacuated during the emergency due the damage at various floors of the buildings.

The S. Salvatore complex is an hospital serving a region of 1800 km2, with more than 500 beds and 13 wings,. The S. Salvatore Hospital is constituted by a number of R.C. buildings constructed between the 60th and the 90th, with heterogeneous structural typologies and systems of 3 to 5 storeys, with 1 or 2 basements. Infill walls are made of unreinforced hollow clay blocks, and all the buildings are covered by an exterior veneer of unreinforced brick masonry.

The EUCENTRE Mobile Unit and inspection teams have been involved in the usability assessment of the hospital structures during the post-earthquake emergency. Moreover, a careful survey has been carried out at the hospital facilities right before the reconstruction, in order to make a deeper assessment of the quality of employed materials, of the structural layout and of the level of damage suffered by the different buildings.

After the 6th April earthquake, only 3 buildings out of 15 in the San Salvatore complex suffered considerable structural damage. This was limited to small areas and primarily was due to evident issues that will be described in detail. There was slight and relatively limited non-structural damage and significant non-structural damage only in a few buildings. In these latter inner partitions significantly helped the lateral resistance by dissipating the earthquake's energy and suffering critical damages.

From the point of view of the human safety, the most widespread and relevant non-structural damage was that of exterior façade bricks, covering the entire surface of all the buildings. Such a coating, not linked to the interior walls, in many cases was partially or totally detached. No significant damage was observed on the equipment and internal mechanical devices inside.

A thorough analysis of the hospital facilities has shown besides the essentially non-structural damage also some aspects of the construction of the hospital complex, which might be crucial for the seismic response to future shocks and therefore open more critical damage scenarios.

Figure 1. The EUCENTRE Mobile Unit.

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Development of Performance Based Tsunami Engineering (PBTE)

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ABSTRACT

In 2005, the US National Science Foundation funded a project through the Network for Earthquake Engineering Simulation to develop Performance Based Tsunami Engineering, PBTE. The primary objective of this study is the development of design guidelines and computational tools that can be used by engineers to design coastal structures to resist tsunami loads. This involves improved inundation and scour modelling, as well as determination of structural loading time histories and loading expressions for use in structural design. This paper presents an overview of the laboratory experiments performed to determine tsunami bore loading on structural components. The resulting loading expressions will be incorporated into PBTE design guidelines being developed for future design of multi-storey coastal buildings for tsunami loading.

Coastal buildings, bridges, highways, and harbour facilities that are at risk of tsunami inundation may suffer significant damage if the structures are not adequately designed for the fluid loading.

The author led a team of researchers in a four-year NSF-funded project to develop Performance-Based Tsunami Engineering, PBTE. This includes the methodology and validated simulation tools for implementation of site specific PBTE for use in the analysis, evaluation, design and retrofit of coastal structures and facilities, as well as the development of code-compatible provisions for tsunami-resistant structural design.

The project's experimental component focused on the following aspects of tsunami design: 1) run-up and inundation, including fluid velocities and energy dissipation; 2) sediment transport and scour as a result of inundation and drawdown; and 3) fluid loading on structural elements. The experiments were performed during 2007 and 2009 utilizing the Tsunami Wave Basin and Large Wave Flume at the Hinsdale Wave Research laboratory at Oregon State University.

The primary objective of the experimental component of the project was to provide data to validate numerical models for each of these three areas. Riggs et al. [1] describe the different experimental setups in some detail.

This paper provides an overview of the progress made to date on this project, and provides references where more information can be obtained about the various aspects of this study.

Based on the experiments performed in this program, the following conclusions were drawn:

- Multi-storey coastal buildings can be designed to withstand the hydrodynamic loads imparted by a tsunami bore advancing onshore.
- The effects of scour due to fluidization during drawdown must be considered in designing building foundations.
- Hydrodynamic uplift forces on floor slabs depend on the amount of flow restriction provided by columns and walls below the slab.
- Impact loads due to floating debris such as shipping containers, boats and other large objects may cause damage or failure of individual columns or walls. Design provisions to prevent progressive collapse should be included in coastal building design to prevent disproportionate collapse.

ACKNOWLEDGMENTS

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Seismic behavior of prefabricated concrete sandwich panels

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ABSTRACT

In this paper the seismic performance of an innovative prefabricated structural system comprising large RC sandwich panels (RCSPs) is investigated. Despite the use of prefabricated RCSPs have been introduced in the construction industry for more than 40 years, the last have been used in practice primarily as gravity load bearing structural elements. The last decade, many companies from the international precast construction industry have started manufacturing RCSPs commercially with the aim of developing a quick and permanent building system which is supplemented with a satisfactory earthquake resistance. Consequently the investigation of the seismic performance of prefabricated RCSPs still remains a challenging task, which is addressed in this study through full-scale seismic testing.

A total of 8 full-scale RC panel specimens, comprising panels with and without openings and a 2-storey full-scale H shaped structure, were constructed and tested under cyclic uniaxial flexure with constant axial load. The geometries of the single storey structural panels are shown in Fig. 1. A series of parameters on their seismic behavior were investigated, namely: panel length; level of the axial load; and the presence or not of openings in the panel. In brief, the notation of panels is PL_A_O, where the letter P denotes the panel specimen, L defines the panel length (3 m or 4 m), A denotes the level of the applied axial load during test (150 kN or 300 kN) and O denotes the presence of opening on the panel (W for window, D for door).

The response of all panels and the H structure tested is given in Fig. 2 in the form of load-drift ratio loops. Key results about panels' general behaviour are also summarized in Table 1. The performance and failure mode of all panels tested revealed strong coupling between flexure and shear due to the squat-type geometry of the panels. However due to their well-detailed reinforcement, all panels exhibited only a relatively gradual strength and stiffness degradation and in no case did any panel suffer from sudden shear failure. From the results obtained in this study, the authors believe that the presented structural system with prefabricated RCSPs comprise a promising construction system for regions of moderate and high seismicity.

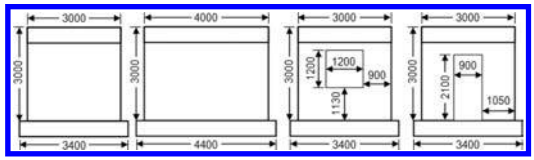

Figure 1. Geometry of panels. (Dimensions in mm).

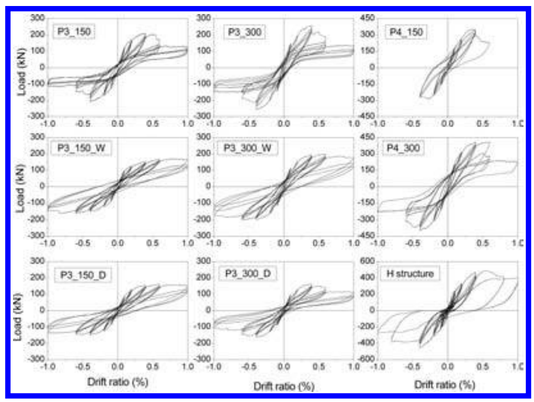

Figure 2. Load versus drift ratio curves for all specimens tested.

Table 1. Summary of test results.

Specimen notation	Peak force (kN)	Drift at yielding (%)	Drift at peak (%)	Drift at failure (%)	Disp. ductility $\mu_\Delta = \theta_u/\theta_y$
P3_150	210	0.18	0.40	0.52	2.9
P3_300	257	0.17	0.39	0.50	3.0
P4_150	306	0.22	0.32	0.43	2.0
P4_300	403	0.21	0.40	0.61	2.9
P3_150_W	166	0.23	0.70	0.99	4.3
P3_300_W	200	0.23	0.59	0.99	4.3
P3_150_D	155	0.21	0.48	0.99	4.7
P3_300_D	162	0.22	0.39	0.60	2.7
H Structure	467	0.26	0.44	0.90	3.5

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Relationship between seismic index of structure and seismic response of retrofitted RC structure

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ABSTRACT

It is well known in Japan that a lot of reinforced concrete storied buildings built before 1981 by an old earthquake resistant design code were destroyed in the 1995 Hyogo-ken Nambu Earthquake. Therefore, Japanese Government has been adopted several significant politics concerning this issue since 1995 in order to reduce a lot of earthquake damages for RC-storied buildings built before 1981 as quickly as possible.

At the present time, a seismic resistant performance of RC building structure can be judged from a seismic index of structure, I_s, evaluated by some criteria established by the Japan building disaster prevention association (2001). This seismic index of structure, I_s, can be obtained from the floor plan and various investigations of RC building structure and does not require any seismic responses of RC building structure against a large earthquake. However, it is very important for a structural engineer to take account of a seismic response of RC building in the seismic retrofitting design process, because the relationship between the seismic index value, I_s, of structure and seismic response of RC building structure can play a key role to propose an effective countermeasure in the seismic retrofitting design process. The number of steel brace or RC wall required for the seismic retrofitting work of RC building structure can be decided to satisfy a given seismic judgment index value, I_{so}.

In this paper, the seismic response analysis of 3 storied RC building structure retrofitted by RC walls is carried out using an acceleration data in the 1995 Hyogo-ken Nambu Earthquake. In particular, the effect of seismic retrofitting countermeasure on the seismic response behaviour of RC building structure is numerically investigated from a viewpoint of the number of RC walls. The acceleration, velocity and displacement responses and the bending moments of RC columns are numerically evaluated by 3-D elasto-plastic seismic response analysis in order to investigate the relationship between the seismic index value, I_s, of structure and seismic response of a retrofitted RC building structure.

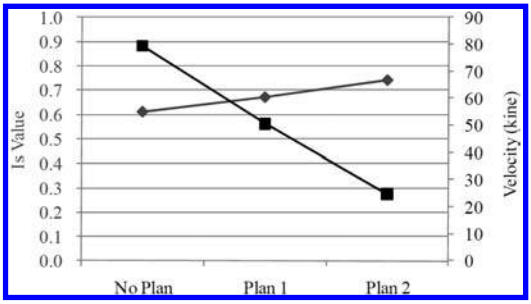

Figure 1. Relationship between seismic index of structure, I_s, value and velocity response value

Figure 1 shows a relationship between the seismic index of structure, I_s, and seismic response for one nodal point. "No Plan", "Plan 1" and "Plan 2" mean "No Retrofitting Work", "Seismic Retrofitting Plan 1" and "Seismic Retrofitting Plan 2", respectively. The seismic index of structure, I_s, seems to linearly increase with the number of RC walls, while the maximum velocity response value greatly decreases in comparison with the seismic index of structure, I_s.

On the other hand, a relationship between the seismic index of structure, I_s, and bending moment response of RC column is obtained. The maximum bending moment value decreases with increasing the seismic index of structure, I_s. Consequently, it should be found that the seismic retrofitting countermeasure can be decided from not only the seismic index of structure, I_s, but also the seismic non-linear response analysis.

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Definition of conservative design values of the residuals of seismic response parameters of RC frames

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ABSTRACT

Performance based seismic design (PBSD) provides for evaluation procedures of a probabilistic type (ATC 58 2007) to account, among other things, for the uncertainties that characterize the values of the seismic response parameters. These uncertainties regard both the values of the global response parameters and the values of the local response parameters obtained from dynamic analyses of nonlinear models of the structure subjected to a certain number of seismic inputs, representative of the site risk. Numerical simulations lead, indeed, to values of the response parameters characterized by a large scattering. In the evaluations of the damage levels it is moreover necessary to consider both the maximum values of the response parameters and the residual values (residuals) (FEMA 356 2000). The residuals generally present a larger scattering with respect to that of the maximum values.

Codes (EC8 2004) provide for design values of the seismic response of a structure with a nonlinear behaviour computed as the mean value of the response to at least seven accelerograms or as the maximum value of the response to at least three accelerograms. These modalities of evaluation do not allow to define a probabilistically significant design value of the seismic response, that is a value characterized by a predefined probability of not being exceeded by any response (D'Ambrisi and Mezzi 2005).

A probabilistic approach to define a conservative design value of the peak seismic response of reinforced concrete (RC) frames has been proposed in D'Ambrisi and Mezzi (2009). In the present paper this method is extended, with the appropriate modifications, to the definition of conservative design values of the residuals of the global and local seismic response parameters of RC frames. These values, characterized by predefined non-exceedance probabilities, are calculated using a limited number of accelerograms.

The method is based on the definition of an amplification factor of the estimate of the residuals evaluated as the maximum value of the response to a limited number of accelerograms. The amplification factor is calibrated as a function of the scatterings characteristics of the considered parameter.

The method is validated with reference to four RC model frames characterized by different aspect ratios and natural periods, designed according to the EC8 (2004) provisions. It results more efficient than the methods prescribed by the EC8 (2004), that provide design values whose non-exceedance probability can not be controlled and is function of the number of the accelerograms used in the analysis.

In the presence of response parameters characterized by different scatterings, the proposed method, contrarily to those provided by the EC8 (2004), allows to obtain design values characterized by the same non-exceedance probability level.

The proposed methodology allows to define similar non-exceedance probability values for the global and the local response parameters.

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Seismic analysis method of structure with passive energy dissipation devices

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ABSTRACT

In this paper, a approach is developed based on the force analogy method (FAM) to analyze the dynamic response of structure with energy dissipation devices (EDDs). The proposed algorithm is applicable to variety of the EDDs through including the EDD to the equal force imposed on the joints of frame.

Compared with the conventional analytical methods, the force analog method (FAM) as a new algorithm was first proposed by Lin, The original concept of the FAM theory was actually for the application of stress and strain in continuum mechanics with the inelastic behavior defined by the plastic strain. Unfortunately, this concept only found limited acceptance because it was developed at approximately the same time that researchers were focusing their attention on studying the deformation of solids using a FEM with the inelastic behavior defined by changing stiffness. In 1999, Wong and Yang applied the FAM to dynamic elastic-plastic analysis of structure using a time history method in civil engineering. Further, Wong built the energy response model of structure based on the FAM, especially, it showed the detailed solution and calculated procedure for the energy analysis of response and performance of structures subjected to severe ground excitations. Then, Wong applied the FAM to predictive instantaneous optimal control (PIOC), in which this research greatly simplifies the computation and makes the inelastic analysis readily applicable to the PIOC algorithm. Moreover, Wang and Wongbuilt a stochastic dynamic analysis method using the FAM. Apart from these, a modified FAM for simulating the nonlinear inelastic response of reinforced concrete structures proposed in 2007, can efficiently simulate the deteriorating stiffness of structure subjected to repeated load, with the addition of the $P - \Delta$ effect of the structure. All the researches found that FAM can reduce the stiffness storage spaces, simplify the computation, and enhance the calculation speed. However, most of research work focus on the beam of structure or the plastic deformation of the column plastic hinge utilizing the FAM. There are no applicable theoretical models for the FAM aiming on simulating the EDD.

Compared to the conventional method for a nonlinear deformation analysis using changing stiffness, this analytical method is superior with features such as, significantly reduces the integration times owning to the nonlinear characteristic of the EDD, dramatically enhances the velocity of calculation, remarkably expands the amount of the solution information. Moreover, besides the dynamic analysis of structure, the new proposed algorithm also accurately captures the distribution of the plastic hinges and the plastic displacement at any time before and after the EDD installation. In comparison with the numerical results obtained by use of the finite element software ANSYS, the approch here is verified to be accurate and efficient.

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An experimental investigation of the seismic behavior of semi-supported steel shear walls

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ABSTRACT

A semi-supported steel shear wall (SSSW) system has been developed in the recent decade. The steel wall is connected to secondary columns that do not carry vertical loads and are used to enable the plate to enter into the post buckling region and develop a tension field. A single storey of a multi storey building is shown in figure 1. Theoretical research on this system has been performed and an algorithmic method has been developed, which enables the determination of the ultimate capacity of the wall.

This paper shows preliminary results from an experimental investigation of a SSSW wall-frame. The main aim of the experiment is to investigate the cyclic behavior of one intermediate SSSW storey within a multi-storey structure. An intermediate storey will approximately deflect in an asymmetric manor about the wall midpoint. Therefore we can cut out one storey including both upper and lower secondary beams and obtain an asymmetric behavior by constraining the midpoints of the secondary beams to have zero deflection. The constraint is achieved as shown in figure 2 by adding two auxiliary center line columns, one on each side of the wall, connecting the midpoints of the upper and lower secondary beam.

For the wall plate and surrounding frame a low yield steel (fy = 130 MPa) and mild steel (fy = 280 MPa) was used respectively. According to Eurocode 8, cyclic loading was applied to the system with a hydraulic jack with ±100 mm capacity. Each nonlinear cyclic displacement was repeated three times. Figure 3 shows the hysteresis loops that have been obtained from the test. The hysteresis loops have “S” shape and dissipate energy well. This shows that, the SSSW system has sufficient ductility to be used as a good lateral force resisting system.

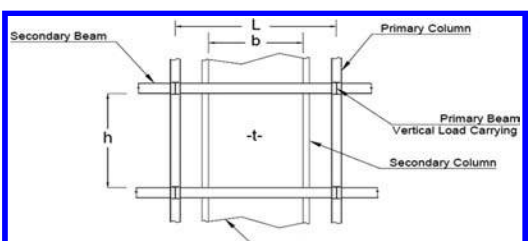

Figure 1. General shape of SSSW.

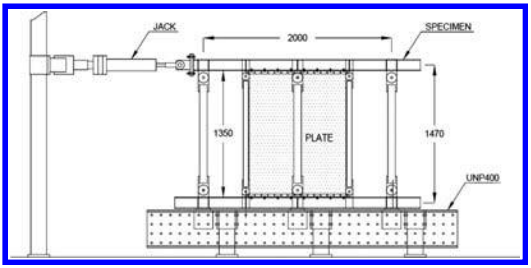

Figure 2. The considered test setup for the wall/frame.

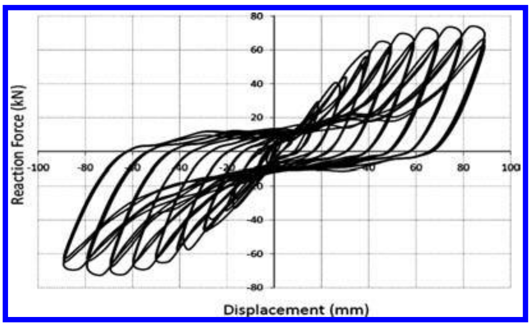

Figure 3. Hysteresis loops for the system during cyclic loading.

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Stiffness and damping aspects of fiber reinforced elastomeric bearings

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ABSTRACT

Fiber reinforced elastomeric bearings (FREB) without mounting provide cheap and robust solutions for anti seismic devices. Compared to seismic isolation systems based on sliding isolation pendulums the stiffness and damping of FREB systems is difficult to determine. But the exact tuning of the system in relation to the oscillation characteristics of the structure is fundamental for the effectiveness of the anti seismic device.

One of the most important advantages of FREB compared to conventional (steel reinforced) bearings is the high lateral deformation capacity for unfixed and therefore cheap bearings. Due to the bending flexibility of the reinforcement sheets the lifting off from the contact planes and the rolling movement of the bearing do not cause damage in the reinforcement.

The intention of this paper is to describe the influence of the rolling movement on the horizontal stiffness and equivalent viscous damping of the bearings. The investigation is based on material tests which are used for the calibration of numerical material models. They are implemented in numerical models of the bearings to carry out parametric studies. Based on these results the influences on the mechanical bearing properties can be described.

The choice of the hyperelastic material model used for the numerical calculations is fundamental. It depends on the aspect of the rolling effect that shall be explored. To explore the influence on the horizontal stiffness a material model with a constant shear modulus is preferred to study the geometrical aspects only. To explore the influence on the horizontal damping a more sophisticated model including nonlinear behavior, material degradation and hysteretic behavior is used.

For the calibration of the material models extensive material tests have been carried out on two different elastomer materials. The test results given in the paper show the strong dependency of the stiffness and damping on the actual shear deformation and the material

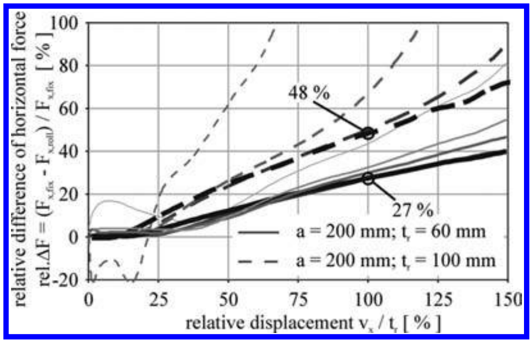

Figure 1. Numerical results of bearing models with variable height ($t_r = 60$ mm and 100 mm) and compression p_z from 1.6 to 15 N/mm^2 (the thicker the line, the lower the compression); shear modulus $G = 0.45$ N/mm^2; form factor $S = 10.6$.

degradation due to former deformations. For an accurate bearing design these aspects have to be considered in the calculations.

The material models are included in numerical bearing models which provide the basis for the parametric studies. The first study on the difference of the horizontal reaction force of fixed and unfixed bearings leads to reduction factor with linear dependency on the horizontal deformation v_x and the side length a (see Figure 1).

The parametric study is on the influence of the mounting on the equivalent viscous damping. It shows that the reduction of the horizontal reaction force is proportional to the reduction of the inner work. Both effects balance each other out. The mounting has no effect on the damping ratio.

The studies lead to the conclusion that the nonlinearity of the material behavior and its mechanical degradation as well as the rolling effect have a significant influence on the bearing behavior and should be considered in the bearing design. This should be taken into account in the in the current revision of the design codes for elastomeric bearing.

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The design, experiment and application of coupling beam dampers

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ABSTRACT

Coupled shear wall structures are usually used in high-rise buildings both in domestic and abroad. It is demanded that the coupling beams yield before the wall legs to meet the criterion that “the wall legs are stronger than the coupling-beams” according to the conceptual design of shear wall structures with the structural ductility demands. The plastic hinges on the coupling beams are formed to dissipate the input seismic energy under major earthquake actions, meanwhile, the structural predominant frequency is diminution when the coupling beams break in shear wall legs connection and departs away from the resonant region of the predominant seismic frequency of ground motion, therefore the structural response becomes smaller and the wall legs will be more safe under major earthquake excitation. Design of the coupling beams in the shear wall structures should maintain the dual roles. The first one belongs to its stiffness, with which the coupling beam connects the wall legs reliable and ensures the structure sufficient lateral stiffness under normal service conditions and the minor earthquakes. The other one belongs to its energy consume contribution to the structures, which demands that the plastic hinge on the coupling beam appears earlier than the wall legs under the moderate earthquake and major earthquakes and dissipate the earthquake input energy to insure the security of the main structural wall legs. It dissipates little input seismic energy and it is difficult and uneconomic in post-disaster restoration based on the traditional seismic resistant design method with the reciprocating plastic deformation and accumulates damage of coupling beams (Paulay and Binney, 1974). Based on the previous study, a new type of coupling beam damper is proposed in this paper, which can provide lateral stiffness to structures under normal service condition and share the input seismic energy with main structure when moderate earthquake or major earthquake occurs. Simultaneously, the coupling beam damper can even effectively alleviate the common problems of over-reinforced beams for engineering designs.

In this paper, the mild metallic dampers which yield in plane are used as the coupling beam dampers in the coupled shear wall structures. The construction of coupling beam dampers (Teng, et al., 2009) is proposed (China national invention patent number: 200710124547.2, Figure 1). All specimens are made of Q235B steel plate with the same outer contour dimensions of 1070 mm × 500 mm. 11 specimens have been tested as pseudo-static test and the optimal parameters of coupling beam dampers have been chose. The pseudo-dynamic substructure tests (Wang, 2007) have been conducted also and the energy dissipation ability of coupling beam dampers has been verified. The case study of vibration control using coupling beam dampers has been introduced and the structural elastic-plastic dynamic time history analysis has been simulated with the software ABAQUS/Explicit (Li, 2009). The usage of the coupling beam dampers and the design method of the dampers in the real shear wall structures are proposed in this paper also.

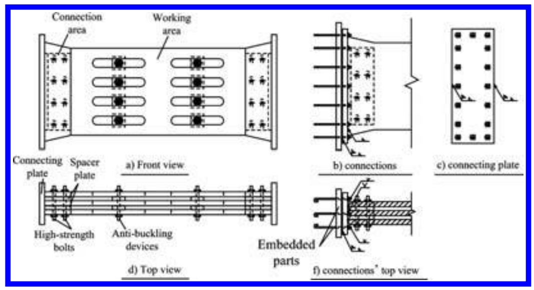

Figure 1. Construction of coupling beam damper.

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Different seismic behavior of RC frame structures modeled with and without masonry infills

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ABSTRACT

The object of this study is the influence of the masonry infills on the seismic behavior of RC frame structures, with particular reference to the case study of a building located in L'Aquila which was subjected to the seismic event of the 6 April 2009.

On the basis of the data observed by the authors in different inspections in L'Aquila, it was noted that most part of reinforced concrete buildings remained substantially intact and just some masonry infills was more or less damaged. Only in few cases it was observed severe damage to structural elements. Rare were the cases in which failure was reached.

The aim of this paper is to comprehend the reasons why structures, designed for accelerations lower than those recorded during the L'Aquila's earthquake, have shown better performances (with low structural damage) than design ones. It was hypnotized that the masonry infills, generally non considered in the design phase, acted as horizontal resisting elements, causing a reduction of the seismic demand for the structural elements. This occurred, naturally, in the cases characterized by no irregularities in plan or in elevation of the masonry infills which can cause concentration of demand and damage. The work was, initially, characterized by the identification of the case study and by the collection of the fundamental data (Fig. 1).

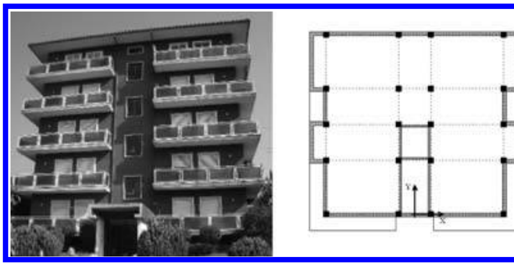

Figure 1. Photo and plan of the building under study.

In absence of the original design it was carried out a simulated design based on the code provisions of those days for the identification of the reinforcements in the structural elements. After that, the seismic behavior of the structure under study was evaluated through non linear dynamic analyses by applying the acceleration record obtained during the L'Aquila's earthquake. Linear and non-linear models of the structural elements and of the masonry infills were developed. A comparison between the response of the structure without the masonry infills and with the masonry infills was performed. Finally, the level of damage evaluated through the analyses was compared with the actual damage observed in the structure.

Through the comparison between the level of damage evaluated trough time-history analyses (developed with linear and non-linear models of structural elements and masonry infills) and the actual damage observed in the structure, it can be noted that for the correct characterization of the seismic response of the RC structure it is necessary to include in the model the masonry infills (contrary to current design strategies). In fact, they did not reach their ultimate deformation capacity and they were able to act as a bracing system with limited economic cost and great structural effectiveness. The contribute of the masonry infills, not designed for acting as a strut, allowed to maintain the structure in the elastic range under a strong earthquake.

This aspect can offer a new way for the seismic design: it is convenient to provide specific structural requirements for the masonry infills, in terms of stiffness, strength and ductility instead of considering them as sacrificial elements.

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Nonlinear seismic response reduction of different types of structures using two type dampers

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ABSTRACT

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When using the method of nonlinear step-by-step time history analyses to calculate the seismic responses of a building, the structural models are usually simplified as storey model. The storey shear model and shear-bending model are designed according to dynamically equivalent standard, and the time-history analysis programs for different computing models are compiled. For the commonly used passive energy dissipation devices, velocity-based devices and displacement-based devices are considered in this paper, which have different characteristics. On the premise that the equivalent damping of two kind dampers is equal, studies focus on the effect of type and quantity of dampers on structure control effectiveness of different computation models and comparisons are done.Considering the relative displacement of inter-storey as the objective function, the optimal displacement of the viscoelastic dampers (VED) and the metallic yielding dampers (MD) are obtained by genetic algorithms. The control effectiveness of the structure with different kinds of dampers can be shown by the figures and the results in the tables.The numerical analysis results show that the control effectiveness of the two type dampers is different for different computing models. According to the analysis of the examples in this paper, some suggestions are proposed. For the shear-bending model, under the condition of the same seismic performance requirement, the MDs are better in storey drift control. As far as the maximal acceleration is concerned, the control effectiveness of VEDs is better. For the shear model, MD may enlarge the acceleration of some floors, so it is safe to choose VED as the energy dissipation devices. Under the condition of the same control effectiveness of storey drift, fewer VED are needed than MD. When the space of the structure is limited, VED can be considered as the first choice.

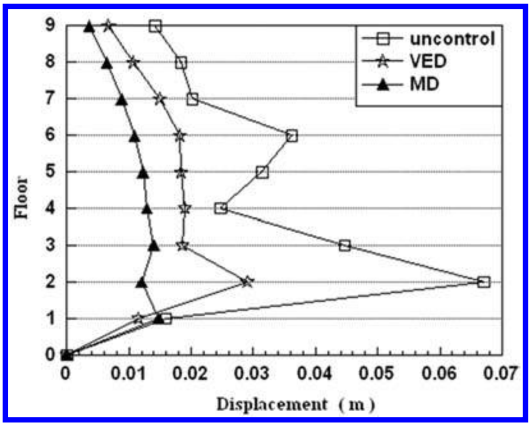

Figure 1. Envelope diagram of maximal drift for the shear-bending model.

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Parametric study on structural behavior of shear walled systems

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ABSTRACT

Reinforced concrete coupled shear walls are widely used in countries under earthquake risk for tall building systems to provide lateral resistance against seismic demands. For elastic design of high-rise buildings, the '*structural behavior factor*' R is used which is computed according to the global ductility level of the whole structural system to include its nonlinear behavior.

According to the Turkish Seismic Code of 1997, (TEC'1997) the selection of the suitable R value is based on the ratio of the base moment carried by the structural walls and the other frame elements to the total overturning moment, for considering the systems with high, normal or mixed ductility level. The coefficient α_m had been introduced named as '*shear wall participation factor*' for this purpose which is a deciding factor for the selection of a suitable R value. On the other hand, in TEC'2007, the definition and computation of above mentioned '*shear-wall participation factor*' α_m has been changed to α_s that is defined as the ratio of the base shear carried by the structural walls to the total base shear of the building, which is a more practically computed factor for the designers.

The aim of this study is to examine the range of the contribution of the additional moments that produced at the edges of coupling or connecting beams to the α_m coefficient which plays an important role for determining the system behavior factor R, as well as to examine the relation between α_m and α_s coefficients and R values obtained with these two definitions. Additionally, their effects on buildings with torsional irregularity are also investigated at various types of shear walled case study systems.

Thus, a parametric study has been carried out about the coefficients α_m computed based on moments and α_S based on shears to see their different effects on structural behavior factor R at different multistory reinforced concrete buildings.

Three 10 storey case study buildings are selected as systems having free shear walls without connected to beams, framed and shear-walled systems and coupled shear walled systems having the same floor plans. First, symmetrical systems are selected so that they do not include any torsional effect, then the walls are shifted on the same multi-storey systems and the computations are repeated to see the effect on torsionally irregular systems.

It is concluded that the moment contributions of coupled or connecting beams should be included in the computations of α_m since they are not too small to be neglected. The new definition of shear wall participation factor α_s, produces higher R values for the buildings with and without torsion, as compared to the previous definitions of α_m when the structures are designed with mixed ductility levels. The change of the participation factors α_m, α_s and R values of the considered buildings are shown in the Table 1.

Table 1. Computed participation factors α_m, α_s and R values of investigated buildings with mixed ductility level.

	α_m, α_s and R values			
Structural system	α_m (1997)	α_s (2007)	R (1997)	R (2007)
No Torsional Irregularity				
Solid Wall (A1)	0.642	0.864	6.89	7.00
Framed- SW (A2)	0.642	0.837	6.89	7.00
Coupled SW (A3)	0.642	0.826	6.89	7.00
With Torsional Irregularity				
Solid Wall (B1)	0.625	0.838	6.81	7.00
Framed-SW (B2)	0.625	0.801	6.81	7.00
Coupled SW (B3)	0.625	0.785	6.81	7.00

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Seismic assessment of a gravity-load designed R/C frame with masonry infills

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ABSTRACT

The main purposes of this study are (1) to evaluate the effects of the presence and of the mechanical properties of uniformly distributed masonry infills on the seismic response of a R/C frame without seismic detailing and (2) to use simplified procedures for the seismic assessment of existing infilled R/C structures. Based on results of experimental tests carried out at the JRC Elsa Laboratory, numerical models were developed in order to properly simulate the seismic response of a R/C frame designed only for gravity loads.

Numerical results from non-linear pushover and time-history analyses are presented for different frame configurations: a) bare frame (BF) (no infills); b) partially infilled frame (PIF) with openings. A fully infilled model (FIF) without openings was also considered. Two variants of masonry infills were investigated, aimed at simulating weak and strong infill panels.

The bare and infilled frames were modelled using a fibre modelling approach. The four-node masonry panel elements were used to represent the behaviour of infill panels in the frame. The effect of the openings was taken into account by reducing the strut area and thus the infill panel stiffness.

A simplified approach, extended to infilled R/C frames, was applied for the seismic assessment of the investigated structure, comparing deformation capacity and demand. The pushover curve was idealized as a multi-linear force-displacement relationship and inelastic spectra were determined by using specific reduction factors appropriate for infilled frames. The demand point was defined as the intersection of the capacity curve and the inelastic demand spectrum.

The expected contribution of the masonry infills in terms of both strength and stiffness was evident when comparing the response of the different frame configurations under monotonic loads. The higher stiffness provided by the masonry infills led to anticipate, in terms of drift, the development of global inelastic mechanisms in the infilled frames with respect to the bare frame.

The presence of uniformly distributed infills considerably changes the distribution of damage throughout the frame. In the bare frame a soft-storey mechanism occurred at the third storey due to a change of cross-section and reinforcement details of the strong column. In the infilled frame an extensive damage in the infill panel was registered at the first storey. The sudden reduction of strength due to the damage of the infills can lead to the formation of a soft-storey mechanism for severe seismic input motions.

The influence of uniformly distributed infills on the seismic response of the investigated frame was beneficial according to the simplified assessment method. For the infilled frames, the deformation capacity at LSSD (Limit State of Significant Damage) was large enough to accommodate the demand and a significant reduction of the DCR values was registered compared to the bare frame. The mechanical properties of the infill panel affected the distribution of damage throughout the frame.

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Response analysis of base isolated railway bridges

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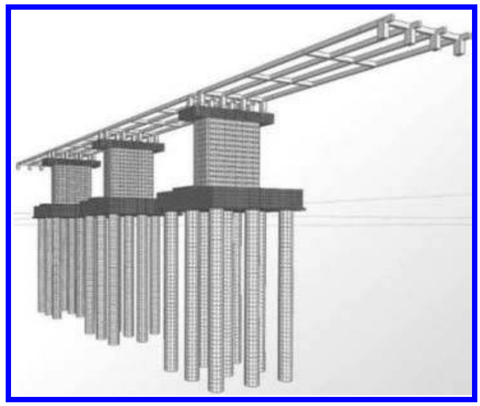
ABSTRACT

Railway bridges generally require large piled foundation systems because of the large earthquake demand imposed on the piers supporting the viaducts. On the other hand, base isolation systems may be employed to lower the seismic forces transmitted to the superstructures. Notwithstanding, the isolation devices may lead to large lateral displacements that endanger the functionality of the railways bridge system. The present paper assesses the studying of feasibility of the application of base isolation systems to typical railway bridges. Special emphasis in on the refined modeling utilized to assess the seismic structural performance. The *"Cintura" Viaduct*, i.e. *High Speed Railway Milano-Bologna* viaduct recently built in Italy, is considered as sample structure. The viaduct was design in compliance with the 1996 seismic design rules (DM, 1996) for areas of medium earthquake hazard. The existing viaduct was assessed in compliance with the new Italian code NTC2008. The system was then retrofitted with base isolation and the seismic performance was analyzed through detailed nonlinear dynamic analyses.

The analytical study carried out demonstrates that:

1. The re-assessment of the bridge structure, built in compliance with the old Italian seismic code, has shown a ductility demand at the base of the pier due to high tensile action effects. The estimated ductility value is in compliance with the values of the behaviour factor implemented in the newly issued national design standards for bridges designed for low ductility. The checks of the piles of the foundation system are not satisfied because of a significant increase of the action effects for the axial load-bending moment interaction;
2. The application of lead rubber bearings improved significantly the seismic bridge response, nevertheless, the isolation devices do not comply with serviceability requirements due to the presence of the rail;



3. The introduction of steel dampers isolators seem to be a good strategy both in terms of serviceability and ultimate limit states.

The above discussion shows the viability of using of *ad hoc* type of base isolation systems for railway bridges. Further studies are deemed necessary to characterize the optimal properties for the most suitable isolation system. It is also vital to achieve reliable earthquake performance assess to investigate the rail-bridge seismic behaviour of the bridge system encompassing the fastening system and his interaction with structure.

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A predictive FE model for a simple precast concrete panel wall subjected to moderate shaking

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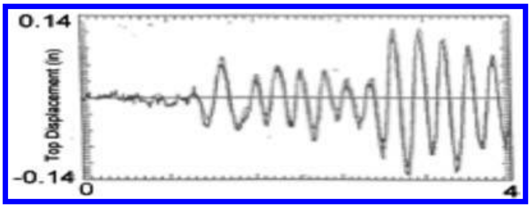
ABSTRACT:

During recent earthquakes, large precast panel walls played a very positive role in the performance of multistory panelized structures, exhibiting a sound seismic resistance. To investigate the behavior of such walls, shaking table tests were preformed on a 1/3 scale three-story assemblages. The results of these tests allow for proper assessment of the seismic performance. The results are also used to develop analytical models. The various recorded response parameters can be effectively used to confirm the ability and reliability of proposed analytical models to predict the complex dynamic response of such structures. The main objective of this work is to develop elastic analytical models that correlate favorably with the moderate shaking table tests results.

The focus of the research presented in this paper is directed towards the evaluation of an existing algorithm. This paper reports on such model that was developed during the correlation studies, using the results from the tests of the simple wall, subjected to moderate intensity shakings The DRAIN computer program has been chosen partly for this purpose, and partly because it has been used in analytical studies of precast panel walls for which many discrete spring models to idealize the joints, have been developed and incorporated in the program. The correlation was carried out against the top displacement time history. Three different analytical models were developed until adequate correlation was achieved.

The first model was a linear elastic model that focused on establishing the correct dynamic characteristics of the system. The computed top displacement time history for this model is showed good performance up to 2.8 sec. The model then exhibited a stiffer behavior, with complete deviation from the recorded response. It was clear that the significant deviation occurred when the recorded response is exhibiting large displacements. The second model included a precracked zone in the lower panel. This second model was unable to correlate adequately with the full time history. It did well early on but deviated and showed softer behavior later in the signal. The third model included a permanent cracking mechanism that allowed for the lower panel to be initially uncracked.



The third model results have shown that a permanent stiffness loss due to cracking leads to a model which is too soft in the latter part of the response. It also helped reveal that the test wall experiences a stiffness recovery in the latter part of the signal. The fourth correlation model was called the stiffness recovery model. The stiffness recovery can be attributed to the continuous closing and opening of the cracks as the wall vibrates back and forth. In this model the repeated occurrence of the opening and closing of pre-existing cracks is to numerically modeled and incorporated in the wall model. With this model, Successful correlation is accomplished with 22 in. cracking zone. The model correlated well for the 20 second duration. The response of the wall was expected to be linear elastic. However, the correlation studies have revealed that response is more complicated than anticipated, involving the cracking of the joints, the cracking of the lower panel, the flexibility of the foundation system, and the pitching of the shaking table.

The conclusions are: (a) The analysis under moderate shaking should take into account the effect of cracking on the elastic stiffness; (b) The fluctuation of stiffness due to continual cracks closing and opening should be accounted for in the model. The dynamic response is extremely sensitive to minor fluctuation in the elastic stiffness.

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Maximum seismic rotational response of multi-storey structures

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ABSTRACT

Asymmetric structures, characterized by non-coincidence between centre of mass and centre of stiffness, when subjected to dynamic excitation, develop a torsional response which increases the dynamic response. Recent studies (Trombetti and Conte 2005, Trombetti et al. 2008, Silvestri et al. 2008), have shown that it is possible, through a structural parameter, called ALPHA parameter, to evaluate the torsional displacement from the longitudinal displacement for asymmetric one-story systems. Such parameter is the base for a simplified method (called ALPHA method) for the evaluation of the maximum rotational response, valid for one-story structures. Such simplified method has been extended also to multi-story structures, more common in the design experience. In the first part of the paper a brief explanation of the ALPHA method for one-story structures, with the individuation of the fundamental parameter, will be carried on. The objective of this paper is to analyze the torsional behavior of multi-storey structures as subjected to earthquake ground motions and try to extend the validity of the ALPHA method to such structures.

Several numerical analyses have been performed using 20 earthquake ground motions as base input and considering a wide range of values for the basic structural quantities (stiffness, mass, eccentricity and ratio between the radius of inertia of the masses and the radius of inertia of the stiffnesses, for each storey). Due to the effectiveness of the ALPHA method for one-storey structures, it has been decided to capture the torsional behavior of each different inter-storey of a multi-storey structure with that of an equivalent single-storey system. The fundamental result lies in that, in most common cases (e.g. replication of a given storey n times to build a n-storey structure), the equivalent single-storey systems that better represents the behavior of all storeys is characterized by the total mass and the in-series stiffness of the multi storey structure.

The study of the different typology of multi-story structures (2-, 3-, and 6-story structures) leads to the determination of an equivalent single-story which is able to simulate the behavior of the *i-th* story on the analyzed structure. The most important result of the present research is the identification of a unique model for representing the multi-story-structures having identical stories or variations of mass and stiffness, having: mass m_{ss} equal to the total mass of the multi-story structure; stiffness k_{ss} equal to the sum in series of the stiffness of the n stories of the structures; relative eccentricity e_{ss} and parameter $\Omega_{\vartheta,ss}$ equal to that of the i story.

$$
\begin{aligned}
m_{ss} &= m_{tot} = \textstyle\sum_{i=1}^{n} m_i \\
k_{ss} &= k_{tot} = \left(\textstyle\sum_{i=1}^{n} \frac{1}{k_i}\right)^{-1} \\
e_{ss} &= e_i \\
\Omega_{\vartheta,ss} &= \Omega_{\vartheta i}
\end{aligned}
\qquad (1)
$$

It was not possible to determinate an unique model when either the relative eccentricity e and the parameter Ω_ϑ varies; in such cases the single-story previously obtained is not able to hit the behavior of all the stories of the structures

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3D time history analysis of RC structures versus commercial methods with attention to the modeling of floor slabs and near versus far-fault earthquakes

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ABSTRACT

Commercial softwares such as ETABS and SAP, commonly used for the analysis of apartment buildings, assume the slabs as a rigid or semi-rigid membrane and only roughly allow for the slab's flexural stiffness using the concept of effective width. These assumptions when further simplified adopting a 2D frame method that ignore the torsional effects may produce results that are very different to the full 3D finite element modeling in particular when time-history nonlinear dynamic behavior is sought. The errors could be larger in near-fault earthquakes that often excite higher vibration modes. Recent major earthquakes (Northridge 1994, Kobe 1995, Chi-chi 1999 and Bam 2003, etc.) have shown that many near-fault ground motions possess prominent acceleration pulses that result in different structural responses for common medium to high-rise buildings. This paper presents results of an analytical study performed on the effects of floor slabs in the seismic behavior of wall-frame systems under near-fault and far-fault earthquakes.

Two different plans are used to investigate the influence of the flexural stiffness of slabs. Plan type A is a typical framed structure and plan type B is a wall-frame system structure with shear walls. In order to include the effects of the floor slabs in the analysis, the slabs are to modeled by subdividing the slabs into many shell elements. To show the floor slab effect more clearly, the gross section is used for the slab stiffness.

For the dynamic response of the structure, four different types of earthquakes, El-Centro, Bam, Tabas and Manjil are applied. The selected sets of earthquake records are chosen in order to investigate the nonlinear structural response to excitations with different frequency content and duration. The equivalent static and nonlinear dynamic analyses are performed with the framed structures and the wall-frame system structures. Two plan types are analyzed for 14-story and 19-story structures.

The results show that, in all cases, the lateral displacements are reduced when the flexural stiffness of slabs is included in the analysis. The effects are more pronounced, however, in the taller 19-story wall-frame system structures. The roof displacement of the 19-story framed structure with plan type A was reduced by 24% when the flexural stiffness of slabs is considered. The roof displacements of wall-frame system structures were reduced, however, by 67% with plan type B.

Comparing the results of near-fault and far-fault records shows that the effects of the floor slabs are more significant for near-fault records. The roof displacement of 19-story framed structure with plan type B subjected to near-fault records was reduced by 37% when the flexural stiffness of slabs is considered.

Also, results show that in all cases the natural period is shorter when the flexural stiffness of the slab is included. The floor slab effects are more noticeable in taller wall-frame system structures. The differences in natural periods are more significant in the first mode which is the most important mode for the seismic response of a structure.

The model with rigid diaphragm has longer natural periods and thus lower spectral accelerations than those of model with flexural stiffness of slabs included. Therefore, if the flexural stiffness of the floor slab is ignored, the seismic loads scaled to the code base shear could be underestimated. Even though the difference in the periods is small, the difference in the spectral acceleration becomes large in the shorter period region, because the slope of the response spectrum is steep in that region. Therefore, in order to obtain more accurate results, it is important to include the flexural stiffness of slabs adequately based on the actual behavior of a building.

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Modeling and design issues of non load-bearing permanent shuttering systems with concrete under seismic loads

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ABSTRACT

Since the end of the 60s non load-bearing permanent shuttering systems based on the use of formwork blocks filled with concrete or panels of insulating material covered with spray-concrete were used across Europe for buildings in non seismic areas. The formwork blocks are laid dry, thereby eliminating the various drawbacks caused by the use of mortar, and subsequently filled with concrete, thus ensuring an adequate bearing structure. This construction method with a single operation resolves the problems related to the thermal and acoustic insulation, the thermal inertia and the structural behavior; this last taking advantage of the characteristics of the concrete used for the filling. These characteristics provide living comfort and savings on the heating costs. Same advantages characterize the permanent shuttering systems realized spraying concrete, which performs as load-bearing element, on panels of insulating material (such as polystyrene).

Extension of the use of such construction techniques in the seismic regions is now envisaged: for this reason, series of experimental tests, both quasi-static and dynamic, are currently under development together with numerical studies. This paper presents the part of the work which faces several issues among which the development of guidelines for the finite element modeling, the evaluation of the internal actions and the design. Key problem is the sought balance between simplicity and accurateness of the model and between the straightforwardness of the design procedure and the resultant structural safety. The research presented in this paper deals with different possible modeling solutions (i.e. frame macro-elements and bi-dimensional shell elements), analyzing their drawbacks and advantages at local and global level. The main objective is to investigate if the limitations of a simplified (i.e. computationally efficient) model lead to reasonable results for design purposes. Several pseudo-static experimental tests on single panels under different constraint and load conditions have been performed to validate the implemented numerical models. Shake table tests of large scale 3D buildings are under development.

Figure 1. (a) Formwork blocks filled with concrete; (b) model of the torsionally-unbalanced building.

Finally, a software for the computer-aided design, is briefly described. Such software allows the user to define a geometrical 3D model of the structure, automatically generating the finite element model with the macro-element formulation. A post-processor is implement in the software performing most of the checks required by the new Eurocode-based Italian code “Norme Tecniche per le Costruzioni”.

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Effect of shear wall and steel bracing as lateral load resisting system on a ten storey building

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ABSTRACT

Earthquake causes shaking of the ground in all three directions. All structures are primarily designed to carry the gravity loads. Since factor of safety are used in the design of structures to resist the gravity loads, usually most structures tend to be adequate against vertical shaking. However, horizontal shaking along X and Z directions remains a concern. Hence, the existing structures need to be evaluated and strengthened based on evaluation criteria before an earthquake.

Depending upon the general availability of buildings for commercial purpose, most of the buildings range around 9 to 12 storeys. Thus, the present study is concerned with the structural behavior of different lateral load resisting systems (with shear wall and with steel bracings) for a regular ten storey building under the action of gravity and lateral load both quantitatively and qualitatively. Types of 3D models considered are as given below:

1. Basic 3D Frame 1×3 bays (7.5×3 m) (BF) [Fig. 1].
2. Frame in (1.), with Shear Wall in X direction & in Z direction through out the length in end frames only (ISRB-XZ) [Fig. 2].
3. Frame in (1.), with A Steel Bracings in X direction at end frame & at mid frame in Z direction in all storeys (FASB-XMZ) [Fig. 3].
4. Frame in (1.), with A Steel Bracings in X direction & in Z direction at end frames in all storeys (FASB-XZ) [Fig. 4].

The above mensioned models are considered in zone V of Seismic zones of India. Size of columns are 750×300 mm, Beams size are 300×750 mm, 300×375 mm along X and Z axis respectively. Plinth beams size are $300\,\text{mm} \times 450\,\text{mm}$, 300×300 mm along X and Z axis respectively. The shear wall thickness considered is 300 mm and the steel bracing section considered is ISLC225 for all cases. All the

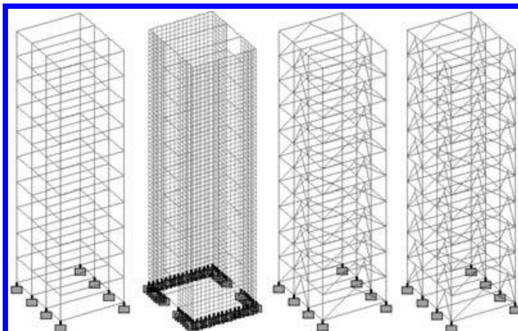

Fig 1. Fig. 2 Fig. 3 Fig. 4

models are subjected to 4 types of primary loads and 13 types of load combination. The live load considered is 4 kN/m^2 as adopted for medium office, hospital or hostel building. The Equilibrium Static Method of Analysis is adopted for the calculation of the lateral load at each floor level as per IS 1893 (Part 1): 2002.

The results show reduction between 11 to 97% in maximum support reactions, 14 to 80% in column forces, 11 to 50% in beam forces and 90 to 97% in displacement. But in columns and beams there is increase in torsion which requires some consideration.The study is hoped to be helpful during retrofitting of such structures which are initially designed only for gravity loads and found unsafe for seismic loads and or any combination of loads.

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Distribution of peak inter-story drift for estimating non-structural elements vulnerability

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ABSTRACT

Development of performance-based earthquake engineering has made consideration of seismic performance of nonstructural building components necessary. Even if the structural components of a building achieve desirable performance level after an earthquake, failure of its nonstructural components can seriously endanger the performance of the entire building. For the purpose of seismic design and assessment of nonstructural components, determination of realistic seismic demand has a critical role, and should be studied carefully. In this study we tried to consider general or local nonlinear effects in seismic response of equipments. All present design codes that have been written in order to design nonstructural elements, use some simple methods to control seismic inter story drift in height of structure stipulating linear and elastic deformation. These methods are mostly qualitative than quantitative. Equipments on the roof level and base level of structure can be examine for the same drift, nevertheless, measurements show that higher modes effects increase with height of structure. In this research the linear and elastic nonstructural displacements are studied under nonlinear behaviors of their supportive structures. In this regard the nonstructural components are investigated in two phases. In the first one the dynamic interactions between structures and non-structures are ignored. In the second phase the effects of dynamic interactions are considered and the non-structures of different masses with the same periods of supportive structure's main mode are located on the structures' roofs. Then, the maximum relative displacements are compared in two phases. For this purpose, four different models of 4,8,12 and 16 stories are constructed designed in the forms of elastic and rigid bending frames. Then, an ensemble of thirty-two different ground motions, representing hazard levels of 2, 10, and 50% probability of exceedance are used as input to the building models and nonlinear dynamic analysis conducted. In order to examine the methods effect and integral parameters, some sensitivity analysis have been done on the methods. This is important to not to override the validity of analysis. In this research the structures and non-structures parameters that affect on the height wise distribution of inter story drift are investigated. according to the results obtained in this research, the seismic demand of relative displacement feature depend mainly on the parameters such as: number of stories, rigidity or elasticity of supporting structure, relative stress of earthquakes, vibration period of supporting structure and the behavior joints patterns.

Therefore, any exact estimation of seismic demand relative displacement should be conducted with respect to these parameters. According to the observed results, the same pattern cannot be used in all structures for relative displacement distribution; yet the relative displacement distribution in the elastic domain which is mostly considered in the ordinary codes cannot be generalized in all structures.

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Suppression of wind induced vibration using elevated service reservoir as tuned mass damper

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ABSTRACT

Passive vibration control devices, such as Tuned Mass damper (TMD) have been widely used in multi storied buildings [1–3]. These devices do not require any external power supply for their operation. However, heavy mass and space allocation to install TMD sometimes restrict their uses and throws preference to active and semi active control devices. In the present paper, in built service reservoir in a multistoried apartment building has been utilized as TMD to examine its effectiveness in controlling wind induced vibration effect.

An asymmetrical planned building has been chosen for the dynamic analysis. The water tank has been modeled as TMD taking into account of the sloshing of water during wind excitation. When walls of the tank accelerate back and forth during dynamic excitation, impulsive force on the wall as well as reactive force is due to convective mode of water at fundamental mode of oscillation of water have been considered in modeling TMD.

The building considered here is a general 'n' storied (asymmetric in plan) R.C.C. framed structure. The girders on the floors are infinitely rigid as compared to column and the deformation of the structure is independent of the axial force present in the columns. Each floor is associated with two translational (longitudinal and transverse) degrees of freedom and one rotational degree of freedom.

The power spectral density function of the response due to stochastic wind field has been obtained and variances of the responses have been studied with different levels of water in the tank. For choosing the optimum parameter of TMD, the variance of response is minimized with respect to mass and stiffness. The constraints are sizes of columns supporting the tank and its height above the roof level.

A RC framed asymmetrically planned apartment building of 10 storeys is considered in the present study. The analysis has shown that the building with water tank has significant reduction in amplitudes of the displacement due to tuned mass dampening. However, it is also found from other results that due to rotational effect involved due to greater eccentricity in other direction, effectiveness of TMD slightly reduces.

The present analysis demonstrates that service reservoir that is generally provided at the roof of the building, can be effectively utilized to suppress the vibration effect on the building. The analysis has included a sloshing behavior of water in the tank during lateral vibration under wind and thus improved TMD model. It was concluded that for the particular ten storied building, when water storage tank of dimension $3.0\,\text{m} \times 4.5\,\text{m} \times 2\,\text{m}$ is placed along the long side of the building at a height of 1.8 m to 2.4 m above the roof of the building, the acceptable reduction of vibration energy can be obtained when the minimum water level is 0.5 m in the tank under wind induced motion.

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Evaluation of damping matrix for buildings

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Damping is one of the important factors to predict the response of the building under the lateral loads too. In this paper, the nature of damping matrix associated with multi degree of freedom simple shear building models is investigated. The various methods of creating damping matrices in structures are summarized and the obtained results are compared to illustrate the inconsistencies among them. Numerical examples are also used to illustrate the significance of non-zero off diagonal terms in the transformed damping matrix obtained after pre and post multiplication with mode shape matrix[Φ]. Analytical results show that the incorrect formulation of damping matrix results in highly incorrect responses. Several formulations for damping matrices are then proposed and evaluated in an effort to better predict the response of these structures. In this paper, two, five and ten story buildings are investigated and the results presented.

In recent years, significant research has been performed to study structures incorporating supplemental damping devices, base isolation, and tuned mass dampers to mitigate the seismic risk. Buildings are subjected to earthquake and other natural events in whole over the world. The dynamic response of these structures must be evaluated in the design stage. Damping in a structure has an important effect on the magnitude of its response (lateral displacement, internal forces, drifts...). Although the consideration of the structure classification is done, the first damping ratio has been set at 2%, 3% and 5%, etc. regardless of the condition of the building. This setting seems to base on experience and practice in the past, and the reason which is clearly physical is not shown. It is very difficult to evaluate the damping ratio by theoretical methods such as common transforms. The database of building damping ratio is made on various experiments and observation results done in the past, and the statistical examination of damping ratio has been made. This data is analyzed from many points such as characteristics of buildings and the influential factor to damping ratio is analyzed. The correlation is being found on some factors, but clear tendency can not be found because of large dispersion. From these results, large dispersion is included in damping ratio. It seems to become a situation in which structural designers must use practice value. Damping matrices for structures are typically created by assuming damping ratios for each mode, which may in turn be evaluated experimentally by impulse hammer testing of the entire structure or a least squares solution of test data from transfer functions obtained from sine-sweep tests.

In this paper, the different techniques of evaluating damping matrices are investigated and compared to each other.

There are several methods for formation of damping matrices in a building including, Damping Matrix in Simple Shear Buildings, Damping Matrix using Rayleigh's formulation, Damping Matrix by Superposition and Damping Matrix Using Mode Shapes.

In this paper, 3 models including 2, 5 and 10 story steel buildings are considered. Lateral loadings are calculated according to the standard 2800 and then the models designed. Consider first a two story two bay plane frame and story heights and bay lengths of 3 m and 4 m respectively. The frame consists of HEB260, HEB220 and HEB180 column sections for the first and second stories respectively and IPE200 and IPE120 beam sections for the first and second stories respectively. The frame was also excited by the impulse loading. The assumed damping value is 2%. It is concluded that:

1. The higher the mode number, the higher modal damping ratio.
2. The higher the building, the smaller modal damping ratio.
3. The damping matrix which is created by using mode shapes is differing from other damping matrices. Because in this method, a simple beam (only with two translational degrees of freedom) in order to represent a column placed in a shear model structure is considered.
4. The other techniques including simple shear buildings, Rayleigh's formulation, superposition and mode shapes are close to each other and their difference in maximum range is about 20%.
5. The higher the building, the bigger difference in discussed various methods.

4. Material modelling, numerical methods, numerical formulations, FEM modelling

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Particle dynamic methods for microsystem applications

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ABSTRACT

In this paper we present an overview of our work on particle method based simulations of dynamics in MEMS (micro-electromechanical systems).

We consider both, process simulation and device simulation. The first example is the micro powder injection moulding (PIM) of powder filled polymers, where simulation is confronted by the complex rheology of a largely deforming feedstock with free surfaces. In the second example, the micro casting of metal alloys, large differences in length scales occur. Here we show that it is useful to couple the particle method to a grid-based approach. This technique is used to simulate thermal conduction between the alloy and the mould efficiently. The hybrid approach is also useful when we require coupling to electrostatic fields, which is illustrated in the third example where we show how the simulation of conveying a micropart (see Fig. 1) by means of electrowetting can benefit from particle methods. Eventually we give an example from device simulation with particle methods where we consider carbon nantotube resonators (CNT). We perform a molecular coarse-graining that results in a dissipative particle dynamics model. Without considering all the molecular details, this method naturally includes thermal fluctuations, which cannot be ignored on this length scale.

Particle-based simulation methods exhibit two major advantages over standard mesh-based approaches such as finite elements: they perform well under large deformations of materials with free surfaces, and they cope with a larger range of length scales. The latter becomes clear when thinking about atoms to be easily represented by Newtonian particles, while it is difficult to interpret a finite element on this scale. This flexibility makes particle methods also good candidates for the simulation of problems related to MEMS.

The two methods used are smoothed particle hydrodynamics (SPH) and Dissipative Particle Dynamics (DPD). SPH is based the concept of interpolating an arbitrary function f (r) by a sum over a finite set of points or "particles" distributed in space (Gingold and Monaghan 1977).

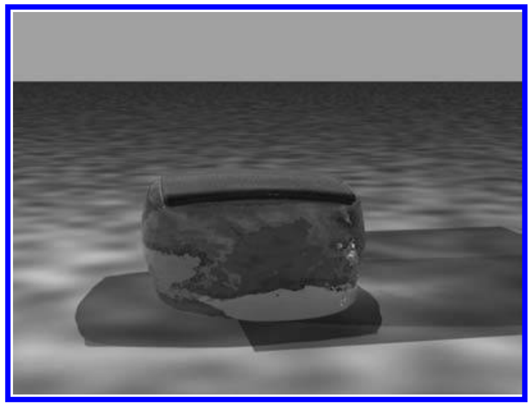

Figure 1. Micropart conveyed through electrocapillary forces. The strong deformation of the droplet is due to increased importance of surface forces on the microscale.

DPD is a particle method based approach with microscopic background DPD (Hoogerbrugge and Koelman 1992). As in SPH, there is an equation of motion for the particle's velocities. The main difference is due to the dissipative and the stochastic force acting between pairs of particles.

The examples given show in detail the usefulness of the simulation methods and their respective field of application. The possibility to couple these methods easily to other simulations methods make them very promising tools for the simulation of soft matter and its coupling to solid materials and objects.

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Introduction of multi-scale modeling techniques for the analysis of long span FRP composite bridges

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ABSTRACT

An innovative solution to safely bridge very long spans is proposed in this paper by introducing a hybrid type cable stayed bridge as a competent system. The new system leads to huge reduction in deck weight and critical stresses in the pylon zones by using a hybrid advanced composite deck. It also reduces the stiffness losses of the stay-cables due to the catenary action by using carbon fiber reinforced polymer (CFRP) cables. The paper proposes a consistent and systematic analysis and design procedure (ADP) that optimizes and precisely simulates the proposed bridge system. It recognizes the changes in structural behavior of the cable-stayed system, and accordingly clearly defines the ultimate and serviceability limit states for such a new structural system, consistent with limit state design philosophy. The design steps of the ADP represent a multi scale design approach, while the analysis steps characterize a multi-scale modeling technique. They represent material design at the micro/macro-level, and accurate homogenization of the advanced composite components properties and evaluation of the resulting anisotropic characteristics. The method provides three dimensional simulation of the hybrid bridge system, involving all its nonlinearities. The multi-scale modeling technique connects two phases, the micro-macro level phase and the structural level phase. The modeling procedure accurately homogenizes the advanced composite individual components' properties and quantitatively evaluates the resulting composites' anisotropic properties in its first phase. The procedure then precisely simulates the resulting hybrid system, in the structure level, by employing three-dimensional non-linear modeling techniques.

A multi-scale modeling is performed as a numerical example. For the first phase, the micro/macro level, two programs were developed and tested in comparison with some experimental data, using Visual FORTRAN. The first program was the stochastic finite element model for the advanced composites with random inclusion, while the second program represents the analytical procedure for the periodic advanced composites. However, in both computer programs, the slip-bond, crack initiation and crack propagation phenomena at interface between fiber and matrix were not considered in the analysis. For the second phase which is related to modeling the bridge in the structure level, the results obtained from the first phase are used as input to define the material. The ABAQUS general purpose finite element software is used for this phase of the analysis. It is worth to integrating the first phase results, the second phase procedures and the finite element analysis with ABAQUS to get the benefit of such top-state-of-the-art finite element software.

The three dimensional nonlinear-anisotropic finite element models presented in this research are more consistent with the actual material and geometrical properties and the complicated system nature of the proposed hybrid cable-stayed bridge. Many properties, which have never been noted before or studied individually, are well illustrated in the present modeling technique, for example, the cables actual deformation shape during the deck vibration, the internal stress distribution and the distribution of the stress failure criterion (Tsai-Hill for instant) of the advanced composites in deck parts and the stress concentrations.

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High-performance finite element and coupled Eulerian-Lagrangian simulations of pile installation processes

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ABSTRACT

In recent years, several researchers used the finite element method to simulate the pile driving process. These simulations lead to a better understanding of the influence of pile installation on the surrounding soil and adjacent structures. Even the mechanisms of soil plugging depending on the installation method can be investigated. An explicit time integration scheme implemented in Abaqus/Explicit is used. The soil body is discretised using a hypoplastic constitutive law. Regarding these analyses, the capabilities and limitations of finite element method in context with pile installation process simulations are discussed.

In Abaqus, Version 6.8 and 6.8-EF a Coupled Eulerian-Lagrangian (CEL) approach is implemented to investigate problems involving large deformations. This approach is used to simulate the pile installation process, too, to overcome limitations of the afore described finite element simulations.

Therfor, pile jacking of a circular pile with a diameter of 30 cm into granular material is simulated using the two different approaches (CEL and classical finite element method). The granular material in the numerical analyses is discretised using a hypoplastic constitutive model. The considered finite element models for pile installation simulation have been successfully validated by comparison with insitu measurement results (Henke, 2008 or Mahutka, 2008).

In this paper, the void ratio distribution and the stress state around the pile after installation are compared for the different modelling techniques and it can be stated that both methods show similar results.

Thus, it can be stated that CEL is well suited to simulate pile installation processes. Furthermore, this approach has some advantages compared to finite element method. Especially, the simulations are more robust such that even high friction values between soil and pile can be considered.

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Identification of material properties for a unified visco-plasticity model and application to isothermal and anisothermal experimental data

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ABSTRACT

An experimental programme of cyclic mechanical testing of a 316 stainless steel and a P91 steel at temperatures up to 600°C, under isothermal conditions, was carried out in order to identify the material constants for a constitutive model. The constitutive model adopted is a modified Chaboche, unified viscoplasticity model. This model was fist proposed by Chaboche and Rousselier (1983) and is base on the viscoplastic flow rule as shown by equation (1):

$$\dot{\varepsilon}_p = \left\langle \frac{f}{Z} \right\rangle^n sgn(\sigma - \chi) \tag{1}$$

where f represents the model yield criterion, described by equation (2):

$$f = |\sigma - \chi| - R - k \tag{2}$$

This constitutive model can deal with cyclic effects, such as combined isotropic and kinematic hardening, described by equations (3) and (4) and equation (5), respectively, as well and rate-dependent effects, associated with viscoplasticity, shown by equation (6).

$$\dot{\chi}_i = C_i\left(a_i\dot{\varepsilon}_p - \chi_i\dot{p}\right) \tag{3}$$

where i = 1, 2

$$\chi = \chi_1 + \chi_2 \tag{4}$$

$$\dot{R} = b(Q - R)\dot{p} \tag{5}$$

$$\sigma_v = Z\dot{p}^{1/n} \tag{6}$$

The characterisation of the materials is presented and compared to the experimental results obtained from cyclic isothermal tests as well as in-phase (IP) and out-of-phase (OP) anisothermal tests. Linear interpolation between consecutive values as f(T) for each of the isothermal material constants was used to predict the material behaviour under these anisothermal conditions.

A least squares optimisation algorithm has been developed and implemented for determining the material constants in order to obtain the best fit of the model to experimental data, using initially obtained material constants, from experimental data, as the starting point in this optimisation process. The process for determining the initial estimates for the material constants has been described in previous publication by Hyde et al (2009, 2010). Also described within these publications is the subsequent optimisation process which takes account of the stress-strain loops and hardening/softening behaviour by use of two separate objective functions. The model predictions using both the initial and optimised material constants are compared to experimental data showing excellent correlation. Typical examples of these comparisons are shown by Figure 1 and Figure 2.

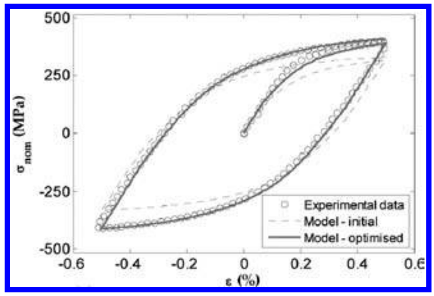

Figure 1. Initial tensile curve and first loop for P91 steel under isothermal conditions at 500°C.

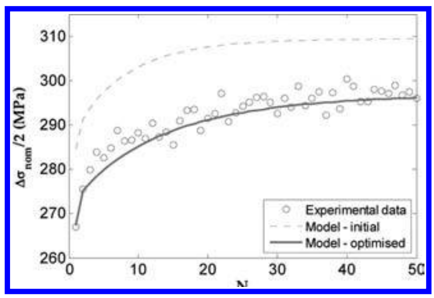

Figure 2. Hardening behaviour for 316 stainless steel under isothermal conditions at 500°C.

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A nonlinear micropolar continuum theory for initial plasticity

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ABSTRACT

The mechanical behaviour of metals is influenced by dislocations already in the elastic range. This is often named anelastic effect and in coherence with grain size and structure. As a consequence, a decreasing shear modulus of 1 up to 23% has been observed experimentally, see. e.g. Nowick & Berry (1972). Corresponding to the driving forces dislocations move and produce distortion within the lattice. Depending on the shear stress τ dislocations unsnap from fixed joints and lead to irreversible distortion. However, below a certain limit for τ dislocations just bow between these joints in accordance to driving forces. This is well known as inceptive Frank-Read mechanism. In a simple microscale model, the shear stress τ causes the dislocation to slip within the unpinned dislocation length L_0 yielding the macroscopic strain quantity $\varepsilon_{\mathrm{ae}} = (L_0^2 \tau \Lambda_d)/(6\mu)$, whereas Λ_d is the scalar dislocation density and μ the shear modulus. Due to the curved configuration of dislocations higher dislocation energy results. Assuming the same magnitudes for self-equilibrating stresses caused by a straight and a curved dislocation, the increase of dislocation energy is proportional to the line elongation. Then, an anelastic strain potential can be defined by

$$W_{\mathrm{ae}} := \frac{3\,\mu}{\Lambda_d^2\, L_0\, \mathrm{V}}\, \varepsilon_{\mathrm{ae}}^2 \,. \tag{1}$$

We deal with a micropolar model and decompose the deformation gradient $\mathbf{F}$ locally in two parts: pure elastic deformation represented by $\mathbf{F}^{\mathrm{e}}$ and pure anelastic deformation $\mathbf{F}^{\mathrm{ae}}$. Thus, $\mathbf{F} = \mathbf{F}^{\mathrm{e}}\mathbf{F}^{\mathrm{ae}} = \mathbf{R}^{\mathrm{e}}\ \mathbf{U}^{\mathrm{e}}\ \mathbf{F}^{\mathrm{ae}}$, $\mathbf{U}^{\mathrm{e}} \in \mathrm{PSym}$ applying the polar decomposition of $\mathbf{F}^{\mathrm{e}}$. Apparently, for $\mathbf{F}^{\mathrm{ae}} = 11$ the rotation $\mathbf{R}^{\mathrm{e}}$ departs from the rotation of the polar decomposition of $\mathbf{F}$. Allowing anelasticity $\mathbf{F}^{\mathrm{ae}} = 11$ asks for independent rotations $\mathbf{R}^{\mathrm{e}}$ where we meet the spirit of a Cosserat theory: we identify $\overline{\mathbf{R}} = \mathbf{R}^{\mathrm{e}}$, $\overline{\mathbf{U}} = \mathbf{U}^{\mathrm{e}}\ \mathbf{F}^{\mathrm{ae}}$ with independent rotations $\overline{\mathbf{R}} \in \mathrm{SO}(3)$ and $\overline{\mathbf{U}} = \overline{\mathbf{R}}^T\ \mathbf{F}$ with the first Cosserat strain. Next, we define anelastic deformation via $\mathbf{F}^{\mathrm{ae}} := 11 + \varepsilon^{\mathrm{ae}}$ and elastic deformation via $\mathbf{U}^{\mathrm{e}} := 11 + \varepsilon^{\mathrm{e}}$, $\varepsilon^{\mathrm{e}} \in \mathrm{Sym}$. Neglecting terms of higher order and splitting the first Cosserat strain into $\mathrm{skew}[\overline{\mathbf{U}} - 11] \approx \mathrm{skew}[\varepsilon^{\mathrm{ae}}]$ and $\mathrm{sym}[\overline{\mathbf{U}} - 11] \approx \varepsilon^{\mathrm{e}} + \mathrm{sym}[\varepsilon^{\mathrm{ae}}]$ allows to say: Anelasticity accounts for nonsymmetric strains within a Cosserat theory. Further, anelastic strain can be decoupled from elastic strain by neglecting terms of higher order in deformation and by setting $\mathrm{sym}[\varepsilon^{\mathrm{ae}}] = 0$. This yields the anelastic deformation $\mathbf{F}^{\mathrm{ae}} = 11 + \mathrm{skew}[\varepsilon^{\mathrm{ae}}] \approx \mathbf{R}^{\mathrm{ae}}$ to become a rotation in first order. However, small rotations $\mathbf{R}^{\mathrm{ae}}$ are consistent with initial plasticity. A two field problem results from the assumed kinematics, which is posed in a variational setting. The task is to find a pair $(\varphi,\ \overline{\mathbf{R}})$ minimizing the energy functional $\Pi(\varphi,\ \overline{\mathbf{R}}) = \int_{B_0}\ \mathrm{W}_{\mathrm{mp}}(\overline{\mathbf{U}}) + \mathrm{W}_{\mathrm{curv}}(\mathfrak{C})\mathrm{dV} + \Pi^{\mathrm{ext}}(\varphi,\ \overline{\mathbf{R}})$ combined with boundary conditions. The curvature measure is defined via $\mathfrak{C} := \epsilon_{aik}\overline{R}_{jn}\overline{R}_{ji,a}\ \mathbf{e}_n \otimes \mathbf{e}_k = \mathrm{Curl}_\sharp[\overline{\mathbf{R}}]\ \cdot\ \overline{\mathbf{R}}$ and covers Nye's curvature, see Neff & Münch (2007). Comparing $W_{\mathrm{mp}}(\overline{\mathbf{U}})$ with Eq. 1 we obtain $\mu_c = (3\mu)/\Lambda_2^d\ \mathrm{L}_0\ V)$. Nowick & Berry (1972) estimate typical values in crystals about $\Lambda_d \approx 1 \cdot 108/\mathrm{cm}^2$ and $L_0 \approx 3 \cdot 10^{-5}$ cm. Using this estimation the Cosserat couple modulus μ_c approaches zero.

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A new material model for concrete within microplane framework. Part I: Constitutive formulation and computational algorithm

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ABSTRACT

The purpose of this two-part paper is to develop an accurate and reliable concrete material model used for nonlinear analysis of plain and reinforced concrete structures. To this end, the well known microplane theory is chosen for describing the fracture and damage of concrete under various stress states.

The performance of an advanced and reasonably widely used microplane constitutive model for concrete – model M4 developed by Prof. Bažant et al. (2000) is carefully examined. Numerical experiments indicate that this model still exhibits some undesirable behavior, such as

- The model response is sensitive to the strain increment unless the strain increment is smaller than a critical value;
- The model fails in correctly describing the tensile fracture of concrete, characterized by a residual tensile stress at even quite large crack opening and spurious excessive lateral strain in the postpeak regime;
- The model gives less accurate predictions for the Poisson's ratio of concrete under uniaxial compression and tension;
- The model overestimates the ultimate strength of concrete in biaxial and triaxial tension as well as in combined tension-shear due to the erroneous representation of the lateral behavior in uniaxial tension;
- The model is not able to realistically reproduce the increase of the strength, the deformation and the postpeak ductility for concrete under lateral confining pressure.

These problems must be solved to accurately predict the responses of reinforced concrete structures under general loadings with this model.

A new model, called microplane model M4L is then formulated in this paper to overcome the weaknesses of model M4. The new model enhances model M4 both in constitutive formulation and in computational algorithm. The main contributions are:

- A new concept of macroscopic stress-adjusted effective microplane elastic moduli, making it possible to realistically capture the increased deformation capacity in triaxial compression;
- An improved normal stress boundary allowing the tensile crack bridging stress approach zero at sufficiently large strain;
- Two novel deviatoric stress boundaries with strength limits and slopes of the softening branch scaled with the macroscopic stress, benefiting for simulating the enhancement of ultimate strength and postpeak ductility under low confinement;
- An enhanced shear stress boundary for accurately representing the variation of the Poisson's ratio in uniaxial loadings;
- An adoption of Di Luzio's (2007) idea of coupling a microplane model with V-D split with a model without V-D split for tensile fracture, making it possible to correctly reproduce the postpeak lateral deformation in uniaxial tension;
- An innovative and efficient loop for the calculation of the microplane volumetric and deviatoric stresses, which is helpful for achieving more stable results.

An extensive numerical validation and application of the new model to structural analysis will be given in the second part of this paper.

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A new material model for concrete within microplane framework. Part II: Numerical validation and application to structural analysis

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ABSTRACT

This part presents the numerical validation as well as the application of the model M4L formulated in Part I of this paper to structural analysis. The model parameters are calibrated through best fitting of typical test data through inverse considerations.

The model responses are compared with a wide range of test data in the literature. Numerical experiments indicate that model M4L is able to realistically describe many complicated inelastic behavior of concrete under different loading conditions. The model exhibits significant improvements over model M4 in correctly representing the lateral deformation in uniaxial tension and compression, the strength reduction in bi- and triaxial tension, the increase of ultimate strength, deformation capacity and the postpeak ductility under low and high confinements, and the stress-strain response for concrete subjected to combined tensile and shear loadings, while most of the successful aspects of model M4 remain.

After the verification of the accuracy and reliability at the material level, model M4L is applied to simulate the mechanical response of a complex reinforced concrete structure tested at Leipzig University (Tue & Schwarz 2009), which is a high strength concrete (HSC) column – normal strength concrete (NSC) slab system subjected to constant column load while progressively increased slab load, as shown in Figure 1. The failure of the structure is found to be punching failure of the NSC slab.

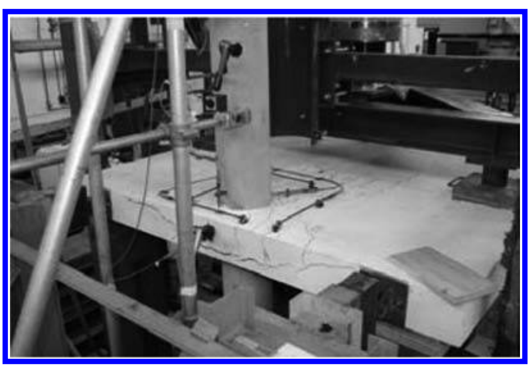

Figure 1. Specimen: HSC column-NSC slab.

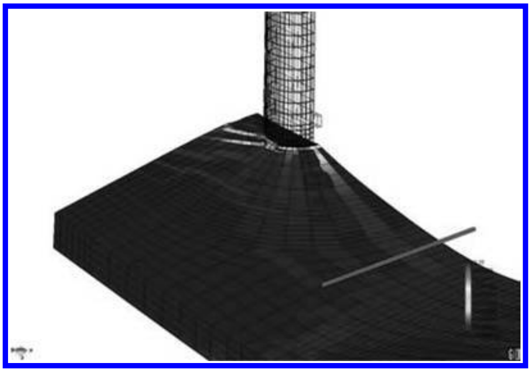

Figure 2. Predicted crack patterns in the slab.

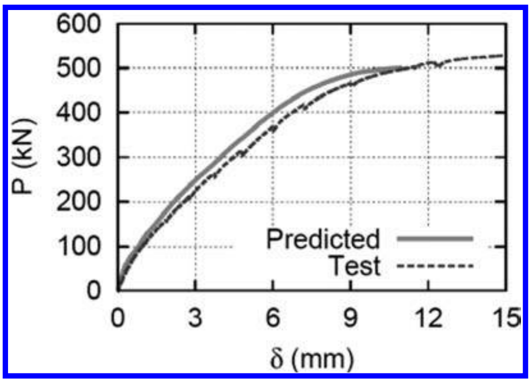

Figure 3. Load-deflection curve of the structure.

The predicted responses from finite element analysis with model M4L are shown in Figure 2 and Figure 3. It can be seen that the numerical model correctly reproduces both the crack patterns (due to bending and punching, respectively) in the slab and the load-deflection response of the structure.

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A micromorphic theory and its application to elastic size-scale effects

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ABSTRACT

Classical theories of deformation do not provide internal length scales and do therefore not account for size-scale effects of heterogeneous materials as observed in experimental work. The size of material constituents in relation to the dimensions of the specimen or structure cannot be considered as neglible and the interaction between the constituents needs to be addressed. In this context so-called generalized continuum formulations have proven to provide a remedy as they allow for the incorporation of internal length scale parameters which reflect the micro-structural influence to the macroscopic material response.

Based on a generalized continuum framework (Sansour, 1998) a multiscale approach is derived and a micromorphic deformation description is formulated which introduces additional degrees of freedom. Making use of the micromorphic deformation description, new strain and stress measures are defined which lead to the formulation of a corresponding generalized variational principle. Of great advantage is the fact that the constitutive law is defined in the generalized space but can otherwise be considered classical. This limits the number of the extra material parameters necessary to those needed for the specification of the micro-space. In contrast to classical and other generalized continuum approaches, e.g. strain gradient theories, the micromorphic approach benefits from the fact that size-scale effects exhibited in uniaxial compression characterized by a constant deformation field can be addressed.

The theory is applied to model uniaxial compression experiments on concrete members. Cylinders with 80 mm diameter and 75 mm height are presented. The specimen size as well as the volume ratio of coarse aggregates to mortar matrix were kept constant for all specimens while the aggregate size changed from 6.7 mm to 26.5 mm. The Youngs moduli of the different specimens were calculated according to the outlined theory and compared with experimental results.

The outcome of the model and the experiments (Fig. 1) showed that (i) size scale effects were captured in this specific example and (ii) by increasing the aggregate diameter, the material stiffness was increased.

The presented micromorphic model with two micro spaces could qualitatively describe size scale effects in concrete specimens. However, further experiments with a variety of materials needs to be carried out to enable prediction of the right input parameters needed for the model.

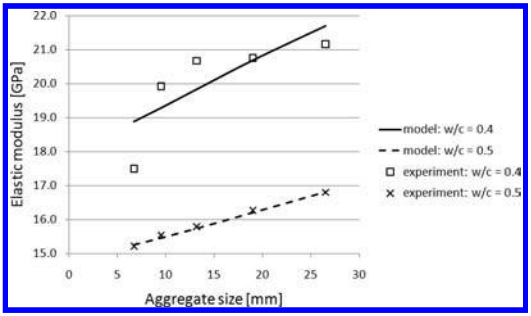

Figure 1. Variation of the elastic modulus by changing aggregate sizes.

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Nonlinear analysis of large concrete structures using a finite element shell model

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ABSTRACT

For analysing large reinforced concrete structures with nonlinear finite elements the engineer can choose between a high resolution model with discrete crack formation and a low resolution model with smeared cracks. The former method leads to a highly sophisticated analysis with an expensive remeshing procedure if necessary. In order to analyse large structures within moderate computation time smeared crack models can be used where the cracked solid is considered as a continuum and nonlinearities are treated in terms of stress-strain relations. A common method is using a tension stiffening approach for reinforced concrete. Tension stiffening comprises the strain-softening of concrete and the effects resulting from the interaction between concrete and reinforcing steel (i.e. friction, bond slip and load transfer across cracks).

In this contribution a new tension stiffening approach is presented and applied to flexural members. The cracking and deformation behaviour in the tension zone is accounted for by considering the influences of shrinkage restraint and random strength distribution. The tension stiffening formulation is derived from the Tension Chord Model (Marti et al. 1998), a physically consistent discrete crack model that can depict the cracking and deformation behaviour of structural members. At every significant load stage (i.e. initial cracking, stabilised cracking, yielding of reinforcement in a crack and failure of the reinforcing bar) the tension force $F = F_c + F_s$ is subdivided into the contributions of concrete and steel by fulfilling the equilibrium condition at every crosssection (Fig. 1).

The resulting expressions for average concrete stresses and corresponding average strains along the tension chord are transformed to the tension stiffening formulation. Furthermore, initial and residual concrete strength values are modified to allow for the effects of shrinkage and an increase in total tensile response during crack formation (Wasner & Sigrist 2010).

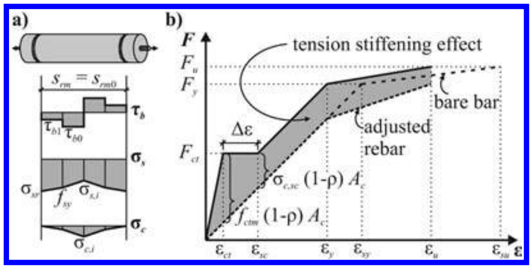

Figure 1. Tension Chord Model: a) tension chord with maximum crack spacing; b) load-strain diagram for tension chord.

The tension zone of a cracked reinforced concrete member is dominated by the behaviour of the reinforcing bars and the surrounding concrete. This shallow band of concrete embedding the reinforcement forms the tension chord of a flexural member. Since a single constitutive law for concrete has to be defined in a non-layered (finite) shell element, the depth of the tension chord is accounted for in the expressions for the reinforcement ratio.

Model computations are compared to test results of two-span slab strips, and it is shown that the proposed model is capable to yield sufficiently good results with respect to failure loads. In a next step, the ongoing research will concentrate on general geometries and loading situations as well as on physical and numerical strain effects.

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Compact RC structures with textile strengthening: Computational models and application

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ABSTRACT

Concrete constructions with textile reinforcement provide an opportunity of increasing load-bearing capacity of existing and potentially damaged structures. New textile technologies permit the effective production of textile surface structures with several layers of filament threads made of glass or carbon. Although this new type of reinforcement applied to strengthening of existing structures is effective, clarification is still required concerning the modified load-bearing behaviour and an assessment of structural safety.

This paper is mainly devoted to enhanced computational algorithms to account for the load-bearing reserves of compact reinforced concrete structures with textile reinforcement under quasi-static loading processes. It is essential for the numerical model, to take into account the physical nonlinearities of concrete (i.e. cracking) and of the reinforcement (yielding of steel, damage of the textile roving). The three-dimensional endochronic material law by Bažant in combination with a smeared crack model is used for the concrete. The smeared crack model consists of up to three non-orthogonally fixed cracks.

A multi-reference-plane model (MRM) is developed for the realistic numerical simulation of the load bearing behaviour of reinforced concrete folded plate structures with textile strengthening (Möller et al. 2005). For compact reinforced structures like columns or T-beams, it is necessary to use 3D finite elements.

Hybrid MRM-elements have been applied for the realistic numerical simulation of the load bearing behavior of RC folded plate structures with textile strengthening. Hybrid eight-node hexagonal solid elements for the physical linear analysis are described in (Pian & Wu 2006). For the physical nonlinear analysis of reinforced concrete (RC) and textile reinforced concrete (TRC), respectively, two kinds of reinforcement are introduced – single fibers and fiber layers (Fig. 1).

The formulation of the hybrid eight-node hexagonal solid element with embedded (textile) reinforcement based on the functional of Hellinger-Reissner in differential notation

$$\Pi_{HR1,NC} = \int_V \left(d\underline{\sigma}^T (\underline{G}\, d\underline{v}) - \frac{1}{2} d\underline{\sigma}^T d\underline{\varepsilon} - d\underline{\overset{+}{p}}{}_V^T\, dv \right) dV - \int_{O_p} d\,\underline{\overset{+}{p}}{}^T\, dv\, dO$$

with $d\sigma$, $d\varepsilon$, dv – differential stress, strain and displacement increase in the Volume V, $d\underline{p}_V$ – external forces in V and $d\overset{+}{p}{}^T \pm$ external forces along the boundary surface O_p and the differential operator $\underline{G}$.

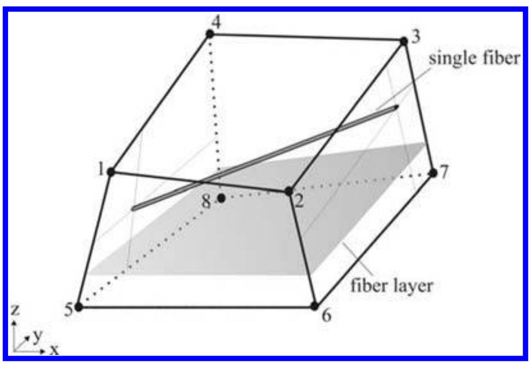

Figure 1. Eight-node solid element with embedded reinforcement.

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Isogeometrical analysis of functionally graded materials in plane elasticity problems

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ABSTRACT

In this article a unified modeling and analysis approach to address the functionally graded plane problems is presented which is making use of B-Splines and NURBS for the definition of geometry and material properties as well as the analysis.

FGMs are relatively complicated composites in which the volume fraction of constituent materials varies gradually, giving a non-uniform microstructure with continuously graded macro properties [1].

The recently developed isogeometric analysis numerical method is concisely explained and the functionally graded materials are briefly introduced. It is shown that the difficulties encountered in the Finite Element analysis of the FGMs are to a large degree alleviated by employing the mentioned method. Finally, a few examples are presented to demonstrate the efficiency of the method.

The main idea of the proposed method is inspired by the core idea of the isogeometrical analysis. That is, the components of the recovered stress field are considered as an imaginary (hyper-) surface. This surface is constructed by the same spline basis functions and NURBS shape functions that were employed for approximation of the components of the unknown displacement field vector. In two dimensional elasticity problems, similar to the isogeometrical analysis method [2–5] itself, the *x* and *y* coordinates of the control points, which are used for the analysis, are considered as before and their *z* coordinates are calculated based on a criteria which is constructed by using the well known notion of superconvergent points in finite elements.

The performance of the method is demonstrated via an example solved with different variations of the modulus of elasticity. It is shown that the proposed method has superior performance in comparison with the conventional FE method. A square plate with unit thickness subjected to two equal point loads at the free end corners, as illustrated in Figure 1, under plane stress conditions is considered. The obtained results for the centerline vertical displacements are depicted in Figure 2.

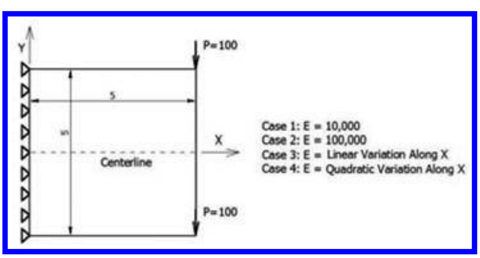

Figure 1. Problem definition.

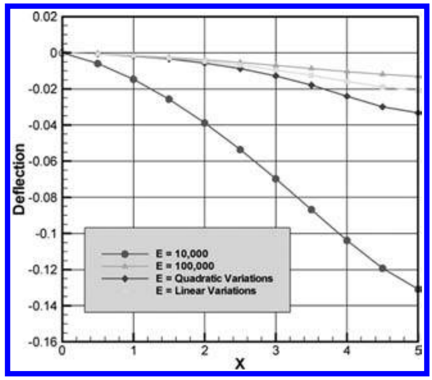

Figure 2. Variations of the centerline vertical displacements obtained by IA for different cases.

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Investigation and modeling of the glued steel-glass connection used in hybrid beam

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ABSTRACT

New hybrid steel-glass beam consists of steel flanges and glass web bonded together by polymer adhesive, see Figure 1.

Glued joint is the key element of whole composite structure, therefore also the key aspect of research, development and numerical modeling. This paper deals with the research pointed on the adhesive behaviour, ultimate carrying capacity of the area glued joints and calibration of the FE models of polymer adhesives, transferred from industry to glass construction design (epoxies, acrylics, polyurethanes, silicones).

Pure glass beams always fail suddenly and without extensive previous warning. In case of composite structures (hybrid beams), stiff member, assembled to the tensioned part of the beam, is able to work as a consumer of the break energy even if the first cracks in the glass pane are visible. Hybrid glass structures can this way achieve higher stiffness, load carrying capacity and also residual capacity, which depends mainly on used kind of adhesive and type of glass (Louter, 2007).

Well known Möhler's method was modified and applied to describe the behaviour of the hybrid beam and to determine the normal-stress distribution along the cross section of this composite structure including semi-rigid horizontal shear connection realized by polymer adhesive (Netusil, 2009)

For the purpose of hybrid beams, in general, adhesively bonded joint should be rigid enough to provide an optimal interaction between both connected elements, but whole connection has to compensate different temperature deformations of steel and glass. Wide range of adhesives with different properties was involved to experimental program. Set of adhesives includes almost all common used material types of polymer adhesives - starts with a very stiff epoxy resin, goes down via acrylics and polyurethanes to very flexible silicone. Special emphasis was devoted to the UV stability and long-time behaviour.

Volume models of tested glued connections were created and calibrated by software package Ansys 11. Material models were found for each of the chosen adhesives. Tougher adhesives, for example epoxy resins or acrylics could have been modeled as multilinear elastic, with using appropriate input material parameters. There is a possibility to use phenomenological material models, which are included in most of the common used finite element software. Hyperelastic models, i.e. Mooney-Rivlin, Arruda-Boyce or Ogden, which can be calibrated by curve fitting, can very accurately predict the behaviour of compliant adhesives like silicones or polyurethanes achieving very big elongation or shear slope and therefore common material models are not properly working and fail too early.

Figure 1. Hybrid steel-glass beam.

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Coupled hygro-mechanical stress analysis on adhesive material under hot/wet environments

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ABSTRACT

The advantages of adhesive bonding over traditional joining techniques have been proved to be smoother load transition, weight savings to the whole structure and ability to join different materials. They have been used in automotive, aerospace and also civil engineering industry. However, the long-term performance of adhesive materials under hot/wet environments is not clearly defined and the durability modeling and lifetime prediction of the adhesively bonded joint are still issues which designers and engineers have to face. In this paper, moisture diffusion in the adhesive material is discussed and analyzed numerically. Moisture diffusion coefficients and moisture-dependent mechanical properties of adhesive were deduced from experimental studies. The moisture concentration distribution of a dog-bone adhesive model in different time intervals was obtained by using FE software ABAQUS, which can subsequently be used as input for the coupled hygro-mechanical stress analysis. Figure 1 shows the comparison between the moisture concentration distribution and stress distribution throughout the thickness of the adhesive specimen model at the aging time equal to one day. Due to the mechanical degradation of the adhesive material under moisture effect, stiffness of both external parts is lower,

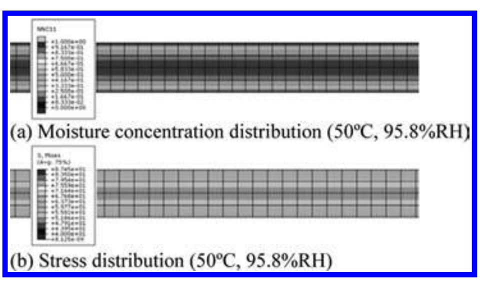

Figure 1. Comparison between the results of moisture diffusion analysis and coupled hygro-mechanical stress analysis.

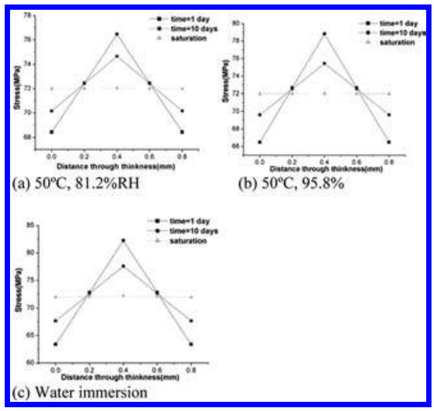

Figure 2. Stress distribution in different time intervals.

and correspondingly higher stress distribution will concentrate on the middle layer.

Three time intervals (1 day, 10 days and saturation level) are selected to investigate the changes of stress distribution on the adhesive specimen model under different aging conditions. From figure 2(a) (b) (c), the same conclusion can be drawn that, at the initial moisture ingress period (time = 1day), the stress distribution in the through-thickness direction is rather abrupt with the stress values in the middle are 11.75%, 18.54% and 29.77% higher than the external parts. But when moisture content reaches the saturation level, stress distribution is much balanced.

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A Genetic Algorithm based procedure for the constitutive characterization of composite plates using dispersive guided waves data

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ABSTRACT

Considerable work has been done in the field of identification of elastic constants of either isotropic or orthotropic materials by means of elastic waves (Vary A. 2007). Numerous approaches based on Bulk Waves (BW) were proposed in the past (Castagnede B. et al. 1990). More recently, researchers are exploiting the dispersive properties of Guided Waves (GW) to characterize material properties in waveguides such as beam-like or plate-like structures. Compared to BW, the use of GW for material characterization provides a set of information at several different frequencies that can be proficiently used for the identification. In fact, for given geometry and material properties of the waveguide, the waves behavior changes with the frequency of propagation (dispersion).

Here the group velocity dispersion curves (c_g) are exploited for the characterization of the elastic constants C_{ij} in anisotropic plates. The characterization is performed by minimizing an objective function, based on the discrepancy between experimental (c_g^{exp}) and numerical (c_g^{num}) group velocity curves, via Genetic Algorithms (GA). As known, c_g^{exp} can be extracted from a RF signal by means of time-frequency transform (TFR) (De Marchi L. et al. 2009). A TFR yields a contour plot that retains the time-frequency energy content of the propagative waves. For a known distance of propagation (source-receiver), the group velocity curve for each wave can be obtained by taking the time of arrival of the corresponding TFR peaks at various frequency values. In this pilot study the c_g^{exp} curves are simulated for a set of known material properties (C_{ij}^{exp}).

In brief, GA iteratively updates the material properties (C_{ij}^{i}) into a Semi-Analytical Finite Element (SAFE) formulation (Bartoli I. et al. 2006), used to compute c_g^{num}, until the designed objective function is minimized.

The use of GA coupled with guided waves for material characterization is not new. For instance, a GA procedure based on the Lamb waves speeds at a single frequency-thickness value computed at different angles of propagation (using a circular array of receivers) was proposed to reconstruct all nine elastic moduli of orthotropic plates (Vishnuvardhan J. et al. 2007). The novelty here consists in extracting guided waves over a frequency range for a reduced number of directions of propagation (reduced number of sensors), to build the GA objective function. Results of the procedure tested on an orthotropic plate seem to be promising.

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A viscoplastic constitutive equation for the modelling of tailings heaps consisting of rock salt

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ABSTRACT

A numerical constitutive equation with the name CAPCREEP was developed at the Institute and Laboratory of Technische Universität Darmstadt, Germany, for the three-dimensional and time variant investigation of the displacement behaviour and stability of tailings heaps consisting of granular rock salt. Within the scope of the development the numerical model was implemented in the Finite- Element-Program ABAQUS. Such tailings heaps (Fig. 1) develop in the course of potassium fertilizer production.

Intact rock salt, as it is mined for example for table salt production, has a low pore volume. The density of intact rock salt corresponds roughly to the grain density. The mechanical properties of rock salt, such as the strain and strength behaviour, are particularly depending on time and rate respectively, stress and temperature.

Index tests as well as approximately 200 triaxial tests of different execution kind were performed for the macroscopical investigation of the mechanical properties of rock salt.

The strain ε of a rock salt specimen consists of a spontaneously and instantaneously respectively elastic and plastic part in the early stage as well as a viscous part as a result of temperature and stress dependent creeping. The appearance and the magnitude of the single strain parts depend on several conditions, especially from the stress state. The instantaneously plastic material behaviour is influenced by the stress ratio (minimum to maximum principal stress), the strain rate and the compaction state.

Since the investigation of the material behaviour of rock salt was carried out by use of macroscopic geotechnical laboratory tests, an empirical and phenomenological approach is used for the mathematical formulation of the numerical model. Using this approach it is possible to introduce the decisive strain parts separately and to connect them additively.

The numerical model CAPCREEP as a user defined material routine is been implemented via an interface in the Finite-Element-Program ABAQUS. In the numerical model the stress and strain state, the material stiffness as well as further solution dependent state variables are computed as a component of the system stiffness. Therefore, the material behaviour is actualized in every integration point of an element.

For verification and proof of the suitability of the numerical material model CAPCREEP, element tests of different execution manners were considered at first. Thereby, it was possible to verify the single mathematical material approaches. Following this, the developed material routine was inserted for the simulation of the stockpiling of the tailings heap recognizable on the left margin of Figure 1.

On basis of element tests, the single approaches are verified as accurate and effective. The Finite-Element calculation, in which the complex stockpiling of the structure was modelled as threedimensional and time variant, shows the suitability of the new numerical model CAPCREEP with respect to the results of the investigation drillings.

Additionally, the introduced numerical calculations displayed a robust and an efficient solution behaviour, which is an important and essential requirement for a user-defined material model.

Figure 1. Tailings heaps.

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Advances in strongly-coupled fluid-structure interaction by space-time finite elements

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ABSTRACT

This contribution discusses recent developments in simultaneous (monolithic) analysis of fluid-structure interaction based on the space-time finite element method. A weighted residual-based approach to numerical analysis of fluid flow around flexible thin-walled structures, enabling the investigation of flow-induced vibrations of strongly coupled systems involving large structural motion and deformation, is presented. Within the simultaneous solution procedure, velocity variables are used for both fluid and solid, and the whole set of model equations is discretized by a stabilized time-discontinuous space-time finite element method. Flexible structures are modeled using a three-dimensional continuum approach in a total Lagrangian setting considering large displacements and rotations. In the flow domain the incompressible Navier-Stokes equations describe the Newtonian fluid. A continuous finite element mesh is applied to the entire spatial domain, and the discretized model equations are assembled in a single set of algebraic equations, considering the two-field problem as a whole. The continuous fluid-structure mesh with identical orders of approximation for both solid and fluid in space and time automatically yields conservation of mass, momentum and energy at the fluid-structure interface. A mesh-moving scheme is used to adapt the nodal coordinates of the fluid space-time finite element mesh to the structural deformation.

A monolithic approach to fluid-structure interactions is chosen in order to develop an accurate and efficient solution procedure for coupled systems exhibiting strong interactions. The governing equations for both solid and fluid are formulated in velocity variables and discretized with the time-discontinuous space-time finite element method (1). A continuous finite element mesh is applied to the entire spatial domain, and the discretized model equations are assembled altogether in a single set of algebraic equations, considering the multi-field problem as a whole. The space-time finite element method provides a consistent discretization of both space and time, avoiding semi-discrete formulations. Since the future cannot influence the past, the space-time domain Q is subdivided into a sequence of space-time slabs Q_n which are solved successively.

The continuous fluid-structure mesh with identical orders of approximation for both solid and fluid in space and time automatically yields conservation of mass, momentum and energy at the fluid-structure interface. For large structural displacements new coordinates have to be defined for the mesh nodes inside the fluid domain. In order to avoid overlapping nodes and to reduce distortions of extremely stretched elements a pseudo elastic continuum with constitutive equation of neo-Hookean material is used as a mesh-moving scheme.

The highly non-linear system of discretized model equations for solid, fluid and fluid mesh dynamics has to be solved iteratively. To enable a computation in acceptable time steps, the Newton-Raphson method is used.

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Tools for modeling fluid-structure interaction via Lagrangian FEM

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ABSTRACT

An increasing development of numerical methods can be observed in the field of fluid-structure interaction in last decades. These methods are applied to various types of engineering problems involving hydrodynamics of floating objects, structures subjected to water flow, free-surface flows or casting processes.

Typical approach in modeling fluid flow consists in the use of Eulerian description of motion. The computational mesh is fixed and the material points flow moves with the respect to the grid. In Lagrangian description each node of the computational mesh is associated with the material particle during the motion.

The Particle finite element method – PFEM (Oñate et al. 2004) belongs to the class of methods based on Lagrangian formulation. The main advantage of Lagrangian approach consists in absence of convective terms causing numerical difficulties in other formulations and requiring stabilization. On the other hand it is necessary to pay attention to movements of nodes. Through their large displacements it is essential either to renew the finite element mesh or build a new one in each solution step in order to avoid numerical instability as consequence of large distortions.

Many algorithms for mesh generation are based on Delaunay triangulation. It consists in such triangulation of a general set of points, that no point lies inside circumscribed circle of any triangle except three points forming triangle.

There is a lot of algorithms constructing Delaunay triangulation. One of the most used was introduced by A. Bowyer (Bowyer 1981) and D. F. Watson (Watson 1981). It is an incremental algorithm based on sequential adding of points into the triangulation. The points are inserted one by one into the current triangulation. Triangles are in each step checked, if their circumscribed circles enclose the new added point. All triangles violating the Delaunay property are removed and the vacant space is retriangulated.

The particle finite element method presented above is intended to extend a finite element package OOFEM (Patzak & Bittnar 2001) developed at Czech Technical University in Prague, in order to enable FSI-analysis.

A pilot C++ code using Bowyer/Watson algorithm was programmed for testing purposes. Application of the code on several examples has proven its functionality and reliability. On the other hand, a high time demand for large problems was observed. It was caused by the search through the whole database of the existing elements.

The algorithm has been currently implemented into the OOFEM package in order to profit from the usage of all its existing tools. To speed up the spatial search for elements violating Delaunay property during the new point insertion an octree-based search has been implemented.

Octree is used for spatial sorting of temporary elements during the mesh build-up phase. Each octree cell contains elements, whose circumcirles are inside the cell or contain the cell. By the insertion of a new point into the triangulation, a terminal cell is determined from point coordinates and search for affected elements is performed only through cell data and not through the whole database.

Testing of the algorithm and the examination of its CPU time consumption compared to full data search is the objective of present research.

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Simulation of permeation tests on articular cartilage under different testing conditions

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ABSTRACT

Articular cartilage is composed of a charged solid matrix phase consisting of charged proteoglycan macromolecules and collagen fibers, an interstitial fluid phase, and an ion phase. The tissue is characterized by a very low permeability which is responsible of the support and distribution of loads in diarthrodial joints and of the transport of nutrients to chondrocytes. The permeability of articular cartilage varies through the depth of the tissue, and, due to the compaction of the solid matrix under compressive loads, decreases for increasing deformation. The study of the permeability is very useful to understand the onset and progression of osteoarthritis and to regenerate cartilage by tissue engineering. Permeation studies in vitro are conceptually easy but practically difficult to be performed. In particular, it is not trivial to guarantee the sample seal while maintaining a uniform deformation.

Furthermore, the measured variables are fluid pressure and flow through the sample, whereas it is not possible to measure the local strain, which otherwise can be evaluated by coupling computational studies of articular cartilage behaviour during permeation test with experimental results. We therefore performed numerical simulations, based on the poroelasticity theory, of permeation tests varying the method used for positioning the sample into the test chamber (ideal, glued sample and o-ring on top). For each design we evaluated the local strain distribution which is essential to a precise calculation of the permeability from experimental tests. The results for each model showed a central area where pressure and strain displayed a linear distribution with depth, whereas close to the constraint (glue or o-ring) non-linear distributions are evidenced. Our method will be very useful for the design of a novel testing chamber to measure articular cartilage permeability.

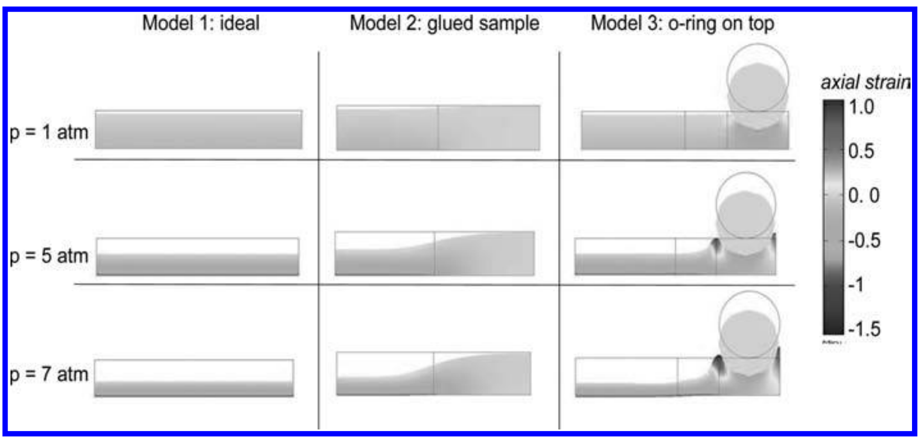

Figure 1. Strain distribution at equilibrium inside the models and deformed plot. The center for all of the models is on the left side (axial symmetry). Results are shown for fluid pressures imposed on top of 1, 5, and 7 atm. Flow direction is from top to bottom through the sample. Radial displacement is impeded and the lateral walls are impermeable. Other boundary conditions are shown in Fig. 2.

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A posterior error estimation in the isogeometrical analysis method

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ABSTRACT

A new approach for improvement of stresses and estimation of solution errors based on the isogeometrical analysis method is presented in this paper. This estimator falls in the category of the stress recovery error estimators. The main idea of the proposed method is inspired by the core idea of the isogeometrical analysis. That is, the components of the recovered stress field are considered as an imaginary (hyper-) surface. This surface is constructed by the same spline basis functions and NURBS shape functions that were employed for approximation of the components of the unknown displacement field vector. In two dimensional elasticity problems, similar to the isogeometrical analysis method [1,2] itself, the x and y coordinates of the control points, which are used for the analysis, are considered as before and their z coordinates are calculated based on a criteria which is constructed by using the well known notion of superconvergent points in finite elements [3].

The performance of the method is demonstrated by comparison of the exact and approximate energy norm errors for two examples that an exact solution is available for them. It seems that the suggested method can be used as a suitable approach for error estimation in the isogeometrical analysis method.

The first example is a cantilever beam subjected to a shear load at its free end. The contours of the exact and approximate energy norm errors are illustrated in Figure 1 and are compared in Figure 2.

The calculated effectivity index for this problem is 0.86.

Second example is an infinite plate with a hole subject to horizontal tensile tractions. A square portion of the infinite plate is taken into consideration and due to symmetry only a quarter of the problem is modeled. The exact and approximated error norms are depicted in Figure 3. The calculated effectivity index for this problem is 0.81.

Figure 1. Distribution of errors in cantilever beam.

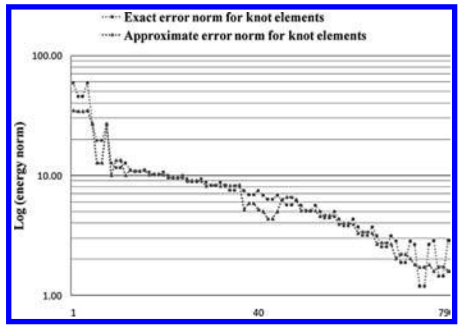

Figure 2. Comparison between the exact and approximate energy norm errors for knot elements of the cantilever beam.

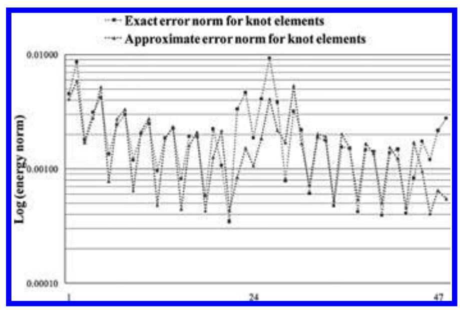

Figure 3. Comparison between the exact and approximate errors of knot elements for infinite plate with a hole.

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Numerical design of geothermal projects

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ABSTRACT: The application of geothermal energy as a part of renewable energies is a multidimensional process of fast growing improvements. For all geothermal projects the design of every single system component is most important to avoid thermal interactions and to optimize the total degree of efficiency. Therefore different analytical and numerical tools for the design of geothermal systems were developed. In this report information on geothermal energy application as well as actual improvements of their analytical and numerical design methods are given. Geothermal basic principles needed for application orientated improvements are shown.

1 INTRODUCTION

In times of global warming renewable, green energies are getting more and more important. The development of application of geothermal energy as a part of renewable energies in Germany is a multidimensional process of fast growing improvements.

2 GEOTHERMAL DESIGN

For every kind of geothermal project the modeling of the geothermal energy transport processes is most important for the design of each system component to avoid thermal interactions and to optimize the total degree of efficiency. Therefore different analytical and numerical modeling tools were developed.

Analytical modeling tools such as on g-functions based programs as well as numerical modeling tools based on the finite-element-method (FEM), the finite-difference-method (FDM) and the boundary-element-method (BEM) are introduced in the full paper.

3 CURRENT RESEARCH

For a proper numerical modeling of the energy transport processes of geothermal systems in the underground the subsoil has to be modeled as a three phases system, which contains solid, fluid and gaseous phases. Therefore the process of solid phase energy transport (conductivity), fluid phase energy transport (convection) and gaseous phase transport (radiation) as well as their interaction has to be considered separately.

German universities and state aided organizations are developing numerical programs for a detailed use of application on geothermal systems. The Institut für Werkstoffe und Mechanik im Bauwesen (IWMB) at the TU Darmstadt is developing a Finite-Element-multiphase model, verified by experimental investigation and laboratory tests. This research is supported by the Federal Ministry of Economics and Technology (BMWi).

In this multiphase model the subsoil will be analyzed as a three-phases-model with separated consideration of conduction, convection and radiation and their subsequent interfaces, considering the theory of porous media as described in the full paper.

4 CONCLUSIONS

With the described research project of the IWMB supported by the (BMWi) a multiphase model is developed analyzing the subsoil as a three-phases-model with separated consideration of conduction, convection and radiation and their subsequent interfaces, considering the theory of porous media.

The lessons learnt in Germany could be adapted for international purposes. In that way an efficient environmental friendly green geothermal energy production could be performed all over the world, not even in high-enthalpie regions. With that possible high number of decentralized geothermal power plants as well as shallow geothermal projects the world energy demand could be satisfied to a considerable extent by geothermal energy.

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Numerical modeling of the behavior of multiple-cable-pylon cap joint in a cable roof structure

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ABSTRACT

This paper presents a detailed finite element modeling of a multiple-cable-to-pylon cap joint of a real project. Figure 1 shows the layout of the roof, supporting structure of which is of our primary concern of this study.

For the set of internal forces obtained from modeling of the structure as a beam-cable system, the substructure of the upper part of the pylon is selected for the detailed analysis of localized behavior of the pylon cap joint. The joint accommodates guiding CHS elements for the roof cables anchored in the pylon and the pylon upper CHS spire element provided for a safe load transfer from the roof cables onto the compound pylon composed of three chords and battens of tubular sections.

Finite element modeling technique is presented that allows for prediction of the behavior of pylon cap joint. All the steel tubular components of the analyzed pylon substructure are modeled with use of thin shell four node and three node elements.

Three different joint solutions are analyzed:

1. The original solution with the short conical shell.
2. The modified solution with the original short conical shell reinforced from outside by a longer conical shell of 16 mm in thickness welded to the spire tube and the joint bottom plate.

Figure 1. Roof and pylons of amphitheatre structure considered.

Table 1. Summary of numerical model.

Solution	No. of nodes overall	No. of elements	
		4 node	3 node
1*	36732	37007	84
2*	39967	40075	91
3*	41089	41348	92

* As identified above the table.

3. The modified solution 2 with an additional reinforcing cover shell of 20 mm in thickness welded to the longer conical shell.

Summary of numerical model features applied for analyzed joint solutions are given in Table 1.

Three different methods of analysis are employed and numerical simulations supported by commercial software ABAQUS. In order to find the sensitivity of analyzed subsystems to buckling effects and describe a possible profile of initial imperfections, the linear buckling analysis (LBA) is performed. The first buckling modes are considered for the description of imperfect geometry in the pylon unloaded state. Load-displacement characteristics are evaluated using the Riks type analysis in two versions: geometrically and materially nonlinear analysis – GMNA (perfect model) and GMNIA – geometrically and materially nonlinear analysis with imperfections (imperfect model).

The original joint solution was developed without the detailed finite element analysis. FE prediction of the behavior of this joint has proven the joint being overstressed for which the failure load factor was below unity (failure occurs under the design load level).

The insufficient strength of the pylon cap joint was the reason for the improvement proposals of joint detailing. Reinforcements were provided to the original solution in terms of conical shells, longer in the solution 2 and in the solution 3 – additional shorter and welded to it. Reinforced joint solutions give the ultimate load above unity, therefore providing acceptable reliability from the view of the ultimate limit state.

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A "simple beam" revisited

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ABSTRACT

Kamel & McCabe stated around 1980: "The finite element method is an approximate method based on discretization of both geometry, loading and boundary conditions, as well as the use of elements derived using various assumptions of which the user is often not quite aware." At about the same time Bathe prophesied that "within a period of ten years the powerful tool of finite element analysis would be available on every analysis engineer's desk, but not enough people would be trained sufficiently by then to correctly and safely apply this method."

This paper is a follow-up of two previous papers, using additional boundary conditions, but using only quadrilateral elements, with 4, 8 and 9 nodes.

Since 1986, the writer has presented a one-week course "The Application of the Finite Element Method in Practice" about every two months. In almost every course some of the participants, when modelling a simply supported deep beam with shell elements, make the same mistake: They add unnecessary restrained freedoms (translation DZ as well as the rotations RX, RY and RZ) to all node points of the structure. This causes the maximum stress results in the horizontal direction of shell elements to be 24% lower than in a structure without extra supports.

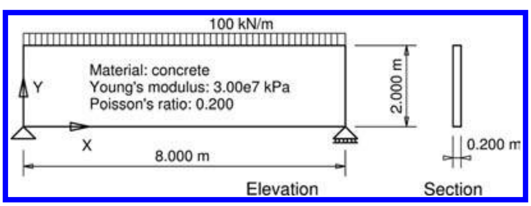

Figure 1. Test beam.

Users of finite element analysis should be warned to avoid these pitfalls.

This investigation covers four different mesh types and five different restraint arrangements.

The results of 74 finite element analyses are summarised in a table.

Four-noded shells (Kirchhoff elements) in Strand7 should never have any default restraints. Such unnecessary restraints can increase the stress error by 24 %. This does not apply to 8- and 9-noded shells (in Strand7: Reisner-Mindlin elements), but unnecessary restraints should in principle never be used - it can only cause confusion and lead to errors.

It would be interesting to find out why, in two analyses with no default restrained freedoms, RZ rotations do occur. These two analyses have sixteen 4-noded elements each. This phenomenon cannot be found in the analyses with four 4-noded elements each.

It would further be of interest to all finite element users to carry out these 74 test runs on the software of their choice. The author of this paper would be very pleased to receive the results of such analyses.

The findings of this paper will be useful to all users of finite element analysis: students as well as practicing engineers.

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Finite element analysis modeling of full scale 5-ton electric overhead travelling crane and the crane supporting structure

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ABSTRACT: The current codes of practice for the design of structures do not explicitly account for the flexibilities and interactions of the Electric Overhead Travelling Crane (EOHTC) and the crane supporting structure. This leads to analyzing member forces in the EOHTC and the crane supporting structure separately for ease of computation. Thus, the interaction of the various components of the EOHTC and crane supporting structure are ignored, which results in an incorrect assessment of the member forces.

This paper describes how a computationally efficient Finite Element Analysis (FEA) model of a full scale 5-ton experimental model was developed to predict the global responses of EOHTC and crane supporting structure when subjected to various loading conditions. The FEA model was developed through many simplifications taking cognisance of the limitations of the simplifications. Finally, a comparison between the experimental and FEA end buffer impact forces are provided and discussed.

Keywords: finite element modeling, crane supporting structures, electric overhead travelling cranes, end buffer impact forces

1 INTRODUCTION

Since the design codes of practice do not account for the interactions between the EOHTC and its supporting structure, a full scale experimental model was constructed to investigate the above as a coupled system. During the experimental investigation, a number of parameters were discovered, which has a significant effect on the end buffer impact force history. These parameters were difficult to control in the experimental model. This lead to the development of a FEA model where these parameters could easily be adjusted and controlled. To obtain a computationally efficient model, many simplifications were required. The paper describes how these simplifications were developed and implemented, taking cognisance of the limitations of the simplifications.

2 RESULTS

After the simplifications were implemented the FEA model was calibrated with the experimental end buffer impact force histories for the case of the EOHTC without payload. Figure 1 shows the superimposed experimental and FEA impact force histories. The difference in magnitude between the peak impact forces is less than 2%; while the time difference of the impact peaks varies by less than 3%. This implies that the FEA simulation provides a good correlation to the experimental impact force history. This FEA model was used in further research work which included the payload, due to its good correlation with the experimental model.

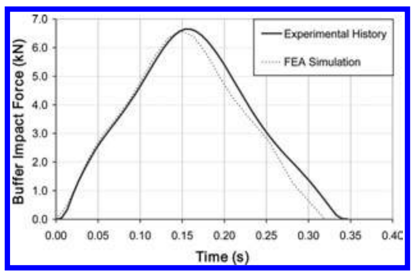

Figure 1. Comparison of the FEA and Experimental end buffer impact force histories without payload.

3 CONCLUSION

These simplifications can be incorporated in any commercial FEA program by practicing engineering practitioners to obtain a detailed analysis of crane supporting structures. Such a detailed FEA model will also eliminate the need to develop costly experimental models in future.

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Numerical models of lightweight roof structures

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ABSTRACT

The paper presents several new concepts of the lightweight structural systems designed for large span roof covers like e.g. crystal tension-strut structure, VA(TH)No2 tension-strut structure, JR Tetra System or Lenticular Girder. All these systems were recently developed by the author and most of them were proposed as the main support structures for various types of roof covers starting from flat covers, through barrel vaults till dome covers shaped also as geodesic domes. For all these structures are prepared numerical models defined in the programming language Formian. Numerical models defined in the programming language Formian can be relatively easy modified during the design procedure by means of parameters optionally chosen by the programmer (Nooshin & Disney & Yamamoto. 1993). These models are the base for numerous and comprehensive analyses of systems proposed for the lightweight roof covers. All the proposed systems of the tension-strut structures are worked out by the author on the basis of the transformations of chosen types of simple forms of structural configurations. One of these developed systems is the VA(TH)No2 structure. Structural concept of this system is that the single layer grid of the compression bars is supplemented by means of oppositely directed tetrahedron bar modules, arranged respectively onto inverse sides of the grid. The top vertices of these modules are suitably connected by means of tension members used for the pre-tensioning of the system and the whole structure is fastened in the perimeter ring like it is presented in Figure 1. (Rębielak. 2005). Because of complexity of this geodesic form of that structure and also of the procedure of preparation of numerical model the defining process of it has to be divided into several separated stages.

Numerical models of all the presented structural systems are defined in the programming language Formian. These models can be determined by means of appropriately selected sets of parameters, due to which they can be easy modified what is very important for initial studies and later for the further various and comprehensive analyses. Application of Formian accelerates the entire procedure of design of sometimes very complex spatial structures of various forms of roof structures and makes easier co-operation between all participants of the investment process, in particular between architects and engineers.

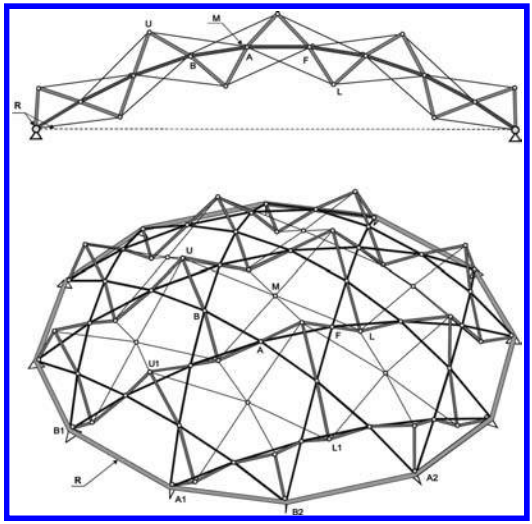

Figure 1. Basic schemes of VA(TH)No2 tension-strut structure.

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5. Damage mechanics, fracture, fatigue, blast, impact, damage modelling

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A thermodynamic approach to nonlocal strain damage

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ABSTRACT

The realistic modeling of inelastic behaviour of ductile or quasi-brittle materials is essential for the solution of numerous boundary-value problems occurring in various engineering fields.

Therefore, a main issue in engineering applications is to provide realistic information on strain, stress and damage distributions within elements of such materials.

In order to describe the gradual internal deterioration of solids, within the general framework of continuum thermodynamics of irreversible processes, several continuum damage models have been proposed which are either phenomenologically based or micromechanically motivated.

Nonlocal regularization methods introduce the characteristic length as an additional material parameter which describes the nonlocal micro-interactions produced in media suffering micro-decohesion or damage processes.

Nonlocal theories are defined either in a strong form (integral-type), see e.g. Borino et al. (2003), Jirásek & Rolshoven (2003) or in a weak form (spatial higher gradients), i.e. Polizzotto (2007).

In this contribution, attention is focused on a geometrically linear model of nonlocal damage of integral type and a loading function, controlling the evolution of damage, is introduced.

According to the damage theory in the strain space, it is assumed that the arguments of the loading function include the strain (Jirásek & Patzák 2002). An important advantage of models with strain-based loading functions and explicit damage evolution laws is that the stress corresponding to a given strain can be evaluated directly, without any need for solving a nonlinear system of equations. As a consequence the numerical implementation of the nonlocal strain damage model is relatively straightforward. Of course, equilibrium iteration on the structural level cannot be avoided, same as for any other nonlinear model (see e.g. Pijaudier-Cabot & Bažant 1987, Lemaitre & Chaboche 1994).

Moreover it is formulated a nonlocal model of strain damage in which the stress decomposition consistently follows from the thermodynamic analysis based on a first principle written in a global form and on a second principle written in a pointwise form (Edelen & Laws 1971, Polizzotto 2007). Hence it is not necessary to introduce a damage tensor as an additional internal variable of the model.

A variational formulation for the finite-step elastic model with nonlocal damage is then provided on the basis of conjugate functions (Rockafellar 1970).

The resolution of the finite-step elastic model with nonlocal damage can be properly assessed by neans of a FE algorithm which is not based on ad hoc extensions of procedures pertaining to local damage.

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Fracture mechanical based assessment of partial penetration welds in T- and cross joints

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ABSTRACT

Design and welding codes require full penetration in case of complete butt splices. Anyway, manufacturing defects, metallurgical inhomogeneities or welding imperfections are discontinuities which may occur in welded connections. HY-or DHY-welds can exhibit incomplete penetrations which transversal dimension is referred to root gap (Fig. 1) in the following. These root gaps often end up to time- and cost-intensive repair work for the executing steel company whenever the weldments are produced with partial penetration only.

A classification method for the assessment of such partial penetrations, also to be considered right from the weld design or planning stage, is actually missing. Recently a fitness-for-purpose-strategy has been developed within a German-research project based on an accurate detection of root gaps by non-destructive testing methods (NDT) and based on fracture mechanical techniques in order to guarantee the required structural reliability of a welded steel member despite of leaving a weld that actually is to be rejected due to insufficient penetration according to the conventional codes.

Therefore, 12 large-scale specimens of steel type S355J2 with cross joints and existing root gaps have been tested in tensile and 4-point-bending tests. The root gaps were transformed into sharp cracks at first due to cyclic loading in order to meet the fracture mechanics requirements. Subsequently, the specimens were subjected to quasi-static tensile loading at very low temperatures about −120° to −130°C in order to force brittle fracture. Moreover, extensive parametric boundary-element-analysis (BEA) was carried out in order to verify and extend the experimental results to a range of typical cross joints with various geometric dimensions.

It was shown, that the fundamental techniques of the brittle fracture avoidance concept which is the basis for the choice of steel in EN 1993-1-10 in due consideration of an adequate NDT-technique was expanded successfully to partial penetrated welds. From now on sufficient bearing capacities can be verified for partial penetrated HY- and DHY-welds in cross- and T-joints as for full penetration.

Tolerable root gap sizes are shown in extracts in Fig.2 for static and cyclic loading for cross joints with DHY-seams subjected to tensile loading.

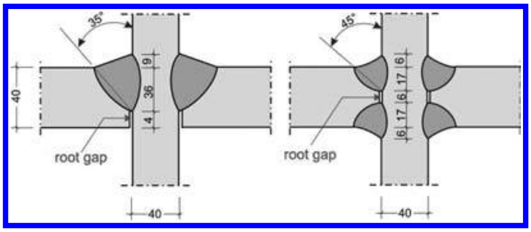

Figure 1. Partial penetration welds (root gap) for single and double-bevel butt joints with broad root face.

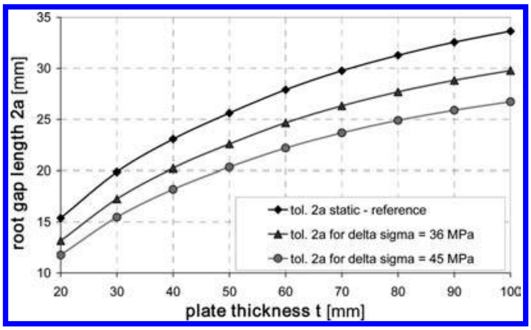

Figure 2. Tolerable root gaps subjected to fatigue loading for DHY-seams in cross joints.

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Fracture criterion for all concretes: Normal, lightweight, high- and ultra-high-performance concrete

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ABSTRACT

Multiaxial states of stress occur in nearly all areas of reinforced concrete structures, even if they are rarely taken into account during measurement. Multi-axial strength of high performance concrete (HPC) with and without fibres, high performance lightweight concrete (HPLC) and ultra-high performance concrete (UHPC) were investigated at TU Dresden in the recent years.

Independent of the concrete type the same failure modes occurs. Depending on the state of stress, four fracture forms are to be observed for concrete: compressive, splitting, shear or scrunching fracture (Figure 1). Among the various fracture mechanism, modifications of the concrete mixture affect the strength in different ways. For fibre free concrete, an increase of strength leads to the reduction of the strength increase in per cent compared to the uniaxial compressive strength among all fracture forms. In contrast to that, a strength increasing effect of the fibres only occurs for those fracture forms at which the concrete fails under lateral stress, i.e. at compressive and splitting fractures.

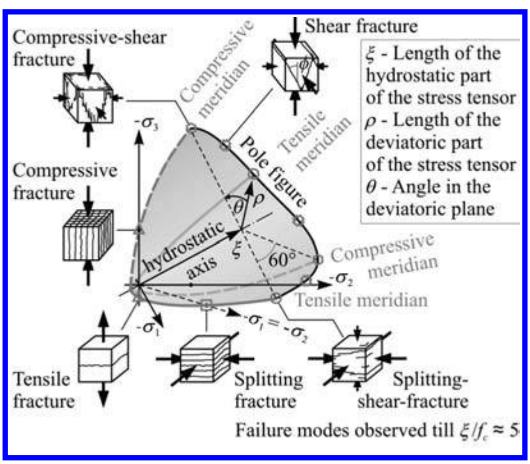

Figure 1. Fracture mechanisms, as shown in the range $\sigma_2 > \sigma_1$.

These constitutive relations were implemented in a fracture criterion based on criteria by Ottosen (see Equation 1 below), in use for instance in CEB-FIP Model Code 1990 (CEB 1991). The criterion is smooth, partly concave and, with the exception of the compressive meridian, differentiable. The criterion can be adjusted to the considerably more brittle behaviour of (U)HPC.

$$A\cdot\frac{J_{2\sigma}}{|f_c|^2}+\lambda\cdot\frac{\sqrt{J_{2\sigma}}}{|f_c|}+B\cdot\frac{I_{1\sigma}}{|f_c|}\le 1 \qquad (1)$$

where $I_{1\sigma}$ = linear invariant of the stress tensor and $J_{2\sigma}$ = quadratic invariant of the deviator stress tensor. The coefficient λ is defined as a function of the angle θ, which is defined in the deviation plane with 0° on the tensile meridian and 60° on the compressive meridian, and with the constants K_1 and K_2.

The constants A, B, K_1 and K_2 are calibrated on concrete parameters. Uniaxial tensile and compressive strength, biaxial compressive strength and strength under a suitable state of stress on compressive meridian were used to calibrate the criterion, because these values characterise the behaviour of the concrete under the various fracture mechanisms in such a profound way that the structural behaviour for the whole of the multi-axial stress area can be deduced from them. Calibration values were calculated from more than 3000 test by one's own and by other researchers for concrete with characteristic strength till 120 N/mm^2 and for lightweight concrete till 80 N/mm^2, respectively. Values for concrete with fibres and UHPC have to be determined by test, because no adequate database exists.

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Fracture mechanics test specimen design for cementitious composites

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ABSTRACT

Fracture mechanics properties of cementitious composites like concrete or fibre reinforced concrete are subject of investigation of various research institutions for years. Experiments are being held on beam shaped specimens with some sort of notch modelling the initial crack, mostly this notch is sawn into the specimen. The process of sawing deals some amount of damage to the surrounding material, resulting in material behaviour different from theoretical premises given by the fracture mechanics theory and also requires specialised heavy machinery and trained personnel.

With the macroscopic granular structure of concrete, another problem arises: to comply with the characteristic material length of concrete being in the proximity of 100 mm, test specimens fulfilling this size criterion are quite large, heavy and difficult to manipulate in laboratory conditions.

New type of beam test specimen was developed, trying to submit to these conditions: perform in accordance with fracture mechanics theory, be small and light enough for manipulation, to be easy to prepare and to be affordable in terms of machinery and personnel required for production.

The principle of the test specimen preparation is to completely skip the sawing phase of the manufacturing process and to introduce the initial hairline crack into the specimen while being cast.

The specimens introduced have dimensions of $100 \times 100 \times 400$ mm with a 20 mm notch in one side of the prism. Material of all specimens tested was concrete grade C30/37 (according to EN 206-1). The testing procedure involved subjecting the specimens to three-point bending with displacement-controlled load cycle and measuring loading force, vertical displacement at midpoint and crack opening, using inductive displacement sensors. After tensile tests all resulting halves of cracked specimens were subjected to compressive strength measurement, so that was made sure the material properties of all specimens were in accordance.

There were three groups of specimens used in the testing procedure, consisting of four specimens each. First group was used as reference for testing tensile strength and Young modulus of the material and no notch was formed in these specimens.

Second group had a 20 mm deep and 5 mm wide notch sawn to a side of the specimen. The orientation of the specimen during testing was rotated 90°, so that the notch was facing downwards and the rough surface of cast concrete was on the side and not interfering with testing equipment. Tensile strength of notched specimen was measured together with crack opening during loading.

Third group had 20 mm deep notch formed during casting by presenting a thin plastic foil into the casting form. The foil was not removed from the specimen until the testing was finished. The orientation, instrumentation and testing procedures on the specimens from third group were identical to those from second group.

Resulting calculated fracture toughness of the specimens from third group was about 20% lower than of those from second group, while the notch behaviour of the specimens of second group was more resembling a reduced-profile beam than a sharp notched beam. Thus, the proposed test specimen geometry is not only cheaper to manufacture but also provides more relevant fracture properties data.

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Perturbation problems for interfacial cracks

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ABSTRACT

We consider the perturbation problem of a Mode III interfacial crack. The perturbation is of geometrical type and can be both perturbation of the crack faces and perturbation of the interface, which can deviate from the initial straight line configuration. Asymptotic formulae are derived for the first-order perturbation of the stress intensity factor. It is shown that, due to the unsymmetrical nature of the problem, the Mode III skew-symmetric weight function derived in Piccolroaz et al. (2009) is essential for the derivation of the correct asymptotic formulae.

The notion of weight functions for cracks, as stress-intensity factors associated with a point force load, was introduced in Bueckner (1985). Another approach, viewing weight functions as special singular solutions of homogeneous problems for cracks, was used in Willis and Movchan (1995). Further studies, involving both symmetric and skew-symmetric weight functions, are included in Bercial-Velez et al. (2005) and Piccolroaz et al. (2007). It has been shown by Piccolroaz et al. (2009) that the lack of symmetry inherent in the interfacial crack problem induces the appearance of a skew-symmetric component of the weight functions, so that, for instance, the skew-symmetric loads generate non-zero stress intensity factors even in the case of two dimensions (plane strain, plane stress or anti-plane shear). We note that the problem of finding the variation of the stress intensity factors induced by a small geometrical perturbation of a plane crack placed at the interface between two dissimilar elastic materials requires the use of not only the symmetric weight functions but also of the skew-symmetric components.

We make use of the integral formula derived by Piccolroaz et al. (2009) to compute both the leading term and the first order variation of the stress intensity factor. The integral formula reads

$$K_{III} = \frac{\mu_+\mu_-}{\sqrt{2\pi}(\mu_+ + \mu_-)} \int_0^\infty [u]^{(+)}(x_1)\, x_1^{-3/2}\, dx_1 - \sqrt{\frac{2}{\pi}} \int_{-\infty}^0 \left\{ \langle\sigma\rangle^{(-)}(x_1) + \frac{\eta}{2}[\sigma]^{(-)}(x_1) \right\} (-x_1)^{-1/2}\, dx_1$$

where $\langle\sigma\rangle^{(-)}$, $[\sigma]^{(-)}$ are average and jump of the prescribed loading on the crack faces, whereas $[u]^{(+)}$ is the prescribed discontinuity of displacement along the interface, $\eta = (\mu_- - \mu_+)/(\mu_+ + \eta_-)$ is the contrast parameter. Note that the kernels $(-x_1)^{-1/2}$ and $x_1^{-3/2}$ play the role of the symmetric weight functions, whereas the kernel $\eta/2(-x_1)^{-1/2}$ plays the role of the skew-symmetric weight function.

In this paper, we show the use of the symmetric and skew-symmetric weight functions in evaluation of the stress intensity factors for regularly perturbed interfacial cracks in problems of anti-plane shear. The influence of asymmetry of the applied load as well as geometrical perturbations is illustrated by the numerical simulations based on the explicit asymptotic formulae.

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Fatigue performance of pre-stressed and reinforced concrete

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ABSTRACT

High stress ranges in structures like bridges or crane-ways can result into accelerated crack propagation, higher deflections, structural stiffness reduction and consequently into fatigue failure. The introduced mathematical tool for describing the strain development, i.e. the increase of deflections, in a concrete specimen subjected to cyclic loading – the fatigue damage function - is verified on both pre-stressed and reinforced concrete specimens.

The developed function gives a reduced value of modulus of elasticity in every particular moment of cyclic loading. The reduced value of modulus of elasticity is then used for calculating deflections increased by damage accumulation caused by cyclic loading.

The instant value of modulus of elasticity after n_i load cycles is:

$$E_{n_i} = \omega_{F_i} \cdot E_{n_o} \tag{1}$$

where E_{ni} modulus of elasticity of concrete after n_i load cycles; ω_{Fi} = fatigue damage function after n_i load cycles; and E_{n0} = modulus of elasticity of concrete at the start of the cyclic loading.

The fatigue damage function is tested on pre-stressed slab specimen and on two reinforced concrete specimens. The comparison of measured and calculated deflections for reinforced concrete specimens can be seen in Figure 1, 2.

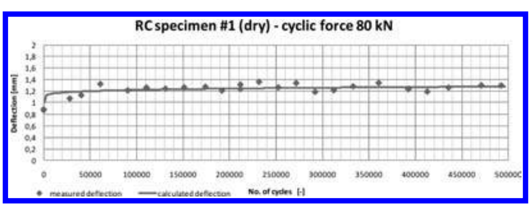

Figure 1. The increase of deflections of a dry reinforced concrete specimen subjected to cyclic loading. Comparison of calculated and measured deflections.

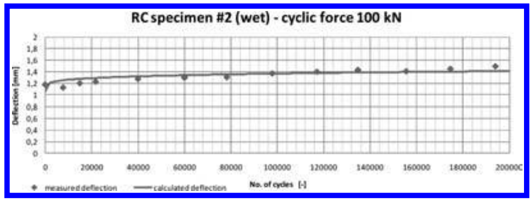

Figure 2. The increase of deflections of a wet concrete specimen subjected to cyclic loading. Comparison of calculated and measured deflections.

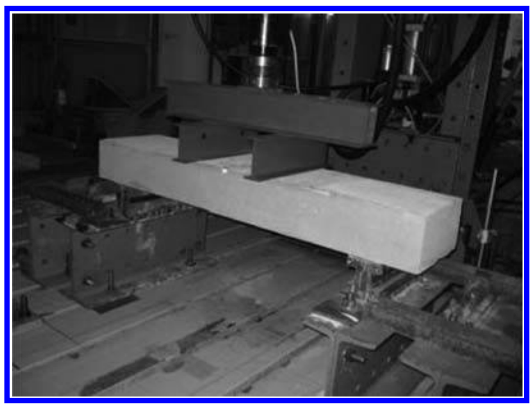

Figure 3. The arrangement of experiments with reinforced concrete specimens.

The arrangement of experiments with reinforced concrete specimens can be seen in Figure 3.

The comparison of measured and with the use of fatigue damage function calculated deflections shows satisfactory agreement.

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High frequency hammer peening: Enhancement of lifetime of fatigue loaded steel structures

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ABSTRACT

The application of High Frequency Hammer Peening Methods, like HiFIT or UIT may increase the fatigue strength of welded structures significantly. In contrast to ordinary hammer peening methods with frequencies of 10 to 80 Hz metal pins of the new peening methods cold work the weld toe with a frequency of 200 Hz and more. Due to the pin impacts the weld toe is deformed plastically to a uniform weld toe geometry. The surface is hardened and high compressive residual stresses are produced up to a depth of 1 mm.

Analyses regarding the early crack initiation and crack propagation using beach marking and thermal imaging show that the crack initiation phase of the cracks leading to failure is increased and the propagation rate up a crack depth of 1 to 1.5 mm is decreased.

Investigations have been carried out regarding the beneficial effects of the high frequency peening methods on the fatigue strength of fatigue preloaded structures. Specimens have been preloaded at different load levels up to their calculated lifetime based on the experimentally derived fatigue strength. The results prove that the fatigue life after the HiFIT/UIT-treatment of specimens with no detectable crack corresponds to the fatigue life of virgin treated ones, so that the fatigue strength is enhanced strongly. Even the fatigue life of treated specimens with cracks up to a depth of 0.5 mm show an increased fatigue life compared to the fatigue life of as welded specimens with no initial crack. The studies confirm that the increase of the fatigue strength is caused by compressive residual stresses in the surface layer which lead to a retarded crack propagation. In case where no with common NDT-methods detectable fatigue cracks are present the fatigue life of existing structures can be as highly increased as for new structures.

In order to establish these methods for practical use quality control systems have to be developed which ensure a continuous efficiency of the treatment. Investigations prove that the constant treatment parameters result in constant treatment quality. Further, the treatment process should be continuously controlled as it is the case for the HiFIT-device.

Above that the effects of the treatment regarding the compressive residual stresses and the surface hardening should be controlled by work sampling which allow hardness and residual stress measurements.

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Steel frame structures under blast loading

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ABSTRACT

Explosions in buildings due to terrorist attacks have raised major concerns in many countries and governments. Blast mitigation measures have been proposed, which are being incorporated in new design standards and existing buildings are being retrofitted.

The response of a steel frame building under blast load is studied by means of finite element computer models. Attention is paid to the various aspects of the problem: blast load; load transfer and failure of the facade and structural elements; material behavior and failure. The failure models are based on the experimental knowledge existing at TNO. The results of this investigation will provide more insight into the main issues and the knowledge gaps regarding steel frame structures against blast; and finally learn how to build them more efficiently.

A simplified multi-storey steel building has been considered, with typical values of spans and heights. The building façade is made of glass and a reinforced concrete parapet; the floor and roof are made of prefab cellular concrete and the beams and columns are made of structural steel. Standard materials are used, which are modelled by means of suitable material models with failure criteria. Three blast scenarios are considered: 5 kg(TNT)-1 m; 100 kg(TNT)-10 m and 1000 kg(TNT)-25 m. Simulations have been performed with LS-DYNA. The blast load is modelled using ConWep, which is based on TM 5-855-1. After detonation, the shock wave reaches the façade, reflecting back (faced-on pressure). Then it surrounds the building, with the side-on pressure acting on the sides and roof.

Figure 1 shows the final state of the building after 100 kg-10 m TNT explosion. All the glass has been blown away and there is damage and failure to the concrete parapets of the bottom floors. Plasticity is also observed in some of the most exposed steel column and beam elements. Obviously, the amount of damage is proportional to the blast intensity.

The tougher the façade, the more energy is absorbed by the structural elements. As a consequence, the damage increases. Hence, although a tough façade will reduce the number of direct human casualties due to fragment impact, it has a negative on the structure. This consideration should be taken into account when designing a building against a blast load.

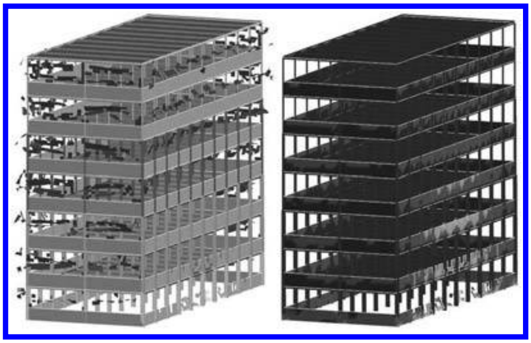

Figure 1. Damage caused by a 100kg-10m explosion. (left) glass and concrete façade; (right) damage to concrete façade.

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Performance of reinforced concrete columns under the vehicular impacts

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ABSTRACT

Frontal columns in buildings and columns in car parks are vulnerable to vehicular impacts. This paper treats the impact response of such concrete columns under vehicular loads and the use of polymer wrap to enhance their impact capacity. Comprehensive dynamic computer simulation techniques are used along with strain rate effects and hour glass control to evaluate the impact response. Results indicate the effectiveness of wraps in enhancing the impact capacity of these columns.

A comprehensive finite element model has been developed to assess the performance of ground floor columns of five storey building and effectiveness of a retrofitting method to mitigate adverse effects under vehicle impacts. The model was validated using existing experimental results in literature.

In order to strengthen existing vulnerable structures against impact loads; glass fiber reinforced composites (GFRP) were used. Fiber reinforced polymers has superior properties such as high strength, high energy absorption, light weight, low disturbance, easy and fast application.

The finite element model is validated by a horizontally positioned column impacted by a 1.14 t dropping weight (Louw et al. 1992). The time history of mid-span deflection curves obtained from the numerical simulation and the experiment were compared. The maximum deflection, residential deflection and the duration of the impact provided a reasonable similarity.

The validated finite element simulation was adapted to model ground floor column in a five story building to assess the vulnerability against vehicle impacts. The columns were designed according to Australian Standards (AS3600, 2004).

The research was also focused on strengthening of vulnerable concrete columns. Glass fiber based unidirectional sheets was used to enhance the impact capacity of the columns. Efficient material models for concrete and reinforcement with strain-rates under high dynamic loads were adopted in the simulation.

Column was fixed from both of its ends and was subjected to impact load directly at its front surface. An idealized triangular shaped time-history diagrams of impact loads were used in this study. The duration of the impact was kept constant as 100 msec. For different vehicle masses and impact velocities, the magnitude of the peak force can be increased during the analyses until the columns reach failure (ultimate stage). In the present study, the impact capacities of the reinforced concrete columns were thus obtained from the simulations. Existing and retrofitted columns failure forces were used to make comparisons between their impact capacities.

The results of the analyses approved the vulnerability of the axially loaded five story building columns that can be damaged by the medium velocity of car impact. The results approved that strengthening by GFRP at RC columns can significantly improve the performance of the column to withstand the transverse impact loads and to mitigate their adverse effects. Therefore this research is still in progress. Further information will be presented in the presentation.

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Spatial statistical analyses for time-dependent corroded surfaces of carbon steel plates in various corrosive environments

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ABSTRACT

Severe corrosion damage has been observed in the structural members of many steel bridges, with some plate girder bridges having failed completely at the ends of a girder. It is important to devise a method of predicting how corrosion develops over time to ensure the safety of such structures. However, no es-tablished method of predicting such behavior in var-ious corrosive environments is currently available.

Here, we describe a method for evaluating the time-dependence of corrosion at the surfaces of carbon steel plates in various corrosive environments. Atmospheric exposure tests were carried out on steel plates on the island of Okinawa in Japan, where the environments of each corroded surface were monitored using atmospheric corrosion monitor (ACM) sensors (Shinohara et al. 2005), and the resulting data were subjected to spatial statistical analysis.

Atmospheric exposure tests were conducted on unpainted specimens for periods of 0.5, 1, 2 and 3 years on Okinawa (Latitude 26°15′N, Longitude 127°46′E). The specimens were $400 \times 60 \times 9$-mm JIS G 3106 SM490A (Japanese Industrial Standard) structural steel plates, and the chemical composition is shown in Table 1. The specimens were mounted on racks at angles of 0°, 45° and 90° to the horizontal. This range of angles was not used to investigate the influence of angle on the corrosion behavior as such, but rather to obtain data on different corrosive environments. The skyward- and groundward-facing surfaces were monitored using Fe/Ag galvanic couple ACM-type sensors to evaluate the relationship between the mean corrosion depth and the atmospheric environment. The galvanic corrosion current from the sensors and temperature and humidity in the test site were recorded every 10 minutes throughout the tests. The mean corrosion depth of the specimens was measured using a laser and also calculated from the weight loss of the samples.

The spatial autocorrelation of each corroded surface was analyzed using a variogram (Cressie 1985) (Wackernagel 1998). In this study, the spherical model was used as a covariance function (Cressie 1985). Spatial regression analysis of the variograms was performed by applying a nonlinear least squares method to the covariance function. The spatial autocorrelation structures were estimated using the values of the range and the sill obtained from the analytical results. The equation for the weighted least squares standard was applied as a nonlinear least squares approximation (Cressie 1985).

Figure 1 shows simulated surfaces when q is 0.05 C/day and t is 50 years. The size and depth of the corrosion pits increase with time. q can be calculated from the output of ACM-type sensors (Kainuma et al. 2009) glued on the steel plate.

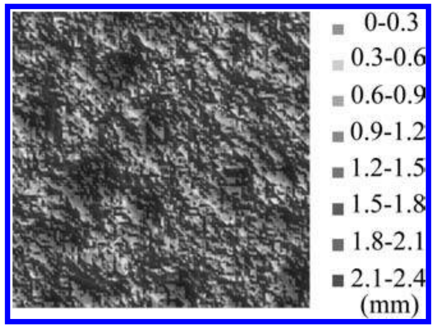

Figure 1. Simulated surfaces after exposure to a corrosive environment corresponding to of $q = 0.05$ C/day for 50 years.

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Prediction of whole process of concrete cracking under combined steel corrosion and service load

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ABSTRACT

As a global problem for reinforced concrete structures located in chloride and/or carbon dioxide laden environment, the reinforcing steel corrosion in concrete costs around $100 billion per annum world-wide for the maintenance and repairs primarily of premature concrete cracking and spalling. The continual demands for greater load for infrastructure only exacerbate the problem. This paper attempts to examine the whole process of cracking in concrete structures under the combined effect of reinforcing steel corrosion and applied load, from cracking initiation to surface crack width. In the paper, a model for residual stiffness of cracked concrete is derived using the concept of fracture energy as follows:

$$\alpha = \frac{\sigma/\varepsilon_\theta}{E_{ef}} = \frac{f_t\, exp(-\frac{f_t}{G_f} w)}{\varepsilon_\theta E_{ef}} \tag{1}$$

where σ is the post-crack (residual) stress, ε_θ is the residual tangential strain of cracked concrete, f_t is the tensile strength of concrete, G_f is the fracture energy and w is the crack width of cracked concrete. The stress-crack opening relationship is obtained from the laboratory tests on plain concrete cylinders prepared according to RILEM recommendations as shown in Figure 1.

Based on the stress analysis in concrete an analytical model is developed to predict the final crack width. The developed model is verified by numerical results as shown in Figure 2.

From the worked example it is found that the corrosion rate is the most important single factor that affects both the time to surface cracking and crack width growth. The paper concludes that the developed model is one of few models that can predict with reasonable accuracy the crack width on the surface of reinforced concrete structures under the combined effect of steel corrosion and service load. The developed model can be used as a tool to assess the serviceability of corrosion affected concrete infrastructure.

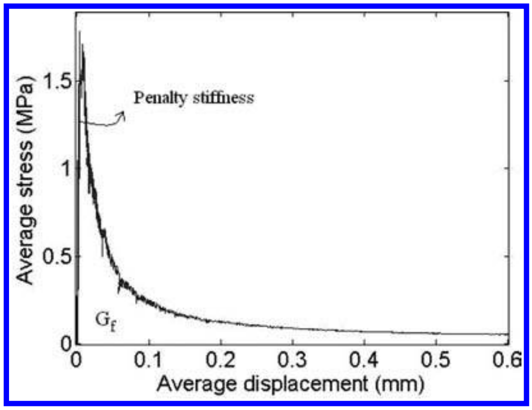

Figure 1. Stress-displacement curve for cohesive material.

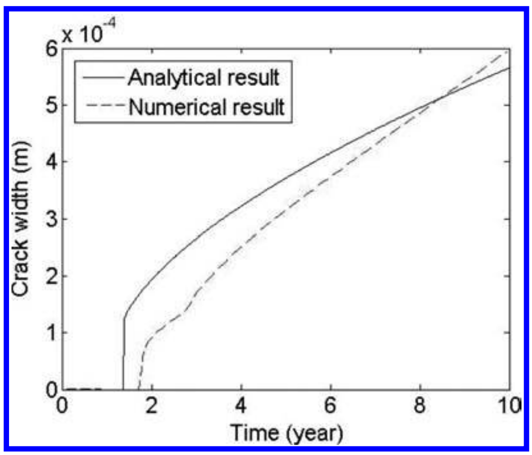

Figure 2. Comparison between analytical and numerical results.

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6. Loading on structures, structural analysis, testing, practical structural design

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Probabilistic basis of present codes of practice

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ABSTRACT

Recently revised national and international standards for structural design are systematically based on probabilistic concepts, mathematical statistics and on the theory of structural reliability. The International Standard ISO 2394:1998 and EN 1990:2002, based on materials of JCSS, play a key role to constitute a common basis for defining design rules relevant to the construction and use of the wide majority of buildings and civil engineering works (ISO 13822:2001, ISO 22111:2007).

The theory of structural reliability provides rational basis for specifying partial factors and other reliability elements. Figure 1 shows the variation of the partial factor γ for variable actions with reliability level β and design working life characterised by period ratio N (ratio of the design working life and the basic reference period). This graph can be used for assessment of partial factors for a given reliability level and design working life in case of new as well as existing structures.

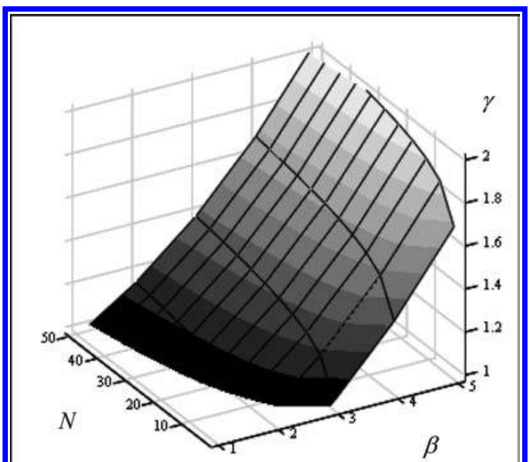

Figure 1. Variation of γ_Q with the reliability index β and period ratio N for the coefficients of variation $w_Q = 0.4$ assuming Gumbel distribution of Q and the time-sensitivity factor $\alpha_T = 0,6$ (wind action).

Similar results are available for other reliability elements used in partial factor methods. The theory of structural reliability becomes a powerful tool used for the development of new standards, and alternatively for the direct verification of new and existing structures (Holicky 2009, Retief and Wium, 2010).

Further research is however needed to specify the optimum reliability level taking into account cost of structures, relative costs of safety measures and possible consequences of structural failure.

ACKNOWLEDGEMENT

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Application of computational mechanics in design of tall buildings

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ABSTRACT

The increasing rate of urbanization has seen an accelerated trend in construction of tall buildings worldwide. Whether it is in the choice of the lateral load resisting structure (LLRS), or in the approach for integrating the structure in the overall form and architecture of the building, the decisions made by the structural engineer have a dramatic impact on the cost, amenity, constructability, and sustainability of tall buildings.

As multi-storey buildings get taller and more slender, their design becomes increasingly influenced by the factors such as the dynamic response of LLRS to wind loads both in the ultimate and serviceability limit states (ULS and SLS), the differential shortenings of vertical elements under gravity loads, and ULS and SLS performance of highly repetitive typical floors. Therefore, appropriate application of computational mechanics tools and methodologies plays a fundamental role in efficient design of tall buildings.

In this paper key issues encountered in the design of tall buildings are first discussed. Descriptions are then given on generic methodologies used for structural modeling and analysis of LLRS that respond to the need for analytical simplicity in the schematic design stage and the demand for increasing analytical refinement for the verification of final design.

Case studies in the application of computational mechanics tools in detailed design of tall buildings are then presented. They include assessment of the ductility of (load redistribution capacity between) the header beams of a 92 storey building under differential shortening effects (Fig. 1), and evaluation of the structural adequacy of a reinforced and post-tensioned concrete transfer wall in a 500 m tall building under the combined effects of gravity and wind loads (Fig. 2). In these case studies, nonlinear finite element analysis (FEA) methodologies are used to simulate the behavior reinforced concrete elements after cracking of concrete and yielding of steel reinforcement.

Finally, current trends in the application of computational fluid mechanics (CFD) in the design of tall buildings are discussed.

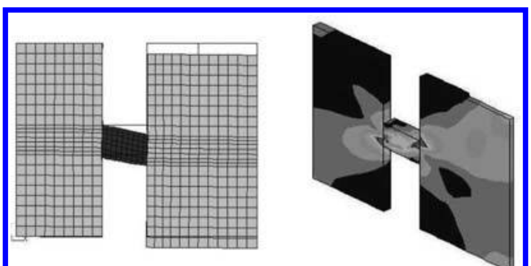

Figure 1. Nonlinear FEA of header beams under shear deformations due to differential shortenings in the building.

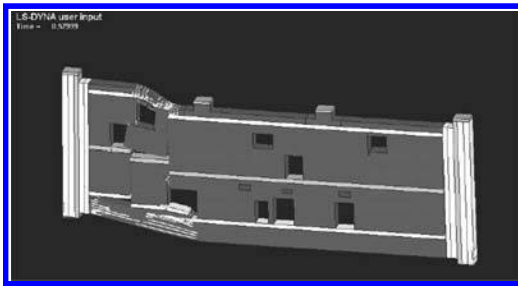

Figure 2. Collapse mechanism of a transfer wall under gravity and wind loads.

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Impact of engineering seismicity and cyclic load on a prefabricated planar structure of a multi-storey building

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ABSTRACT

Prefabricated planar systems are characterized by a deformation and failure mechanism under which planar elements shift in joints disintegrated by cracks, i.e. at so-called contact interfaces. Due to the fact that the deformations of elements as compared to the deformation of their joints are small, they can be solved for standard, mainly design loads, under the conditions of linear elasticity theory and planar stress state. We may, therefore, formulate an assumption that the limit state of the structure as a whole is preceded by joint disintegration, or that the structure tends to pass from linearly elastic behaviour to non-linearly elastic to plastic state by exceeding the proportional elastic limit in joints.

The results of experimental tests (Witzany 1982) manifested that the joints of concrete elements are extremely sensitive to the effects of variable repetitive and cyclic loading, the joint failure occurring due to cyclic loading amounting to 40–70% of the joint limit load. Time-related non-linear changes in joint properties, in relation to the loading history and plastification, may be of principal importance for the structural safety and serviceability of the bearing system in time.

In this context, residual structural safety of existing buildings exposed to the effects of technical seismicity must be evaluated with a special focus on the condition of joints and potential occurrence of cracks in the joints of load-bearing units.

Research oriented towards the effects of dynamic impacts caused by technical and induced seismicity involved several measurements of the dynamic response of multi-storey prefabricated and masonry buildings situated in the vicinity of a railway track, road and light rail traffic routes or close to tunnel boring works for the metro.

The measurements performed to-date have manifested that mainly the buildings in which the so-called safe distance in compliance with the national regulation had been observed do not require special assessment in relation to dynamic effects.

Research of resistance and residual structural safety performed within Research Plan "Reliability, optimization and durability of building materials and structures" included research on a model of a 7-storey prefabricated load-bearing wall system assembled in a 1:3 scale (Witzany et al. 2007).

The experimental and theoretical research completed to-date has manifested a relatively high resistance of prefabricated wall systems of multi-storey buildings with discrete arrangements of reinforcement securing the joints between individual load-bearing prefabricated units at the floor slab level (Witzany et al. 2009).

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A beam element for the analysis of framed structures with multiple discontinuities

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ABSTRACT

The analysis of beams in presence of different types of discontinuities is usually conducted, both in the static and the dynamic contexts, by means of classical procedures relying on integration of the governing differential equations between discontinuities. In this case additional integration constants, besides those depending on the external boundary conditions, are introduced at each discontinuity. According to the latter approach the computational effort depends on the number of discontinuities. Alternative and more efficient procedures, based on the theory of distributions and the application of the auxiliary beam method, although providing single response functions over the entire beam span, require the enforcement of a single continuity condition at each discontinuity. The computational effort is reduced, however, it is still dependent on the number of discontinuities.

An interesting approach relying on the so called transfer matrix method allows a treatment of different types of discontinuities providing a convenient recursive solutions in the sense that the solution at a generic cross-section at abscissa x is dependent on the response at the discontinuity immediately preceding the abscissa x. The latter approach leads to a sequential evaluation of the solution.

Recently, one of the authors, within the context of the theory of distributions, proposed an integration procedure to treat the Euler-Bernoulli and the Timoshenko beams with multiple discontinuities of different types. Explicit closed-form solutions, dependent on four integration constants only, were provided in the static field. The latter satisfy implicitly any continuity condition required by the discontinuity.

The above-mentioned closed-form solutions are exploited in this work to formulate a beam element to be adopted for a finite element discretisation of framed structures.

All different types of discontinuities are included in the proposed beam element and an arbitrary number of discontinuities can be accounted for without increasing the computational effort. The shape functions, for the static case, are formulated and the explicit expression for the stiffness matrix of the element is proposed. By assembling, with classical procedures, the stiffness matrices of the elements adopted for the discretisation, the global stiffness matrix of the structure is obtained without any increment of the degrees of freedom due to the presence of the discontinuities.

Dynamic analysis of framed structures is also conducted by means of the proposed finite element discretisation. In fact, the mass matrix consistent with proposed shape functions is evaluated.

Analyses of framed structures in presence of concentrated damages are presented and discussed. Furthermore, a sensitivity analysis of the response due to changes of the damage parameters is also conducted in order to investigate on possible solution procedures of inverse problems such as damage identification.

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Design of sport stadia: Wind action perspective

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ABSTRACT

This paper presents a review of the most relevant issues related to the structural and architectural design of large sport stadia, with a particular concern of wind loading aspects of these types of structures.

Due to the ever increasing competitiveness of sporting events, the designs of all modern sports facilities are governed by the provision of optimal conditions for the athletes/players. In most cases, this necessitates measures to reduce negative external environmental effects and to introduce enclosed or semi-enclosed roofs and façades of sport venues.

Traditionally, for large-span structures, the main drivers of formulating their shape and appearance were related to structural engineering limitations and the feasibility regarding the weight and costs of elements.

This paradigm has changed due to the recent development and use of modern lightweight materials and elements (e.g. cable roofs, fabric membranes) which are often integrated in a structurally innovative manner. An example of this is presented in Figure 1, which depicts the roof of the Port Elizabeth stadium, in South Africa, which combines intermittently, aluminum sheeting and fabric.

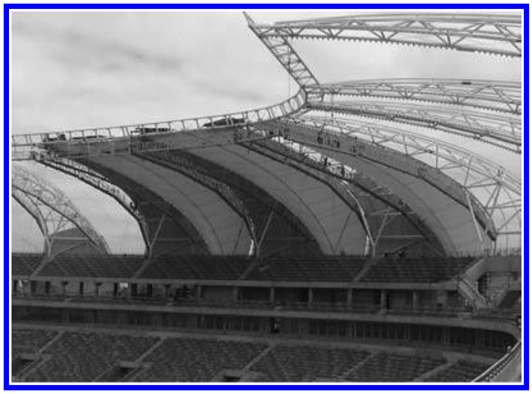

Figure 1. Construction of the Port Elizabeth stadium.

Furthermore, in view of some tragic crowd panic events which took place in recent years at several facilities across the world, this is followed by critically important issues related to the accessibility, fire safety and crowd control. Finally, functionality issues became important as most modern sport venues are designed as multi-purpose facilities, with some of them designed to be partially deconstructed in the future, with their elements to be re-erected at smaller satellite facilities.

All large sport-related structures are subject to climatic loads that can be grouped into the various types of influences, i.e. temperature, rain, wind and snow. (The latter is irrelevant to South Africa but is of great relevance in northern hemisphere.) A huge problem in predicting wind action is posed by the instantaneous application of the loading (due to the non-steady nature of flow), the dependence of the wind response of structure on the specific flow regime which is generated and also possible dynamic/fatigue effects.

The quantification of risks of wind induced failure is also difficult due to the seasonal characteristics of wind climate which could be profound like in many areas of South Africa. From a risk and structural reliability point of view, the robustness of the structure should be considered in terms of a joint probability of occurrence of an extreme wind event in combination with the extensive utilisation of the facility at the same instance of time (i.e. the occurrence of a specific large sport event attracting a large number of spectators).

Following the stipulations included in old loading codes, the current version of the South African loading code SANS 10160-1989 includes a stipulation on grand stand roofs. This information refers to isolated structures and is not applicable to grandstands encircling entire stadiums. This paper provides the background information regarding the importance of wind action and the relevance of wind-tunnel technology as a tool to address design challenges.

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Numerical method for live load distribution in road bridges

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ABSTRACT

In the design or assessment of bridges many numerical or empirical methods are employed. The availability of a wide range of computer programs for structural mechanics facilitates the exact bridge analysis.

In the paper, the old idea of transverse load distribution of wheel loads for bridge decks is solved by the grillage analogy. For the load distribution purposes even coarse grillage mesh is sufficient, but numerical idealisation should simulate the bridge deck by having its longitudinal beams coincident with centre lines of deck's girders. The same is for the transverse beams. The beams should be given the flexural and torsional properties of the girders. The grillage analysis is applied with the help of conventional computer programs, so all numerical mathematics is done by computers.

For the purpose of load distribution, the bridge girders are loaded at the critical cross section with a load P. Due to the torsional stiffness of the bridge members, the concentrated load P makes deformations of the cross section. The higher the torsional rigidity of bridge girders, the more uniform the girder deflections are. The nonuniformity of girder deflections is a qualitative indicator of transverse load distribution. The deflection of girder i due to load P on girder k is defined as f_{ik}. The deflection line of bridge cross section at midspan of simply supported deck under the load at the external girder is presented in Figure 1.

Using the old formula for transverse load distribution coefficient κ and adopting Maxwell's reciprocal theorem for deflection $f_{ik} = f_{ki}$, the transverse load distribution coefficient κ is described by the formula:

$$\kappa_{ik} = \frac{f_{ki}}{f_N} \tag{1}$$

where: κ_{ik} = distribution coefficient for girder i at point k; f_{ki} = deflection of girder k due to force at girder i; f_N = total deflection in cross section which is the sum of deflections of all girders.

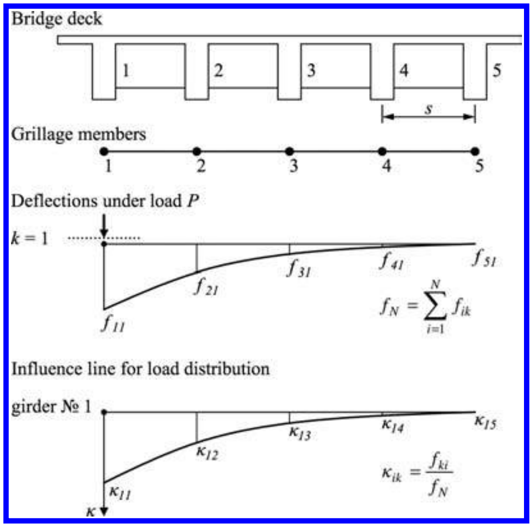

Figure 1. Five-girder bridge: cross section, grillage members, deflections due to force at external girder and load distribution graph for external girder (No 1).

The graphical presentation, preparing the influence line of transverse load distribution for external girder is presented in Figure 1. For a bridge with N girders, the solution requires N distribution coefficients for each girder computed according to the formula (1). All these coefficients are dimensionless and can be used in the same way as a usual influence line.

Using grillage approach for bridge decks analysis requires some experience and basic knowledge to model the behaviour of bridge systems. For checking the results, the Maxwell's theorem can be applied, and the sum of distribution coefficients κ_{ik} for girder i must be 1.000. This confirms that no errors have been made. The grillage approach is adopted, however, other numerical method can be introduced as well.

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The computational estimating of safety load factors and silo wall strength according to experimental and theoretical loads of pressure of bulk materials

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ABSTRACT

The authors of this paper who are members of the research team in the Institute of Building Engineering of the Technical University of Wrocław, for more than 30 years have been carrying out researches on silos for cement, limestone powder, wheat, rape, sand, ash fly, flour, coal, gypsum, etc. They had big influence on preparing polish silo codes. These investigations were carried out on silo batteries, free-standing silos and silo models. The researches include the evaluation of value of pressure and temperature field on the silo wall as well as temperature gradient, registration of simultaneity of these loads. The researches include either the evaluation of influences of the wall stiffness and wall inclination on pressure changes, parameters connected with silo strength and geometry etc. According to the data of the mentioned above parameters values the evaluation and estimation of density function of these random values and first, second order statistics were calculated based on Level-1, Level-2 and Level-3 Reliability Methods. These experimental data were subjected to statistical accounts. The data obtained from calculations can be used in designing similar silos and in silo codes. The estimation of safety factors and values of partial safety coefficients are presented in this paper. According to situation in Polish industry, where many old buildings are now renovated, it is urgent estimated the safety of this construction in terms of Level-1, Level-2 or even Level-3 Reliability Methods. It implicates necessity of find minimum of safety function as distance of weakest point to central point. Methods described in paper are mostly connected with experimental data, so safety functions take into account experimental mistakes and random values of equipment. Some experimental results of values of bulk solids pressure, proposals of safety functions and some results of safety coefficient are presented.

Probability of non failure can be written in terms of safety margin $\Delta = R - S \geq 0 \Delta = R - S$ can be written in linear form: $\Delta = a_0 + \Sigma_i(a_i x_i) \geq 0$. Then two first moments are $\Delta_{\acute{s}r} = a_0 + \Sigma_i(a_i x_{mi})$, $\mu_d^2 = \Sigma_i(a_i^2 x_i^2)$. The limit condition $\Delta^* = 0$ implicate reliability index $\beta \leq \Delta_m/\mu_d$ and $\Delta = (R_m\text{-}S_m)/((\mu_R^2 + \mu_S^2)^{1/2})$, when R and S with index m denotes mean value of R and S, μ denotes standard deviation. If are used standardized variables $\xi_i = (x_i\text{-}x_{im})/\mu_I$ then β is the shortest distance to limit surface $\beta = (R_{mr}\text{-}S_m)/\sqrt{(\mu_R^2 + \mu_S^2)}$. When R and S are separated than index β in Level-2 is divided to two indexes for resistance and action consecutively $(S\text{-}S_m)/\mu_S \geq \beta_S$ and $(R\text{-}R_m)/\mu_R \geq \beta_R$. Another solution for $\Delta = R - S$ is evaluate global reliability index $\boldsymbol{\varpi}$: in equation $R^k \geq \boldsymbol{\varpi}\ S^k$, where S^k, R^k are characteristic value of S and R. It is simple method FORM with using calibration method in SORM. Value of this index verify from 0,4 to 1,4. Another coefficient of construction safety is described by β, the reliability indices interval: $\beta = z_m/\sigma_z$ and reliability index of risk failure $p_f = \Phi(\text{-}\beta)$ where z_m i the mean value of function Δ, σ_Δ is standard deviation of Δ, $\Phi(\text{-}\beta)$ is Laplace function. For $\Delta_m \geq \beta^*\sigma_z$ construction work in safe area. Coefficient β is equal in European standards $3{,}7 \div\ 6{,}7$, $p_f = 10^{-3} \div 3 \cdot 10^{-8}$. Coefficient β can be separated to two β_R and β_S connected with strength and loads. The value of safety global factor $\boldsymbol{\omega}$ for the investigated and new designed silos is included within the range 0,9 - 7 which depends on the silo exploitation mode and "silo conditions", Kaminski & Maj 2006. Reliability index β depends on kind of stored material, construction geometry etc should be individual calculated. It is useful to separate b to β_R and β_S,

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Comparative study of wind loads on gable roof buildings with and without attached canopies

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ABSTRACT

It is a general practice to provide a flat or sloping slab called canopy at the entrance door of the building at the ground level wherever space permits. Indian Standard Code of Practice on Wind Loads gives values of wind pressure coefficients on gable roof buildings with unsupported canopy for wind direction normal to ridge only. Similarly, the Australian code recommends certain values of net pressure coefficients on canopies/awnings and carports. The information available in such codes is very scarce. The present paper describes the results of the wind tunnel study carried out on the models of gable roof buildings with attached canopies. Pressure points are made on roof as well as wall surfaces to measure external wind pressures. In case of canopies, pressure points are placed on upper as well as lower surfaces. Wind pressure coefficients are calculated from the measured values of mean pressures on wall and roof surfaces in all the models and compared in order to observe the influence of canopy on wind pressure distribution.

Three models of rectangular plan gable roof building are made with Perspex sheet. One model is made without canopy, while other two have horizontal and inclined attached canopy. On each surface of building model, pressure tappings are provided for measuring the surface pressure. The gable roof slope is kept as 20°. The width of attached canopy is 50 mm and length is kept as 75 mm. The slope of inclined canopy is kept same as roof slope i.e. 20°. The models are named as A (without canopy), B (horizontal canopy) and C (inclined canopy). The isometric line diagram of the model is shown in Figure 1.

All the models are tested in the closed circuit wind tunnel of Civil Engineering Department, Indian Institute of Technology Roorkee, India. The cross section of wind tunnel is 1.15 m (width) ×0.82 m (height) and length of the test section is 8.25 m. Models are placed at a distance of 5.8 m from the upstream edge of the test section. Although tunnel is constructed for uniform flow only but in the present study the models are tested in uniform flow as well as boundary layer flow. The flow of the wind tunnel is then made boundary layer by placing a grid of hollow circular tubes at the upstream end of the test section.

It is concluded from the present study that the value of pressure coefficients in BLF are small compared to UF between angles 0° and 15°. The pressure equals at angle 30°. For rest of the angles the pressure goes on increasing. The inclined canopy produces low negative pressure on upper and lower surfaces of the canopy.

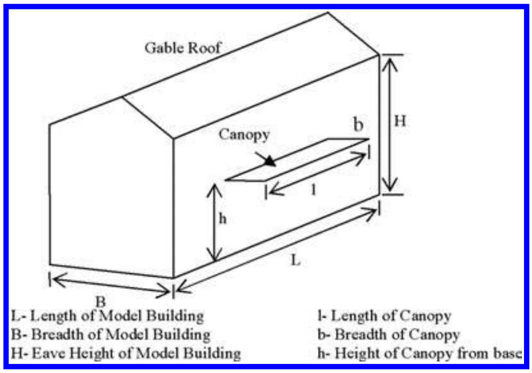

Figure 1. Isometric diagram of the model with attached canopy.

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An ACS algorithm for the formation of subminimal-suboptimal cycle bases

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ABSTRACT

Cycle bases of special properties have many applications in applied science, and particularly in the mesh method of system analysis. If the selected basis is minimal, the corresponding cycle adjacency matrix, which is pattern equivalent to the mesh matrix but α fold smaller, will be sparse leading to a sparse mesh matrix. α can be 3 or 6 depending on the mesh being planar or spatial, respectively. However, sparsity can further be increased if the basis is optimal, i.e. overlaps of cycles are minimal (Kaveh 2006).

There are efficient algorithms for finding minimal cycle bases among which algorithms developed by Kaveh (1976 and 2006), Horton (1987) and Berger (2004) are noteworthy. Greedy algorithm was first employed by Kaveh using the cycle space of a graph. Horton used a subspace of this cycle space to reduce the size of the search space. Unlike minimal cycles, algorithms developed for finding suboptimal cycle bases are limited to those of Kaveh (1988 and 2004). With the advent of heuristic algorithms such as ACO, it is interesting to apply them to the formation of subminimal cycle basis of a graph.

In this work, an ACS algorithm is developed for the formation of cycle bases that are suboptimal besides being subminimal. As a typical example, this ACS algorithm is applied to the graph of Figure 1. This graph consists of 32 nodes and 64 members and a typical cycle basis should have 33 cycles. The minimal basis is of length L(C) = 132 and the associated cycle adjacency matrix has $\chi(\mathbf{D}) = 233$ nonzero entries. Application of ACS algorithm results in a minimal basis with cycle adjacency matrix having $\chi(\mathbf{D}) = 183$ nonzero entries. This graph has 42 cycles of length four and the algorithm selects 33 of them excluding those shaded in Figure 2.b.

The patterns of cycle adjacency matrix for these two cases are shown in Figure 3.

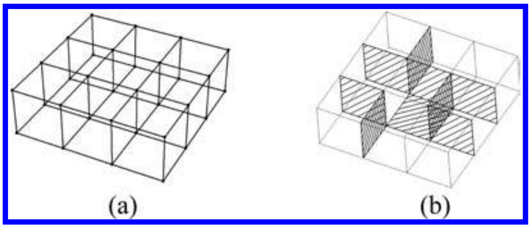

Figure 1. (a) A graph consisting of 32 nodes and 64 edges, (b) Shaded cycles are excluded.

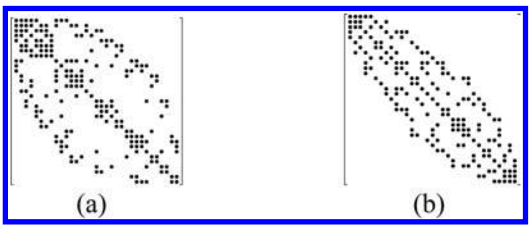

Figure 2. Pattern of the cycle adjacency matrices for (a) A subminimal basis, (b) A suboptimal basis.

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Excel for advanced structural analysis

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ABSTRACT

The aim of this paper is to illustrate the use of Excel in solving advanced engineering problems.

Solutions to the following structures are demonstrated:

Natural frequencies for tall structures are solved using classic theory of un-damped free vibration applied to a multi-degree-of-freedom system. A Visual Basic program is used to find the value of $[B] - \omega^2[I]$ for a specified value of ω^2. The mode shapes are calculated for each value of ω.

Transient temperature analysis is applied to a tall box structure subject to solar radiation. The method of finite differences in used to solve the partial differential equation $\partial T/\partial t = \alpha\ \partial^2 T/\partial x^2$. A Visual Basic program is used in an iterative mode and a solution is found when the temperature regime is the same at 24:00 as at 0:00 h.

Beams on elastic foundations: A free ended, finite difference model is used to solve for different loadings and various sub-grade reactions. Excel matrix routines are used to solve for the displacements, bending moments and shear forces. A strip foundation for travelling cranes is used as example.

Airy stress functions: Finite differences are applied to the governing partial differential equation

$$\partial^4\varphi/\partial x^4 + 2\ \partial^4\varphi/\partial x^2\partial y^2 + \partial^4\varphi/\partial y^4 = 0$$

An iterative VB program is used to solve for φ, the Airy stress function, at each node. The values of φ are then used to calculate the stresses in orthogonal directions.

Tapering circular slabs: Finite differences are applied to the governing differential equation

$$d^2\varphi/dr^2 + 1/r\ .\ d\varphi/dr - \varphi/r^2 = -\ Q/D$$

where φ is the nodal rotation. An Excel matrix solution is used to solve for the nodal rotations, the vertical deflections and bending moments.

Plane frames: An analysis of an eight member portal frame is given. Virtual work is used to solve for the restraining forces. The x and y displacements at each member end are then calculated using the principles of virtual work. Finally, graphs of the bending moments are given.

Trusses: The stiffness method of analysis is used. For specified loading and material properties a combined stiffness matrix is formulated using a VB routine after which Excel matrix routines are used to establish the nodal deflections, the local deflections and member end forces

Block type foundations under dynamic loading: Natural frequencies in the vertical direction and rocking modes are calculated. A simple comprehensive method, whereby the coupled rocking modes are reduced to a single degree of freedom, is demonstrated.

CONCLUSIONS

The above examples illustrate the ease and effectiveness of an Excel solution for complex problems.

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Theory for large complicated space bar structures with algorithms and programs

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ABSTRACT

This kind of investigations was made almost parallel with early world computer methods. From beginning (1970), theory was oriented on large space bar structures – single- double- and multi-layers. Moreover, from app. 1972, it was oriented on covering structures as large span flat roofs, cylindrical vault and spherical domes. Next about 1975 investigations were focused on more complicated shapes as conical, barrel, toroidal and shapes defined by any functions known in analytical geometry, Obrebski 1972, 1979, 1980, 2008.

Further, in cooperation with A.H. Fahema (1999) (Libya) the FEM algorithms were extended on space bar structures including space bar wavy domes etc.

The global geometry for authors programs was consequently derived by means of vector calculus. There were elaborated detailed mathematical relations and geometrical objects for following kinds of net of points: orthogonal; translational; rotationally-translational; ring net of points with particular or separate cases: barrel-, cylindrical-, spherical-, conical- and toroidal; elliptical and wavy structures.

There is assumed, that all nodes of structure are inscribed into regular net of points and located in intersections of three, curved parametrical lines ξ_i, from which two lines are lying on some equi-distance smooth surfaces. The position of particular node is defined in proper formulae and next in input data by integer three numbers x_i .

There the most general description of complicated bar structures gives theory elaborated by author – including numerical algorithms by means of DMEM (Difference Matrix Equation Method) and applied (by author, too) in programs for computers. There, the particular bars can have any boundary conditions by nodes and analysis can be static, dynamics etc). Similarly, supporting and loading systems of space bar structure can be very general.

There, were elaborated by Obrębski some computer programs for different destinations: from single bar strength analysis to analysis and synthesis of complicated space bar structures. It were some such programs, which names can be found in paper Obrębski 2008, with specification of its possibilities.

In all author's programs, the description of structure global geometry and topology is defined by introduction in input data the following groups of information:

- kind of applied net of points,
- list of nodes inscribed into declared net of points,
- definition of repeatable nodes,
- definition of repeatable bars,
- given support system,
- external loading.

Even now, when we observe on market many sophisticated commercial programs, presented theory, algorithms and programs are still interesting and competitive.

The above theory and computer programs have literature in form of some books (see below), papers and numerous conference presentations.

The paper and presentation are illustrated by many drawings and computer graphics of calculated structures.

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Design assisted by testing: A tool for the design resistance determination

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ABSTRACT

Methods of the design assisted by testing can be used as the tool for the determination not only of the material properties but also of the design resistance of structural members or components namely in the cases if no other tools are usable. In the last period many times the design using test results only allows to determine design resistances of structural members, details or parts. On that account it is available to deal with the problems of the design resistance evaluation from the viewpoint of the influence of statistic characteristics of parameters determining the design resistance value. Helping the parametric study, on several examples the design resistance evaluation with respect to the test number, complexity of the resistance model and to the variability of the basic variables is shown.

Philosophy and basic principles of methods of the design assisted by testing described in EN 1990: Annex D (2004) and going to standard procedures for the design resistances determination are known generally. The standard procedure for the design resistance determination is generally based on: (i) the comparison of experimental resistances $R_{ex,i}$ and theoretical resistances $R_{th,i}$ calculated according to suitable resistance model, (ii) the statistic evaluation of the design resistance model with the respect to the test number statistical variability of the quantities X_j and uncertainties of the resistance model.

From the viewpoint of the statistical evaluation the design resistance is influenced by the statistic characteristics of the various parameters, namely by the variation coefficients apposite the resistance variability and the uncertainties of the resistance model. Helping the parametric study the influences of some parameters are shown on the examples of selected representative resistance models. The parametric study was elaborated for usual typical functions of the resistance model given by the general equation $g_{Rth} = X_1^p \cdot X_2^r$. For the evaluation of influences mentioned above various combinations of the powers p and r were considered. For the comparison the design resistances were calculated according to exact and simplified forms, for the various test numbers n and the variation coefficients v_δ, v_{X1}, v_{X2}.

For the illustration only, the example of the design resistance (related to mean values) for the selected resistance model is shown on Fig. 1.

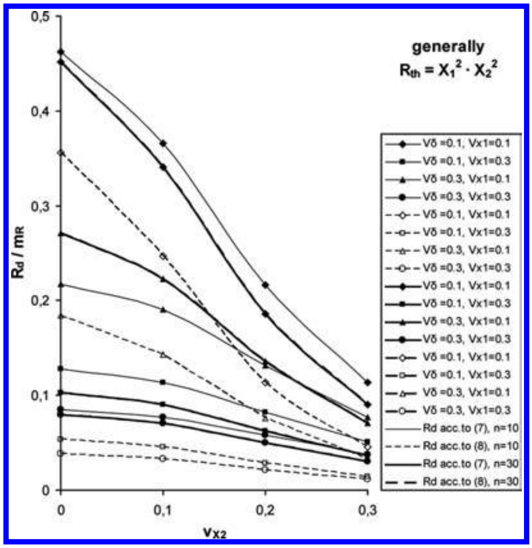

Figure 1. Design resistance in dependence on the test number n, variation coefficients v_{X1}, v_{X2}, v_δ for $R_{th} = X_1^2 \cdot X_2^2$.

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Design, construction, and shake table testing of a full-scale seven-story apartment building

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ABSTRACT

In July 2009 a full-scale 1600 square meter mid-rise apartment building, shown ready for testing in Figure 1, was subjected to a series of earthquakes at the worlds' largest shake table in Miki, Japan. The test program consisted of two major phases: the first phase building consisted of a single story of steel with six stories of wood on top, and the second phase consisted of locking down the steel story to test only the six-story light-frame wood building. This paper presents the details of the performance-based seismic design, construction, and testing of the building. The objectives of the test program were to (1) demonstrate that the performance-based design procedure developed as part of the NEESWood project worked, i.e. validate the approach; and (2) gain a better understanding of how mid-rise light-frame wood buildings and mixed-use buildings respond in a major earthquake while providing a landmark data set to the seismic engineering research community.

The seismic test program consisted of multiple shake table tests during three days. During the first test day a steel special moment frame (SMF) at the base of the building was not braced and therefore participated in the testing. The ground motion was scaled to the peak ground acceleration levels to represent seismic hazard levels with 50%, 10%, and 2% probability of exceedance in 50 years, which corresponds to return periods of 72, 475, and 2500 years, respectively.

The building performed very well with maximum averaged inter-story drifts on the order of 2% and suffered only minor non-structural damage. Peak shear wall drifts in one corner of the building slightly exceeded 3%. Based on the results of this test the following conclusions can be drawn: (1) Design of a light-frame wood building using PBSD likely enabled better performance than has been observed for force-based designed buildings in previous testing. However, it should be noted that buildings previously tested were not designed to the current IBC or using PBSD; (2) Although the floor plan was approximately symmetric and the seismic mass was evenly distributed, torsional response was still present during the seismic tests. Torsion needs to be included in PBSD procedure for wood frame buildings, e.g. Direct Displacement Design; (3) A steel SMF appears to be a good option for commercial space below a mid-rise light-frame wood building in seismic regions; and (4) as a mid-rise mixed use building, it is possible to design for and achieve performance that is better than what is expected from current design codes and guidelines.

Figure 1. The NEESWood Capstone test building ready for testing at the worlds' largest shake table in Miki, Japan in July 2009. The building represents the largest building ever tested on a shake table.

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Gautrain Southern Viaducts designed to Eurocode

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The Gautrain Rapid Rail Link in South Africa joins Johannesburg to Pretoria and to the Oliver R Tambo International Airport. It consists of some 80 km of track, 10 km of viaduct and 50 bridges.

The Southern Section Viaducts consist of nine viaducts totaling some 4.3 km in length. The bridge decks are simply supported precast segmental concrete box girders with both internal and external prestressing. Two elevated stations have platforms cantilevering off the viaducts. One of the stations is shown in figure 1.

The South African bridge design code, TMH7 does not adequately cover segmental construction, external prestressing, rail-structure interaction and rolling-stock dynamic analysis. All of these are covered in the Eurocode. It was therefore decided to design the viaducts to the new Eurocode.

In Eurocode, a single equation covers all of the possible permutations of multiple combinations. This leads to significantly more combinations than required in TMH7. Eurocode also have 7 limit states. Although this leads to many combinations, they were easy enough to apply in a FEM program or spreadsheet.

Figure 1. Viaduct under construction at ORTIA airport, showing elevated station platform.

Rationalization of the number of combinations will lead to reduced design time with no loss of accuracy.

In Eurocode, seismic actions are only considered at Ultimate Limit State (ULS). This leads to more realistic load combinations than in the TMH7.

Shear design to Eurocode is more flexible and less complicated than TMH7. For prestressed sections, a simple enhancement is allowed. This is significantly simpler than TMH7's complex handling of Class 1, 2 and 3 prestressed sections.

A crucial weakness in shear design is the definition of z – the inner lever arm. For a section with axial compression, z will decrease, requiring more shear reinforcement. If the bending moment is increased on a section, z will increase, requiring less shear reinforcement. Both are counter-intuitive. Furthermore, z is meaningless in a class 1 prestressed section where there is no tension in the section.

One of TMH7's durability and appearance requirements is to control concrete crack widths under full service loads. The Eurocode approach is quite different in that it only controls crack width under permanent loads.

The coverage of dynamic behaviour of bridges in Eurocode, especially footbridges and railway bridges, is comprehensive and clear. This includes rolling stock dynamic analysis.

Rail-structure interaction (RSI) is also well documented. Eurocode sets limits for the additional stresses in the rails caused by this effect. It also sets limits on the displacements and rotations across an expansion joint.

For bridges, Eurocode as it is currently laid out, is significantly more complex to follow than TMH7. Eurocode also includes the latest research which will lead to a reduction in cost of bridges.

If we choose to adopt Eurocode in bridge design in South Africa, we need to look carefully at how other countries deal with the code in their National Annexes. A possible approach to adopt is to write the National Annex such that it is a stand-alone document for the simpler bridge forms, along the line of the relationship between TMH7 and BS5400.

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Buitengragt structural steel pedestrian bridge: A continuous asymmetrical box girder

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ABSTRACT

Using over 3500 welded steel plates, the asymmetric box girder of the Buitengragt structural steel pedestrian bridge is an unconventional form that responds to the challenges of its location. The bridge was designed as a simple but visually bold line across the busy urban back drop of Cape Town's City Centre. Its fabrication required a high degree of dimensional accuracy and carefully planned welding procedures to ensure the fit up of the plated sections. Its location, partly over the roof of a buried parking garage, influenced the deck design and required the development of bespoke foundations details.

The localised deformation of an asymmetrically supported bridge can disturb pedestrians, even though it may be structurally irrelevant (Adao Da Fonseca, 2007). The development of the deck section therefore took its direction from a sensitivity analysis of the serviceability performance of the structure. Beams of various dimensions and torsion constants were modeled using a 3-D beam model. The resulting transverse deflections were then reviewed in conjunction with the economy of the section.

A dynamic analysis that applied oscillating loads to the structure was carried out following the recommendations of the SETRA Technical Guide, Footbridges: Assessment of the vibrational behavior of footbridges under pedestrian loading, 2006. This was judged necessary because the bridge's first vertical mode of vibration is in a range susceptible to excitation from pedestrian loading. The vertical and lateral accelerations were, however, found to be well within acceptable limits.

Although the detailed design followed a conventional approach, the fabrication process did not. The design team invested significant effort in the detailing of all plated sections to ensure the constructability of the bridge. Throughout the construction process no dimensional errors were found in the design. This is because the plates were dimensioned both numerically and graphically using a three dimensional model. This was, however, only the first part of the process. During the fabrication process the welding procedures were also critical in preventing plate distortions that translated to fit up problems on site. In this regard the presence of the transverse diaphragms was important. In placing additional diaphragms at closer centres, the time saved in the fit up of the sections outweighed the cost of the additional steel.

The final design of the bridge is a direct response to the site and the required functionality for pedestrians. It is hoped it also achieves the goal of creating a positive landmark in the City.

Figure 1. Photograph of box section during the main span erection.

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Study for the structural behaviour of low/medium/high rise reinforced concrete residential buildings in Kuwait

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ABSTRACT

Local authorities in Kuwait recognized the need to revise regulations to allow for more building height and recently released a new version allowing for more heights and more built-up area for residential buildings. The main concern of this paper is to provide an overview of the impact of the different design factors on the structural behavior of low/medium/high rise residential buildings aiming to deepen structure and architect designer understanding for such type of buildings. The study is performed on reinforced concrete since it is the most common material of construction in the Middle East and emphasized only on Kuwait City design parameters for wind, seismic and soil conditions. In this paper, an extensive study using six different buildings of a regular and symmetrical layout plan is carried out to clearly demonstrate the building height impact on the structural analysis and design. All buildings have the same layout plan and different building height ranging from five to fifty typical residential stories. Building loadings as per the common engineering practice in Kuwait are applied to the structural models. Serviceability study is performed to ensure that buildings have sufficient stability to limit lateral drift and peak acceleration at the highest occupied level within the acceptable range of occupancy comfort. Ultimate strength study is carried out to verify that buildings are designed to withstand factored gravity and lateral loadings in a safe manner according to the international building codes. Three dimensional finite element analysis using ETABS software (ETABS2005) is used for conducting analysis and design of different structures presented in this study. This software is originally established with emphasize of tall buildings analysis and design and widely used in the engineering design practice all over the world and especially in the Gulf area. The studied factors in this paper include the building slenderness ratio and the building core size and location since they are the key drivers for the efficient structural design of such type of buildings. Slenderness ratio ranging from 1.15 to 7.37 is selected to represent low/medium and high rise building cases. Both cases of centric and eccentric building core of the same size are studied to investigate the impact of building core location in the structural design of buildings with different slenderness ratios. Effect of building core size on the building wind drift is investigated by using different core depth to building height ratios ranging from 4.425% to 9.19% for the same 30 stories residential building. Results for study of the impact of these design factors are presented. Conclusions are summarized as guidelines for the structural professions in Kuwait when performing design for reinforced concrete residential building with different height ranging from low to high rise.

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Design of the Nasrec Transportation Hub roof

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ABSTRACT

On the Southern side of Soccer City at Nasrec, the new Pedestrian Promenade and Transportation Hub were recently completed for the World Cup. The Transportation Hub is a collection point for people coming to the 2010 World Cup stadium and is part of the Bus Rapid Transport development. Pedestrians travel from the Transport Hub, over the new Pedestrian Bridge and Pedestrian Promenade, to the southern side stadium entrance gates. Various upgrades to the area totaling R270 million were undertaken by the Johannesburg Development Agency (JDA).

The 27 m wide Pedestrian bridge has inclined columns and handrailing as part of the architectural concept. The precast bridge side panels (over the existing railway line) presented numerous challenges to the design team with regards to construction sequencing. There was an overlap between the roof foundations and the inclined retaining walls which form part of the bridge abutments – this meant that meticulous co-ordination of setting out was required between these two structures.

The 211 m long, boomerang shaped Transportation Hub Roof, designed by Goba and Boogertman Urban Edge Architects (BUEP), is a sheeted steel structure, resembling an aircraft wing in section, incorporating bullnosing, I-beams and circular hollow section columns. Four main column grid lines support the roof structure over and diverge from each other – these are spaced at 15 m centres down the length of the roof. An interesting aspect of the roof is that every column is a different length due to the columns gradually going from being purely vertical to having a maximum inclination of 10 degrees to the vertical on the Northern end – the idea being that the columns should point or lean towards the stadium. The roof also steadily climbs at a 2 degree angle from its start height on the Western end of 2.8 m above ground level to its final height of 14 m above ground level.

All the above factors meant that the roof first had to be analyzed as a 3-dimensional structure to test whether it could in fact be built in the manner intended by the Architect (with inclined columns). The analysis revealed that this was indeed possible and that an axial tension force was generated in the main longitudinal beams due to the column inclinations. In effect, the longitudinal beams restrain the columns from falling over. The longitudinal beam axial forces eventually die out as one transitions from the Northern roof leg towards the Western roof leg (columns also getting shorter and more vertical).

The roof structure houses a Ticket Kiosk underneath the transition between the North/South roof leg and the West/East roof leg which plays a critical role in ensuring the roof's stability. The roof has several beam connections which tie back to this point as a result of this.

The roof is covered with a conventional 'Kliplock 406' sheeting on its top surface and incorporates translucent sheeting material over approx. 40% of the roof surface area to allow sunlight to penetrate through the roof structure. The ceiling material is also made of two materials – Brownbuilt sheeting and an expanded steel mesh material. Numerous challenges were experienced with the layout of the sheeting as the sheeting had to transition around curved shapes in plan and in section. It therefore was decided to have the two roof legs meet at a ridge line which solved the sheeting problem in plan. Radiused bullnosing at the transition between roof sheeting and the inclined ceiling material was used to achieve a rounded edge along the entire perimeter of the roof.

Construction of the roof commenced in June 2008 and final completion was achieved in March 2010.

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On the calculation of Eurocode 3 buckling interaction factors for combined bending and axial compression

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ABSTRACT

For steel members subjected to a combination of axial compression and bi-axial bending, Clause 6.3.3 of EN1993-1-1 (Eurocode 3 Part 1-1) gives two verification criteria that must both be satisfied, namely (Comité Européen de Normalisation, 2005):

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \le 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \le 1$$

where N_{Ed}, $M_{y,Ed}$, $M_{z,Ed}$ are the design values of the compression force and the bending moments about the *y-y* and *z-z* axes at any given cross-section, and N_{Rk}, $M_{y,Rk}$, $M_{z,Rk}$ are the characteristic resistance values. The k_{yy}, k_{yz}, k_{zy}, k_{zz} are the buckling interaction factors. Other symbols are as defined in EN1993-1-1.

The focus of this paper is the k_{ij} interaction factors. Eurocode 3 specifies two alternative methods (referred to as Methods 1 and 2) for the calculation of the interaction factors. Method 1 is described in Annexure A of EN 1993 Part 1.1, while Method 2 is described in Annexure B. Eurocode 3 does not recommend the preferred method. Previous studies undertaken on these approaches include the work of Boissonnade et al (2004), Gonçalves & Camotim (2004), and Greiner & Linder (2006).

In this paper, we compare the two methods through a parametric study, based on steel sections that are available in South Africa. The results are presented in plots of the type shown in Figure 1. With regard to the degree of matching of the two methods, it has been found that sections belonging to the same cross-section shape category (I, H or RHS) generally displayed similar trends. However, within each cross-section shape category, Class 3 cross-sections showed notable differences to their Class 1 & 2 counterparts.

In general, the methods tend to agree better for axial compression combined with bending about the *y-y* axis (as opposed to bending about the *z-z* axis), with the exception of Class 3 I- and H-sections. The closest agreement was generally found with the rectangular hollow sections (RHS). The higher-slenderness range generally showed better agreement of results for I- and H-sections, but less agreement for RHS sections.

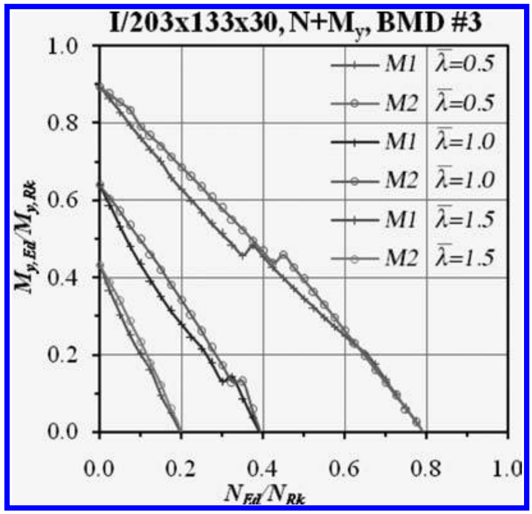

Figure 1. Results for an I/203×133×30 section under compression and major-axis bending. M1/ M2 refer to Methods 1 and 2.

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7. Plates, shells, pipes, laminated composites, sandwich structures

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Finite rotation finite element analysis of layered composite plates and shells

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The present paper deals with geometrically nonlinear static and stability analysis of composite laminated shells in the framework of the first-order transverse shear deformation theory (FOSD). The theory and the finite element method account for full geometrical nonlinearity, thus they account for arbitrary, finite rotations.

The FE developments in the present paper are based on the work of Lentzen (2009), who derived a finite element code for nonlinearly coupled thermo-electro-mechanics using the four-node element of Gruttmann & Wagner (2006). Finite rotations are treated by Rodrigues parameterization. The Riks–Wempner arc-length control method is used to trace equilibrium paths in the pre- and post-buckling range of deformation.

We consider standard benchmark problems of isotropic and composite laminated shell structures that serve in literature as most demanding tests for finite rotation analysis. One of the examples treated in this paper is the hemispherical shell with an 18° hole shown in Figure 1, subjected to two inward and two outward forces. Radius and thickness are $R = 10$ in and $h = 0.08$ in, respectively. The material data are: $E_{11} = 20.46 \times 10^6$ psi, $E_{22} = 4.092 \times 10^6$ psi, $G_{12} = G_{13} = 2.53704 \times 10^6$ psi, $G_{23} = 1.26852 \times 10^6$ psi and $\nu_{12} = 0.313$. A discretization of 24×24 elements for one quarter of the shell was used along with appropriate symmetry conditions. The results for the radial inward displacements of the composite hemisphere are shown in Figure 2. An excellent agreement is obtained with our earlier results in Kreja & Schmidt (2006), where a mesh of 12×12 eight-node isoparametric shell elements based on the FOSD finite rotation shell theory (LRT56) was used.

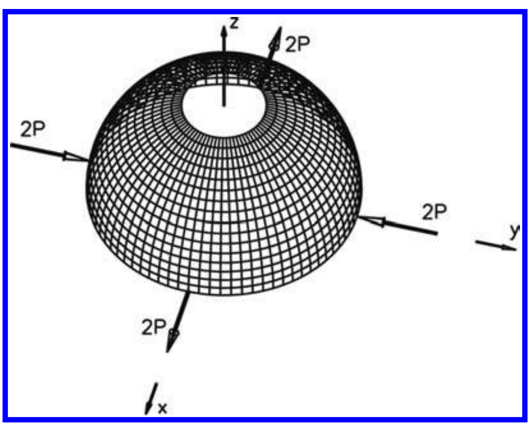

Figure 1. Pinched hemispherical shell with 18° hole.

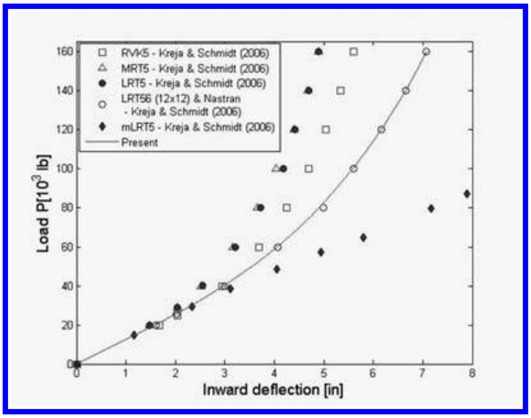

Figure 2. Inward deflection.

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Hybrid active-passive laminated structures: Modeling, optimization and identification

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ABSTRACT

In this Lecture, recent developments on modeling, optimization and parameter estimation in sandwich structures with hybrid damping are discussed. A finite element model has been formulated using a mixed layerwise approach, by considering a higher order shear deformation theory to represent the displacement field of the viscoelastic core and a first order shear deformation theory for the displacement fields of adjacent elastic laminated face layers, as well as for the sensor/actuator piezoelectric laminae or patches. The layerwise model is based on an eight nodded plate/shell finite element, with 17 mechanical degrees of freedom per node, after imposing inter-layer displacement continuity, and 2 electric degrees of freedom per element. The complex modulus approach is used for the viscoelastic material behavior, and the dynamic problem is solved in the frequency domain, using viscoelastic frequency dependent material data for the core. Active control is applied using proportional displacement and negative velocity feedback control laws, through the exteriorly bonded sensor and actuator layers.

Optimization of passive, active and hybrid damping treatments is conducted, using as design variables the viscoelastic core thickness, the constraining elastic face laminae thicknesses, orientation fiber angles, as well as piezoelectric actuators/sensors locations (Araújo et al. 2009). The optimization problems are solved using gradient optimization and/or non-gradient algorithms as appropriate. Comparative frequency response functions are presented in Figure 1, regarding a rectangular simply supported sandwich plate with viscoelastic core, carbon fiber laminated elastic face layers, and four pairs of co-located piezoelectric sensors and actuators. The optimization effect on modal damping is clearly depicted from the figure.

Parameter estimation of viscoelastic core material properties is also presented and discussed. The inverse problem is formulated as a constrained optimization problem, by fitting the response of the finite element numerical model to the corresponding experimental response of the viscoelastic sandwich structure, considering specific parametric models for frequency dependent damping material behavior (Pritz 1996).

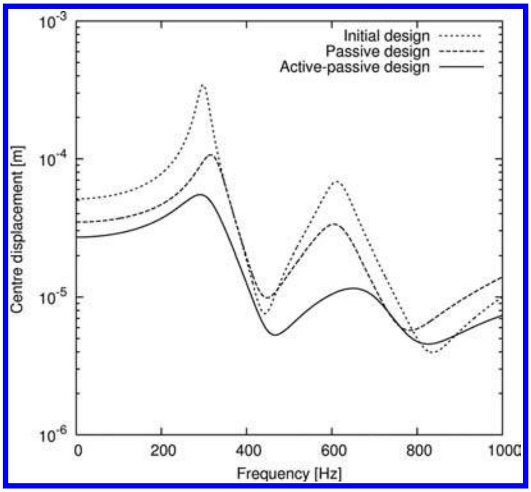

Figure 1. Effect of optimal designs in frequency response of a hybrid active-passive sandwich

Constraints are imposed on the design variables, arising from thermodynamic restrictions on isothermal linear viscoelasticity, and gradient based optimization techniques are used to solve the optimization problem (Araújo et al., in press).

Both optimization and parameter estimation applications will be presented and discussed.

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Stability of sandwich barrelled shell under pressure

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ABSTRACT

The present work is devoted to the elastic stability of sandwich barrelled shell. Such shell may be an alternative for the classical cylindrical monocoque shell used as a main part of tanks for storage or transport of liquid materials. Strength and buckling resistant of the sandwich barrelled shell in comparison with the classical solution is much higher due to meridional curvature and thicker cross section. Since the cross section consist of two thin faces and a light core thicker cross section does not increase the mass of the shell significantly. Stability problems of the sandwich structures were analysed e.g. by (Grigolûk and Čulkov 1973). The review of works devoted to many aspect of sandwich structures is given by (Noor et al. 1996).

In the paper the mathematical model of the sandwich barrelled shell is presented derived on the basis of the broken line hypothesis presented e.g. by (Volmir 1967). The linear strain relations are assumed. The shell is defined in a curvilinear coordinate system and consist of three layers: upper and lower face of the same thickness and a light core (see Fig. 1). Thickness of the shell is constant along the whole meridian. Model of the shell is simply supported at both ends and loaded with uniform external pressure.

In the analytical description the integrals representing the strain energy U_ε and the work of external load W are presented. The integrals are functions of displacements. The total potential energy can be written as follows: $\Pi = U_\varepsilon$-W. Applying the Rayleigh-Ritz method five linear equations are obtained in the form $\partial\Pi/\partial q_a = 0$, where q_a represents unknown coefficients of displacements functions assumed before. Solving the system of equations the critical value of external pressure may be determined.

The numerical investigation have been carried out with the use of the ABAQUS system. Finite elements as well as procedures available in the system were used. The sandwich shell was modelled as an assembly of three components: the core, for witch a C3D8R brick element was used and two faces modelled with the use of S4R5 thin shell element. The material properties of the core and faces were: $E_c = 100\,MPa$, $\nu_c = 0.16$, $E_f = 71000\,MPa$, $\nu_f = 0.33$. A family of barrelled shells of the same capacity and length was generated including the reference cylindrical shell. Different values of the core thickness were considered. The reference monocoque shell had the thickness equals two times face thickness.

Results of the investigation obtained for different shells show a significant increase of buckling load for the monocoque shell with large meridional curvature. This increase however is much smaller for sandwich shells – the thicker the core the smaller increase of buckling load. The number of circumferential waves due to buckling is smaller for shells with thicker core what influence the value of stresses in the buckled structure.

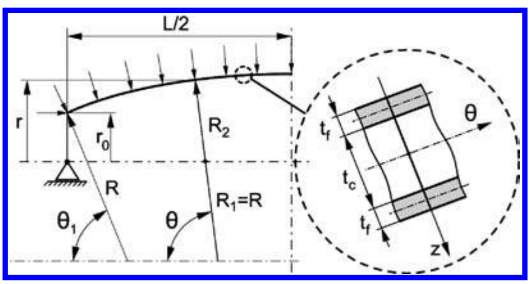

Figure 1. Model of the barrelled shell.

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Power towers: On structural engineering problems of solar updraft chimneys

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ABSTRACT

Solar Updraft Power plants (SUPPs) are the most economic techniques for renewable power generation in arid zones. They work free of CO_2-emissions, using solar irradiation as fuel. Their CO_2-footprint delivers 10 g to 20 g CO_2/kWh of produced electricity, depending on the plants' service-lives.

Their working principle is illustrated in Figure 1. (Schlaich 1995) Such SUPP consists of the collector area (CA), the turbo-generators as power conversion units (PCUs), and the solar chimney (SC). In the glass-covered CA, solar irradiation heats up the enclosed air which thermally expands and streams towards its center. There, in the PCUs, its kinetic energy is transformed into electric power. The kinetic energy is due to the uplift of the warm air in the SC.

A SUPP with CA diameter of 7.000 m and SC height of 1.500 m delivers a maximum electric power of ≈400 MW_p, on mid-days in summer time. For a

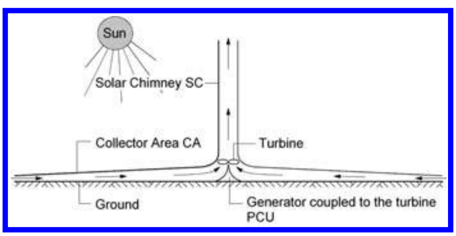

Figure 1. Schematic working scheme of a SUPP.

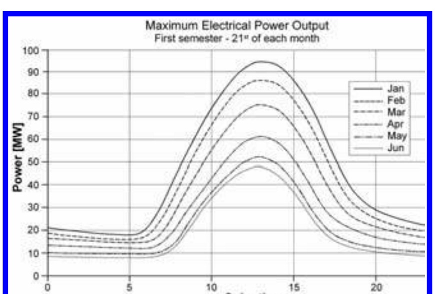

Figure 2. Daily power output of a SUPP in different months.

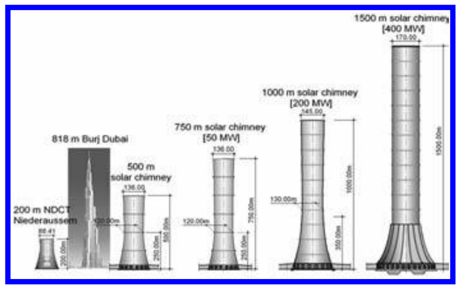

Figure 3. Different planned SCs compared to Burj Chalifa.

smaller SUPP the typical power output over 24 hours is shown in Figure 2 (Pretorius 2007).

The characteristic landmark of SUPPs is the huge central SC or power tower, which stands in the center of this paper. Depending on the power output, this thin RC shell reaches to enormous heights, as Figure 3 elucidates. Such structure is loaded by dead weight D, Wind W, Temperature T and Earthquake excitations E for seismic active locations. Wind is by far the most important loading component.

The lecture will first cover design aspects for economically optimized SCs, then turn to their static as well as dynamic instability and vibration behavior, and finally explain non-linear responses (Kraetzig et al. 2009). It closes with remarks on the great importance of these power plants for mankind's future energy supply (Backström et al. 2008).

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Structural optimization of solar towers to minimize wind induced effects

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ABSTRACT

Solar Updraft Power Plants (SUPPs) represent a totally new generation of renewable energy sources, converting solar radiation into electric power.

The working principle is simple (see figure 1): a SUPP consists of a collector area to heat the air due to the wide banded ultra-violet solar radiation, the high-rise solar chimney to updraft the heated air to the atmosphere, and in between the power conversion unit, where a system of coupled turbines and generators transform the stream of heated air into electric power. The only problem is that efficiency will only be reached with extra-large dimensions of the tower and the collector area. In fact, the amount of power delivery is a question of the size of both, collector and solar chimney. Therefore, the potentials of an unrivalled economical energy production can be achieved only through the highest degree of optimization of the structural behaviour, the thermodynamic efficiency and the construction costs.

Within the SUPPs technology, for the feasibility of such a new, ultra-high tower structure, the modeling of the actions and the stochastic analysis of the structural response need to be up-graded, since the tower height and the loading models are above the current experience. Solar towers reach far beyond the Prandtl layer into the Ekman layer. Here, the shear and the turbulence decrease, whereas the Coriolis force becomes important. It increases as the wind velocity increases and tends to align the flow in the direction of the isobars. In the Ekman layer, experimental meteorological data are scarce. However, theoretical considerations available in literature provide adequate models for the mean and fluctuating wind components.

From the structural viewpoint one of the major objectives is to construct the solar tower as thin as possible. This can be achieved by using high-strength concrete and/or by installing stiffening rings along the chimney height and on top. They can be realized in several ways, e.g. with classical RC-beams, with composite steel-concrete and with spoken wheels. The stiffening rings improve the structural behaviour of the tower by avoiding ovalling deformations of the cross section. As a consequence, a better distribution of internal forces is achieved, without peaks of tension at the windward side. Wind induced tensile stresses are partly balanced by compressive stresses due to dead load. Therefore, only the remaining tension requires reinforcement. The structural optimization procedure presented in the paper aims at minimizing wind induced tensile stresses through the introduction of suitably stiffened ring beams along the height.

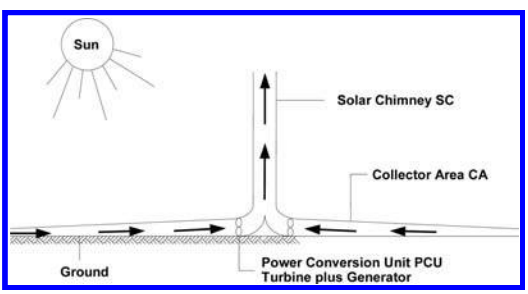

Figure 1. Working principle of SUPPs.

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New direction of cooling towers monitoring and diagnostics

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ABSTRACT

The article comes out of the results of the long-term technical structural researches made on such a real cooling tower, by which the regular technical structural researches were executed in the longest time-interval and the results of which were available. The results were processed into the form of "the degradation curves" describing the real time dependence of the degradation.

The dependencies gained from the experiments and the degradation curves can be summarized into the following table:

From the summary of the researches the problem of the vertical cracks in the cooling tower envelope arose. Pursuant to this fact the metodics of technical structural research focused only on the monitoring of vertical cracks effect on the structural envelope was worked out.

The exact determination of the dependencies of in-lab simulated aggressive environment and the reality is possible to be considered as a very difficult one or almost impracticable and its general formulation is conditional on the long-term monitoring of the materials and underlayers of totally identical properties and technologies used for their application. The methodics for monitoring of vertical crack effect on the cooling tower envelope was also formulated in this work. By the help of proposed procedure is possible to say whether the cracks are or are not imminent to static function of the cooling tower.

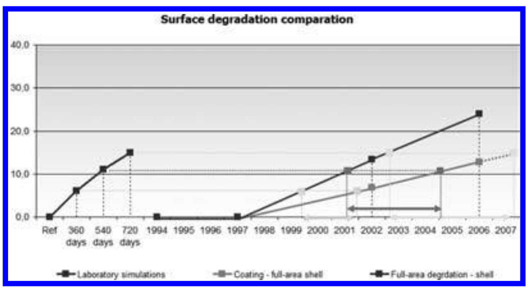

Chart 1. The comparison of the technical structural research results and the laboratory testing of the paint Sikagard 6802S durability.

Table 1. The dependencies between in a lab simulated and real environment degradations.

Parameter	Time of deposit in a lab [days]	Corresponding time on real structure [years]
The surface degradation of the coating composit.	360	5–7
	540	7–10
	720	10–13
Cohesion with underlayer	360	10–11
	540	11–12
	720	12–13
The depth of carbonatation	360	0
	540	4
	720	8

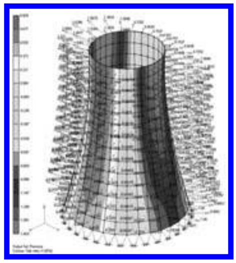

Figure 1. The wind load effect on the cooling tower. [pressure kPa, gallery KN/m]

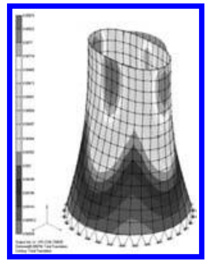

Figure 2. The shape and isograms of the deformation from the wind load.

ACKNOWLEDGEMENTS

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Application of sandwich-structures in steel bridges

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ABSTRACT

An SPS-sandwich consists of a lower and an upper steel plate and a non-porous, rather stiff polyurethane ($E \approx 800\,N/mm^2$) between the plates. The lower steel plate may be an old one of an existing structure to be refurbished or a new one. The mechanical thermal and endurance properties are such that this sandwich system can be used for a variety of structural applications, e.g. for ship engineering or bridge design. In the field of refurbishment of steel bridges, the technique has proven effective in several cases, e.g. the refurbishment of a major motor-highway-bridge near Krefeld, Germany (Feldmann et al. 2007a). Within the research project 'Development of steel decks with reduced welding' (Feldmann et al. 2007b), sandwich-type cross-sections have been developed for new bridges. In one solution, hollow profiles are provided that are embedded in the polyurethane sandwich core between two deckplates made of steel, thereby reducing weight and contributing to the stiffness of the structure (Fig. 1).

In order to determine the complex stress distribution over the bridge's cross-section, the so-called Generalised Beam Theory (GBT) (Schardt 1989) was extended for a bridge construction as illustrated in Figure 1 assuming thin-walled structural elements controlled by membrane stresses (Moeller 2006). The calculation method is based on an energy formulation and uses principal axis rotation to derive a set of orthogonal warping functions ${}^{k}\tilde{\omega}(s)$ showing symmetry and antisymmetry with regard to the vertical centroidal axis of the cross-section as well as to the shear elastic sandwich core of the bridge deck (Fig. 2). The related weighting functions ${}^{k}U'(x)$ can be obtained by solving a differential matrix equation and allow for the final determination of the stress distribution in any location of the bridge girder.

By summing up the membrane forces in the upper and lower deck plates ('stress areas') and relating them to the theoretical stress distributions obtained under the assumption of a linear stress distribution over the cross-sectional depth, it is possible to describe the bearing behaviour of the structure by means of stiffness dependent 'effective widths' for the upper and lower plate. This simplified approach corresponds to the standards and allows for an easy design.

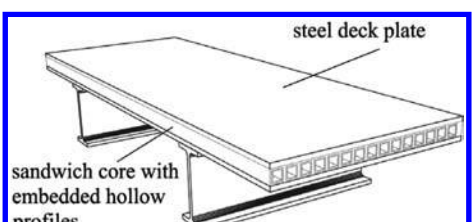

Figure 1. New bridge design with sandwich deck.

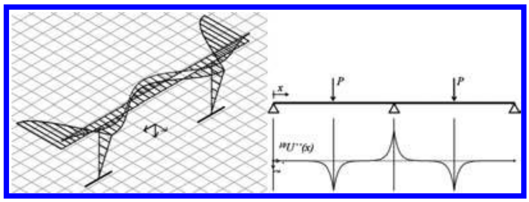

Figure 2. Orthogonal warping function and derivative of corresponding weighting function.

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Optimum design of a composite morphing leading edge for high lift wing

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ABSTRACT

Conventional high lift devices are designed to gain the maximum lift at a certain flight condition. By applying morphing technology, it would be possible to achieve an optimal aerodynamic efficiency and performance in multiple flight conditions. This paper presents an investigation into the design and optimization of a composite wing seamless morphing leading edge (LE), integrated with an actuation mechanism, for a large aircraft. The 3 mm thick LE skin was made of glass fiber composite and was reinforced with I-shaped stingers.

In order to enhance the take-off and landing performance, a LE fully deformed shape was specified, based on aerodynamic calculations. An internal LE actuation system was designed to meet the demanding shape requirements. The actuation mechanism consisted of an actuator mounted on the front spar which drives an eccentric beam to rotate and force the LE to deform. The eccentric beam was designed with a curvature to meet the specified LE deformed shape at any rotational angle. A number of discs with varying radius were mounted along the eccentric beam. The discs connect the skins and the beam and ensure that the LE deforms in the specified shape as shown in Figure 1.

A parametric study was conducted to set the optimum number of discs and their chordwise location. A finite element model of the morphing LE structure was created using PATRAN/NASTRAN. The specified displacements set by the eccentric beam and the discs were imposed to the model. The aerodynamic pressure was also applied to the LE skin. The calculated reaction forces were taken as the actuation loads required to deform the LE to the fully deflected position. There were eight possible positions for the discs and fourteen different arrangements were analyzed to find the optimum combination. Figure 2 shows the force required from each disc for the various design cases.

The optimum disc arrangement when five discs and the beam tip were used to impose the required displacements (case 11). The total actuation force required to deflect the structure was 5068 N. A nonlinear static analysis was carried out and the LE mechanical behavior under both actuation and aerodynamic load was studied. The stress results show that some of the laminate plies in off-fiber direction failed due to high tensile and shear stresses. Different layup combinations were therefore used to reduce the critical stresses. An alternative design using aluminum was analyzed and the stresses for this case were lower than the material allowable.

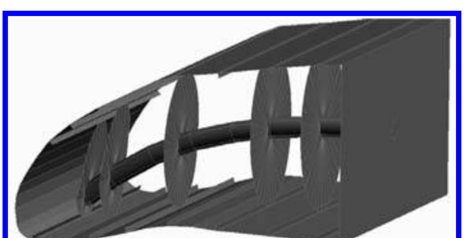

Figure 1. EBAM in the fully deployed position.

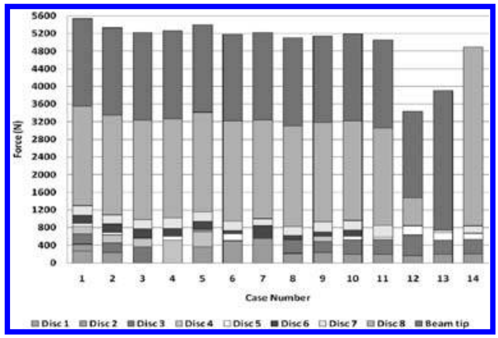

Figure 2. Force required from each disc to deflect the LE.

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Localized buckling in sandwich beam-columns

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ABSTRACT

Beam-columns are elements that resist the combined action of axial force and moment. The combined effect of these actions can increase the stress within the element, taking it to the limits of its load bearing capacity or structural stability. Sandwich beam-columns offer excellent bending performance with high weight efficiency; they are used extensively in various applications in astronautics, aeronautics, marine and civil engineering. However, this loading combination is liable to lead to complex failure modes in certain configurations such as the interaction between overall (Euler-type) and local buckling (Hunt & Wadee, 1998).

The effect of applying an axial force offset from the panel's neutral axis is investigated currently. Earlier works provided information on critical loads and eigenmodes (Drysdale *et al.*, 1979). Nonlinear, or post-buckling behaviour is currently studied; two models based on different bending theories are presented (Wadee *et al.*, 2010), where the effect of pure bending is incorporated within the displacement functions for the overall buckling behaviour. The first model is based on Timoshenko beam theory (TBT) and the second model is based on the higher-order Reddy–Bickford theory (RBT). These are formulated analytically and the resulting governing equations are solved using the numerical continuation package AUTO (Doedel *et al.*, 1997) for a number of panels. The resulting post-buckling equilibrium paths and corresponding deformations are presented with discussions of the destabilizing effects. The results are validated with a finite element (FE) model created with the commercial package ABAQUS (2006).

Sandwich beam-columns with aluminium face plates and softer cores are studied in the detailed investigation. Results for different load eccentricity ratios show that the TBT and RBT models produce similar pre-buckling paths, both triggering the local-ized mode beyond their respective limit loads. As expected from the strut models, the RBT model produces higher limit loads at a lower level of overall deformation due to the increased bending stress in the more compressed face plate, and as a result of the employment of a more flexible cross-sectional deformation field. Since the eccentricity increases the bending stress in the face plate closer to the load application point, both the limit load and minimum midspan lateral deflection are lower.

As the post-buckling deformation increases, the localized interactive buckling modes of the two models converge since the overall mode component dominates, hence diminishing the importance of the cross-sectional deformation which represents the principal difference between the two models. However, the dif-ference between the two analytical models regarding the onset of interactive buckling becomes more pro-nounced as the panel depth is increased.

The analytical models exhibit excellent correlation with the results from the FE simulations in the distribution of the limit loads and the corresponding level of overall buckling displacement with increasing load eccentricity; the RBT model being superior. Owing to its nonlinear in-plane core displacement, the same model also seems to be superior in estimating the onset of interactive buckling as the compressive stress threshold to cause localization is exceeded earlier. Conversely, the TBT model is marginally better in estimating the limit loads and the load carrying capacity in post-buckling, which suggests that a linear in-plane displacement distribution may suffice to model the combination of bending and buckling in the post-buckling response of beam-columns. In both cases, the analytical models have shown excellent agreement with the FE simulations for the localized buckling profiles.

Further work on interactive buckling in sandwich panels is continuing by focusing on the combination of independent end moments and axial force as well as investigating the response of sandwich struts with functionally graded cores.

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Application of generalized differential quadrature method to bending analysis of laminated thick plates with mixed boundary conditions

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ABSTRACT

In this paper, the accuracy and convergence of the generalized differential quadrature (GDQ) method in the bending analysis of laminated thick plates with different types of mixed boundary conditions is studied. Of particular interest is the plates with free edges. The governing equations of the problem, assuming first order shear deformation theory (FSDT), include thirteen first order partial differential equations in terms of displacement, rotations, plane forces, bending and twisting moments and shear forces. After discretization of the domain using zeros of the Chebyshev polynomials, the GDQ technique is directly used for both internal and boundary nodes. Results revealed that the method offers similar order of accuracy for both deflection and moments. It is also demonstrated that results for plates with different mixed boundary conditions including free edges show very good agreement with the FEM and also published analytical results without anymore need to modifications in the method or mesh distribution.

Employing First order Shear Deformation Theory (FSDT), the governing equations could be written as 13 partial first order governing differential equations with the all displacement, rotations, plane forces, shear forces and bending and twisting moments as unknowns. Using GDQ technique the first governing equation descretized as follows:

$$
\begin{aligned}
N_{xx}(i,j) =\ & A_{11}\sum_{k=1}^{Nx} W_x(i,k).U(k,j) + A_{12}\sum_{k=1}^{Ny} W_y(j,k).V(i,k) \\
& + A_{13}\sum_{k=1}^{Ny} W_y(j,k).U(i,k) + A_{13}\sum_{k=1}^{Nx} W_x(i,k).V(k,j) \\
& + B_{11}\sum_{k=1}^{Nx} W_x(i,k).\varphi(k,j) + B_{12}\sum_{k=1}^{Ny} W_y(j,k).\psi(i,k) \\
& + B_{13}\sum_{k=1}^{Ny} W_y(j,k).\varphi(i,k) + B_{13}\sum_{k=1}^{Nx} W_x(i,k).\psi(k,j)
\end{aligned}
$$

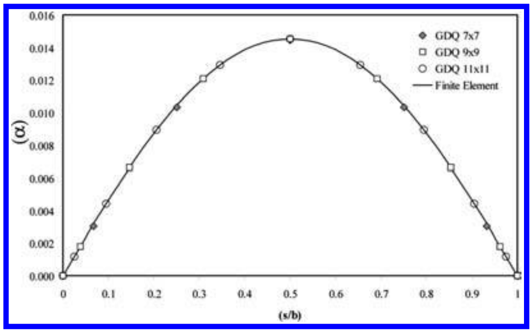

Figure 1. Deflection of SFSF plate analyzed using first order form of the governing equations.

Results of the SFSF boundary condition plate which is carried out by applying GDQ method to the first order (13 equations) form of the partial governing equations is shown in Figure 1 while the FEM results are included. As could be realized from the graphs, both convergence and accuracy is provided for the GDQ results comparing with FEM.

It is demonstrated that the method offers relatively accurate results for both deflection and moments of the plate with the same order of convergence and accuracy. Furthermore, it was shown that the method provides accurate results for plates with different combinations of simply supported, clamped and in particular free boundary conditions with relatively small number of grid points.

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Stress wave propagation in shape memory alloy reinforced rectangular composite plates

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ABSTRACT

In this paper, stress wave propagation in a rectangular composite plate with Shape Memory Alloy fibres embedded in the plate is studied. The spectral element method (SEM) based on the First order Shear Deformation Theory (FSDT) is used [1]. The effect of the SMA wires in the composite plate is taken into account by introducing internal force in fibre directions [2]. Thermal loading is also introduced in the plate. Graphical results for the wave velocity versus the frequency are illustrated. The results are obtained through analytical methods and they are compared with those obtained using the SEM. The influence of the percentage of the Shape Memory Alloy in two directions of the plate on the wave velocity is investigated.

In this paper, the group velocity for different volume fraction of SMA wires in the composite plate for 25 kHz frequency is analytically obtained. The same value for the group velocity in the FEM analysis is approximately obtained.

The comparison of the results obtained from the theoretical method and those from the FEM solution are given in Table (1). The correlations of the two sets of results are very satisfactory.

As it is observed in Figure (1) and Table (1), the addition of SMA wires in one direction, the increase in density overcomes the influence of other parameters, which causes reduction of wave velocity, whereas, in the perpendicular direction increasing stiffness overcomes the influence of other parameters and causes an increase in wave velocity.

Time domain spectral element method is used for modeling and simulation of rectangular fibre-reinforced laminated composite plate with embedded shape Memory Alloy wires. The effects of SMA wires are taken into account as in-plane internal forces in the motion equations. The problem is also solved by a theoretical method by which group velocities of wave propagation is calculated and the two sets of results correlate very well.

Table 1. Comparison of theoretical and FE methods for group velocity for $V_{sx} = 0$.

	C_{gx}		C_{gy}	
V_{sy}	FEM	Theory	FEM	Theory
0	1137.40	1144.58	1051.71	1078.44
0.25	1403.32	1433.80	1024.02	1033.33

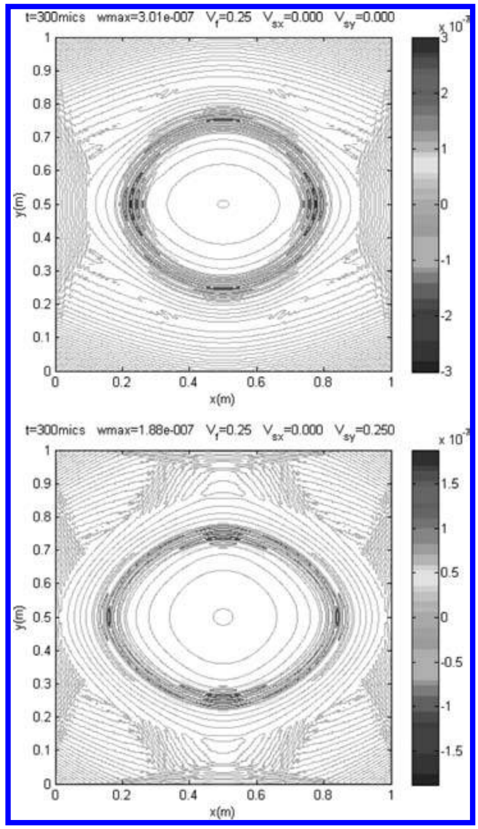

Figure 1. The influence of the SMA wires on the wave propagation for $V_{sx} = 0$.

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Residual capacities of uniaxially compressed composite panels under buckling driven delamination

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ABSTRACT

Fibre-reinforced composites are increasingly being used in many engineering applications owing to their high strength to weight ratio. Despite their advantageous in-plane material properties, laminated composites usually lack performance perpendicular to the fibre orientation owing to their relatively weak inter-laminar interfaces. Thus delamination, or layer interface cracks, may arise, which reduce a component's load carrying capacity and stability considerably (Chai & Babcock, 1985). Delamination propagation can, moreover, lead to dangerous consequences if the growth is uncontrolled or unstable; it is therefore imperative to account for it to maintain design safety. Many applications in structures, particularly in the aerospace industry, contain thin rectangular plated panels subject to in-plane stresses, and hence the current investigation focuses on a composite plate with an embedded pre-existing delamination. Since the current design procedures in aircraft construction do not allow for growth to occur, the structure may not be exhausted to its possible limits; hence, it may be desirable to find threshold parameters for a safe design scenario; an attempt to identify such parameters is presented herein.

The model presented is nonlinear, such that large deflections are included which allows critical and postbuckling behaviour to be analysed. The model is formulated using potential energy principles in conjunction with the Rayleigh–Ritz method that has been successfully employed in previous studies (Kim & Kedward, 1999; Hunt *et al.*, 2004). A cohesive zone model is also introduced for the delaminated region to include its growth. Linear eigenvalue analysis shows that the critical buckling loads decrease with the delamination size and that they also vary with the delamination depth. Additionally, different modes, *i.e.* closing or opening, can be identified depending on the parametric configuration. Different modal intensities are also identified, *i.e.* local, mixed or global, depending on the parametric configuration. This change can be observed to occur in the neighbourhood of a transitional depth of delamination ctwhich is located approximately 10% from the top of the panel.

For the current system, nonlinear analysis reveals that thin-film and mixed mode buckling lead to delamination growth and potentially to premature overall structural instability depending on the configuration. Buckling-driven delamination growth hence only occurs where the pre-existing delaminations are closer to, or precisely located at, depth ct. However, from investigations for a particular panel with material properties taken from Turon *et al.* (2007), it is concluded that a maximum permissible delamination size must be smaller than a particular value or contained within certain boundaries to avoid complete debonding of the laminates due to progressive damage. It is also found that growth commences when the ratio of the delamination length to the overall length of the panel is larger than or equal to the ratio of the delamination depth to the overall thickness. Moreover, it is established that thin-film buckling leads to stable delamination growth and mixed mode buckling to unstable growth. With the criteria presented, a panel can be sized and a level of tolerance may be introduced that could allow for an acceptable delamination to exist in-service. It should be noted, however, that the results presented incorporate a unique fracture mode, which has been shown to dominate where thin-film or mixed mode buckling is triggered (Rhead & Butler, 2009), and only allow for uniform growth of the delaminated patch. Further work is being conducted to establish whether direction specific growth would significantly affect the mechanical response.

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Effect of distributed attached patches on fundamental natural frequency of thick cylindrical and spherical shells using higher-order shell theory

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ABSTRACT

In many engineering applications such as aerospace, finding the vibrational response of the system is important and many investigators focused on this field. In many cases, some substructures are added to the faces of the shell. These substructures can be some mass-points, patches or even a layer which are added to the main system. These substructures affect the response of the system and understanding these effects is important in designing and applications.

Nowadays, weight sensitive engineering applications are widely used in many industrial structures. So, fiber-reinforced materials in the form of shells are so common in these applications. Some common types of the composite shells are in the form of cylindrical, spherical and doubly curved shells with two distinct and constant radii of curvatures. Various theories for shells are proposed based on the displacement field. Classical and First-order shell theories are such theories. These two theories reflect a high percentage error in the cases of thick composite shells with stiff facings or distributed attached masses.

The use of higher-order shear deformation theory (HOST), which includes a realistic parabolic variation of transverse shear stress through the laminate thickness and warping of the transverse cross-section, is very important for the free vibration analysis of laminated composites. Shell theory used in this paper is also the higher order shell theory with 12 independent variables. Although, so many publications are available in the cases of mass points added to the beams, rods, plates so few investigations are available in the case of a distributed attached mass added to the plates. However, none of them included the stiffness of distributed attached mass. In this paper, free vibrations of cylindrical and spherical shells carrying a distributed attached patch on the top face are investigated. In this paper, the stiffness effect of the patch is also included.

Results obtained for the structures with and without a distributed attached patch are compared with related references. Some parameters which influence the natural frequency of the structure are investigated in this study.

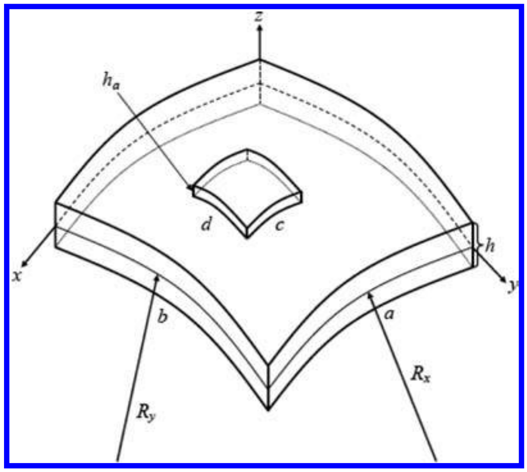

Figure 1. Uniform thickness laminated composite curved shell with a patch.

These parameters are included the thickness of the patch, location of the patch, variations of density and elasticity moduli of the patch.Thicker patches affect the natural frequency more. Patches located near the center of the shell, affect the fundamental natural frequency more, as compared to those which are located near to the edges. By increasing the elasticity moduli or decreasing the density of the patch, its effect on natural frequency is increased. Variations of forgoing parameters are presented analytically in this paper, via derived tables and plotted figures.

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A new triangular buckling pattern of twisted inextensible sheets

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ABSTRACT

When twisting a strip of paper or acetate under high longitudinal tension, one observes, at some critical load, a buckling of the strip into a regular triangular pattern (see Figure 1(a)). The deformation is reversible and the pattern, consisting of helically stacked nearly-flat triangular facets, seems to be nature's way of achieving global twisting by means of local bending and minimal stretch. There appears to be no record of this buckling pattern in the literature.

Very similar triangular facets have recently been observed in solutions to a novel set of geometrically-exact equations describing the equilibrium shape of thin inextensible elastic strips. An example is the equilibrium shape of a Möbius strip, obtained using these equations, shown in Figure 1(b). Here we formulate a boundary-value problem for these equations that 'cuts out' this triangular (more precisely, trapezoidal) region and use symmetry to construct a periodic pattern in good agreement with experiment (see Figure 1(c)).

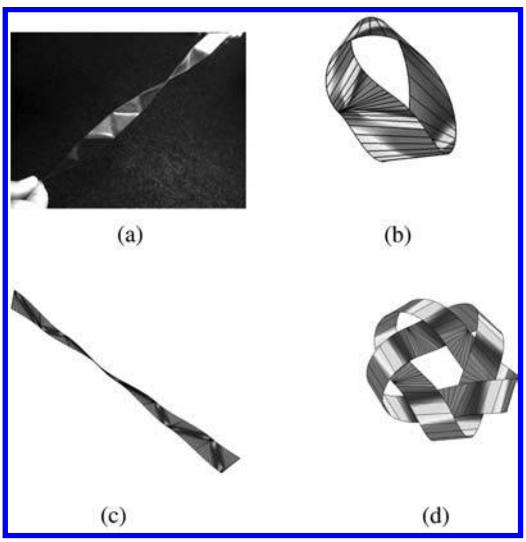

Figure 1. Shapes of strips. (a) Acetate model strip. (b) Möbius strip. (c) Computed twisted-strip solution with $n = 8$. (d) Example of an elastic ribbon knot.

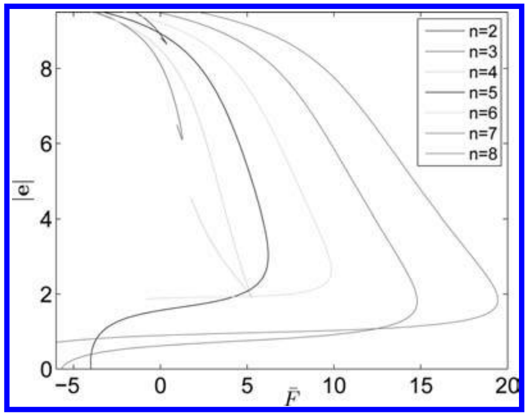

Figure 2. Force-extension curves at constant twisting moment showing fold instabilities under increasing force.

We also study the force-extension and moment-twist behaviour of these strips by varying the mode number n of triangular facets; see Figure 2, where parts of the curves with positive slope are expected to be stable. Our results predict that, for $n \geq 5$, under increasing tension solutions jump to a higher mode as the force is increased beyond the folds.

By applying periodic boundary conditions closed strips may be obtained, including knotted ones. We find a whole family of torus knots (see example in Figure 1(d)). An essential feature of one-sided inextensible sheets turns out to be the existence of (an odd number of) singular points of infinite bending energy density. The location of these singularities may be of interest for graphene and other ribbons currently being fabricated and studied in nanotechnology.

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Initial curvature effects on the behaviour of cylindrical shell structures

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ABSTRACT

Thin and thick shells as structural elements occupy a leadership position in civil, mechanical, architectural, aeronautical and marine engineering, since they give rise to optimum conditions for dynamic behaviour, strength and stability. These structures support applied external forces efficiently by virtue of their geometrical shape [1]–[3].

The present work is based on the FSDT including initial curvature effect. The governing equations are put into generalized displacement form by the use of strain-displacement relationships and constitutive equations. Numerical applications refer to a moderately thick cylinder and the effects of transverse shear deformation are taken into account.

The basic configuration of the problem considered here is a circular cylinder of uniform thickness h. The co-ordinates along the meridian and circumferential directions of the reference surface are x and s, respectively (Fig. 1). The distance of each point from the shell mid-surface along the normal is ζ.

Considering the curvature effect in the Reissner-Mindlin theory [1], more elements are added to the fundamental system of equations. It should be noted that the study of initial curvature effect on the behaviour of cylindrical shell structures is the main novelty of this paper.

An analytical solution is found considering an axial-symmetric circular cylinder. So, the circumferential displacement u_s and the rotation around x-axis can be neglected. Also, the derivatives with respect to the circumferential coordinate are zero. In the end, it is considered the static case and $q_x = 0$, and the meridian displacement u_x is negligible. In this paper two different type of boundary conditions are developed: 1) both clamped (C-C) and 2) clamped-free (C-F).

As loading condition, a uniform pressure on a circular cylinder is considered. The structure is made of a homogeneous and isotropic material and the numerical results are given in table forms like table 1.

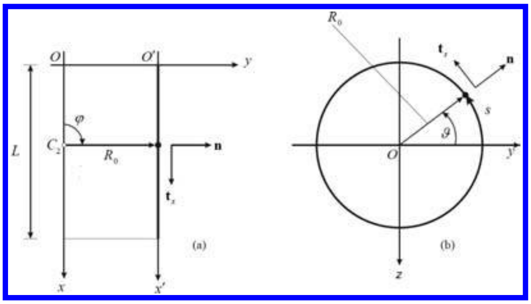

Figure 1. Circular cylindrical shell: meridian section (a); parallel section (b).

Table 1. Results for (C-C) cylinder with $h = 5$ mm.

R_0 (mm)	50	500
h/R_0	1/10	1/100
R_0/L	0.05	0.5
h/L	0.005	0.005
w (CET)	2.1199	128.0947
w (FSDT)	2.1217	128.0953
difference	−0.0018	−0.0006
T_x (CET)	31688.5342	291072.9071
T_x (FSDT)	31701.7350	291073.7929
difference	−13.2008	−0.8858

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Effects of varying thickness on the buckling and post-buckling behavior of thin-walled cylindrical steel shells subjected to combined external pressure and axial compression

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For economic reasons, large fluid storage tanks are made with varying thickness in height. In this study, non-linear buckling and post-buckling behaviour of thin-walled cylindrical steel shell with varying thickness are obtained by using ANSYS, which is a general-purpose finite element program designed specifically for advanced structural analysis. To this aim, material and geometrical nonlinear analysis should be undertaken. To trace the equilibrium paths through limit points into the post-critical range, the 'Arc-Length-Type Method' have been used which is the most efficient method for this purpose. Because of imperfection sensitivity behavior of shell structures, geometric imperfections are considered in the numerical models. In order to verify the accuracy and validity of the finite element modeling, the numerical results, have been compared with the results of available experimental data [1]. Figure 1 illustrates comparisons of the general shape and external pressure–radial displacement responses for the specimens.

It has been found that the finite element models is reliable enough to be used to undertake nonlinear analyses. When a large liquid storage is discharged, an internal suction will be applied to its wall and head. The reaction of suction that exert to head, press the wall in axial direction. In this investigation, instability of cylinders with three different thicknesses subjected to external pressure, axial compression and combined loading is studied (Figure 2). Edge support conditions are provide similar to actual tanks, fixed

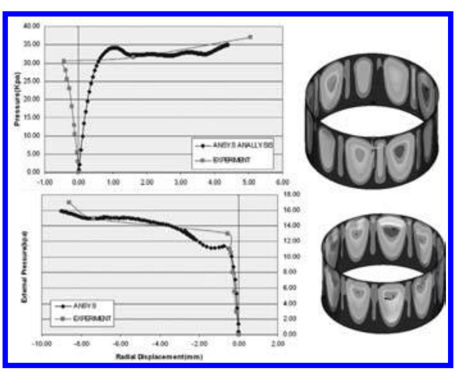

Figure 1. The experimental and numerical external pressure–radial displacement responses.

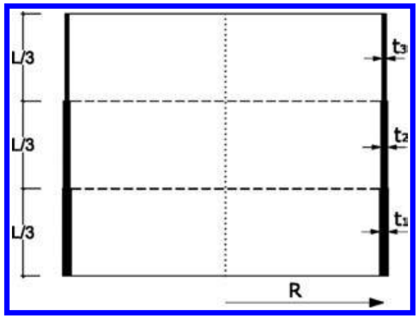

Figure 2. Schematic elevation of a typical specimen.

Table 1. Geometrical dimensions of models.

Model	L/R	R/t_1	R/t_2	R/t_3	t_2/t_1	t_3/t_2
1	0.93	750	833	1875	0.90	0.44
2	0.93	750	938	1875	0.80	0.50
3	0.93	750	1071	1875	0.70	0.57
4	0.93	750	1250	1875	0.60	0.67
5	0.93	750	1500	1875	0.50	0.80

for lower edge and radial resistance for upper edge. Because of symmetry, a quarter of cylinder is modeled. For nonlinear analysis, initial imperfection in form of modal buckling shape applied to each model. Amount of imperfection is assumed percentage of upper part thickness of cylinders. Geometrical dimension of models has been shown in table 1. Amount of imperfection is assumed equal to $0.1t_3$, $0.3t_3$ and $0.5t_3$. These values are in permissible of API code. For realizing the real behavior of the models, a slight imperfection, in the form of modal buckling shape, is applied to them.

The study on the buckling and post-buckling behavior of thin-walled cylindrical steel shell with varying thickness showed that thickness variation has direct influence on the location of final waves in general buckling stage.

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Cylindrical bending of thick isotropic plates using trigonometric shear deformation theory

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ABSTRACT

A trigonometric shear deformation theory for cylindrical bending of thick isotropic plates taking into account of transverse shear deformation effect is presented in this paper. The displacement field of the theory uses sinusoidal function in terms of thickness coordinate to include shear deformation effects. It has only two variables. It is to be noted that the present theory satisfies the constitutive relations in respect of transverse shear strain and stress. The transverse shear stress can be obtained directly from constitutive relation, satisfying the shear stress free boundary conditions at top and bottom surfaces of the plate. Hence, the theory requires no shear factor. Governing equations and boundary conditions of the theory are obtained by using principle of virtual work. The results from closed-form solution obtained for inplane and transverse displacements and bending and shear stresses for three load cases, *viz;* single sine load, uniformly distributed load, and concentrated load are presented in this paper. The results are compared with those of classical plate theory, first order shear deformation theory, higher order and other refined plate theories to validate the accuracy of the theory.

The displacement field of the theory is:

$$u = -z\frac{\partial w}{\partial x} + \frac{h}{\pi}\sin\frac{\pi z}{h}\phi(x)$$
$$w = w(x)$$

where u and w are the inplane and transverse displacements in x and z directions respectively. ϕ is an unknown function of x only and is associated with the shear slope at neutral plane. The variationally consistent governing differential equations (Euler-Lagrange) and the associated boundary conditions of the theory are obtained using principle of virtual work.

Table 1. Comparison of non-dimensional maximum transverse displacement $\bar{w}$ for aspect ratio, $S = 10$ at ($x = 0.5L$, $z = 0$).

Source	Model	SSL	UDL	CLL
Present	TSDT	1.153	1.461	2.351
Reddy	HSDT	1.153	1.461	2.350
Kirchhoff	CPT	1.121	1.422	2.275
Mindlin	FSDT	1.153	1.461	2.352

The closed form solution, satisfying the governing equations and boundary conditions exactly, is used for the bending analysis of plate. The typical results obtained for transverse displacement from this solution are shown in Table 1.

The major conclusions drawn from the flexural analysis of plate subjected to cylindrical bending with different loading cases are as follows:

1. The results of inplane displacement, transverse displacement and stresses obtained by present theory are in excellent agreement with those of higher order shear deformation theory.
2. Theory is capable of predicting the effects of stress concentration due to concentrated loads acting on the plate.

Thus the validity of present trigonometric shear deformation theory is established for the static flexural analysis of thick plate subjected to cylindrical bending.

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Free vibration of thick isotropic plates using trigonometric shear deformation theory

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ABSTRACT

In this paper results of frequency of free vibration of thick isotropic plate using trigonometric shear deformation theory are presented. The displacement field of the theory uses sinusoidal function in terms of thickness coordinate to include shear deformation effects. It has only three variables. The theory is as simple as first order shear deformation theory. The present theory satisfies the constitutive relations in respect of transverse shear strain and stress. The theory satisfies the shear stress free boundary conditions at the top and bottom surfaces of the plate. Hence theory obviates the need of shear correction factor. Governing equations and boundary conditions of the theory are obtained by using dynamic version of principle of virtual work. The results obtained for natural frequencies of flexural mode and thickness shear mode are compared with those of classical plate theory, first order shear deformation theory, higher order shear deformation theories and the exact theory. The results for natural frequencies in bending mode and shear mode are presented in the nondimensional form in this paper. The present displacement based theory is variationally consistent. The number of unknown functions involved in the theory is three, similar to Mindlin's theory. The theory assumes displacements such that transverse shear stress variation is realistic. The theory obviates the need of shear correction factor. The frequencies obtained using TSDT are in excellent agreement with those of exact theory.

The Displacement field

The displacement field of the present refined plate theory for free vibration is as follows:

$$u(x,y,t) = -z\frac{\partial w}{\partial x} + \frac{h}{\pi}\sin\frac{\pi z}{h}\phi(x,y,t)$$

$$\boxed{\begin{aligned} v(x,y,t) &= -z\frac{\partial w}{\partial y} + \frac{h}{\pi}\sin\frac{\pi z}{h}\phi(x,y,t) \\ w(x,y,t) &= w(x,y,t) \end{aligned}}$$

Table 1. Comparison of non-dimensional natural predominantly bending mode frequencies $\overline{\omega}_w$

m	n	TSDT	Reddy	FSDT	Shimpi	Exact
1	1	0.0930	0.0931	0.0930	0.0930	0.093
		(−0.22)	(−0.11)	(−0.22)	(−0.22)	(0.00)
1	2	0.2219	0.2222	0.2219	0.2220	0.222
		(−0.32)	(−0.18)	(−0.32)	(−0.27)	(0.00)
1	3	0.4151	0.4158	0.4149	0.6525	0.417
		(0.20)	(−0.31)	(−0.53)	(−0.48)	(0.00)
2	2	0.3406	0.3411	0.3406	0.3406	0.342
		(−0.44)	(−0.29)	(−0.44)	(−0.44)	(0.00)
2	3	0.5210	0.5221	0.5206	0.5208	0.523
		(−0.55)	(−0.34)	(−0.63)	(−0.59)	(0.00)
2	4	0.7457	0.7481	0.7446	0.7454	0.751
		(−0.72)	(−0.40)	(−0.87)	(−0.76)	(0.00)
3	3	0.6842	0.6862	0.6834	0.6840	0.688
		(−0.68)	(−0.39)	(−0.80)	(−0.71)	(0.00)
3	4	0.8913	0.8949	0.8896	0.8908	–

The numerical results obtained for natural frequencies of square plates $(b/a = 1)$ with $h/a = 0.1$ are represented in Table 1.

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Dynamic response of a circular plate resting on a tensionless Vlasov foundation

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ABSTRACT

Plates supported by elastic foundations present very common technical problems in structural and geotechnical engineering. These problems are usually analyzed by assuming that the foundation (one- or two-parameter) reacts in compression as well as in tension. However, it is well known that the foundation sometimes cannot provide a tensile reaction, and under some conditions, some parts of the plate may lift off. Therefore, a tensionless foundation, in which the reaction is only compressive, is modeled. The case of plates on two-parameter foundations has received less attention because of the model complexity and the difficulties in estimating the parameter values. Recent studies in this field include those of Güler (2004), Yu et al. (2007), and Celep & Güler (2007). In this study, the axisymmetric dynamic contact response of a circular plate resting on a two-parameter tensionless elastic foundation and subjected to a concentrated harmonic load is investigated. The elastic foundation is assumed to be of the modified Vlasov type, in which the material properties of the soil layer are used to compute the coefficients (K, C) of the subgrade reactions. These coefficient depend on the value of the mode shape parameter γ, which is obtained by using an iterative method. The tensionless character of the foundation results in the creation of lift-off regions between the plate and the foundation. Although there is no nonlinear term in the equations, the problem appears to be nonlinear because the contact region (radius) is not not known in advance. Closed-form solutions of the differential equations of motion in each of the contact and noncontact regions are determined. The boundary and continuity conditions are then satisfied, which leads to a system of algebraic equations that are linear in certain unknown coefficients and nonlinear in the unknown contact radius. Eliminating the linear coefficients, the contact radius of the plate can be determined from the resulting transcendental equations. It was found that the contact radius decreases and displacement increases as the forcing frequency increases. However, for a fixed value of the forcing frequency, the contact radius increases as the depth of the soil layer increases. It should be noted here that the coefficient K decreases and C increases as the depth of the foundation increases. When the second parameter (C) in the foundation model is not taken into consideration (Winkler foundation), the contact radius and the displacements become larger.

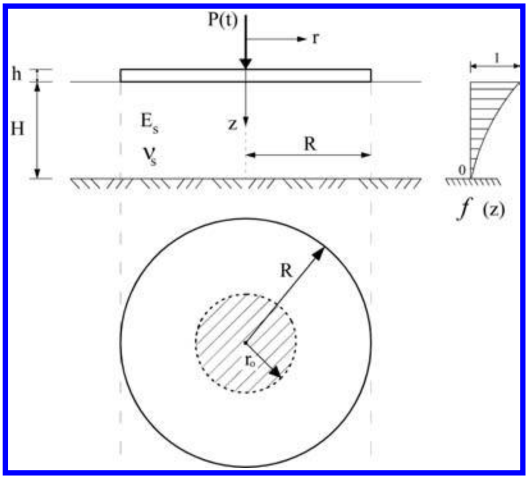

Figure 1. Circular plate-foundation system.

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Leakage evaluation in longitudinally cracked pressurized pipes

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ABSTRACT

Due to aging and deteriorating of water distribution systems, leakage is often the principal cause for water losses (Farley 2001). As it is well known, one of the main factors affecting leakage is the pressure in the distribution system. A classical correlation between leakage and pressure is given by the orifice equation, which assumes the flow rate proportional to the square root of the pressure head, but several studies showed that leakage in water distribution systems can be much more sensitive to pressure. One of the main reason for this is the pipe deformation, which is not accounted for by the orifice equation. Longitudinal cracks, for example, show expansion under pressure, which increases the leakage area (Ilunga et al. 2008). An extensive experimental investigation, involving different pipe materials and crack geometries, which highlights the effect of the pipe deformation on the leakage can be found in Greyvenstein 2004. On the other hand, a simple and reliable methodology to determine the relationship between pressure and leakage is a crucial issue for Water authorities in view of water loss control strategies in large water reticulation networks.

This paper presents a simple model for the estimation of the leakage area in a pressurized pipe affected by a longitudinal crack. The pipe is modelled based on the theory of beams with elastic constraints. In particular, in the cracked part of the pipe, it is assumed that the elastic constraints respond to the displacements in the radial and tangential directions and to the rotation over the longitudinal axis. The stiffness of the constraints is given by the transversal rings. The model predictions are compared with three dimensional finite element simulations for different crack lengths and diameter ratios. Numerical results show that the model, notwithstanding its simplicity, allows a reliable evaluation of the radial and tangential displacements in the cracked zone of the pipe (directly related to the leakage area) in

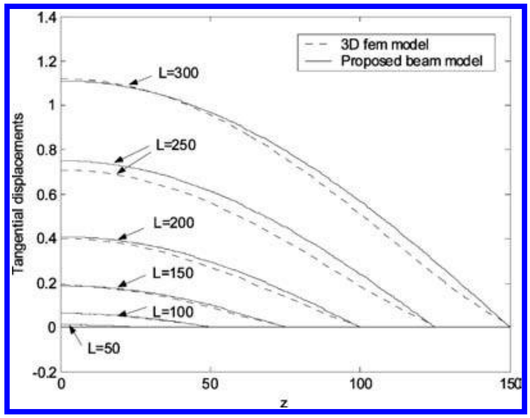

Figure 1. Tangential displacements for different crack lengths.

longitudinally cracked pressurized pipes. In Figure 1 the tangential displacements for different crack lengths are reported and compared with a 3D fem analysis. As it can be noted there is a good agreement of the simple model developed and the fem analysis.

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8. Thin-walled sections, steel structures, steel connections, steel-concrete composite structures

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The mechanics of post-local buckling and overall bending interaction in thin-walled I-section compression members

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ABSTRACT

The structural performance of thin-walled compression members is governed, essentially, by the effects of buckling and post-buckling behaviour. There are a number of possibilities with regard to the mode of buckling that can occur and this would depend, of course, on the geometrical details of the member cross-section, the member length, end conditions with regard to support and loading and also the member structural material.

The buckling mode is characterised, in essence, by half-wavelength such that local buckling of the section walls is associated with a short half-wavelength, distortional buckling by an intermediate half-wavelength and torsional, flexural and torsional-flexural buckling are the overall modes at the longer half-wavelengths. Clearly, interaction is possible and coupled instabilities can be encountered in design whereby local-distortional buckling can occur (Schafer 2002), (Dinis et al 2007), or, indeed, the interaction between local buckling and overall flexural behaviour may be of influence (Loughlan & Howe 1987), (Loughlan 1993).

In this paper, finite element simulation is employed to examine the post- buckled response of thin-walled I-section struts and columns giving due consideration to the influence of geometric imperfections and to elasto-plastic material behaviour.

The findings from this work highlight the complete loading history of the compression struts from the onset of initial compressional loading through the non-linear elastic and elasto-plastic phases of behaviour to final collapse and unloading.

The influence of overall column bending is illustrated in the paper and some results are presented for the longer length compression members which highlight the interactive post-buckling response of the I-section columns.

The compressive failure of the thin-walled I-section struts is found to be associated with considerable material yielding and yield propagation prior to ultimate conditions and then to an elasto-plastic unloading phase of behaviour. It is shown that there is a steady reduction in post-buckled compressional stiffness under elastic conditions due to change in the post-buckled shape as loading progresses. It is clear, also, that material nonlinearity has a significant effect on the post-buckled stiffness and this is particularly the case for near simultaneous buckling and yielding designs.

For longer columns, the weakening effects of local buckling have been shown to result in reduced global elastic Euler buckling loads and the influence of geometrical imperfections and material nonlinearity have been shown to be of significance in further reducing the ultimate carrying capability of the columns. In particular, it has been shown that for longer columns whose local buckling stress is close to the material yield stress, geometrical imperfections can result in ultimate load levels which are somewhat less than the elastic local buckling load of the members.

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Restrained distortional buckling and postbuckling behavior of steel-concrete composite beams under hogging bending

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ABSTRACT

Composite beams may generally be used in a simple construction or in recent years more frequently in continuous or semi-continuous structural systems. In case of a simple construction, composite beams are simply supported and subjected to sagging bending only. The concrete part of the composite section of such beams is under compression and the unrestrained bottom flange of their steel section is under tension. In contrast, continuous or semi-continuous structures are subjected to both sagging and hogging bending; therefore the concrete slab of the composite beams is exposed to cracking and the bottom flange of steel I-section to RDB phenomenon. The latter aspect in relation to the behavior of castellated composite beams is considered in the paper in terms of experimental investigations, numerical modeling and analytical formulation.

Beams tested experimentally consisted of two equal spans, one castellated and the other one plain webbed. The summary of tested specimens is given in Table 1.

Considered different shapes have the same opening area and equal c/c spacing. Specimens were loaded in such a way that the entire tested specimen was under hogging bending as shown in Figure 1 for a circular composite beam.

Table 1. Summary of tested castellated composite beams.

	Span		Openings	
Specimen	Length* mm	Steel –	No. –	Type –
C4S355	2116	S355	4	Circular
C4S420	2116	S420	4	Circular
H4S355	2116	S355	4	Hexagonal
H4S420	2116	S420	4	Hexagonal
R4S355	2116	S355	4	Rectangular
R4S420	2116	S420	4	Rectangular
C2S355	1058	S355	2	Circular
C2S420	1058	S420	2	Circular
H2S355	1058	S355	2	Hexagonal
H2S420	1058	S420	2	Hexagonal
R2S355	1058	S355	2	Rectangular
R2S420	1058	S420	2	Rectangular

*Beam length is equal to twice the span length.

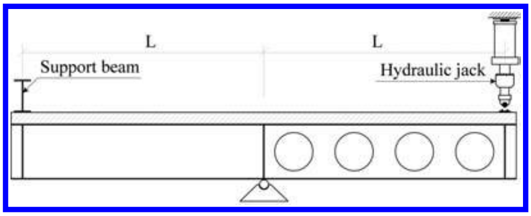

Figure 1. Boundary conditions and applied load.

The FE program ABAQUS of version 6.6 is used to simulate the behavior of tested composite beams. The simulation consists of two steps. In the first step, an elastic buckling analysis, also known as a linear perturbation analysis, is performed on a perfect steel-concrete composite beam model to obtain its buckling modes (eigenmodes). In the second step, a general non-linear analysis is carried out for imperfect composite beam models using Riks method. Material plasticity, concrete cracking and crushing, material imperfections due to welding and geometric imperfections, obtained from the previous step, are included in this analysis in order to obtain the ultimate load, failure modes and the postbuckling behavior of the tested composite beams. The FE results show a good agreement with the test data in tracing the load-deflection path and in capturing all the deformation details of laboratory tested beams.

The closed form solution proposed by Salah & Gizejowski (2008) for the estimation of distortional buckling strength is verified and proven to be a general formulation for both plain webbed and castellated composite beams.

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Reliability-based methodology for the design of support formwork systems by advanced structural analysis

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ABSTRACT

The paper provides an overview of an ongoing research program into the direct design of steel scaffold systems by advanced geometric and material nonlinear analysis. The systems studied are used to support formwork and the weight of wet poured concrete, also referred to as support formwork (scaffold) systems. The research program encompasses the collection and statistical evaluation of field data of member crookedness and frame out-of-plumb, as well as full-scale tests of 3 × 3 bay subassemblies and a comprehensive series of tests on joints to determine statistical distributions of the strength, semi-rigid stiffness and looseness of the joints. Advanced finite element models are calibrated using the full-scale subassembly tests, thereby providing statistical data for the modeling error. Having procured the statistical data for the main random variables affecting the strength of support formwork systems, Monte Carlo simulations are carried out to determine the statistical distributions of strength for a range of geometries of support formwork systems. A trend is emerging from these studies suggesting that the failure mode is mainly determined by the extension of the jacks at the top and bottom of the frame as well as the lift height. The first-order reliability method is employed for calculating the system resistance factor for support formwork systems, leading to the formulation of a design method for support formwork systems based on advanced analysis, which requires no recourse to a steel structures design specification for checking member and connection strengths.

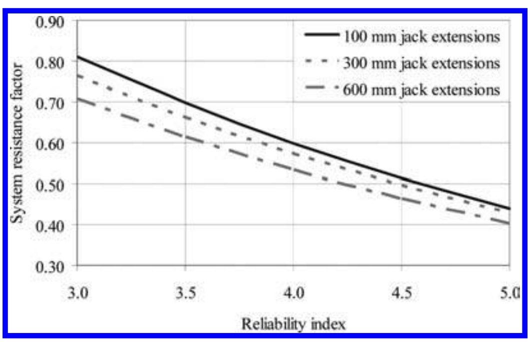

Figure 1. System resistance factor for a 1.0 m lift support formwork system.

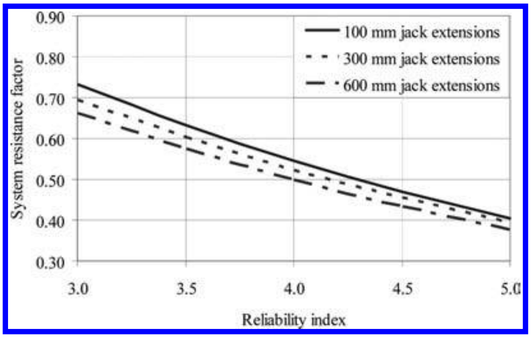

Figure 2. System resistance factor for a 1.5 m lift support formwork system.

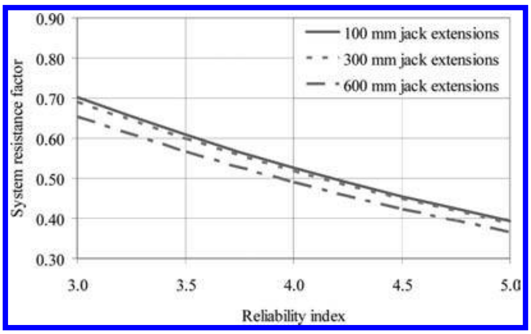

Figure 3. System resistance factor for a 2.0 m lift support formwork system.

The system resistance factors for system reliability indices in the range of 3.0–5.0, as obtained from an FORM reliability analysis for different system configurations, are presented in Figures 1–3. From the figures, the system resistance factor φ_{system} can be as low as 0.37 if the system target reliability index is chosen as 5.0, and as high as 0.81 if the system target reliability index is chosen as 3.0. In addition, longer jack extensions and longer lift heights lead to lower system resistance factors at the same system target reliability index.

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Own theory for thin-walled straight bars: Development and investigations

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ABSTRACT

The theory was formed about 1980, written in form of the book in Polish, by Obręebski 1991. The second edition was in 1999 and now is prepared its third issues (In Polish and than in English). In that shape theory is very general, oriented on bars with open, closed and open-closed cross-sections. There is included theory of first and second order (in it stability), static, dynamics and dynamical-stability of the bars. Moreover, there is taken into consideration interaction of the bar with surrounding media as gas and fluid (e.g. air and water). The media can have certain velocity (wind). There, were included some original approaches to numerical analysis of large space bar structures (including ONE exact finite element for thin-walled bars with any boundary conditions for some kind of 3D analyses- static, dynamics, stability). The theory of second approximation was proposed, too – for improving accuracy of calculations.

In the beginning, the given examples were done in analytical way, only. Therefore, the class of possible solutions was rather limited for the reason of mathematical difficulties, and properties of e.g. hyperbolic functions (hsin x & hcos x; $x<235$!!). So it was strong necessity to find certain methods for extending practical range of solutions on new tasks – much close to engineering practice reality. This way were step by step found new methods of solution:

– the Finite Difference Method (FDM) was used for determination of internal forces in the bars,
– the 3D-Time Space Method with combination of FDM was successfully applied to very complicated tasks for dynamics with any program of loading, (see Obręebski SEMC 2004),
– extension of the possible cross-section on full type – regarded as closed, composite tubes located each into the other Obręebski 1997 to 2009,
– applying above numerical approaches, we can consider simultaneously very general set of four equations, what bring us to much more exact solutions than by other methods including FEM.

The new solutions were concerning applications for: dynamical behavior of tall buildings, bridges and airstrips in airport etc.

For recognition of real accuracy of the considered theory, were performed whole series of serious laboratory tests. They concern comparisons of results obtained in analytical way, Obrêbski1991, numerically, experimentally and by FEM. So, it shows, that good theory can bring us to the results similar as by FEM or even better. Simultaneously such experiments show many real behavior of thin-walled bars, never seen up to the moment in literature.

At last, the development of theory for thin-walled bars permit on observations of the solution methods: from analytical supported by calculators, through own programs, application of known professional commercial programs, including different Math-CADs to use of MS Excel. Some experiences in this domain are mentioned too.

The full list of authors and other references associated with elaboration of the theory can be found in two quoted works of Obręebski 2008 & 2009.

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Iterative inelastic buckling method for estimating collapse loads of steel cable-stayed bridges

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ABSTRACT

This paper proposes a new method for estimating the collapse loads of steel cable-stayed bridges. Based on the fundamental concept of the inelastic buckling analysis previously established by the authors [1, 2, 3], we widen the range of application for the method by suggesting a new criterion of each structural member in the bridge system. The proposed method determines the tangent stiffness of each structural member in the bridge system by iterative eigenvalue computations with the classical tangent modulus theory. In addition, an improved convergence criterion for girder and tower members is proposed to take into account the beam-column interactions. After summarizing theoretical substances, we analyze the example bridges that have center spans of 600-m, 900-m, and 1200-m with different girder depths. To show the validity and applicability of the method, the results of the proposed method are compared with those of the established inelastic buckling analysis and a nonlinear inelastic analysis. Some discussions are also made about the effect of the girder depth on the collapse load and the failure modes of the example bridges.

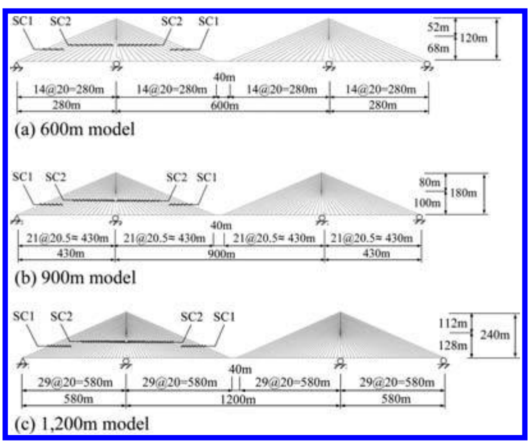

Figure 1. Example cable-stayed bridges.

Table 1. Collapse load factors of example bridges.

		Inelastic buckling analysis		
Model	D(m)	Column criterion	Proposed criterion	Nonlinear elasto-plastic analysis
600 m	1	3.11	2.77	2.67
	2	3.90	3.24	3.15
	3	4.00	3.02	3.19
	4	3.94	2.97	3.04
	5	3.87	2.93	3.15
	6	3.81	2.86	3.10
900 m	1	1.78	1.71	1.69
	2	2.35	2.16	2.23
	3	2.57	2.31	2.50
	4	2.71	2.37	2.47
	5	3.10	2.50	2.56
	6	3.04	2.47	2.53
1200 m	1	1.22	1.18	1.17
	2	1.82	1.78	1.65
	3	2.01	1.86	1.83
	4	2.07	1.95	2.02
	5	2.15	1.98	2.03
	6	2.22	1.92	2.07

Table 1 summarizes collapse load factors of example bridges explicitly with respect to analysis methods, girder depths and center spans for the sake of completeness.

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Local buckling of cold-formed thin-walled channel beams with drop flange

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ABSTRACT

This work is devoted to thin-walled channel beams with open and close drop flanges, names DFB – Drop Flange Beam. The local elastic buckling of these beams in pure bending state is investigated. The study includes simple analytical description and calculations, numerical analysis with the use of the Finite Element Method (FEM), the Finite Strip Method (FSM) and the laboratory tests of beams. Results of these investigation methods are collected, compared and presented in table and figures (Tab. 1). The buckling problems for flanges and webs of thin-walled beams are described in detail. The bend shape of the free edge of the flange significantly affects the critical load of the local buckling. Effective design problem of thin-walled beams is formulated.

The demand for thin-walled structures has been increasing for many years. It results in the development of theoretical basis and experimental investigations of those structures.

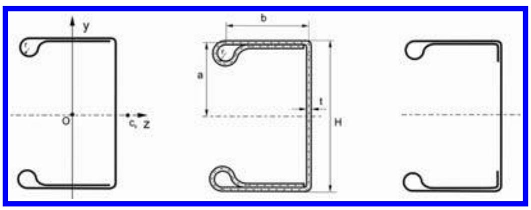

Figure 1. The investigated cross-section of beams.

Table 1. The results of analytical, FSM, FEM and EXP investigation (critical force).

Beams Investigation	Closed DBF [kN]	Open DFB [kN]
Analytical	12.77	3.91
FEM	13.50	3.69
FSM	13.06	3.74
Experiment (EXP)	11.50	3.77

The number of works devoted to the theory of thin-walled structures has been steadily growing in recent years. Materials that are used for building modern structures make it possible to reduce their weight for heavier loads. Cold-formed thin-walled beams tend to buckle locally due to the high ratio of the transverse dimensions to the wall thickness. Similar investigations have been conducted by Pastor and Roure (2008), Camotim et al (2009) who has considered two loading conditions. He has proposed formulas which make it possible to determine the critical load of beams including interaction between global and local stability. The results of experimental investigations of thin-walled beams have been presented by Paczos, Magnucki and Wasilewicz (2009), Cheng Yu et al (2007), Mahendran and Jeyaragan (2008).

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Performance of K-braced cold-formed steel shear walls subjected to lateral cyclic loading

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ABSTRACT

The use of Light-weight Steel Framed structures, LSF, has grown dramatically in recent years all over the world. Compared to other building technologies, such as hot-rolled steel frame buildings, the main advantages are reduction of cost, being light weight, and higher speed of construction. Moreover, LSF structures often are of very high quality as they undergo strict quality control in the factories.

In this study, the performance of different configuration of K-Brace cold-formed steel shear walls is evaluated by testing twelve full scale walls of 2.4 m × 2.4 m under cyclic loading. One specimen included concurrent K-bracing and Strap bracing systems, in order to investigate the compatibility of these two systems. The effects of different components such as: different locations of K-elements, number of K-elements and using double studs, are monitored and investigated in this research by changing them from one specimen to another specimen. Of particular interest are the specimens maximum lateral load capacity and deformation behavior. The study also looks at the failure modes of the system and investigates the main factors contributing to the ductile response of the LSF walls in order to suggest improvements so that the shear steel walls respond plastically with a significant drift and without any risk of brittle failure, such as connection failure or stud buckling.

Based the observation during the tests, the common failure mode for most specimens was plastic local buckling in the K-elements to studs connections following by rivet pull out. Hence, while the tests were progressing, it was decided to add one washer for each rivet to prevent rivet pull out as the main undesirable brittle fracture mode.

According to the current research results, it is concluded that having more enclosed area made by K-elements in the frames causes the better lateral resistant performance in the wall. That is because these enclosed area acts like a formed box and provides cumulative hysteretic energy dissipation. Also, Use of K-braced elements in the side spans are better than the middle spans, as in this case, resistant elements are used more far from the wall shear centre and acts more efficiently.

Studies show that, although using double studs does not increase the ultimate strength, the ductility enhanced due to prevent early buckling in the chords. Moreover, using washer in the K-elements to studs connections improves the lateral performances of the walls considerably; and changes the final failure modes from rivet pull-out to shear failure in the rivets. That causes having more ductility and higher shear load capacity for the walls. In addition testing concurrent K-brace and X-strap lateral resistant system indicates when it is possible to cumulate the lateral resistant of different systems that the lateral stiffness of all systems would be similar. Based on the results, the maximum shear load of a frame using both K braced and X strap braced system is so less than the summation of individual systems. That is because the corresponding drifts of ultimate shear loads for two systems are different.

with compared to the lateral performance of other types of resistant system such as Knee-braced system and X-strap braced system which have done by the authors and other researchers (Moghimi and Ronagh 2009), it is shown that the performance of K-braced cold-formed steel lateral resistant system under cyclic loads is satisfactory; and provides a significant contribution to the frames' lateral resistance. By using proper design, they can be used as a reliable lateral resistant system with adequate energy dissipation in seismic events, especially in low seismic regions where the high lateral resistant systems are not required.

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The coupled post-local buckling and overall bending interaction mechanics of thin-walled plain channel columns

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ABSTRACT

The behaviour of thin-walled columns has been the subject of investigation by a great many researchers over the years. Many different aspects have come under consideration and, as such, our knowledge and understanding of column behaviour today, is at a fairly sophisticated level, although there are still areas which require further research.

The interaction of local buckling with overall column bending has been studied by (Loughlan 1993) using a semi-energy post-local buckling analysis procedure. Theoretical results are presented for the pin-ended support condition and a study is made of the local buckling and overall bending interaction behaviour of compression members with different cross-sectional shapes.

The works of (Rasmussen 1997) and (Young & Rasmussen 1997) considered the overall behaviour of locally buckled thin-walled compression members. One of the major conclusions from these works with respect to channel sections was that for the fixed-ended condition with initial buckling being in the local mode, the overall behaviour of the column can be treated as one of bifurcation of the locally buckled member.

The post-buckling behaviour and interaction of different modes in thin-walled compression members has been discussed by (Rhodes 2004). Of particular interest are his views with regard to the case of fixed-ended plain channel columns. These suggest that for the fixed-ended case, the columns will suffer overall column deflections from the onset of local buckling, a conflicting view to that of (Rasmussen 1997) and (Young & Rasmussen 1997).

In this paper the authors examine the influence of end conditions on the behaviour of plain channel columns in the light of these conflicting views. Finite element simulation is used to determine column response and the study is limited to the interaction of local buckling and overall flexural bending.

For fixed-ended plain channel columns whose initial buckling is the local mode, it is found that overall column deflections are initiated from the onset of local buckling. This is due to the fact that the local buckling mode shape is associated with amplitude modulation along the column length as a result of the rotational constraint imposed at the column ends and thus there is a varying degree of neutral axis shift along the compression member. Although the neutral axis movement is counteracted at the fixed or loaded ends of the column, the variation in neutral axis movement at different sections along the column length is such that the constant line of action of the applied axial load will cause combined bending and compression after the occurrence of local buckling.

It is clear therefore that, precluding the overall torsional-flexural mode of buckling, the failure of fixed-ended plain channel columns is, more than likely, to be associated with overall bending of the locally buckled members and, strictly speaking, since column deflections are in evidence from the onset of local buckling, the failure mode should not be considered as one of bifurcation of the locally buckled members.

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Compressed steel members with combined cross-sections

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ABSTRACT

The paper presents fundamental information about realized experimental research of the local stability and load-carrying capacity of thin-walled compressed steel members with hybrid cross-sections. This research has been distinctively oriented on the investigation and analyses of local stability and post-critical behavior of the slender and ultra-slender member webs and their interaction with compact flanges in the process of their loading and failure.

All tested members are divided into 4 cross-sectional groups. Each group consisted of three similar members.

The web dimensions are designed as thin-walled at the compression loading. At the same time the flange dimensions are designed to be compact, when subjected to the elastic region of loading. The general layout of the tests is illustrated in Figure 1.

The results mention the effect of the local buckling and interaction of the web and flanges subjected to compression. This effect is very significant in the places of the direct transmission of the loading. Good consonance can by found from the comparison of determined experimental limit loads $N_{u,exp}$ and the theoretical limit loads $N_{ul,ep}$ and $N_{u,z}$. Obtained knowledge and results encourage to more consistent analysis of very slender webs influence.

Some of reached results are presented in the following figures:

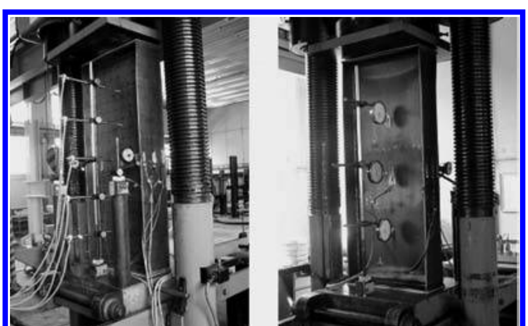

Figure 1. General layout of the tests.

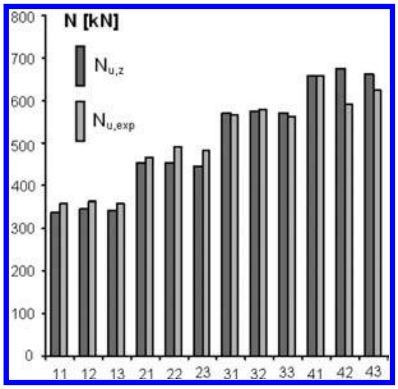

Figure 2. Theoretical and experimental capacities comparison.

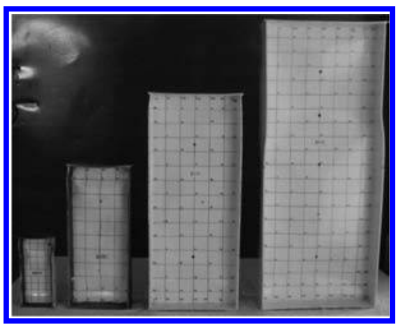

Figure 3. The total failure of tested members.

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Behaviour of curved hybrid steel girders

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ABSTRACT

Hybrid girders make use of different steel grades with the lower grade in the web and the higher grade in the flanges. The hybrid configuration provides an optimum solution of placing the high strength steel where it is most needed. However conventional the design guides place a limit on the maximum steel yield strengths permitted for compact sections and this reduces the efficiency of hybrid girders. These restrictions are placed because most high strength steels do not have sufficient ductility to allow the formation of plastic hinge. Curved girders arise as a solution to alignment constraints mostly in highway and railway bridges. The horizontal curvature of a girder however changes considerably the loading configuration by introducing lateral moments. Despite the increased use of curved girders in bridges, their behaviour and the reliability of the analysis tool is still not fully understood. Fukumoto and Nishida (1981) derived a quartic equation for predicting the strength of curved girders (Eqn. 1).

$$\lambda^4\delta^4 - \left(\left[\left(1 + \frac{p_e(d - t_{cf})}{2M_p}\right)\left(\frac{L^2}{2Rb_{cf}}\right)\lambda^4 + 1\right]\right)\delta^2 - \left(\frac{L^2}{2Rb_{cf}}\right)\delta + 1 = 0 \qquad (1)$$

Richard et al (1994) also derived and ultimate load equation for curved girders. This equation generally gives higher ultimate strength compared to Eqn. 2

$$\delta_u^4\lambda^4 - \left[\left(1 + \frac{(d - t)P_E L^2}{4M_p Rb} - \frac{(d - t)EI_Z L^2}{2M_p R^3 b} - \frac{2P_E}{M_p^2 R^2} + \frac{P_E EI_y}{M_p^2 R^2}\left[\frac{L}{\pi}\right]^2\right)\lambda^4 + 1 - \frac{2L^2}{\pi^2 R^2}\delta_u^2 - \left|\frac{L^2}{2RB} - \frac{P_E L^2\lambda^4}{bM_p^2 R^3} - \frac{L^4}{bR^3\pi^2} + \frac{P_E EI_Y}{2M_p^2 R^3}\left[\frac{L^4}{\pi^2 b}\right]\lambda^4\right|\delta_u + 1 - \frac{2L^2}{\pi^2 R^2} - \frac{2P_E\lambda^4}{R^2 M_p^2} + \frac{P_E EI_Y}{M_p^2 R^2}\left[\frac{L}{\pi}\right]^2\lambda^4 = 0 \qquad (2)$$

Finite element models were developed to investigate the hybrid and curvature effects on girders. The behavior of the curved hybrid girders was compared with that of straight as well as curved homogeneous girders.

The curvature drastically modifies the girder load pattern. Hybrid girder behavior can be approximated

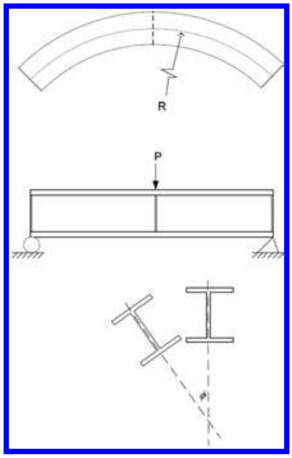

Figure 1. Curved Beam.

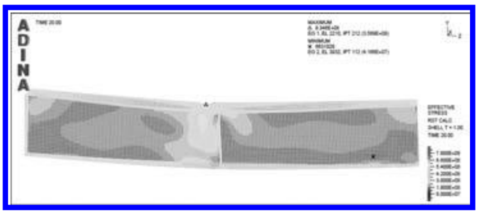

Figure 2. Stress Plot for Curved Girder.

as that of corresponding homogeneous girders with flange steel grade. The equations for determining ultimate load of curved girders are not convenient for design and it is therefore recommend that either FEM modeling or the AASHTO technique of applying a hybrid reduction factor be used for design.

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Experimental investigation of lateral deflection of columns with intermediate slenderness submitted to axial compressive loads

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EXTENDED ABSTRACT

The buckling phenomenon of prismatic column is experimentally investigated in this work, being major structural design concern. A number of tests has been performed on aluminum made prismatic columns with slenderness index λ_{ef} remaining within range of 15 to 460 submitted to programs of multiple in time compressive loads – force and displacement controlled. The experimental setup based on MTS 810 machine with digital controller and tested specimen are shown in Figures 1 and 2, respectively. Failure regimes resulting from structural material plastic deformation – elements of low slenderness, and/or elastic lateral buckling – elements of high slenderness, have been quite profoundly investigated both experimentally and theoretically. The failure behavior of structural elements with intermediate effective slenderness – typically 80–140, frequently met in civil engineering structures remain relatively unexplored. The failure mode of such structural elements seems to be initiated by mutual interaction of elastic deformation, plastic deformation and a third key factor "imperfection". The results of experimental studies and proposition of some theoretical concepts are presented, which could help developing in a systematic way so called "BIY" theory (buckling-imperfection-yield theory).

Figure 1. Collection of prismatic specimen with various effective slenderness.

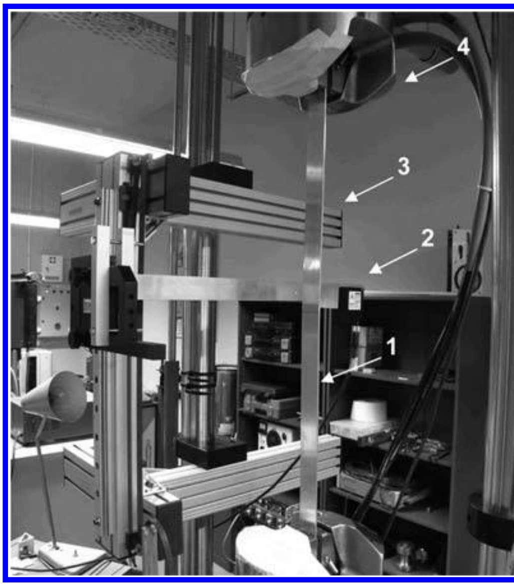

Figure 2. Experimental setup for column stability tests on MTS 810 machine: 1 – specimen, 2 – laser sensor, 3 – moving frame to follow vertical motion of place with maximum lateral deflection, 4 – machine upper grip.

There are presented experimental results showing hysteresis loops formation in post-critical range of columns deformation. There can also be noticed presence of dynamic effects even in a quasi-static strain controlled tests. An effort is undertaken to explain when and why such a phenomena happen.

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A genetic algorithm-based regression to predict resistance of plate girders subjected to patch loading

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ABSTRACT

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The theoretical analysis of web panels of plate girders subjected to patch loading (Figure 1) is quite complicated. This difficulty in theoretical based analysis leads research in this field to be experimentally based and some empirical and semi-empirical formulae have been established, but they still present significant errors (about 20%) when compared to experimental results (Fonseca et al. 2003).

The main focus of this study is to obtain empirical formulation of patch load resistance of plate girders by means of genetic algorithm based on experimental results collected from the literature.

$$P_{u1} = 1.1 t_w^2 \sqrt{E\sigma_w}\left(\frac{t_f}{t_w}\right)^{0.25}\left\{1+\frac{(c+2t_f)t_w}{d_w t_f}\right\} \tag{1}$$

Due to good correlation of the results obtained using Equation (1) (Roberts & Newark 1997, Fonseca et al. 2003) with the experimental data, the initial solution of the evolution may be expressed by the following formulation:

$$P_u = \alpha_1\left(t_w^{\alpha_2}\right)(E\sigma_w)^{\alpha_3}\left(\frac{t_f^{\alpha_4}}{t_w^{\alpha_5}}\right)\left(\alpha_6+\frac{(c+2t_f)t_w}{d_w t_f}\right)^{\alpha_7}\left(\frac{d_w}{b_w}\right)^{\alpha_8} b_f^{\alpha_9} \tag{2}$$

In this study, the variables $\alpha_1, \alpha_2, \ldots, \alpha_9$ are determined as closely describe the problem using symbolic regression. For doing this, the genetic algorithm is applied to minimize the estimated error (Equation (2)).

$$ER = \frac{\sum_1^n\left(\frac{|P_{ex}-P_u|}{P_{ex}}\right)}{n}\times 100 \tag{3}$$

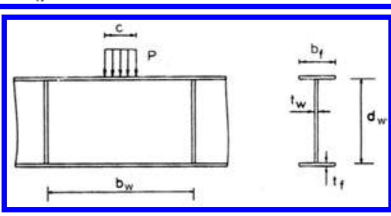

Figure 1. Patch loading and girder dimensions.

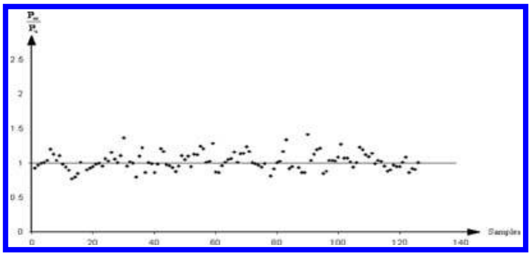

Figure 2. Comparison of proposed GA formulation vs. experimental results.

where P_{ex} *and* P_u are experimental patch resistance and theoretical patch resistance, respectively.

The final equation developed as follows:

$$P_u = 0.043\ t_w^2(E\sigma_w)^{0.66}\frac{t_f^{0.31}}{t_w^{0.4}}\left(0.93+\frac{(c+2t_f)t_w}{d_w t_f}\right)^{1.49}\left(\frac{d_w}{b_w}\right)^{0.0241} b_f^{0.09} \tag{4}$$

The overall results of the proposed GA formulation versus experimental results are presented in Figure 2.

Figure 2 verifies a good performance of the results obtained by the proposed formulation when compared with experimental data. Statistical parameters of the proposed GA-based formulation in terms of mean ratio of test/predicted = 1.05, standard deviation = 0.19 and estimated error = 9.9% indicate that the proposed formulation performs well in comparison with existing models and experimental results.

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Behavior of non-prismatic steel members

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ABSTRACT

A change in the cross-sectional dimensions leads to changes in the moment of inertia (I-values), torsional constant (J), warping constant (I_w) of the cross-section, consequently affecting the elasto-plastic, stability and torsional behaviour of the steel member. The geometrical arrangements and degree of non-uniformity are paramount. Timoshenko and Gere (1961) suggested that when the member geometry closely resembles the bending moment envelope, the load bearing capacity increases by up to 16% for a tapering ratio of $h/H = 0.12$. Where h is the smaller and H is the larger section's dimension. Mueller, Werner and Osterrieder (1999) reported a 15% waste reduction in material usage while Saka (1997) observed a 10% weight reduction. A comparative study considering the various factors which affect the behavior of non-prismatic members was conducted. Factors such as the degree, type and position of non-uniformity were simulated under the same constraint conditions. The effects of different member types, boundary conditions, tapering ratios (h/H), loading conditions and tapering positions of non-prismatic members were investigated. The resulting deflections, stress distributions and critical buckling loads were used in the comparative analysis. The section types simulated were I and H due to their warping phenomenon which influences instability behaviour. The FEM package ADINA/M 8.4.2 (2006) was used for the computational modelling.

The behaviour of non-prismatic members in bending (Fig. 1) and in compression (Fig. 2) was

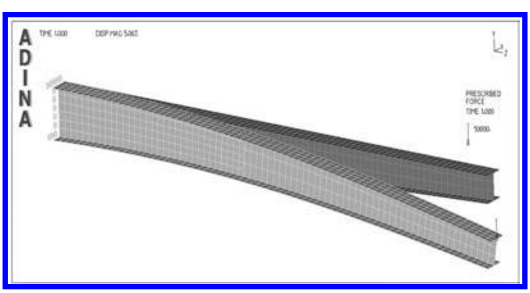

Figure 1. Deflected Web Tapered Cantilever Beam.

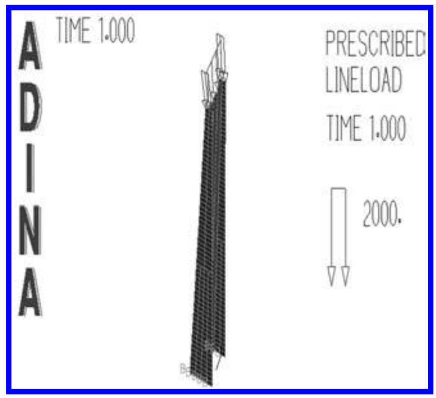

Figure 2. Compression Member (Flange Tapered) Larger Base.

investigated and the results obtained from the simulation of tapered cantilever beams and columns show that, in general, non-prismatic steel members provide some advantages in the load bearing capacity. However not all geometrical arrangements of non-prismatic members provide for an economical design. In this investigation the material volume and weight was kept constant for the tapered members for comparison purposes and the real benefits would be evident if the changes in the member cross-sectional dimensions leads to material savings. There are cases where tapering does not bring any benefits, such as the case of the column with a larger base This means that a process of optimization would need to be carried out in order to determine the cross-sectional tapering that would be of benefit.

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Drilling hole effect on tensile properties of stainless steel 316L

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ABSTRACT

The effect of drilling holes on tensile properties is investigated. Experimental work was conducted on stainless steel 316L in L-T orientation. A series of tensile tests are conducted to determine the tensile properties on flat tensile specimens without and with drilling hole in the center of specimens. Operations drilling for many people appear to be simple process, perhaps due to of different parameters effect, it's very complex. Machining parameters of drilling that affect the mechanical behavior and geometrical parameters that also affect directly the mechanical properties (DeGarmo, 1997). The stainless steels remain irreplaceable materials considering their good mechanical characteristics (capacities of resistance, hardness, impact resistance etc). The study of the stainless steel 316L was the subject of several studies in various forms of loading (Polàk et al. 1994, Puchi-Cabrera et al. 2008, Obrtlik et al. 1997). The notch effect and stress concentration on the fatigue behavior are studied by Obrtlik et al. (1997). The aim of this work was to examine the influence of drilling hole on plate for mechanical behavior (mechanical properties).

Variations of true stress function to the true strain for different tensile tests are plotted in Figure 1. We notice a dispersion of tests results in plastic zone curve and the final value of limit of fracture. The obtained results are good agreement to others results in databases of materials.

Figure 2 shows the evolution of the stress according to the strain for drilled specimens. We note that

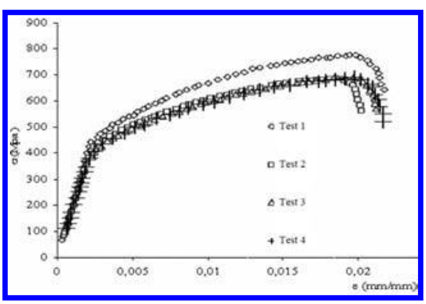

Figure 1. True stress-strain curves of stainless steel 316L.

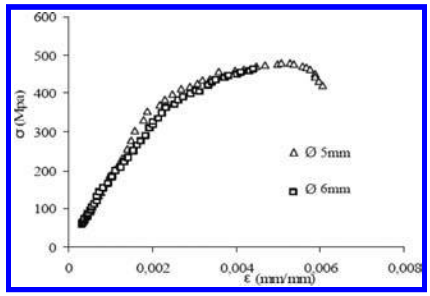

Figure 2. Effect of drilling hole on stress-strain of 316L.

at the beginning of elastic deformation field is the same. From a deformation of 0.003, a shift of the tensile curve resulting in increase of the stress test for drilled of diameter 5 mm from the specimen to diameter 6 mm. For the same deformation in elastic zone of material is significant. In the plastic range a slight variation of plastic strain is recorded. An increasing of drilling diameters decreases the tensile strength. Comparison of figure 1 and 2 shows the effect of drilling of hole specimen from a single specimen. The effect of drilling is significant in the plastic zone which results in a significant reduction in the maximum resistance to fracture (800 MPa to 500 MPa). Moreover, for a simple specimen, the plastic domain is large. We show a decreasing of ultimate tensile strength of fracture (UTS) and yield stress (σ_e) with increasing of diameter hole.

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Assessment of robustness in the design of steel structures

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ABSTRACT

An accidental action is defined as an action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life. Examples of accidental actions are impact, explosions and human error.

Robustness is the ability of a structure to withstand events like impact or the consequence of human error. The leading design principle is that local damage is acceptable; provided that the damage will not endanger the structure.

The South African Loading code SANS 10160:2010, ISO 2394:1998 and the Eurocodes are all codes that specify options that can be followed to increase the robustness of a structure; or to prevent an accidental action from occurring.

It is the aim of this paper to present a systematic process for applying codified procedures to the design of industrial steel structures, as derived from an investigation of the robustness characteristics of selected structures.

The two types of steel structure that were selected were a single storey overhead travelling crane supporting structure and a multi-storey industrial structure housing a process plant. The structures are modelled in 3D in a FEM package.

A structural element is notionally removed from the 3D model of the specific structure. Two types of analysis, buckling and second-order, were run on the structures to identify potential problem areas. Pending the results of the analysis the structural element that was removed was classified as a key element and robustness requirements must be applied to the structure.

Three robustness requirements were discussed in the paper, namely;

- Effective horizontal and vertical tying forces
- Redundancy
- Key element design

It is found that certain structures are better situated for certain robustness requirements. There requirements are a function of cost and feasibility of implementation as such. For example it is not feasible to incorporate tying forces into a portal frame structure. On the other hand, where tying forces are applicable, design of ties provides an effective way to increase the robustness of a multi-storey structure.

Redundancy is an expensive approach option if substructures need to be duplicated, but this is not the case regarding connections where the number of bolts in a connection can be duplicated. The design of a key element is a straight forward procedure.

Human error can be reduced or eliminated by quality management procedures. This will lead to an increase in the reliability level of the structure.

The loss of a structural element will leave a gap in the load path. The loads need a new path to follow and other surrounding structural elements will have to resist an increase in the loads. The resistance of these structural elements must be verified to make sure that the resistance is sufficient to carry the increased accidental ultimate load.

A flow diagram was developed to assist the structural engineer when designing for robustness.

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M-N interaction in beam-to-column joints: Development of a design model

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ABSTRACT

The component method is a nowadays well-known and widely recognized procedure for the evaluation of the design properties of structural joints. It is used as a reference method in Eurocode 3 and Eurocode 4, respectively for joints in steel and composite constructions, but it may apply to many other joint configuration and connection types.

In the component method, any joint is seen as a set of elements (called components). The mechanical properties of these components, in terms of elastic deformation, design resistance and deformation capacity are evaluated through appropriate design models; then the components properties are "assembled" to finally derive the mechanical properties of the full joint, i.e. its rotational stiffness, its moment and shear design resistances, its collapse mode and its level of rotational capacity.

In the Eurocodes, the proposed rules are mainly devoted to the characterization of joints subjected to bending moments and shear forces. It is the reason why, in Part 1.8 of Eurocode 3 dedicated to the design of steel joints, the proposed field of application is limited to joints in which the force (N_{Ed}, also noted N in the paper for sake of simplicity – and the same applies to M_{Ed}, noted M -) acting in the joint remains lower than 5% of the axial design resistance $N_{pl,Rd}$ of the connected beam (and not of the joint what is quite surprising as far as the influence of the applied axial load on the joint response is of concern):

$$\left|\frac{N_{Ed}}{N_{pl,Rd}}\right| \leq 0.05 \qquad (1)$$

Under this limit it is considered that the rotational response of the joints is not significantly influenced by the axial forces. It has however to be stated that this value is a fully arbitrary one and is not at all scientifically justified.

However, in some situations, these joints can be subjected to combined axial loads and bending moments, for instance at the extremities of inclined roof beams or in frames subjected to an exceptional event leading to the loss of a column, situation where significant tying forces can developed in the structural beams above the lost column.

If the criterion of Equation 1 is not satisfied, the Eurocodes recommend to check the resistance by referring to "*M-N*" interaction diagram defined by the polygon linking the four points corresponding respectively to the hogging and sagging bending resistances in absence of axial forces and to the tension and compression axial resistances in absence of bending.

In a previous study (Cerfontaine, 2003), it has been shown that the proposed method is quite questionable. So, an improved design analytical procedure, based on the component method concept, has been (i) developed by Cerfontaine to predict the response of ductile and non-ductile steel joints subjected to combined axial loads and bending moments and (ii) extended to composite joints in (Demonceau, 2008). This method is briefly introduced in the present article; in particular, it is illustrated how this model was validated through comparisons to recent experimental tests performed on steel-concrete composite beam-to-column joints.

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Stability design criteria for steel column splices in non-sway frames

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ABSTRACT

Splices are connections between two structural elements within their length, to form a single and longer element. Splices between parts of columns are necessary to keep individual lengths within manageable dimensions or to provide an opportunity to change the section serial size. Column splices are usually disregarded when design of the columns is being considered. This practice is questionable as the splices may adversely affect the overall behaviour, from a stiffness and strength point of view. The design method for the column is based on the assumption that there is full continuity of stiffness along the column length, and clearly that must be maintained effectively through the splice. The actual splice connection must then deliver performance consistent with such assumption.

The overall objective of this research work is to more closely analyse the buckling behaviour of an axially loaded framed spliced steel column. Specific goals of this study are to (i) gain a better understanding of the behaviour of spliced columns with different end-restraints; (ii) derive a design methodology based on column stability considerations; (iii) propose minimum stiffness requirements for column splices in non-sway frames to ensure full continuity of the member; and (iv) develop a simplified predictive expression for inclusion in design codes.

The current paper starts with an elastic stability analysis of a framed spliced column using an energy-based approach that includes a Rayleigh-Ritz approximation into the total potential energy function. This is a stationary value problem with constraints that is solved by using the method of Lagrange multipliers. A reasonably comprehensive set of parametric studies is then implemented and the results pertaining to the critical load are presented. Results are evaluated with respect to the following variables: (i) splice location and rotational stiffness, (ii) change in the column section serial size and (iii) column end-restraints stiffness coefficients. The behaviour is described in terms of an nondimensional end fixity factor, C, that is defined as the quotient between the critical load of the actual column and that of the Euler column with equal length L. These results are then used to develop regression equations to approximate the end fixity factor by means of an expression of the form:

$$C_{\text{fit}}\left(\alpha, \frac{I_{\text{I}}}{I_{\text{II}}}, k_{\theta\text{a}}, k_{\theta\text{b}}, k_{\theta\text{c}}\right) = C_1\left(1 + C_2\alpha + C_3\alpha^2\right) \times \left(1 + C_4\frac{I_{\text{I}}}{I_{\text{II}}}\right) \times \\ \times \left[1 + C_5 \arctan\left(\frac{k_{\theta\text{a}}}{5}\right)\right] \times \left[1 + C_6 \arctan\left(\frac{k_{\theta\text{b}}}{5}\right)\right] \times \\ \times \left[1 + C_7 \arctan\left(\frac{k_{\theta\text{c}}}{2}\right)\right]$$

where C_i = regression coefficients; α = parameter related to the splice location; $I_{\text{I}}/I_{\text{II}}$ = ratio between second moment of area of the upper and lower column segments; $k_{\theta\text{a}}$ and $k_{\theta\text{b}}$ = rotational stiffness at column ends; and $k_{\theta\text{c}}$ = splice rotational stiffness.

Regression coefficients are first determined for the continuous column, to predict the end fixity factor $C_{\text{fit,cont}}$ as a function of α, $I_{\text{I}}/I_{\text{II}}$, $k_{\theta\text{a}}$ and $k_{\theta\text{b}}$. Then, $C_{\text{fit,spl}}$ is generated for the spliced column, as a function of α, $I_{\text{I}}/I_{\text{II}}$, $k_{\theta\text{a}}$, $k_{\theta\text{b}}$ and $k_{\theta\text{c}}$. The minimum value of the spring constant $k_{\theta\text{c}}$ that ensures continuity of bending stiffness is found by comparing the two cases and applying the so-called 5% criterion:

$$C_{\text{fit,spl}}\left(k_{\theta\text{c,min}}\right) = 0.95 C_{\text{fit,cont}}$$

As an example, consider the case of a fixed-pinned continuous column spliced at $\alpha = 0.2L$ and $I_{\text{I}}/I_{\text{II}} = 1$. The resulting $C_{\text{fit,cont}}$ is 1.89, which is in good agreement with the exact solution $C_{\text{real}} = 2.05$ (error of 7%). The minimum value of $k_{\theta c}$ that satisfies the continuity requirement is found to be 10.

The work outlined in the paper affords some basis to produce design guidance on column splices. The predictive expressions however require further simplification for inclusion in design codes.

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Modeling of HSS endplate connections: Achievements and perspectives

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ABSTRACT

In order to enhance the fire safety of endplate connections, a research is being conducted at Delft University of Technology, using high strength steel (HSS) endplates instead of mild steel endplates. As we know, in endplate connections a thick endplate enhances resistance, but significantly reduces ductility. By using HSS, the needed endplate thickness may be less than that of mild steel endplate. The thinner HSS endplate possesses more ductility at elevated temperatures.

As a basis of investigating the behaviour of HSS endplate connections in fire conditions, their performances at ambient temperature are numerically modelled in the present paper, using finite element software ABAQUS. The detailed FE model is complicated, considering material and geometric non-linear effects, large deformations and contact interactions. The big challenge of modelling contact interactions has been solved successfully, by using several analysis steps with a view to establish contact interactions more smoothly. This allows for a more efficient simulation of the contact interactions between components of a connection when using ABAQUS/Standard.

Validation against Yu et al.'s test results shows that the proposed model can reproduce the behaviour of mild steel endplate connections at ambient temperature with reasonable accuracy. See Figure 1.

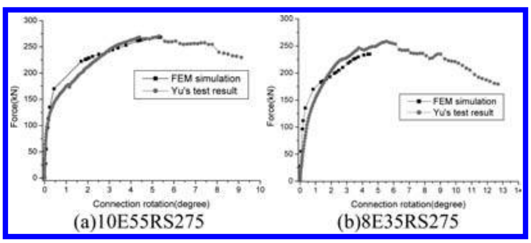

Figure 1. Force-connection rotation curves of connections.

Using HSS instead of mild steel as endplate material, the proposed model is able to predict the performance of HSS endplate connections at ambient temperature. See Figure 2, in which the behaviour of

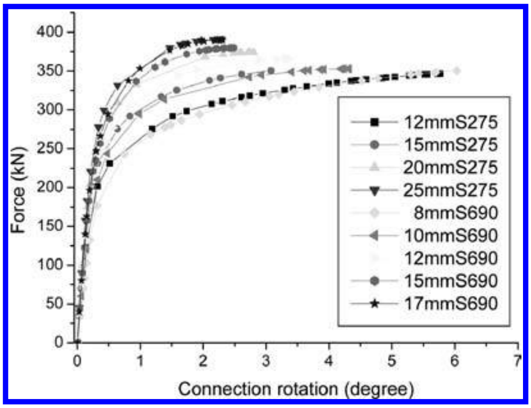

Figure 2. Effects of endplate thicknesses.

S690 endplate connections at ambient temperature is illustrated.

By a parametric study, it is found that a thinner HSS endplate enhances ductility of the connection at ambient temperature, and helps the connection to achieve the same load-bearing capacity as for a mild steel endplate connection. This finding opens perspectives for further improving the robustness of endplate connections in fire.

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Effect of bolt tension variation on the seismic response of cold-formed steel–special bolted moment frame

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ABSTRACT

The American Iron and Steel Institute (AISI) recently published the seismic standard S110 (2007). According to the AISI S110 Standard, high-strength bolts used in a Cold-Formed Steel-Special Bolted Moment Frame (Fig. 1) are to be in a snug-tight condition. Based on the cyclic test results of 9 full-scale beam-column subassemblies (Hong & Uang 2004), it was shown that using a 44.5 kN bolt tension for the 25.4 mm diameter high-strength bolts would provide a satisfactory correlation of the test results in the slip range (Sato & Uang 2009). Therefore, this bolt tension force was adopted in the AISI S110 Standard to compute the slip resistance of a bolt group. Since bolts are to be tensioned to a snug-tight condition, some raised a concern on whether over-tightening or under-tightening will affect the maximum moment that can be developed in the bolted connection. Since the inelastic action in a SBMF is to be developed in the bolted moment connection during a seismic event, a high moment in the bolt connection may over-load the beams and columns and cause the frame to failure in a nonductile manner.

A sensitivity study was conducted to address the above concern, by varying the bolt tension force, a series of nonlinear time-history analyses of a sample SBMF by using a suite of 20 earthquake ground motions was conducted to evaluate statistically the effect of the variation of bolt tension on the seismic response including the story drift and the maximum moment in the bolt connection.

Summary of nonlinear time history analysis are shown in Table 1. The bolt grip length for SBMF application is small, which implies that the increase in bolt tension due to over-tightening is not expected to be high. Even with an assumption that the bolt tension would be increased by 50% due to over-tightening, it can be observed that both the story drift and base shear (or moment in the bolted connection) are not very sensitive to the variation of bolt tension. In this case, over-tightening reduces the story drift by 3%, although the maximum moment in the bolted connection is increased by 11%. Therefore, it is concluded from this sensitivity study that over-tightening is not a concern.

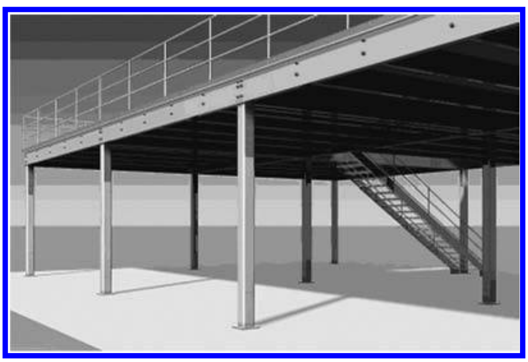

Figure 1. Special bolted moment frame (SBMF).

Table 1. Summary of nonlinear time history analysis results.

Model	Base Shear, kN	Story Drift, %
Under-tightened $T = 31.2$ kN	43.3 [21.3]	3.1 [1.1]
Standard $T = 44.5$ kN	43.4 [15.5]	2.9 [0.93]
Over-tightened $T = 66.8$ kN	48.6 [9.87]	2.8 [0.72]

[]: standard deviation.

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Residual stresses of high strength bolts with large diameters

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ABSTRACT

With the threads rolled before or after the heat treatment two different types of manufacturing methods for high strength bolts exist nowadays. Usually threads of high strength bolts for steel structures are rolled before the heat treatment.

The process order of thread rolling and heat treatment seem to play a significant role regarding the fatigue resistance of the bolts. As the fatigue limit of bolts that have been heat treated before thread rolling decreases linear with increasing preload, the fatigue limit of bolts that have been heat treated after thread rolling stays nearly constant over a wide preload range. This behaviour of bolts with threads rolled after heat treatment seems to be attributed to longitudinal compressive residual stresses in the thread root induced by the rolling process. If the rolling process is followed by a heat treatment it is likely that the residual stresses are reduced hereby.

The investigations that are presented here focus on the question how the manufacturing process of bolts with threads rolled before heat treatment influences a development of a residual stress distribution in the area of the bolt threads. Therefore, different steps of the manufacturing chain of bolts have been tested regarding their longitudinal residual stress distribution. Further details to these results can be found in Marten (2009). Additionally, numerical investigations have been performed regarding the thread rolling process.

The residual stress distribution in the core cross section of the bolts has been determined by the component destructing rotational turning procedure according to Sachs. The principle of this procedure follows a gradually machining of the test specimen with measuring the length variation after every machining step. The residual stresses for each step can be calculated with the determined strains afterwards.

The results of the length variation measurements over the remaining cross section are depicted in Figure 1. The results show only a significant length variation directly after the thread rolling.

So a residual stress distribution is established by the thread rolling with compressive stresses near the surface and tensile stresses in the inner core area. A following heat treatment eliminates these residual stresses. Therefore, it can be stated that bolts with threads rolled before heat treatment are free of residual stresses in the area of the threads.

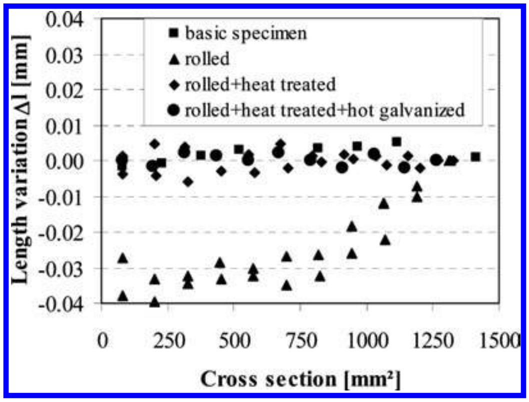

Figure 1. Residual stress distribution for bolts after thread rolling.

The procedure according to Sachs based on the assumption of a rotationally symmetric stress distribution which is constant over the length of the machining range. However, the carried out numerical simulations show that the longitudinal residual stress distribution is not continuous over the whole thread length but higher at the thread root and lower in the remaining thread. Therefore, the peak stress in the thread root determined by the measurements can just be a lower limit. Nevertheless, the quality of the numerical results has still to be validated.

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Compactness requirements of RBS connections

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ABSTRACT

A number of pre-qualified connections have been proposed in FEMA 350 [1] for use in moment resisting frame structures which include Reduced Beam Section (RBS). The idea of RBS connection which was first introduced by Plumier in 1990 [2], has been widely researched after Northridge earthquake [3–11]. Previous research studies on behavior of RBS connections are limited to connections with rolled sections and design requirements have been developed for such sections. Large size rolled sections are not readily available in developing countries like Iran and steel frame structures are usually built using plate girders. In such structures the slenderness ratios of web and flanges could greatly influence the seismic performance of the RBS connection. In this paper the effect slenderness ratios of web and flanges on the behavior of RBS connections is studied by nonlinear finite element analyses. The scope is limited to radius cut RBS connection. Figure 1 shows the analytical model of a typical RBS connection.

The analytical model is first verified with results of a full scale experiment on a RBS connection. Then, twelve RBS connections with various web and flange slenderness ratios are analyzed to evaluate the effect of slenderness ratios on ductility of the connection. Results of this study indicate that compactness requirements of FEMA350 are overly conservative. Connections in which the slenderness ratios of beam web and flanges exceeded the allowable limits by about 30 percent have shown proper ductile behavior in the analyses.

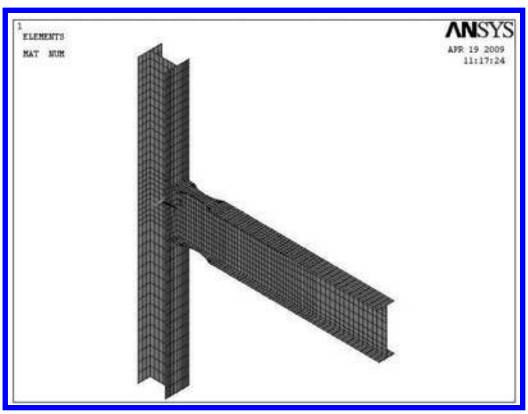

Figure 1. Finite element model of RBS connection.

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The problems of the action of high strength steel elements in structures

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ABSTRACT

The paper is focused on the same application of structural elements made of steel S690 (Weldox 700) in structural details – end plates of beams in joints of steel structures and end plates of steel beams in connections to concrete columns.

The application rules of standard documents EN 1993-1-8 and EN 1993-1-12 can be used for design of steel-to-steel joints or steel-to-concrete joints – column bases with base plates; rules were formulated for application of a steel material with value of yield strength up to 460 MPa (700 MPa).

The comparative project of real behavior of steel joints with an end plate was realized. The exploration of the influence of the end plate variable strength on the joint characteristics (the moment resistance and the initial rotational stiffness) was the main target of the project. The joint is represented by a single side connection of a beam with the end plate to a steel column. The end plate was alternatively made of steel S235, S355 and S690 (Weldox 700).

The set of the static load tests of beam-to-column joints was realized, determination and verification of behavior of single-side joints with the end plate made of various strength steels were the aim of these tests. The analysis of a joint by the finite element method was the next part of the comparative project, the program system ANSYS was used. 3D models on the basis of member and plane wall elements were applied to the given solution.

Connections of steel beams or cantilevers to concrete structures were the second domain of possible application of high strength steel elements. These connections can be realized through the use of steel bonded anchors, additionally installed threaded bolts or preinstalled anchor bolts.

The comparative project of real behavior of these joints steel-to-concrete was realized. The effect of high strength steel application for the end plate and the effect of different kinds of anchoring on the moment resistance and rotational stiffness were the aim of this project. The set of the static load tests of beam-to-column joints (in the steel-to-concrete configuration) was carried out. The main target was to determine and verify behavior of analyzed joints with the end plate made of various strength steels and different kinds of anchoring.

The basic parameters of joints (moment resistance $M_{j,Rd}$ and the initial rotational stiffness $S_{j,ini}$) were determined in accord with design rules of the standard documents EN 1993-1-8 (EN 1993-1-12). These parameters were compared with the results of realized experimental tests and the numerical model.

It is possible to observe, on the basis of the results, that the moment resistance of a joint is affected by the strength of a steel material and by the thickness of the end plate, while the initial rotational stiffness is affected by the thickness of the end plate only. The moment resistance of a joint can be increased by the application of high strength steel for the end plate with the identical thickness. At the other side, a lesser thickness of the end plate made of high strength steel, secures the sufficient moment resistance of joints with the negative effect on the initial rotational stiffness.

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Composite beams with innovative shear connection

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ABSTRACT

Composite structures combine the favorable features of structural steel and concrete. Taking into account the mechanical properties, the steel carries the tensile forces and the concrete is arranged in the compression zone. Due to the composite action a significant increase in load carrying capacity and stiffness of the beam is achieved, resulting in savings in dead load, construction depth and construction time. So far, composite structures made of high strength steel and high performance concrete have been investigated (Hegger et al. 2001, Feldmann et al. 2007). Within a collaborative research project (SPP 1182) ultra high performance concrete (UHPC) with micro steel fibers is applied for hybrid and composite structures with the puzzle strip shear connector (Figure 1, left).

Several Push-Out Tests have been performed to investigate the load carrying behavior of the puzzle strip in UHPC (Hegger, J. & Rauscher, S. 2008). This paper presents two test series with UHPC and HSC. In UHPC the load carrying capacity of the puzzle strip is twice as high as in HSC and the ductility is also increased significantly. For composite beams under positive bending moments it has to be verified that the plastic theory is applicable, i.e. the steel profile plasticizes before the concrete compression zone fails. Using high performance materials this becomes even more vital since the high strength steel requires

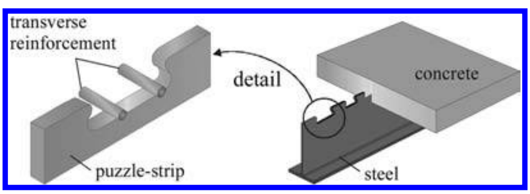

Figure 1. Puzzle-strip shear connectors (left) and filigree composite beam (right).

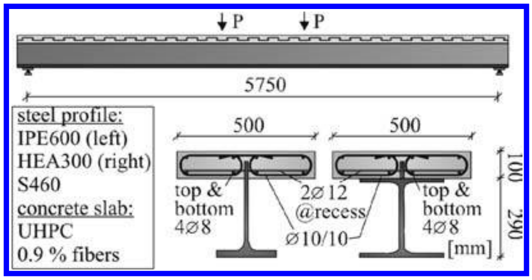

Figure 2. Beam tests under positive bending.

a higher yield strain to plasticize and for concrete the strain at failure decreases with increasing strength.

Therefore two beam tests with UHPC and S460 were performed, a conventional beam with an HEA300 profile and a filigree beam (Figure 1, right) with an IPE600 profile without an upper flange and the shear connection cut into the web of the steel profile. The plastic design is safe for the filigree beam, however, the conventional composite beam did not reach the plastic moment, since the steel profile did not fully plasticize.

A new stress block model is presented taking into account the characteristic material properties of high strength steel and high performance concrete which enables a safe plastic design.

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Modeling of composite slab failure in steel-concrete multistory buildings

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ABSTRACT

The paper presents a study on numerical modeling of composite concrete–steel slabs for the purpose of the collapse analysis of multistory buildings. The problem is investigated using nonlinear dynamic finite element simulations carried out using general purpose program LS-DYNA. The paper focuses on model development for global analysis of subjected to vertical loading and notional column removal.

The composite slab profile considered here is shown in Figure 1. The main FE model shown in Figure 2 and serving for the purpose of global analysis is developed using four node shell elements and is divided into two parts (strips) with different overall cross-sectional properties. The first part has a total thickness of 130 mm and the second 70 mm. The widths for each part are taken in a manner that conserves the original cross-sectional area. Each strip is modeled as a multilayer composite using available in the LS-DYNA elastic-plastic material model with different responses for tension and compression (Hallquist 2006). Additionally, user defined through thickness integration schemes are applied, incorporating different material parameters for the layers corresponding to the concrete, steel deck, and reinforcement. All layers are represented by the same material model but have different properties for steel and concrete.

In the face of the lack of experimental data for the considered composite floor slabs, the 2D model, shown in Figure 2, is verified through comparison with other numerical results obtained for detailed 3D model, presented in Figure 1. Numerical testing was performed for four cases on simply supported 0.6 m by 3.0 m strips of the composite slab cut off along the ribs and in the transverse direction. The strips are analyzed dynamically under four-point bending with sagging (positive) and hogging (negative) moments, caused by prescribed, time-dependent displacements applied at one third of the span.

The largest discrepancy between the 2D and 3D models is for the transverse strip subjected to sagging moments. As shown in Figure 1, in the 3D model, there is a visible concentration of scaled damage measure (Schwer & Malvar 2005) near the convex corners (transition from thickness of 70 mm to 130 mm), indicating premature cracking in the concrete slab. This notch effect cannot be captured in the 2D shell model. Summarizing, it can be stated that the assumed location of the reference plane and material behavior for concrete lead to a rather good approximation of the ultimate bending moment, especially in the primary direction along the ribs. However, the ultimate curvatures of the slab, which depend mostly on the tension softening of concrete, should be considered the main uncertainty of the global FE model (with slabs built of shell elements), requiring experimental calibration. The failure strain in tension for concrete should be treated here not only as a material property but also as a modeling parameter.

Figure 1. Contours of scaled damage measure in concrete fill due to cracking caused by sagging moments.

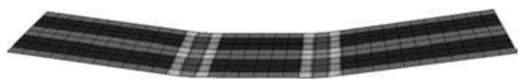

Figure 2. 2D model of the slab strip cut off along the ribs. Contours of plastic strain.

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Finite element modelling of shear connection behaviour in a push test using profiled sheeting

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ABSTRACT

A numerical investigation to study the behaviour of headed shear stud in composite beams with profiled metal decking is reported in this paper. The profiled sheeting ribs were taken as perpendicular to the axis of the beam and headed shear studs were welded through-deck. The analysis of the push test was carried out using ABAQAS/Explicit with slow load application to ensure quasi-static solution. Both material and geometric nonlinearities were taken into account. Elastic-plastic material models were used for all members except concrete slab for which Concrete Damaged Plasticity model was used.

The post-failure behaviour of the push test was accurately predicted, which is crucial for realistic determination of shear capacity, slip, and failure mode. After validation of the model against experiments conducted by various authors as shown in Figure 1 and Table 1, it was used to carry out parametric study to investigate the effect of concrete strength, slab depth and profiled sheeting rib width. It is concluded that as concrete strength, slab depth and sheeting rib width increases, the shear capacity of headed shear stud also increases. Push test with profiled sheeting usually fails by concrete cone failure. The concrete failure cones in finite element model are formed exactly in the same way as in the experiment as shown in Figure 2.

Table 1. Comparison of shear connection resistance obtained from experiments and finite element analysis.

Test Ref.	P_{test} (KN)	P_{FEA} (KN)	P_{test}/P_{FEA}	Tested by
JDT-4	54	50	1.08	Jayas & Hosain (1989)
JDT-5	44.8	44.4	1.01	
JDT-7	46	44	1.05	
G7S	50.7	47	1.08	Johnson & Yuan (1998)
G8S	60.8	55.7	1.09	
—	51.2	51.6	0.99	Hicks (2007)
Mean			1.05	
COV			0.039	

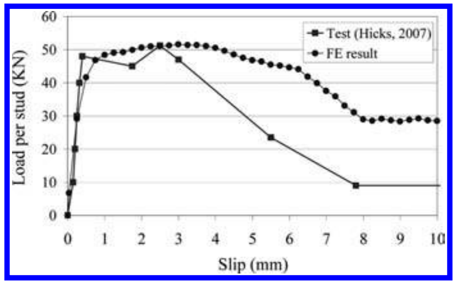

Figure 1. Comparison of finite element analysis with push test.

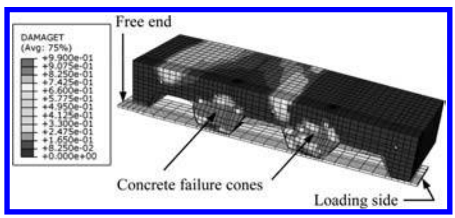

Figure 2. Formation of concrete cones in finite element model.

The separation of steel deck from concrete slab, formation of concrete failure cones, and load-slip behaviour matched with the experimental studies. The model developed in this study is extremely useful in understanding the behaviour of headed shear stud in composite beams with profiled sheeting.

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9. Behaviour of structures in fire, design for fire resistance

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Fire-induced collapses in structures: Basis of the analysis and design

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1 INTRODUCTION

One of the most challenging problems of the modern Structural Engineering is connected to the conception and the subsequent analysis and design of constructions able to face Low Probability – High Consequences (LPHC) scenarios. These situations arise for a lot of different and multifaceted reasons, being possibly followed by catastrophic consequences and it's almost impossible to frame them inside any well-recognized probabilistic format.

To simulate the structural response and, then, to develop the decisional process concerning the construction design and the environment control, one must eventually develop refined complex modeling, able to describe both nonlinear and dynamic aspects. Furthermore, it is usual the case that the structural response must be followed in post-critical range, being the scenario development interactive with the behavior of the construction: generally speaking, one must be able to reconstruct the interaction between action and structure.

A specific situation is represented by fire scenarios. In this case, one must follow a) the development of the fire (from the beginning to the spread inside the construction) b) the thermal diffusion inside the construction; c) the structural response inked to the alterations in the material properties due to the change in time of temperature and to the large displacements and deformations that are usually developed. Finally, the influence of the behavior of people on the accident must be modeled.

In these situations, it is particularly interesting to follow the path of the fire inside the construction, also in relationship with the alterations connected with the progression of failures inside the structural system.

This paper will present simple ideas which in the Authors' experience form the basis to handle fire action in structural analysis and design. One considers then in the following:

1. the characteristics of high probability – low consequences (HPLC) versus low probability – high consequences (LPHC) events;

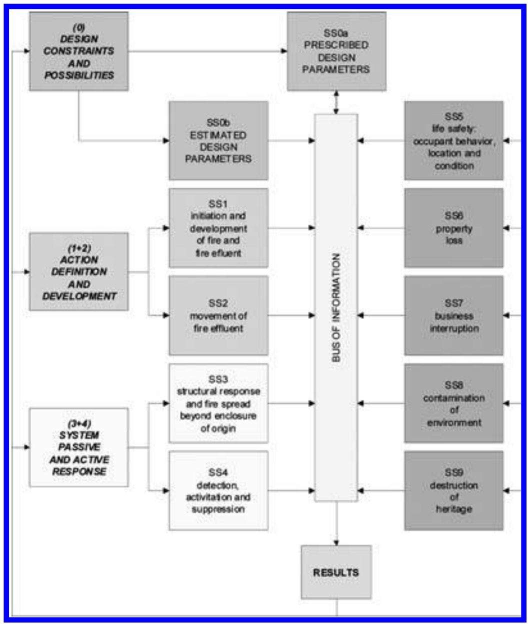

Figure 1. General framework of analysis of fire safety (adapted from ISO 13387).

2. the systemic nature of fire accidents;
3. the concept of risk and the connected activities, as risk analysis, risk assessment and risk management;
4. the scenarios identification and development.

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Fire – material – structure: A holistic approach towards analysis of underground infrastructure

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ABSTRACT

Tunnels and other important underground infrastructure must be designed to withstand hazardous fires according to a prespecified safety level. During the design process, special attention must be paid to realistic representation of the temperature loading in consequence of fire, the behavior of the employed lining material under high temperature, and the structure itself. Hereby, nonlinear effects must be taken into account, both at the material level (temperature-dependent damage, spalling of near-surface layers etc.) and at the structural level (development of plastic regions/hinges, force redistribution etc.).

In the present work, the safety of concrete structures subjected to fire loading is investigated, considering all relevant interdependencies:

Fire: In contrast to using standardized temperature-time curves (RWS, HC, RABT etc.), a more sophisticated approach is needed in case of complex geometries/structures for determination of the surface temperature of the load-carrying structure. For this purpose, Computational Fluid Dynamics (CFD) is employed to model conjugated heat transfer by taking buoyancy-induced turbulent flow (employing suitable k-ε and/or LES-models), radiation, and combustion into account. After determination of the surface temperature, the (nonlinear) temperature distribution within the cross-section of the load-carrying structure is obtained by solving the governing energy and mass-transport equations in a fully-coupled manner.

Material: When concrete is subjected to elevated temperatures, various physical and chemical processes lead to changes/degradation of its material properties and to spalling of near-surface layers. This has a considerable effect on the safety of concrete structures. Realistic description of these processes which involves realistic modeling of the transport and the mechanical material parameters of concrete is achieved by combining experimental investigations with material modeling. As regards modeling the permeability of concrete, a so-called random pore-network model provides first insight into the influence of the (temperature-dependent) pore-size distribution and the arrangement of pores on the macroscopic permeability. The path dependence of the strain behavior of concrete under combined thermal and mechanical loading is modeled by a two-phase composite (cement paste and aggregate), taking the nonlinear behavior of the material phases into account.

Structure: In contrast to state-of-the-art linear engineering models (assuming linear-elastic material behavior), a nonlinear finite element (FE) model (using layered finite elements) is employed, accounting for the nonlinear (temperature-dependent) behavior of concrete as well as for spalling of near-surface concrete layers. Additionally, the numerical model allows for consideration of nonlinearities on the structural level such as the development of plastic regions/hinges and subsequent stress/force redistribution.

The novelty of this approach which enables a realistic assessment of the structural safety of underground infrastructure under fire loading is demonstrated by investigation of real tunnel structures. Comparison of the respective numerical results with experimental data as well as numerical results from state-of-the-art engineering models demonstrates the potential of this approach to achieve a sustainable and economic design of underground infrastructure.

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Calculation of restraint axial forces of concrete cylinders under transient temperature conditions

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ABSTRACT

Objective of the investigation is the calculation of restraint forces of fully restraint concrete cylinders under 100% restraint ($\varepsilon_{\text{tot}} = 0$) during heating up to the temperature of 600˚ C at maximum.

The experimental results are calculated with the ATC-model which takes into account load effects as well as transient creep. The load at t = 0 is realized by given mechanical compressive strain of 30% of the compressive strength at time t = 0 at the beginning of the calculation. The results are compared with measured data. These measured data are a perfect foundation to compare existing high temperature material models for the calculation of the deformations and relaxation of concrete members subjected to fire.

The calculations by FEA were done using the structural code SAFIR. The concrete model is implemented in the FE software SAFIR, which is developed in the University of Liege by Prof. J. M. Franssen for the calculation of structures in fire. This software works with different concrete models, usually the EC2- model is chosen for calculation of concrete members.

The specimens are non-sealed concrete cylinders with 80 mm diameter and 300 mm height. Heating rate is 2 K/min. The compressive strength of the concrete at 20°C is 20 MPa. The moisture content is 1.5% per mass (Schneider 1979). The calculation was done with the ATC-model. It was used a siliceous concrete with high content of siliceous aggregates. The curing reached from storage under water until standard climate condition (20°C/50%):

- Curing under water,
- predried with 105°C and
- standard curing.

In the full paper the results of the calculation with the ATC-model are compared with measured data.

With respect to practical applications it is concluded, that restraint calculations after the EC2 material model lead generally to significant lower restraint forces than those derived from experimental observations, i.e. an underestimation of height of restraint forces up to 100% in the range of 100 to 150°C is possible (see Figure 5). During the simulation with cylindrical specimens at temperatures less than 420°C the different load conditions indicate different restraint axial forces. Above 420°C the curves are nearly identical around a level of 80% of the reference strength at ambient temperature. The higher the load level the higher are the restraint axial forces.

Since the axial stress has a significant effect on the fire resistance of building elements, a realistic simulation is important for loaded structures.

It seems that the calculation with the Eurocode model leads to a significant difference between the real measured data and the calculation results. The EC2-model is not suitable for the determination of concrete restraint during fire exposure.

The examination of a column under fire with respect to restraint shows a high importance of a good concrete model which must are suitable to calculate realistic axial forces due to restraint. The example of the column gives detailed informations about the difference between the two models. Other examinations of restraint structures lead to the same result. As for the interactions between adjacent members, the behaviors (such as displacements, internal forces and redistributions of loads) of concrete structures can only modeled correctly if appropriate material laws are used.

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Different approaches of European regulations for fire design of steel structural elements

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ABSTRACT

Fire has always been a major threat for buildings and other structures, leading to consequences that can affect both the safety of people and the usage or in some cases the very survival of constructions, due to collapse mechanisms induced by fire or fire effects (Bontempi, 2010).

Aim of this paper is to highlight how both safety issues (avoid people injuries and preserve integrity of constructions) are addressed in the framework of European structural fire safety design of steel constructions. Some relevant differences can be found both in the procedures and in the philosophy of national and community regulation in Europe and mainly in:

1. the required fire resistance and modeling of fire by means of nominal or of fully developed fire curves;
2. the modeling of steel resistance by accounting for the degradation of stress proportional limit or by considering only the yielding stress value;
3. the classification of fire as an action and the use of reduction coefficients for the design loads;
4. the type and level of verifications and the required safety objectives.

Due to the counterpoising effects of those aspects, it's not possible to a-priori evaluate which approach leads to the safest or the most economical design.

A punctual analysis of the different aspects and a comparison of the resulting design is therefore of interest and is presented in this paper with reference to the design of a steel car park according to the Scandinavian regulation and to the Eurocodes. Among all the design considered, those resulting from the following design choices have been reported here as most significant cases:

1. Design according the Danish national regulation (DS410:1998), which considers a fully developed fire described by the expression proposed in (Hertz, 2007).
2. Design according the Eurocodes, where the fire is modeled with a parametric curve with a linear cooling phase.

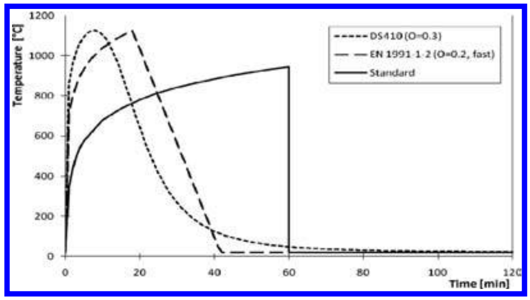

Figure 1. Different modeling of the fire action.

3. Design according the Eurocodes, where a conventional standard fire is assumed and a class of resistance of 60 min is considered for each element.

Results show a good accordance between the designs according the Danish Standard and the parametric curve of the Eurocodes. The design based on the Standard fire results to be the one that requires the highest amount of steel protection. A very strong dependence of this result on the assumed class of fire resistance is though evident, which is a requirement that varies significantly among EU countries: for an open steel car park resistances between 15 and 120 min are prescribed by different regulations, leading to very different resulting designs and safety levels.

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Performance based investigations of structural systems under fire

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ABSTRACT

Behavior of steel structures in fire is dominated by the effects of material degradation, eigenstresses induced by hindered thermal expansion and large deflections and runaway resulting from the action of imposed load on the weakened structure.

If the consideration of material degradation is – even with some differences– well established in all fire verifications, the effects of eigenstresses can be only partially accounted when fire verifications are limited to single elements. The effects of large deflections, due to the difficulties of being integrated in simple verification methods, are mostly completely disregarded in usual prescriptive-based fire design.

It seems difficult to a-priori evaluate if and to which extend the lack of a punctual consideration of these aspects leads to an over-conservative design, e.g. for the triggering of a catenary action (Usmani&al., 2001) or may instead lead to unsafe collapse mechanisms, e.g. in case of a particularly unfortunate combination of actions or after the required time of resistance, when fire design is performed using a nominal fire and a prescribed resistance class for the elements (Hertz, 2006).

Aim of this paper is to highlight how some basic mechanisms can be triggered or modified by the presence of fire on part of a structural system, such as bowing effects, buckling, catenary action.

At first some investigations on single elements are presented and the results obtained are confronted with those obtained in a previous study with a different commercial code (Crosti&Bontempi, 2008).

Subsequently the study of the collapse modality of two different steel frames is presented (Fig.1), which exhibit respectively an inwards (no-sway) collapse and an outwards (sway) collapse (Moss&al, 2009).

The effects of different assumptions in the modeling and in the definition of the collapse are highlighted, as critical aspects of a performance-based investigation.

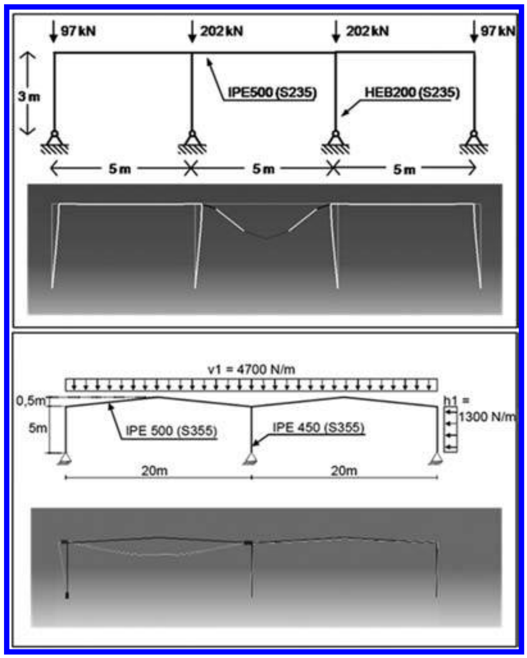

Figure 1. Inwards (sway) and outwards (sway) collapse of the two different steel frame investigated.

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Numerical analyses for Performance-Based Fire Engineering (PBFE)

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ABSTRACT

Modern codes allow two methods in order to design structures subject to fire action: prescriptive approach and performance-based approach. In some cases, the first method may be inadequate. In case of complex structures, where it is impossible to respect all architectonical prescriptions requested by the prescriptive approach, the performance-based approach is more appropriate in obtaining the optimal structural behavior under fire action.

This paper focuses on the application of performance approach to an exhibition pavilion (Figure 1), rectangular shaped with sides of 144 and 72 m. The structure is strongly redundant. It consists of 24 columns in concrete. Horizontal elements are 11 spatial steel reticular beams in the minor direction. The structure has two openings 36×6 m in the centre of the minor sides. Six scenarios have been considered, as it is shown in Figure 2.

The analysis is divided in two steps:

- *Determination of fire action* (Joyeux et al., 2002);
- *Characterization of fire effects on the structure (Buchanan 2008).*

Analyses have been performed by means of a computational fluid dynamic computational code (Bennetsen 2008) FDS (McGrattan et al., 2009) in order to determine the action of fire over the structure in function of requested performance and for realistic scenarios. The action obtained has been applied to a Finite Element structural model in order to establish the structural behavior under fire action

The analyses shown that use of advanced methods for fire action numerical simulation is fundamental in order to obtain reliable results in assessing the structural behavior under fire. Although simplified methods in modeling fire action (use of nominal fire curves) can apparently conduct to the same results (same structural deformed shape under fire) of advanced method (CFD simulation of fire development), a detailed description of the structural response (e.g. detail on vertical displacement time history in critic nodes) highlights the great difference between the structural response obtained by the two methods.

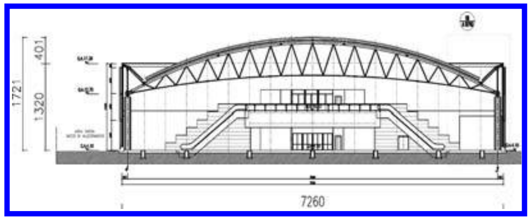

Figure 1. Case study structure typical Section.

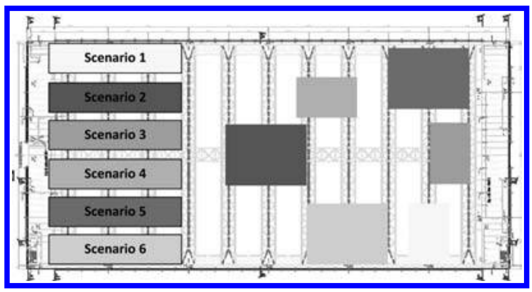

Figure 2. Positions of the six scenarios.

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Fire safety assessment of long tunnels

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ABSTRACT

In the recent years, the affirmation of Performance Based structural Codes and Standards is strong, replacing more and more the traditional Prescriptive Based ones. In modeling complex structures, such as long tunnels, which present exceptional characteristics and performance requirements, there are important aspects that need to be taken into account, especially when setting the boundary conditions of the structural problem as defined by the design environment.

This paper builds on assumptions and findings from a second paper presented at the same conference (Gkoumas, in press). The aim of this research is to evaluate by means of nonlinear non-stationary analyses what happens to the structural elements and to a tunnel construction as a whole, when the fire is not restrained. The presented results are part of a broader research focusing on structural analyses carried out on long tunnels subjected to fire action.

Objective of the analysis is the structural behavior of a temporary steel lining for a tunnel construction, in order to, in a first place, highlight some of the peculiar effects arising from the fire loading, and to some extent, provide a starting point for the characterization of the collapse resistance of the construction. The performed analyses (implemented in a commercial FEM code) account for the material and geometry nonlinearities, thus being able to accurately describe the actual behavior of the structure.

The structural analyses performed are based on prescriptions of International Codes and Standards such as the Eurocode 3 (Commission of the European Communities, 1993).

Observing for example Figure 1, related to a node in the middle of the lining (node n° 16, upper right corner), it is straightforward to follow that, at the initial temperature of 20°C (point A), a vertical displacement takes place dependent only on the applied load. Consequently, in the transition from point A to point B (due to the temperature rising), the steel lining rises due to the thermal expansion of the steel. From point B to point C the steel lining continuous to expand, yet in a less marked manner. This is due to the decrease of the elastic modulus of the material, starting from the temperature of 100°C (corresponding to point B). When the temperature reaches 400°C (point C), after a time of exposition of approximately 106 sec., the displacement of node n°16 of the steel lining becomes once again negative due to the degradation of the steel material at higher temperatures.

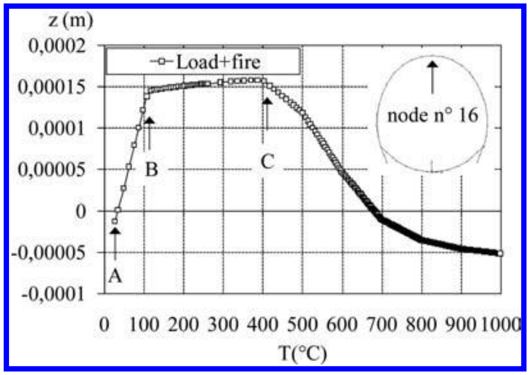

Figure 1. Temperature-displacement diagram on the z-axis.

Findings from this study will be matched in the future with those obtained from computational fluid dynamics (CFD) analyses, aiming at the temperature evolution inside the tunnel.

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Measures for tunnel fire safety

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ABSTRACT

The problem of structural fire safety in the recent years has gained a predominant role in the engineering design. This is because nowadays, always bigger and more complex structures are designed and build, making use of particularly fire sensitive materials such as steel, and also, because there is an increasing belief that structures not only have to resist to the design loads, but to maintain a minimal performance in accidental situations as well. The necessity to pursue these goals, has led to the growth of the branch of Fire Safety Engineering and the concept of Fire Safety Design.

The above considerations are even more valid for tunnels, due to the complexity of establishing such structures. In fact, tunnels and underground structures are becoming more and more essential these days, when installing new infrastructure in congested areas as well as when raising the qualities within the existing urbanizations.

The realization of such structures calls for specific measures regarding durability provisions, commitment to environmental aspects, issues of sustainability and safety assurance. In particular, fire safety in tunnels is challenging because of the particular environment.

In this paper, which is part of a broader research, basic aspects of the measures for tunnel fire safety are covered, with specific reference to a performance-based procedure. The considerations made, lead to further analysis regarding the structural assessment on a long tunnel, and presented in a second paper in the same conference (Gentili et al., in press).

Particular attention is given to two concepts that have found application (the second one, also as a complement to the other):

- The Performance Based Fire Safety Design (PBFSD) of the structure;
- The Fire Risk Assessment (FRA) of the structure, closely related to the complexity of the system (Bontempi 2005).

These concepts are incorporated in recent design Codes and Standards (Miclea et al. 2007).

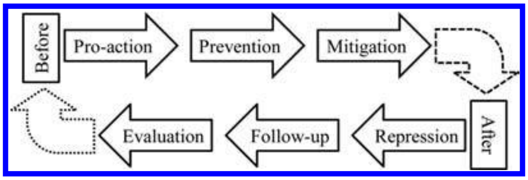

Figure 1. Safety chain for a tunnel fire.

Performance Based Design (PBD) in general, is the design that meets a specified performance level rather than prescribe specific design criteria (see for example, Foliente 2000). Performance Based Fire Safety Design (PBFSD) is a new concept within Fire Safety engineering, and it is based on the premises of PBD. The focus is on the structural performance in the presence of fire and includes requirements of fire resistance for the structural elements or for the structural system as a whole. Regarding the system response, the aim is to prevent critical events that may lead to a fire, and to reduce the consequences in the unlikely event (Fig. 1).

Findings form the basis for additional research, employing computational fluid dynamics (CFD) and finite element modeling (FEM) analyses, in order to assess the system performance.

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When a bridge becomes a tunnel: Fire design of a steel-concrete composite bridge

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ABSTRACT

The increased noise exposure caused by larger traffic volume requires the erection of more and more overhead noise barriers for motorways to fulfill sound insulation requirements and urbanistic and/or architectural aspects. With regard to fire safety the situation in noise barriers consisting of closed walls and a deck is comparable to the situation in a "tunnel".

Existing bridges, which become part of these tunnel-like structures do meet the requirements of a noise barrier but do not provide the appropriate structural requirements in case of fire.

The paper provides general considerations concerning the fire safety of bridges and shows an example where the structural performance of a steel-concrete composite bridge under fire conditions was investigated. The investigated bridge, Figure 1, was initially free standing but will become part of overhead noise barrier with requirement on fire safety in future.

Following the existing prescriptive rules which adopt the fire resistance requirement for noise barrier from tunnels, see Figure 2, a replacement of the bridge by a tunnel-like structure would have been the only solution.

As an alternative the performance based design approach is presented which includes a brief description of the design problem, the definition of design goals, design assumptions, the design procedure and design results.

At least, the examples shows that the existing structure doest not need be replaced (in contrast to prescriptive design) to meet the design goals.

Figure 1. Photo of the bridge.

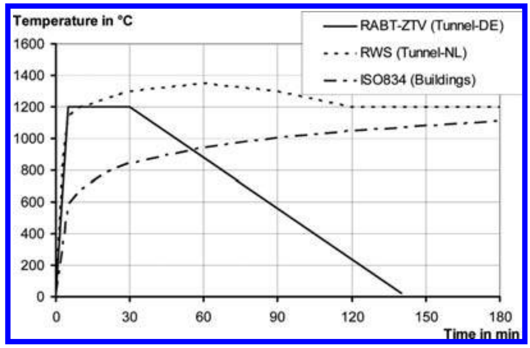

Figure 2. Time temperature curves for tunnel design.

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Behaviour of masonry walls subjected to fire: Experimental tests and analytical model

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ABSTRACT

The response to fire of masonry walls has been the subject of a good deal of past study, only recently have experimental results been compared with those from numerical models. Experimental tests on blocks for masonry walls made by several materials are performed to detect the compressive strength depending from temperature: about 400 cylinders – diameter D = 100 mm height H = 200 mm – are tested from 20°C to 700°C of clay, lightweight concrete, aerate autoclaved concrete, calcium silicate. The data base from tests has been used to calculate the *M*-*N*-*T* collapse domains (*M*: bending moment – *N*: axial force – *T*: time of exposure on nominal fire).

The proposed analytical model has been applied to materials subjected to thermal stress considering lightweight concrete. With reference to the stress-strain curves in EN 1996-1-2, adopting *Prandtl* type constitutive relation, a series of five fields (Fig. 1) for limit domain for strain depending from temperature and M-N values.

Considering masonry wall with, at T = 20°C, strength $f_{cu} = 5{,}0$ MPa, collapse strain $\varepsilon_u = 2.5‰$, and elasticity modulus E = 2800 MPa, *M*-*N* crushing domains (per unit length) have been calculated for different values of total wall thickness and exposure time to the nominal fire (i.e. graphs in Figure 2).

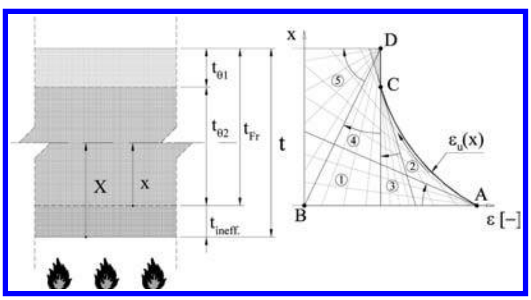

Figure 1. Limit ε fields.

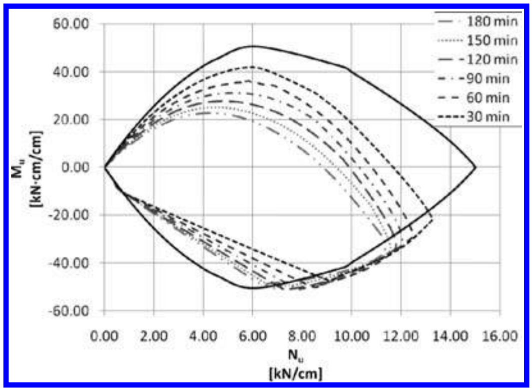

Figure 2. Crushing domains of a 30 cm-thick wall for various exposure times to nominal fire.

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Simplified analytical model for centrically and eccentrically loaded steel columns in fire

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ABSTRACT

The structural stability behaviour of steel columns subjected to fire is strongly influenced by the basic engineering stress-strain relationship of steel at elevated temperatures. In fire the strength and stiffness of steel decrease and the almost linear elastic – perfectly plastic stress-strain relationship at ambient temperature becomes distinctly nonlinear.

Simplified models in fire are usually based on ambient temperature design models considering temperature-dependent reduction factors and do not explicitly include the nonlinear stress-strain relationships of steel at elevated temperatures. These models are easy to use in design practise but have difficulty realistically describing the load-carrying behaviour of steel columns and subjected to fire.

This paper presents a novel simplified analytical model for calculating the buckling strength of centrically and eccentrically loaded steel columns in fire which allows considering nonlinear stress-strain relationship, thermal strains and residual stresses. The model checks equilibrium between the external loads of second order ($N_E = N_{E,II}$ and $M_{E,II}$) acting in the middle of the deflected column and the column internal resistance forces $N_{R,\theta}$ and $M_{N,R,\theta}$ for a given steel temperature θ. The bending moments $M_{E,II}$ and $M_{N,R,\theta}$ depend both on the column deflection w and the axial load NE. The relationships between the bending resistance $M_{N,R,\theta}$ and the exteranal bending moment $M_{E,II}$ and deflection of second order $w_{m,II}$ in the middle of the column is qualitatively shown in the moment-curvature graph in Figure 1. A linear relationship between curvature and deflection is assumed for simplification. The point of intersection A shows the point of equilibrium between internal and external forces. To reach this point of equilibrium a column deflection w_1 is required. For an axial load N_E that equals the buckling strength $N_{b,\theta}$, only one point of intersection is possible (Fig. 1 right: B). However, no point of intersection can be found for axial loads N_E that are larger than the buckling strength $N_{b,\theta}$. For these cases equilibrium is not possible, the axial load N_E exceeds the buckling strength and the column fails.

The simplified analytical model allows considering the influence of several phenomena which can be simply illustrated using moment-curvature diagrams. Therefore the model leads to a better understanding of the buckling behavior and provides a good basis for developing simplified design models.

The simplified analytical model was verified by comparing results obtained for different cross-sections, temperatures, column lengths and eccentricities to results of independent and own full scale furnace tests, numerical results using the finite element approach and results according to simplified design models. The comparison shows a good agreement between analytical prediction, numerical studies and tests results.

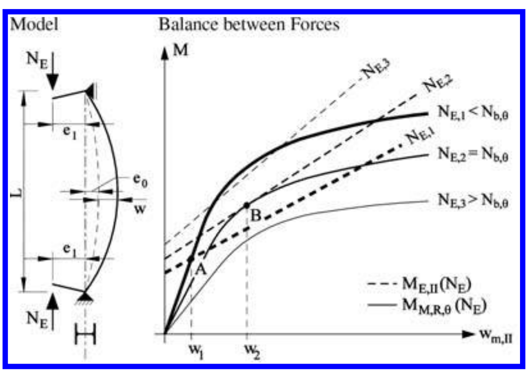

Figure 1. Simplified illustration of the model procedure.

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Experimental and analytical investigation of component column web in shear under bending and axial force at elevated temperature

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ABSTRACT

Structural fire design of steel-framed buildings is traditionally based on the application of fire protection material which reduces the heat transfer to the structure. Extensive investigations during last decades have confirmed that reliability of the unprotected structure can be ensured by a proper fire design. It is required to evaluate the behavior of the structure and its elements which are affected by the deterioration of material properties and by the additional axial forces due to the different thermal expansion at different locations (Buchanan, 2000). Structural performance during the fire strongly depends on connections whereat a large redistribution of internal forces during the heating and cooling phase is required (Wald et al. 2005). A component method which has been established as an analytical method for predictions at ambient temperature can be used also for elevated temperature cases. Poor description of components loaded in shear limit recently published analytical spring models on symmetrical joints with balanced mechanical loading. The component beam web in shear was analysed at NTU in Singapore (Zhenhai, 2007). Behaviour of the component column web in shear was investigated at the Czech Technical University in Prague.

A set of three tests was performed to study the behavior of the component column web in shear. First specimen was tested at ambient temperature (20°C), the others at elevated temperature (600°C). The test specimen consisted of a HEA-cross-section column and a HEB-cross-section beam. An extended end-plate with six bolts M22 Grade 8.8 was designed to connect both parts. The shape of the specimen enabled angle-wise application of the mechanical load that was necessary to subject the joint to interaction of bending moment, shear force and axial force. A hydraulic jack, which was applied on the upper part of the beam, introduced mechanical action. An electric transformer and ceramic heaters were used for heating to desired temperature. Deformation of the column web during the loading was observed optically by two cameras with fixed position. The column web motion was derived based on stereophotogrammetry from regular five-second sequences.

A detailed three-dimensional solid model was developed with the view to extend the experimental results to other temperatures up to 700°C. The specimen geometry is basically symmetrical, therefore only a half of the specimen was modeled with consideration of symmetrical boundary condition. The evaluation of proposed model included two stages whereat experimental results of ambient and elevated-temperature tests were compared. Both comparisons show good agreement with test results.

An experimental investigation carried by (Vimonsatit et al. 2007) show that classical theories for thin plates of plate girders under ambient condition are suitable for elevated-temperature cases. Considering the uniform temperature in the panel zone the behaviour at elevated temperature is similar to the ambient temperature. If the column web is subjected to shear and uniform compression, the resistance at the buckling stage can be calculated using the interaction formula originally established for ambient condition predictions. The comparisons with test results and numerical simulations show very good agreement in term of initial stiffness. The prediction of the resistance using the yield strength of steel at ambient and elevated temperature is conservative.

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Stub column tests on square and rectangular hollow steel sections at elevated temperatures

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ABSTRACT

The cross-sectional capacity of steel sections in fire is strongly affected by the material and the local buckling behaviour at elevated temperatures. During heating the strength and stiffness decrease, and the stress-strain relationship of steel becomes distinctly nonlinear. Due to this nonlinearity the effective yield strength at elevated temperatures $f_{y,\theta}$ is reached for large strains, for instance 2% according to EN 1993-1-2 (2005). The simplified calculation models, however, don't consider these large strains and assume plastic cross section resistance for compact and semi-compact cross sections.

A series of 11 stub column electric furnace tests on hot finished square and rectangular hollow sections (SHS 160·160·5 resp. RHS 120·60·3.6) at ambient and elevated temperatures under uniform axial compression was performed at ETH Zürich. The length of the specimens was three times the nominal height of the cross section. The ends of the stub columns were welded to end plates. The actual area, the initial local imperfections and the material behaviour of the test specimens were measured before the stub column tests. The material behaviour was determined by standard tensile material coupon tests at ambient and dilatometer compression tests (Hochholdinger et al. 2008) at elevated temperatures at ETH Zürich.

The stub column electrical furnace tests were performed using a vertical reaction frame (see Figure 1). Full axial load-end shortening histories were recorded, including the post-ultimate range. Applying the steady-state method, the specimens were first uniformly heated to the target temperatures of 400°C, 550°C and 700°C. During the heating a very low constant axial load of approximately 5kN was applied to the specimens and the thermal elongation was not restrained. After reaching the target temperature, the axial load was applied to the stub columns with three different constant longitudinal strain rates during the entire test. The slow strain rates were used to analyse the influence of the thermal creep on the cross-sectional capacity at elevated temperatures.

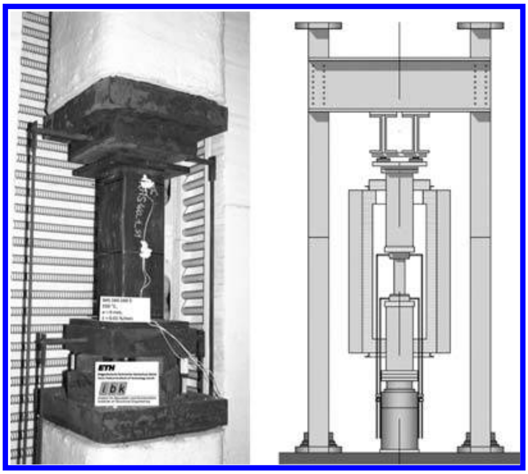

Figure 1. Experimental setup for the stub column tests.

The test results show that the ultimate loads decreased to a greater extent than the temperature dependant stress at 2% strain with increasing temperature. The cross-sectional capacity was additionally influenced by the strain rate and thermal creep effects, not considered by simplified models. The test results show that even compact cross sections develop local buckling before reaching so-called yield strength in fire.

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Interaction diagrams for heated concrete sections using the tangent stiffness matrix

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ABSTRACT

Bending moment axial force interaction diagrams are a commonly used tool in any design office. When designing for fire conditions, the large axial forces which develop place an additional importance on the consideration of the interplay between axial forces and moments. This paper presents a new method for calculating the biaxial bending moment/axial force capacity for a general section through the use of the sectional tangent stiffness. A beam-column section subject to fire is assessed, and comparisons made with simplified design tools.

Structural design codes (e.g. .(EN1992-1-1 1999)) often provide interaction diagrams for use with typical concrete sections at ambient temperature that allow the user to circumvent the cumbersome calculations necessary to determine suitable section parameters directly. However, there are many situations, such as fire loading, where structural engineers may need interaction diagrams for sections which are not covered by the standard cases. In these circumstances it is necessary to produce interaction diagrams from first principles, or to rely on very crude approaches such as the "isotherm" method. This paper presents a method based on the tangent stiffness matrix of a section by which interaction diagrams can be produced accurately and rapidly for arbitrary heated sections.

Although typically used in structural stability calculations as part of Shandley's tangent modulus equation (Bazant 1991), tangent stiffness matrices can also be used to locate failure surfaces. A section's tangent stiffness matrix relates small changes in generalized strains (typically an axial strain and two curvatures are needed for the analysis of biaxial bending of concrete sections) to small changes in the corresponding stress-resultants (an axial force and two bending moments).

The set of stress-resultants that lie on the failure surface of a section are those that arise when an incremental change in the generalized strain vector results in no change in the moments or forces. That is,

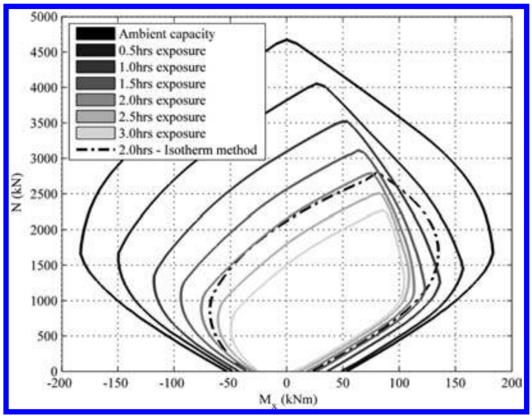

Figure 1. Changes in the interaction diagram caused by the heating of a section; the current method is compared with the 500°C isotherm method.

stress-resultants on the failure surface are those that occur when tangent stiffness matrix is singular.

A 300 × 300 mm section with 4 × 16 mm rebar was exposed on three sides to a 3 hour Standard Fire. The resulting changes in the interaction envelope at half hour intervals are shown in Figure 1.

This paper concludes that it is both possible and efficient to determine the bending moment envelope for a heated section by exploiting the section's tangent stiffness and that the isotherm method offers and unconservative estimate of the section's capacity.

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Calculation of temperature development on concrete members during tunnel fires by CFD modelling

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ABSTRACT

The simulation of fire temperatures with the CFD model Fire Dynamic Simulator showed a good approximation of measured fire curves. The fire scenario has an important influence on the calculated temperatures. The engineer must consider a scenario which is realistic and in agreement with the expectations based on practical experience. Only in this case a fire simulation makes sense.

One to this approach an optimization is possible for the members of the structure. The wall has lower temperatures than the ceiling, it is also not necessary to calculate the whole wall with the maximum time temperature curve. Furthermore it was observed that the ground plate has temperatures which are at maximum 500°C in the area of the pool fire, this must be considered in the calculation of load bearing capacity. For a good adaption of the structural behaviour under to the real fire scenario the calculation of different fire situations should be taken into account. The results of the work show the effect of different temperature boundary conditions on the wall, the ceiling and the groundplate of the structure. The obtained results may be used as a basis for structural and thermal calculations of structures i.e. structures which high temperature and high external loads.

The temperature boundary conditions used in several models for the calculation of load bearing capacity under fire exposure is very important. In case of tunnel cross sections the presented results have a high significance for the fire design. In the given examples the important consideration of temperature boundary conditions by the calculation of the load bearing capacity during heating and cooling and after the fire load is demonstrated.

The relevance of results for practical applications is very high. The following points should provide the possibility of optimizing the fire safety of concrete tunnels due to an adapted CFD simulation for the determination of realistic time temperature curves at different parts at the members, i.e. wall and ground plate. In this case the application of thermal transient creep is in the restraint calculations leads to a considerable advantage (Schneider 2009). By this the fire safety increases in the whole structure. With an adapted CFD simulation an optimizing of the cross section is possible and this may have an important influence to the building materials, the dimension of the members and the building costs.

During a fire simulation at any times a realistic time temperature curve is needed at the boundaries of each member. Figure 1 shows the principle sketch of the practical appliance of different boundary conditions.

When the boundary condition is well known the construction can be adapted to this boundary condition.

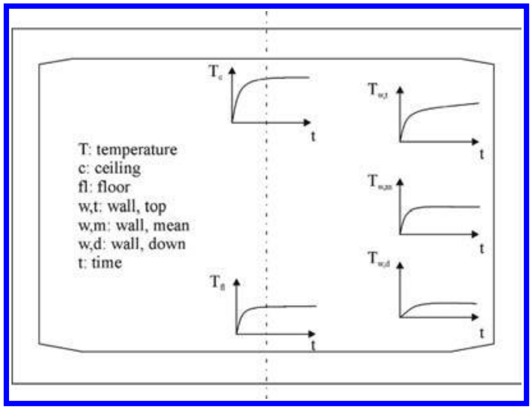

Figure 1. Principle sketch of the boundary conditions.

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Estimation about the residual working life of concrete structural members after fire

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ABSTRACT

Based on the bond strength test of 14 groups of reinforced concrete specimens after fire and cooling, and using the reliability theory, the estimating model of the residual working life of concrete structural members is established. It is used to analyze an engineering example and the analysis results are in agreement with the practice.

The size of the test specimens is $100 \times 100 \times 100$ mm. The plain steel bar with diameter 10 mm is embedded in each specimen that has 6 groups and deformation steel bar with diameter 12 mm is embedded in each specimen that has 8 groups. The test temperatures are 20°C, 100°C, 200°C, 300°C, 400°C, 500°C, 600°C, 700°C. After high temperature, the specimens are cooled for 30 days, and then tested. Based on the test data and the statistics analysis the relationship between the bond force and temperature is got. The formula of the bearing capacity of reinforced concrete members and the relationship between residual working life of members and loading effect after fire are established. The JC method is used to estimate remain working life of reinforced concrete members, and the analysis process of remain working life is given.

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10. Reinforced and prestressed concrete structures

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Finite element design of concrete structures: Differences between theory and practice

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ABSTRACT

Numerical calculations based on the finite element method have become a standard tool for designing of structures. It seems that today, it's only a question of computer capacity, the size of the element mesh, and the modeling of the non-linear material behavior, in order to analyze an arbitrary complex reinforced structures with almost unlimited accuracy. But this is not the case as will be shown in this paper. The main problems are not of numerical or mechanical type but to model the real behavior of a concrete structure and to estimate the required reinforcement. The focus of this paper is not to present highly sophisticated analysis but day-to-day problems a structural engineer has to deal with.

First simple truss and a slab structures will be presented. Even the analysis of a simple continuous girder by means of beam elements is not an easy task when so-called discontinuity regions have to be considered in the model. Next a simple supported rectangular slab will be presented. Here the design for shear and the correct modeling of the support condition is of mayor concern.

The nonlinear analysis of concrete structures is even more complex. Material non-linear finite element design has become popular due to significant improvements in hard- and software. But the accuracy of such complex analysis is very limited as most input data show a high scatter. This aspect will be discussed in the paper from a viewpoint of a consultant engineer working in practice. In a real structure neither the behavior of concrete nor the accurate loading is known. The finite element analysis of a special column of a highway bridge, which was conducted to estimate the load bearing capacity of the under-reinforced member, will be used to demonstrate the problems of such a complex analysis in practice.

Figure 1. Cantilever column.

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Strength and ductility of two-way slabs containing welded wire fabric

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ABSTRACT

The use of low-ductility reinforcement in the form of welded wire fabric (WWF) in one-way slabs loaded to failure has been shown to produce sudden and catastrophic failures caused by fracturing of the tensile reinforcement with very little plastic deformation prior to collapse (Gilbert & Sakka 2007). To date, little research has been directed at the strength and ductility of two-way slabs containing Class L reinforcement.

In this paper, full range load tests are described on 17 two-way reinforced concrete slab panels containing either Class L WWF or Class N deformed bars. The slabs were subjected to transverse loads applied by a deformation controlled actuator in stiff testing frames. The results of the tests are summarized and evaluated, with particular emphasis on the strength, ductility and failure mode of the slabs.

Typical load deflection curves for slabs reinforced with low-ductility WWF are compared with the corresponding curves for slabs containing normal ductility bars in Figure 1. The slabs reinforced with Class L wires demonstrated low capacity to absorb energy in comparison with slabs reinforced with Class N bars. A measure of the ductility of a slab is the ratio W_1/W_0, where W_1 is the total work done in deforming the slab from first loading up to the peak load and W_0 is the elastic work done in deforming the slab from first loading up until first yielding of the reinforcement.

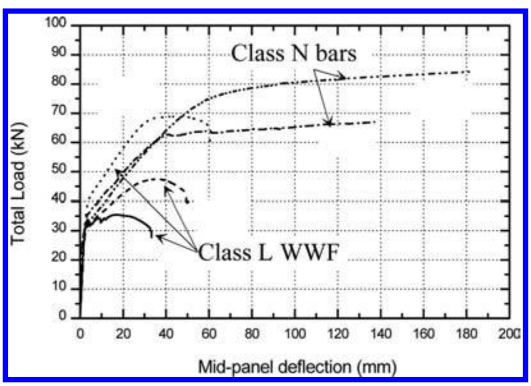

Figure 1. Load-deflection curves (Series 1).

In the analysis and design of reinforced concrete structures, many assumptions are routinely made that are only appropriate and reasonable if the individual elements of the structure are ductile. For reinforced concrete flexural members, a ductility ratio $W_1/W_0 > 2.0$ is usually considered to be necessary to justify the use of elastic analysis to determine the internal actions, or if the effect of support settlement is not to be considered. A significantly higher ductility ratio may be required if plastic analysis is used involving a significant level of moment redistribution. A still higher level is required, if the structure is to be robust or if it is required to withstand dynamic forces, such as seismic, blast or impact loads.

The two-way corner supported slabs reinforced with Class L WWF fail in a brittle mode by fracture of the tensile reinforcement and, generally, not by crushing of the compressive concrete. All slabs containing Class L WWF had ductility ratios W_1/W_0 less than 2.0 (except for one of the slabs supported continuously along all four sides). Unlike the two-way corner supported slabs, none of the edge supported slabs experienced sudden and complete collapse, when the reinforcement in a particular location fractured. The deformation controlled testing arrangements and the highly redundant nature of the slabs improves deformability of these slabs ensuring that alternative load paths are available.

The use of low ductility reinforcement in the form of WWF has a negative impact on the ductility and robustness of two-way reinforced concrete slabs and its continuing use must be seriously questioned.

ACKNOWLEDGEMENT

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Global resistance factors for reinforced concrete structures

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ABSTRACT

The concept of the global resistance factors may significantly simplify reliability verification of reinforced concrete structures when non-linear or dynamic structural analysis is required. The design value of a structural resistance can be determined directly from the resistance corresponding to the mean or characteristic values of basic variables using appropriate global resistance factors.

The presented theoretical procedure for determining the global resistance factors is based on the principles of European standards EN 1990 (2002) and EN 1992-1-1 (2004) and on the probabilistic theory of structural reliability. To illustrate this procedure a beam exposed to bending and short column under compression are analysed. The global resistance factors are derived under three alternative assumptions concerning input values of the basic variables:

1. The mean values of basic variables (the global factor γ^*_M),
2. Characteristic values of basic variables (γ_M),
3. The mean value of the reinforcement yield strength and reduced value of the concrete strength according to EN 1992-2 (2005) ($\gamma^*_{M,EN}$).

Figures 1 and 2 show variation of the global resistance factors γ_M, γ^*_M and $\gamma^*_{M,EN}$ with the reinforcement ratio ρ for the beam and column.

The presented study indicates that the design resistance may be well estimated using the global resistance factors γ^*_M related to mean and γ_M related to characteristic values. In general the type of a member (beam, slab, column), type of exposure (bending moment, shear or axial forces) and reinforcement ratio should be taken into account. In common cases (reliability index $\beta = 3.8$) the following global resistance factors may be considered: $\gamma_M \approx 1.2$ and $\gamma^*_M \approx 1.4$ for beams exposed to bending and $\gamma_M \approx 1.35$ and $\gamma^*_M \approx 1.7$ for centrically loaded columns.

The third approach yields the global factors decreasing with increasing reinforcement ratio from 1.4 to 1.2 for the beam, and in the global factor independent of reinforcement ratio of about 1.15 for the column. The recommended factor 1.27 is conservative for columns of any reinforcement ratio and beams having reinforcement ratio greater than 1%; it is unsafe for lightly reinforced beams.

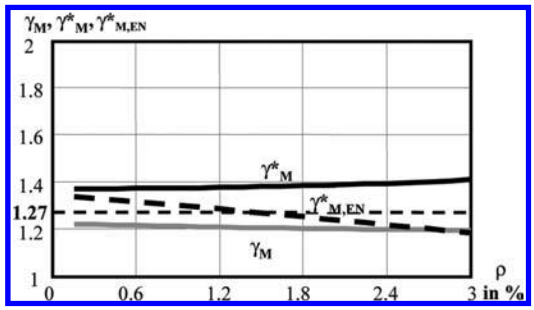

Figure 1. Variation of the global resistance factors γ_M, γ^*_M and $\gamma^*_{M,EN}$ with the reinforcement ratio ρ for the beam.

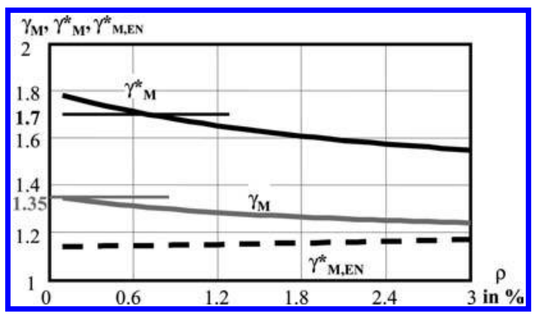

Figure 2. Variation of the global resistance factors γ_M, γ^*_M and $\gamma^*_{M,EN}$ with the reinforcement ratio ρ for the column.

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Reliability analysis of concrete structures applied to strut-and-tie model

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ABSTRACT

An important issue concerning design and detailing of reinforced concrete members is the investigation of the so-called strut-and-tie models (STM). According to this approach, a reinforced concrete member is modeled as a truss-like model, i.e., a set of compressive struts and tensile ties, in order to find a feasible statically admissible transfer mechanism of the applied load to other members or to the structure foundation. This approach has often been used when some kind of discontinuity is present in the concrete element or structure. When applying this methodology, one may take into account the limited capacity of concrete to sustain tensile plastic deformation placing reinforcement steel bars in the tensile regions. The result is a composite material that ensures a ductile behavior, provided that the concrete members in the model, representing by struts and nodes, do not collapse before yielding of steel ties. Therefore, an efficient formulation to verify the safety and ductility behavior of strut-and-tie models is desired.

This work used a Monte Carlo simulation method to perform a reliability analysis of reinforced concrete structures applying the strut-and-tie method. Monte Carlo simulations allow the estimation of the failure probability for any type of random problem. It has two main advantages: a possibility to deal with practically any mechanical and physical model, and an easy implementation without any modification of the mechanical model. With the calculated failure probability, the reliability index can be obtained and compared with a target reliability index proposed by the Joint Committee on Structural Safety (JCSS) model. The JCSS Probabilistic model code gives guidance on the modeling of random variables in structural engineering. In this work, the failure probability of some failure mechanisms is obtained, and modifications will be proposed to assure a ductility behavior of the strut-and-tie model.

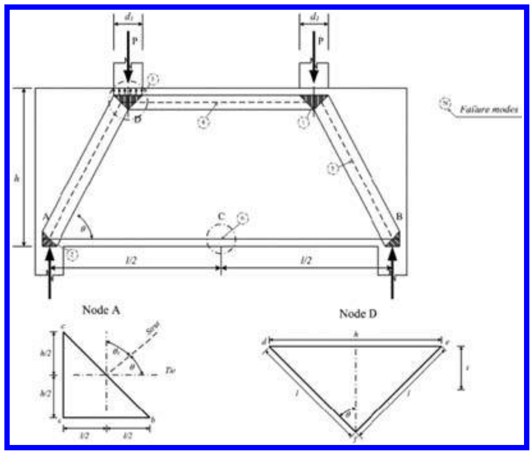

Figure 1. Failure modes in the strut-and-tie model.

Non-deterministic distribution of all modeling parameters, such as values of concrete and reinforcement properties, self weight and live load, were obtained from the JCSS code. For struts and nodes calculations, three different formulations presented in the literature to compute the effective compression resistance of the concrete will be considered: ACI-318, CEB-Fib and Schlaich & Schaefer. Both reliability index value and ductility behavior are considered in this comparison.

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Strut-and-tie model for waffle slabs

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ABSTRACT

Strut-and-tie model is being used increasingly for analysis and design of concrete structures with pronounced D-regions in the recent years. In strut-and-tie model the load carrying mechanism of a structural member is approximated by means of struts and ties representing the flow of compressive and tensile stresses respectively, which enables a direct visualization of the stress flow in the structural members.

Waffle slab is an attractive and economical alternative compared to solid and voided slab system for large spans. Currently numerical and empirical approach is used in the analysis and design of waffle slabs.

The empirical design methods recommended by ACI 318 addresses all types of two way slabs in the same manner. A new 3-D strut-and-tie model is proposed for the design of simply supported waffle slabs In the proposed strut-and-tie model the flow of internal forces in the D-region of waffle slab is transformed into an indeterminate 3-D truss structure which carries the imposed loading to the support. Individual ribs in each direction of waffle slabs form two dimensional trusses connected at rib intersections. Cross bracing at the top simulates the action of concrete slab in providing stability to the ribs in lateral direction (Figure 1)

Load test conducted by Hashim et al. (2000) on 1:4 scale specimens on waffle slabs with square layout of ribs is used for the verification of the proposed strut-and-tie model for waffle slabs. For verification of the proposed 3-D strut-and-tie model analysis of the 3-D truss is performed using ANSYS software. Both linear and non-linear analysis (steel modeled as bi-linear isotropic material) of the truss is carried out. It can be seen from Table-1 that the linear model for reinforcing steel underestimates the ultimate strength by 26.9% and the non-linear model underestimates it 13.5%. The non-linear model allows better redistribution of stresses and hence gives better prediction of the ultimate strength of the waffle slab.

A software program, STWAF for modeling waffle slab using strut-and-tie model has been developed. The software has a user interface for giving inputs such as material properties, overall size, details of waffle slab and loads. The program generates the 3-D truss model and loads at joints and analyzes the truss. The waffle slab is designed using strut-and-tie method as per ACI-318 code.

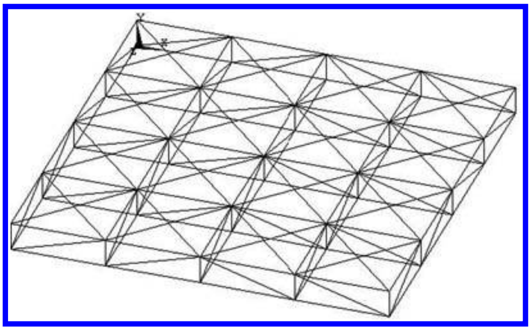

Figure 1. Proposed Strut and Tie model for waffle Slab.

Table 1. Predicted strength and Test results.

Slab name	Pu-proposed $\phi = 0.75$ kN	Pu-predicted Linear kN	Pu-predicted Non-linear kN	Pu test kN
S2	25.0	61.2	72.4	81
S5	34.6	84.4	100.4	120

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Experimental research on reinforced concrete beams under pure torsion and torsion with shear

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ABSTRACT

The paper describes experimental test results of "L" shaped cross-section (beams series BK and BK-TVM) subjected to torsion. Furthermore, as comparison beams of rectangular cross-section were tested (beam series BP and BP-TVM). Some of the beams were subjected to pure torsion only (beams BK-T and BP-T) other complex state of stress: torsion + bending + shear (beams BK-TVM and BP-TVM). In the reinforced concrete construction the loading with torsion is less recognized than other load cases. With the growth of slenderness ration the influence of torsion increases. need to test such slender object occurs. The tests were performed in the Laboratory of Technical University of Wroclaw. The structural respond of beams with different cross-section: L-shaped and rectangular. In the history beams of rectangular cross-section have been tested more extensively, have been more well recognition. In order to perform tests special testing stand was constructed. Multiple use formworks were made, and experiment was performed according to the experiment schedule. The paper presents results of performed tests. Results were compared with theoretical principles (based on existing theories and standardized procedures).

Below a list of results for all tested beams is presented (value of cracking force, breaking force, slope and spacing of cracks) – Table 1.

Table 1. A list of chosen results of beams – series T and TVM.

BEAMS	$P_{f,T}$ crack force [kN]	$P_{f,m}$ crack force [kN]	P_T breaking force [kN]	P_M breaking force [kN]	slope of cracks [°]	spacing of cracks [cm]
BK-T	8	–	27	–	~45	~15
BP-T	8	–	28	–	~45	~13
BK-TVM_1	9	9	31	31	~55	~14
BP-TVM_1	10	10	30	30	~50	~18
BK-TVM_2	9	8	21	42	~(45–80)	~17
BP-TVM_2	9	10	26	52	~(45–90)	~17
BK-TVM_3	9	9	22	66	~(50–90)	~18
BP-TVM_3	9	9	24	72	~(40–90)	~14
BK-TVM_4	10	12	20	80	~(45–90)	~13
BP-TVM_4	9–10	12	26	104	~(45–90)	~15

Where: P_T – are forces producing torque; P_M – forces producing bending moment.

As it can be seen from the above presented table crack formation occurred for very similar values of loading for all elements which proves that concrete strength is crucial in this process. In elements of series TVM, cracks caused by torsion appeared as first (for lower layer of loading).

Crack opening of 0.3 mm was achieved for 70–80% of breaking load. Crack opening increased in final stage of loading for forces close to T_N. It should be noted that according to (Eurocode 2) [8] it is not necessary to check crack openings caused by shearing stress such as for example resulting from torsion of reinforced concrete elements.

According to the experimental studies conducted the following conclusions can be drawn:

- positive influence of shelf on torsional stiffness of angle beams was observed. BK beams are characterised by smaller rotation angle then BP elements, though the last ones are characterised by larger torsional moment of inertia I_T. Decrease of stiffness was 10% less (depends on elements)
- elements only under torque moment – cracks under ~45° and in torsion and bended beams – cracks under ~50–65° (in the middle part of side elements – from torsion) and ~80–90° (in the bottom part of element side surfaces – from bending),
- first cracks in elements under 'pure torque' appeared on side surfaces approximately in the middle of height of longer sides; in elements under torque and bending moments cracks appeared on this side of beam where shear stresses caused by torque and shear summed up.
- in elements loaded by both torque and bending moment decrease of stiffness as larger
- formation of crack on shelf of angle elements occurred at higher load level than for other surfaces of tested beams. (ab. 10–15%).

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Shear strength of reinforced concrete beams without web reinforcement

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ABSTRACT

The shear design method for reinforced concrete members without web reinforcement in the Chinese *Code for Design of Concrete Structures* (GB50010-2002) was developed from empirical expressions proposed in 1980's. These expressions were derived on the basis of shear test data available at that time. Among the test data, only about 3% of them were for specimens with beam depth greater than 600 mm, and more than half of the test beams were heavily reinforced (i.e. $\rho > 2.0\%$). Although a size effect term was incorporated into the GB equation when applied to members with depth greater than 800 mm, however, as more properly scaled tests of large-size beams have become available, it is necessary to reevaluate the empirical GB equation with a more comprehensive database.

With the aim of ensuring safety of members without web reinforcement in shear, such as slabs and footings, the shear equation in the GB 50010 Code for members without web reinforcement was calibrated with a comprehensive shear database (Reineck et al. 2003, Sherwood et al. 2006a,b, Sneez & Ramirez 2008, Lv 2008). Comparisons between test results and predictions by the GB equation are given in Table 1. As shown, the GB equation presents a large scatter of results with an average V_{test}/V_{pred} ratio of 1.12 and a coefficient of variation (Cov) of 32.12%.

The results shown in Table 1 indicate that the GB equation is not capable of adequately accounting for the size effect in shear. For beams with depth greater than 600 mm, the current GB shear design method is dangerously unconservative with an average V_{test}/V_{pred} ratio of 0.67; while for beams with depth less than 600 mm, the GB method is slightly conservative with an average V_{test}/V_{pred} ratio of 1.16.

It can be found that the shear strength of beams with longitudinal reinforcement ratio less than 1.0% are overestimated by the GB equation with an average V_{test}/V_{pred} ratio of 0.74, and that of beams with longitudinal reinforcement ratio greater than 1.0% are underestimated with an average V_{test}/V_{pred} of 1.19.

Table 1. Comparison of ratio of tested to predicted shear strength, V_{test}/V_{pred}.

Beam	No. of beams	GB equation Average	GB equation Cov	Modified equation Average	Modified equation Cov
All	425	1.12	32.12	1.02	16.89
$d < 600$ mm	371	1.16	28.19	1.03	17.01
$d \geq 600$ mm	54	0.67	28.69	1.01	15.94
$\rho \leq 1.0\%$	83	0.74	27.26	1.01	13.89
$\rho > 1.0\%$	342	1.19	27.35	1.03	17.49

Based on statistical analysis of test data, modification of the GB shear equation was suggested. The size effect factor in the GB equation was refined, and a new term accounting for the effect of longitudinal reinforcement ratio was incorporated into the equation. Comparisons between test results and predictions by the suggested equation are also given in Table 1. As shown, the suggested equation fits with the experimental results very good with an average V_{test}/V_{pred} ratio of 1.02 and a coefficient of variation of 16.89%.

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Shear reinforced beams in autoclaved aerated concrete

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ABSTRACT

Shear behaviour in concrete materials is very well documented for normal density concrete materials. This paper presents the results of various tests on low density concrete materials like autoclaved aerated concrete (in the following denoted aircrete) and performs analyses for different combinations of reinforcement and for variable slenderness ratios.

Theoretical approaches will be evaluated and compared with the test results of several test series. The loadbearing capacity of shear reinforced aircrete is highly dependent on the anchorage and bond behaviour of the shear reinforcement. Recently developed formulas covering these areas will be presented and analysed.

The theoretical approach is based on the diagonal compression mechanism method, see Nielsen (1998) simplified by assuming the compression angle of 45 degrees also denoted the standard method.

$$V_w = \frac{A_{sw}}{s} z \sigma_{sw} \tag{1}$$

In this equation the maximum stress in the shear reinforcement is limited by one of the following two expressions:

Method 1:

$$\sigma_{sw} = 250\left(1-0.02\frac{l}{d}\right)\cdot\left(1.2-60\frac{A_{sw}}{s\cdot b_w}\right) \le f_{yw} \le 300\,\text{MPa},$$

for closed stirrups. (2.a)

$$\sigma_{sw} = 150\left(1-0.02\frac{l}{d}\right)\cdot\left(1.2-60\frac{A_{sw}}{s\cdot b_w}\right) \le f_{yw} \le 300\,\text{MPa},$$

for open stirrups, (2.b)

and Method 2:

$$\sigma_{sw} = 0.45 K_1 K_2 f_{co} \frac{\phi_{sl}^2 + K_3 \phi_{sw}^2}{\phi_{sw}^2} \le f_{yw} \le 200\,\text{MPa} \tag{3}$$

The results presented show very consistent conformity with theory, both when considering the mechanical anchorage of the reinforcement or when assuming a pure bond for transferring initial forces between reinforcement and aircrete. Codes for designing prefabricated reinforced components of aircrete structures have adopted these recently developed approaches.

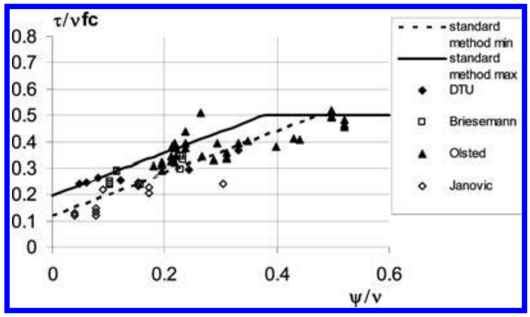

Figure 1. Test results and theory using Equation 2.

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Influence of rebar straightening on bond properties between steel and concrete

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ABSTRACT

Bond between steel and concrete is a basic aspect for the behaviour of R/C structures both in service and ultimate conditions. It is involved into the structural definition of the resistant mechanism for each reinforced concrete element. Generally, in service conditions, a good level of bond ensures a small cracking and a reduced deformability of the structural elements under the action of loads. At ultimate conditions, it is very important to obtain high bond strength both in anchorage zones and in the overlapping zones with the aim to reduce the transmission lengths; on the other hand, good bond properties, after the yielding of the rebars, may be at odds with the plastic deformation and ductility requirements of the structural elements (Eligehausen et al. 1998).

In the past, a direct connection between bond behavior and the relative rib area (bond index) was investigated and minimum values of the latter were defined in order to ensure good bond properties; those values were based on a wide experimental campaign. In the recent years, some structural codes reduced those limits (EN 1992-1-1 2004).

Furthermore building industry increased the use of de-coiled reinforcing steel and bending machine (decoiler) that are used to straighten and shape de-coiled product for use on construction sites. The straightening produces a damage on the ribs that modifies their geometrical properties. It is essentially due to the action of passing through several rollers.

In the present work, the experimental results of pull-out tests according to RILEM recommendation, are shown and discussed. Specimens with diameter of 8, 12 and 16 mm both with mechanical and by-hand straightening, taken from the same coil respectively, were tested. In Figure 1a the scheme of the test is reported whereas Figure 1b shows a picture of a specimen with steel bars diameter of 16 mm.

It is assumed that relative rib area, after the by-hand straightening process remains unchanged. It is possible by means of clamps counteracting against lead elements. Therefore the differences in terms of bond-slip response are only attributable to the mechanical action of the machines.

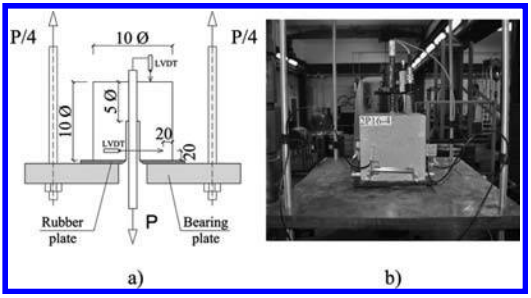

Figure 1. a) Test set up and b) a typical specimen during the test.

The results of the 90 tests show a similar behavior for the two types of specimens about the maximum bond strength and the corresponding slip value. Only for the first series the machined specimens revealed a 5% reduction of the bond strength if compared with the by-hand straightened ones.

It could be observed that this test method does not introduce any confinement in the concrete and this influences the results. A confined test method could reveal a different behavior and highlight the differences between the two types of reinforcing steel and modifications introduced by a mechanical straightening.

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Combined concrete-bond failure of a steel bonded anchor

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ABSTRACT

In spite of the fact that bonded anchors have been used for a long time there is no generally used methodology for the design of such anchoring systems. The ultimate tensile load-carrying capacity is based on the simplified assumption that the concrete or bond will fail separately. This assumption is also used in certification regulations for steel post-installed anchors for concrete e.g. ETAG.

Depending on the mechanical properties of a specific glue type, contact failure arises either at the interface between a steel bolt and the glue or at the interface between the glue and the concrete. Failures in the area of contact between glue and concrete are more frequent for currently-used glues. The dependence of the final load-carrying capacity under a tensile load on bond quality is usually described by the ultimate value of the bond stress on a given contact surface.

Unfortunately the boundary between failure types is not definite and it depends on many parameters. Up to now realized experiments shows that in case of common conditions, when no steel failure occurs, the anchor looses it's bearing capacity with combination of contact failure and concrete failure.

The analysis presented in the paper is based on experimental and numerical research.

Due to cracks propagation from the side of unloaded concrete surface, the bond stress along the loaded anchor bolt is not uniformly distributed. This is the difference between the pull out tests and bond quality tests boundary conditions. Numerical simulations have proved that the distribution of bond stress at the ultimate load depends on the relation between the anchoring length and anchor diameter. This problem can be included to the relation for full contact failure by reduced effective anchoring length as (1).

$$h_{ef} = (h - 1.4 \cdot d_0) \tag{1}$$

where h_{ef} = effective anchoring length; d_0 = diameter of the drilled opening for anchor installation. The relation (2) describes the characteristic value of ultimate anchor load carrying capacity at tensile load given by approximation of experimental results.

$$N_{u,\tau,k} = \pi \cdot 0{,}74 \cdot \tau_{Rk} \left(1 - e^{-1.5 \cdot \frac{f_{c,cube,k}}{\tau_{Rk}}}\right) \cdot d_0 \cdot h_{ef} \tag{2}$$

where τ_{Rk} = characteristic value of bond stress for specific glue

The analysis proved that the simplified methods assuming the separate failures of materials are not accurate and partially unsafe. Real failures always involve a combination of concrete and glue failure.

The presented relation was obtained by an approximation of experimental and numerical model results, and it is based on full contact failure including the influence of concrete characteristics and also the real boundary conditions.

These results were achieved with the financial assistance of Ministry of Education, Youth and Sports project No. 1M0579, within activities of the CIDEAS research centre and project GAČR 103/09/1258.

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Bond between corroded steel rebar and concrete: Experimental tests and finite element analyses

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ABSTRACT

Experimental tests of push-pull type were carried out at the Politecnico di Milano to study the effects of reinforcement confinement, natural corrosion and repeated cyclic loading on bond between steel rebar and concrete.

The different results obtained in the experimental tests can be explained with the different conditions of the concrete cover and with the different levels of corrosion of the transversal reinforcement and of the longitudinal bar. The conditions of the concrete cover and the corrosion state of the stirrups influenced the confinement, while the integrity of the longitudinal bar directly affected bond. In some cases the presence of an effective confinement prevented the formation of splitting cracks and high peak values of bond stress were achieved. The steel yielding preceded the bond failure, influencing the obtained results.

In other cases a marked deterioration of the outer surface of the concrete and a bar with evident signs of corrosion were observed. The premature bond failure prevented the attainment of the bar yielding and therefore a lower value of the slip corresponding to the peak stress was recorded.

The presence of corrosion products on the bar increased the steel-concrete bond for slight corrosion levels, while bond diminished for higher corrosion values. Such results can explain the differences recorded in the load peak values in the tests with the same load history.

Detailed non-linear finite element models of the test specimens were created by using the finite element Abaqus code in order to provide an interpretation of the different experimental results. Three-dimensional axisymmetric elements were adopted to model the steel bar with ribs of annular shape and the surrounding concrete.

In order to model the confinement provided by the stirrups, springs were arranged in the radial direction at the boundary of the concrete volume.

The interface between the steel rebar and the concrete was simulated by using a surface-based interaction.

The volume increase of the corrosion products compared with the virgin steel was taken into account by means of thermal strains imposed for the steel bar. An orthotropic thermal expansion was specified in order to obtain radial expansion, causing a radial and circumferential stress state in the concrete, and radial cracking.

A detailed finite element model was created to reproduce the experimental tests and provide an understanding of the test results. The numerical results obtained, compared with the experimental data, indicate the very important role of confinement as a parameter influencing the bond strength of corroded bars. The average bond strength decreases and the slip increases with increasing corrosion. An initial small increase of bond strength for low level of corrosion was observed, followed by an appreciable reduction, particularly if low levels of confinement reinforcement were used. Bond deterioration was limited by confinement for a certain range of increasing corrosion levels. The application of repeated loads caused an appreciable bond deterioration.

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Behavior of screw anchors in normal strength concrete at pullout failure

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ABSTRACT

For the connection of building components there are several alternatives in the fastening technology like cast-in-place installations (e.g. anchor channels) or post-installed installations (e.g. undercut anchors). The failure load of these fastenings can be calculated with the Concrete-Capacity-Method (CC-Method) with sufficient accuracy.

In the special case of using bonded anchors, the CC-Method is valid only for bonded anchors with a high bond strength which leads to concrete cone failure. For bonded anchors which fail by pullout, a modified CC-Method has been developed at the Institute. In this method the pullout failure load is not only influenced by the embedment depth and the type of bonding material, but also by the anchor spacing for grouped bonded anchors.

Short concrete screws typically fail by exceeding the tensile strength of concrete. However in practice long and thick concrete screws are needed, which can fail by pullout. In this case the accuracy of the CC-Method may not be sufficient enough. With bonded anchors and concrete screws the load is transferred approximately uniformly over the embedment depth. Therefore the design model for bonded anchors may also be valid for concrete screws. However, no results are available to judge the accuracy of this assumption.

For evaluating the accuracy of the bond model for concrete screws, numerical and experimental studies with concrete screws, which are failing by pullout, have been performed. The results of these investigations will be presented in the paper.

The evaluation shows that the modified CC-Method, which is valid for bonded anchors, is not adequate enough for concrete screws. Hence the behavior of concrete screws under tensile loading is not comparable to the behavior of bonded anchors. An explanation for this reason has not been found, yet. The numerical investigation shows that the load transfer of bonded anchors and concrete screws is different.

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Prediction of ultimate stress in unbonded tendon for post-tensioned concrete beams

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ABSTRACT

Post-tensioning is one of the most frequently used technique in designing long span bridges and buildings with large slabs. The method of post-tensioning is also one of the primary methods used in rehabilitation and strengthening of bridges (external unbonded). In unbonded post-tensioned beams, the void between the strand and the duct is left empty. However, many questions have been raised as to what is the level of safety associated with such techniques and how would these structural elements behave at the ultimate limit states. Therefore, there is a need to establish rational, but simplified as well as accurate, equation for the prediction of stress in beams prestressed with unbonded (internal as well as external) tendons.

This paper presents an equation derived by Ozkul et al. (2008) that was used to accurately predict the tendon stress at ultimate in post-tensioned beams. The equation is derived from an analytical study and validated using experimental data. The analytical model assumes that the concrete beam and unbonded tendon are linked at the holding point by a rigid link. The derived equation is as follows:

$$f_{ps} = f_{pe} + \frac{E_{ps}}{196}\left[\frac{e\,\beta_1 f_c' b}{A_s f_y + A_{ps} f_{pu}} k_1\right] \le f_{py} \tag{1}$$

where, $L_h = \dfrac{L}{2} - \dfrac{L}{2f} - (0.5d + 0.05Z)$,

$k_1 = \left[1 - 2\dfrac{L_h}{L} - \dfrac{L_p^2}{L_L L}\right]$, f is a function of loading type, $f = 10$, 3, and 6 for one point, two point, and uniform loading, respectively, Z is the distance from the critical section to the point of contraflexure, and L_p is the plastic hinging zone length.

Results are compared to those from test beams as well as available test data on concrete beams prestressed with unbonded tendons. Figure 1 show the stress at ultimate in unbonded tendon predicted using Eq. (1) in comparison with the experimental values.

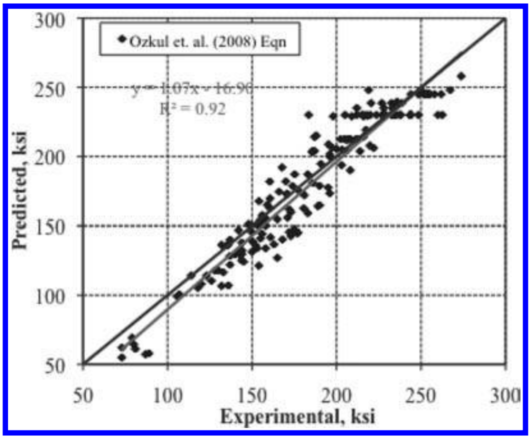

Figure 1. Comparisons of Predicted and Experimental f_{ps} for Eq. (1) by Ozkul et. al (2008).

The correlation coefficient R^2 using f_{ps} for Eq. (3) is 0.92. In comparison, the ACI 318-08 equation exhibited a correlation coefficient R^2 using f_{ps} of 0.73. In comparison with experimental data, the prediction equation demonstrated close correlation and high accuracy in predicting the stress in the tendon at ultimate. Therefore, Eq. (1) is an accurate alternative for predicting the stress at ultimate by encompassing all of the beam parameters. The comprehensive nature of this new equation allows for its application on various types of tendon material, beam geometry, and loading.

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Long-term behaviour of GFRP tendon

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ABSTRACT

Nowadays composite materials are used more often in every parts of industry including civil engineering. Using of these composite materials in civil engineering is innovational and there are many unanswered questions about these materials and relaxation of GFRP (glass fiber reinforced polymers) tendon in prestressed concrete is one of them. Knowing of long term behavior of the prestressed GFRP tendon is very important for the right design. Underestimating of long term changes in the GFRP tendon can lead to the serious problems or collapse of a structure.

The paper deals with long-term performance and material characterization of GFRP tendons in prestressed concrete structures. Regarding pre-stressed GFRP tendon itself relaxation of this composite material has not been fully scanned so far. Therefore relaxation test was carried out in order to describe stress-time relaxation curve for GFRP tendon. GFRP tendon was pre-stressed and deformation was kept constant. Pre-stressing force was measured and value was recorded.

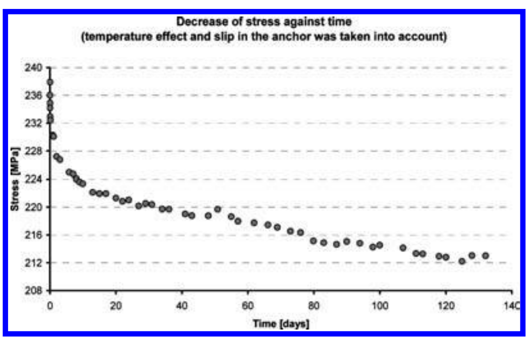

Figure 1. Relaxation test output.

It has been verified experimentally that tensile stress decreases in the GFRP tendon when deformation is kept constant. Stress loss after 28 days reached 7.3%. Loss after 130 days reached almost 10.5% (Fig. 1). Nevertheless the most important characteristic of curve geometry of the pre-stressing force loss in GFRP tendon was carried out. Aim of the research was also to develop a visco-elastic predictive model of stress-decreasing curve through the Kelvin and Maxwell chain. This model should help us to describe numerically a relaxation curve for different fiber reinforced polymers and to predict losses without long-term testing.

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Special charges for thin-walled structures demolition

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ABSTRACT

Thousands of precast concrete panel buildings constructed in the past six decades through out Europe are nowadays attaining their intended lifetime.

There are three general methods for the demolition of the prefabricated panel houses: sequential dismantling with manual joints-disconnection, machine-demolition with hydraulic tools and blasting.

During both manual and machine demolition, the construction system is found in special conditions. The danger lies in the fact that the system was not designed and viewed for these conditions. The panel construction is rigid and interlaced and therefore is able to transfer relatively high surcharge. But then the small impulse can invoke the progressive collapse of whole house with tragic results.

Although in the Czech Republic there is not the blasting technique thought as suitable for the demolition of panel-houses in general, the reality is the other. All the preparative works are made on the non-failed construction using of this technology. That way practically negates the hazard of the progressive collapse and guarantees the maximum volume of the workers-safety. So the blasting technology appears as the optimal for the demolition of the prefabricated panel-house.

Of course the design of the charge can not think the concrete-crushing and the armature-cutting together. Practically the charge is designed only for the concrete-crushing and the armature is subsequently disconnected otherwise. Now the destruction of the reinforced-concrete construction is designed with a view to the change of the static scheme. The blast breaks down the specific concrete section and the left armature can not transfer the concentrated loading and the construction must collapse.

A computer simulation may serve as a useful tool to facilitate a conscious design of safe and efficient deconstruction procedures. However, in a contrast to a standard structural analysis, the main interest here is prediction of mechanical behaviour of a building structure during the phase when it disintegrates and loses static stability.

The proposed methodology appears to be efficient from the viewpoint of computational efficiency, since it separates detailed analysis of complicated fracture phenomena and the overall dynamic analysis of an entire building. It also allows implementation of further enhancements, e.g. extension to 3-D.

The destruction of the reinforced-concrete constructions using explosives is consequently the branch with good chance of the next grow. The workers-safety, work-speed, minimum environment-stress and the high degree of the qualification are undisputed advantages of the blasting technology. One of the possible ways to the rationalisation of the work is using the non-blasting-holes charges. Using the shaped charge we can evoke the building-collapse by the change of the static scheme and not by the direct destruction of the support.

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Modelling of the surface of fair-face concrete structures

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ABSTRACT

Concrete becomes still more frequently used material which makes up visual form of the modern buildings. Architects choose fair-face concrete not only for its excellent physical-mechanical properties (high pressure strength, high durability, etc.) but also for its architectural properties, like the possibility to create all sorts of shapes and variably looking surfaces.

The unique appearance of the concrete structures is due to the fresh concrete composition and the method of its processing. High-quality fair-face concrete could be achieved only when the design is perfect, technological rules are fulfilled during the realization and architect, investor and contractor cooperate together. This article deals about realization draft of the structures made of fair-face concrete.

Visual appearance of the concrete structure is influenced by many factors. The most influencing elements are:

– *Formwork types*. There are three basic types of formwork: beam, frame and individual. Each of them provides a unique appearance of the concrete surface.
– *Formwork coat*. There are various types of coatings which can be used for creation of the concrete surface – plywood with or without additional surface sheet, plastics matrix, wooden planks, metal plates, etc. Each coating creates original look of concrete.
– *Composition of concrete*. Concrete is non-homogeneous mixture therefore it's difficult to achieve uniform color of the fair-face concrete structures. Due to this fact it's necessary to have constant composition of concrete mixture during realization.
– *Weather*. Weather is one of the least predictable and the most influencing factor affecting fair-face concrete appearance. The following climate factors affect view of concrete surface: air temperature, air humidity, rain and snow-shower.
– *Human factor*. To make quality fair-face concrete workers have to fulfill all technological specifications and architects have to make complete and real design.
– *Finishing the concrete surface*. There are various methods of finishing the concrete surface. Most of these methods are based on removing a thick surface layer of concrete and exposing color coarse aggregate. The methods could be divided into following categories: manual treatment, mechanical treatment, technical treatment and brushing and washing.

Figure 1. New library in Hradec Kralove made of fair-face concrete, Czech Republic.

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Computer application of yield line theory in the analysis of solid slabs

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ABSTRACT

The complexity and conservative nature of the Yield Line Theory and its being an upper bound theory have made many design engineers to jettison the use of the analytical method in the analysis of slabs. Before now, the method has basically been a manual or hand method which some engineers did not see a need for its use since there are many computer based packages in the analysis and design of slabs and other civil engineering structures.

This paper presents a computer program that has adopted the yield line theory in the analysis of solid slabs. Two rectangular slabs of the same depth but different dimensions were investigated. The Computer-based Yield Line Theory was compared with two other analytical methods namely, Finite Element Method, which was adopted in developing a computer program called Prokon, and Elastic Theory Method, which was also used by the Reinforced Concrete Council (RCC) to develop a computer program and it was fashioned after BS 8110 1997.

The results obtained for a two-way spanning slab showed that the Yield Line Theory is truly conservative, but increasing the result by 20% caused the moment obtained to be very close to the results of the other two methods. Although it was still conservative, the results showed that the Elastic Theory method that was used in the RCC design generally gave higher results, while the Finite Element Method of Prokon was in between the other two methods, the check for deflections showed that Yield Line Theory is reliable and economical in terms of reinforcement provision.

For a one way spanning slab the results for the Yield Line Theory falls in between the two other methods with the Elastic method giving a conservative results. The paper concludes that the introduction of a computer-based yield line theory program will make the analytical method acceptable to design engineers in the developing countries of the world.

Table 1. Analytical results for 6 × 4 m slab

Edge	Yield-line theory (kNm) Normal	20%	RCC (kNm)	Prokon (kNm)
4c	6.12	7.34	8.20	7.80
3c	7.42	8.90	11.30	9.90
2c	8.40	10.08	12.93	11.40
1c	10.54	12.64	7.38	13.90
Free	12.24	14.69	19.10	16.50

C = Continuous edge

Table 2. Analytical results for 5 × 2 m one-way slab

Edges	Yield-line (kNm) Normal	RCC (kNm)	Prokon (kNm)
2c	20.19	20.40	22.40
1c	27.71	21.30	32.40
Free	40.33	34.00	43.00

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11. *Fibre-reinforced concrete, high-strength concrete, high-performance concrete*

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Ultra high performance concrete structures: Recent developments in research and practice

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ABSTRACT

Ultra-High-Performance Concrete (UHPC) is a challenging new material which enables both durable and lightweight structures. It offers a high potential to replace steel in slender and long-span structures. Therefore, it has been in the focus of research at the Institute of Structural Engineering at the University of Kassel since several years.

The present paper gives an overview about the applications which become possible with this high strength and highly durable material as well as on pertinent research activities. Furthermore, recent research results with regard to the modeling of the behavior of UHPC structural members will be presented.

UHPC has been used within a set of applications in Germany until now. Thereby, it has proven its economical and technical benefits.

Aesthetic and filigree structures, smaller weight, reduction of the construction period, and high durability with smaller maintenance costs are significant advantages of the new material.

In the Kassel area, Germany, a series of pedestrian bridges have been built using UHPC. For one span bridges, the superstructure has been prefabricated as a monolithic prestressed or posttensioned element.

The Gaertnerplatz-Bridge in Kassel is the first multispan hybrid UHPC/steel bridge with about 134 m overall length. The longest single span holds a length of 36 m. The longitudinal girder is a three-chord truss with variable depth (Fig. 1). The top chord is made of UHPC. The bottom chord and the diagonals are formed by steel tubes and are connected with the top chord by high-strength friction grip fastening.

The bridge deck consists of precast slabs with a width of 5 m, which are pretensioned in transverse direction. Due to drainage the slabs have a thickness of 8 cm in mid-span and a thickness of about 11 cm on the edge. After installing the three-chord truss, the precast slabs were connected with each other and with the

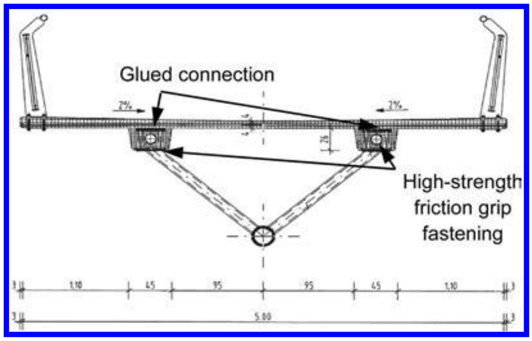

Figure 1. Cross-section of the Gaertnerplatz-Bridge.

top chords only by gluing. No mechanical connectors have been used.

Other actual applications cover the strengthening use of UHPC for strengthening of normal strength concrete beams in shear and/or bending as well as the use of UHPC for in-situ concrete topping for bridges made of prefabricated girders.

Current research on UHPC concerns the topics of combined action of bar reinforcement and fibers in members subjected to bending/shear as well as experimental and theoretical investigations with respect to the use of UHPC for structures endangered by impact loading (e.g. for high rise buildings subject to terrorist attack)

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Nanotechnology: A new approach for high performance concretes and sustainable concrete structures

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ABSTRACT

Ultra-High-Performance Concrete (UHPC) with its very dense microstructure and its steel-like compressive strength of about 200 MPa allows for slender but nevertheless very long lasting and thus sustainable concrete structures. In Germany, a comprehensive research program is performed to make UHPC a commonly available, technologically sound, and standardized material based on individual regionally available raw materials. An extra benefit can be expected by applying nanotechnological approaches to further increase the strength up to 500 MPa and more, and to make concrete an impervious, ceramic-like, multifunctional material, e.g. with the ability to degenerate air pollutants.

To bring this approach to success, a comprehensive 10 Mio. € Research Program on UHPC is currently in progress in Germany (Schmidt 2010). The program is funded by the German Research Foundation (DFG) and coordinated by the University of Kassel. More than 20 research institutes are involved. The individual projects cover the suitability and performance of raw materials including artificial nanoparticles, appropriate mix designs for different applications, the rheological properties of fresh concrete as well as the time-dependent strength and the deformation behavior of hardened UHPC with and without fibers. Further topics are new fitting technologies for precast members – e.g. gluing with epoxy resin or cement-based adhesives – and adequate design and construction principles including new appropriate technologies to build high-performing but yet light and slender sustainable structures suitable for the material. A first large scale practical application of UHPC in Germany is the 130 m long Gaertnerplatzbridge in Kassel, a very slender hybride structure consisting of a steel truss and prefabricated prestressed UHPC-elements which have been fitted just by gluing with an epoxy-resin mortar.

A further increase in performance can be expected by incorporating synthetic nanosilica particles with a carefully controlled particle size distribution extending the packing optimization to the nanoscale (Schmidt et al. 2007) Furthermore, so called "alternative binders" (which are largely free of Portland cement clinker) based e.g. on granulated slag activated by sulfuric or alkaline accelerators (Stephan et al. 2009) – yield a structure that proves stable against most acidic attacks. This is part of current research projects aiming at designing concretes that behave like ceramics.

Also, nanotechnology helps to make concrete a multipurpose "smart" functional material. When concrete is treated or coated with thin layers containing photocatalytic titanium dioxide (TiO_2) particles, it can be used as air-purifying surface. In view of the enormous facades and roof areas, this could make a lasting contribution to the protection of the environment.

According to Stephan (2006), in addition to its photocatalytic properties, TiO_2 also develops a superhydrophilic surface under irradiation by UV light making the surface absolutely self-cleaning.

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Cracking behavior of fibre reinforced concrete beams containing longitudinal reinforcement

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ABSTRACT

One of the important parameters that determines the durability of a concrete structure is the crack formation in that element. One of the often-used techniques to decrease the crack widths is the use of steel fibre reinforced concrete in combination with normal longitudinal reinforcement.

To control crack widths during design, a good calculation method is needed. The formation of cracks is influenced by a large number of parameters. The most important are the dimensions of the cross section, the bond stress-slip relation of the rebars, the tensile and postcracking tensile strength of the FRC. Several calculation methods can be found in literature and are summarized in (Dupont, 2003). These can be divided into semi-empirical relations and physical models. The semi-empirical relations are mostly validated with a large number of experimental results. To overcome this problem, several physical models have been developed. These physical models should have the advantage that the influencing parameters are taken better into account, so that the physical model can be extrapolated more safely to other applications. In this paper, two models, i.e. on the one hand the semi-empirical method in the σ-ε-recommendation of RILEM TC162-TDF and on the other hand the physical model developed by D. Dupont, have been used to predict the crack widths.

Most literature regarding the cracking behavior of SFRC is related to hooked-end steel fibres. In order to investigate if other types of fibres have a similar effect, 12 full-scale FRC beams containing both longitudinal reinforcement and fibres have been tested at the Department of Civil Engineering of the KULeuven. The investigated parameters were the fibre type and the a/d-relation (a : distance between support and load – d: effective depth).

The semi-empirical RILEM model overestimates the experimentally measured crack widths for all types of fibres. The physical model of D. Dupont, however, slightly underestimates the experimentally measured crack widths for the steel fibre reinforced concrete while there is a serious overestimation for the macro-synthetic fibre reinforced concrete.

Figure 1. Test setup for the full-scale beams.

The work presented in this paper is a part of a research program funded by the Federal Public Service "Economy" which is gratefully acknowledged. The partners in this project are KULeuven and BBRI (Belgian Building Research Institute).

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Numerical analysis of concrete beams reinforced with steel fibers

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ABSTRACT

Present research was aimed at experimental and theoretical investigation of tension-stiffening of steel fiber reinforced concrete (SFRC) beams. The paper reports on test results of five beams reinforced with three bars of tensile reinforcement 10 mm in diameter. The beams had different contents of fibers (f). Main parameters of the beams are given in Table 1.

The beams were tested under a four-point bending scheme. Prior to the tests, measurement on concrete shrinkage was performed. Moment-curvature diagrams of the beams are shown in Figure 1.

The technique proposed by Gribniak (2009) has been applied to the test data and *free-of-shrinkage* tension-stiffening relationships were derived. These diagrams are presented in Figure 2. The figure indicates that tension-stiffening increases with growing amount of fiber until the content reaches 1% of the element volume. Under this limit, the efficiency of fiber increases proportionally to its content, whereas, its influence is relatively insignificant above the limit.

Table 1. Main characteristics of experimental beams.

Beam	h mm	b	d	a_2	Age days	f_{cu} MPa	ε_{sh} μm/m	f %
*S*3-2-3	298	284	271	32	47	50.9	−210.9	–
*S*3-2-7*F*	298	284	271	20	49	44.6	−205.3	0.29
*S*3-1-*F*05	302	278	278	29	170	55.6	−311.2	0.47
*S*3-1-*F*10	300	279	276	23	161	48.0	−335.5	1.02
*S*3-1-*F*15	300	279	272	26	159	52.2	−315.7	1.46

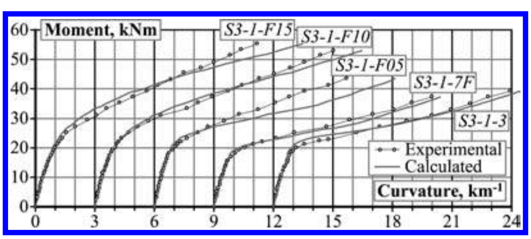

Figure 1. Measured and calculated curvatures of the test beams.

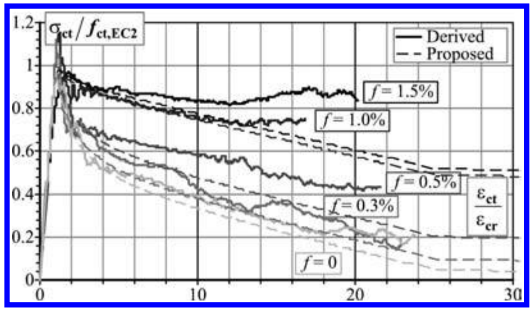

Figure 2. *Free-of-shrinkage* tension-stiffening diagrams.

Numerical analysis of the test beams has been carried out by commercial FE software *ATENA* using the proposed tension-stiffening relationships (shown in Figure 2 by dashed lines). The predicted and the measured curvatures of the beams are shown in Figure 1. Good agreement between the theoretical and test results can be stated.

It can be concluded that the proposed technique for analysis of SFRC beams is capable in deriving tension-stiffening relationships used for numerical modeling.

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Efficiency of steel fibers regarding the shear carrying capacity of UHPC beams

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ABSTRACT: Ultra-High Performance Concrete (UHPC) is a high-tech material opening up new opportunities especially for slender constructions. Within the priority program supported by the German Research Foundation (DFG), 42 shear tests on pretensioned beams have been carried out at the Institute of Structural Concrete at RWTH Aachen University.

Different amounts of steel fibers have been added to the concrete to ensure a sufficient ductility and also to serve as shear reinforcement. Further main parameters were the prestressing, the shear slenderness, the number and position of web openings, the opening diameter and additional shear reinforcement. The investigations, which were carried out between 2006 and 2009, have been presented amongst others by Hegger & Bertram in Kassel (2008), Amsterdam (2008), by Betram & Hegger in Tokyo (2008,) and London (2009). This paper gives an overview of the test results, especially regarding the effect of the fibers. More details will be given in the presentation and in Montreal (2010).

1 INTRODUCTION

A priority program on UHPC with over 20 projects in Germany started in 2005. The shear carrying behavior of pretensioned beams made of UHPC as well as the bond behavior of strands in UHPC have been investigated in one of these projects at the Institute of Structural Concrete at RWTH Aachen University.

The compressive strength of UHPC is about five times the strength of conventional normal strength concrete. Therefore, a high degree of prestressing can be applied and thus, more slender structures are feasible. This leads to significant savings in dead load and transportation costs which is an important issue especially for prefabrication. The stringent production requirements for UHPC restrict the main field of application to prefabricated members, e.g. roof girders of large span. Adding steel fibers to the concrete contributes to the shear resistance and improves the post-cracking behavior. Thus, steel fibers eliminate the need for conventional shear reinforcement. Generally, this will be allowed in Germany according to the guideline for steel fiber reinforced concrete (2010), which contains additions to the German Design Code (2008). In order to accommodate building utilities, web openings influencing the ultimate shear carrying capacity are frequently arranged in the girders. It is therefore necessary to investigate the shear behavior of UHPC beams with and without web openings. In order to investigate the shear strength of such beams, an extensive experimental program consisting of a total of 42 tests on beams with and without web openings was carried out. The investigated test parameters were the fiber content, the grade of prestressing applied, the shear slenderness as well as the location and number of web openings in the beams. Furthermore, different shapes of additional shear reinforcement near the openings were investigated and compared to the bearing capacity of beams without such reinforcement. The tests have shown, that beams with steel fibers but without shear reinforcement have a high shear carrying capacity and a sufficient ductility, which means a adequate failure indication, for example by deflection or cracks. Without steel fibers the ultimate shear stresses in solid beams were about 7–8 MPa, with steel fibers they range between 12 and 22 MPa considering the fiber content and prestressing.

The beams presented in this paper have a cross section with a height of 40 cm and a web thickness of 6 cm. The tests following in 2010 will be focused on size effects. Thus, the height will be increased. Generally, the effective depth is the most used influencing factor for size effects in shear models. In the case of fiber reinforced concrete, a further size effect due to the web thickness occurs which is based on different fiber orientations close to the formwork surface.

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Long-term investigation of concrete elements with FRP

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ABSTRACT

Fibre reinforced polymers (FRP) presents an alternative to the conventional steel reinforcement. High strength, low self weight, thermal and electric non-conductivity and non-corrodibility are the biggest advantages of FRP.

Apart from the high strength there is a relatively small elastic modulus of the FRP reinforcement. Typically this value varies around 40 GPa and 120 GPa for glass FRP (GFRP) and carbon FRP (CFRP), respectively.

Stress strain diagram of any kind of FRP bar is typically linear-elastic up to the sudden rupture. There is no yield domain or any other sigh of ductility. This reflects significantly behaviour of FRP reinforced concrete element mainly in terms of force-deflection relationship. Therefore it can be expected that long term investigation will present some fundamental differences.

It was verified experimentally that concrete slab prestressed with GFRP bars presents enormous increase of deflection and cracking in time. Serviceability limit state (SLS) requirements in terms of deflection were overstepped shortly after sustained load application. Namely deflection reached value of 1/250·L (16 mm) in the 169th day. This is mainly caused by the very small elastic modulus of the GFRP reinforcement (40 GPa). Also GFRP tendons are liable to relaxation, which is the most significant prestressed loss and affects significantly the overall behaviour. Overall deflection in the mid-span reached value of 19.86 mm after a year under the sustained load. This corresponds to the 7.49 in terms of time dependent to initial elastic deformation ratio. Creep rate of the GFRP prestressed slab is specified to 1 mm per 75 days. Utilization of GFRP prestressed concrete elements is therefore possible only in specific sites where everyday loadings are small.

Concrete slab prestressed with CFRP rods presents better results in terms of deflection evolution in time. SLS requirements in terms of deflection were not reached after one year under the sustained load. At the 300th day mid-span deflection showed value 8.61 mm which is circa half of the SLS criteria (16 mm). This corresponds to the 3.63 in terms of time dependent to initial elastic deformation ratio. Overall deflection in the mid-span after a year under the sustained load was 8.90 mm. Creep rate of the CFRP prestressed slab is specified to 1 mm per 98 days. Very good behaviour is also given by the reason that the carbon fibres do not relax as all under the sustain load. Therefore those are more suitable for civil engineering purposes.

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Advantages of textile reinforced concrete applied to a pedestrian bridge

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ABSTRACT

The pedestrian bridge over a state road in Albstadt, Germany, had immense corrosion damages of the steel reinforcement which are caused by too small concrete covers. Thus, the bridge had to be torn down and has actually been replaced by a new bridge. The aim of the design of the new bridge is a slender fair-faced concrete superstructure fulfilling demands on a frost resistant construction. Thus, the innovative composite material textile reinforced concrete (TRC) was chosen as construction material instead of the commonly used steel reinforcement. The main advantage of using textiles is the possibility of reducing the concrete cover to a minimum because the corrosion protection needed for steel reinforcement is not required. The concrete covers can be reduced to a minimum of only some millimeters resulting in slender, light-weight and sharp-edged structures with high-quality surfaces.

In this project a reinforcement structure made of technical textiles in combination with a fine-grained concrete is used. This high performance concrete has a maximum grain-size of 4 mm.

The pedestrian bridge with a total length of nearly 100 m is subdivided into six prefabricated parts, each with a maximum length of 17.20 m and a maximum span of 15.05 m. The cross-section of the superstructure is a 3.21 m wide concrete T-beam with seven webs, each pre-stressed by four single-strand cables and reinforced with three GFRP-bars in addition to the textile reinforcement, Figure 1.

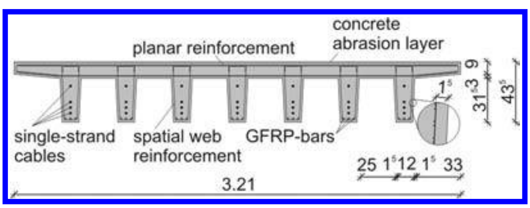

Figure 1. Cross section of the TRC superstructure.

Shear forces are borne solely by the spatial textile web reinforcement. With a height of the superstructure of only 43.5 cm the construction has a slenderness ratio of H:L = 1:35.

Within the scope of the structural analysis, especially the verification process of the oscillation behavior of the slender structure is presented by means of a three-dimensional finite elements model.

Today TRC is not regulated by any standards in Germany, thus, an individual approval of the construction by the building authorities is required. Therefore a large scale testing program was conducted. In the scope of this paper, the load-bearing behavior of the longitudinal beam under shear force is presented exemplarily, describing the main test in longitudinal direction of the bridge. All tests showed that in serviceability limit state, the members remained uncracked, while a maximum crack width of 0.3 mm was allowed by the client. In ultimate limit state the specimen have high deformations and a well developed crack pattern. All in all, a global safety factor of 5.0 is achieved.

This paper shows the application of TRC for large-scale members on the example of a 100 m long pedestrian bridge in Albstadt, Germany. It is possible to design slender and light-weight concrete members by using technical textiles instead of steel as reinforcement. Furthermore, the fine-grained concrete allows for sharp-edged members, which meet the needs of modern architecture.

The authors gratefully acknowledge the Groz-Beckert Group and the town of Albstadt, Germany, for their financial support and their willingness to undertake this pilot project. The authors also thank Sebastian Wochner GmbH & Co. KG Dormettingen (construction company), the Regional Commission Tübingen (construction supervision) and hns architects Stuttgart (architectural design) for the efficient collaboration. Special thanks go to the German Research Foundation (DFG) which financed the Collaborative Research Center 532 at RWTH Aachen University.

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Slender façade structure made of textile reinforced concrete

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ABSTRACT

In the last decades, façade panels made of steel-reinforced concrete have become less attractive as building elements for architects and clients, because many cases of damage occurred due to corrosion of the steel reinforcement. Using mesh-like technical textiles for a new composite material, textile-reinforced concrete (TRC), it is nowadays possible to build light-weight and slender concrete structures which are invulnerable to corrosion.

Today in Germany, façade panels with small ($A \leq 1\,m^2$) to medium ($1 < A \leq 6\,m^2$) element sizes with metal stud-frame-systems are state-of-the-art in applications made of TRC. In a finished research project large-sized façade elements ($A = 12.2\,m^2$) with a thickness of only 30 mm have been developed at RWTH Aachen University in cooperation with Hering Group, a precast concrete supplier.

In a pilot project the developed large-sized panel is used as a façade for a new laboratory building, Figure 1. Due to the large size of these elements fewer mounting parts are necessary, Figure 2, thereby significantly reducing labor and material costs.

To enable an economic production of these structures in precast plants, inherently stable and prefabricated 3D textile reinforcements are required which allow ordinary casting methods. Fabrics made of alkali-resistant glass (AR-glass), coated with thermoset epoxy resins and cured within a mould in an oven can be prefabricated to almost any arbitrary shape, Figure 2. Such prefabricated reinforcements provide tensile strengths of ~1400 MPa at ultimate limit state in the façade-panel.

The paper describes the design of the façade panel, production process of planar and spatial textile reinforcement structures and gives information to the load-bearing behavior or TRC members.

Figure 1. Sample of a figure caption.

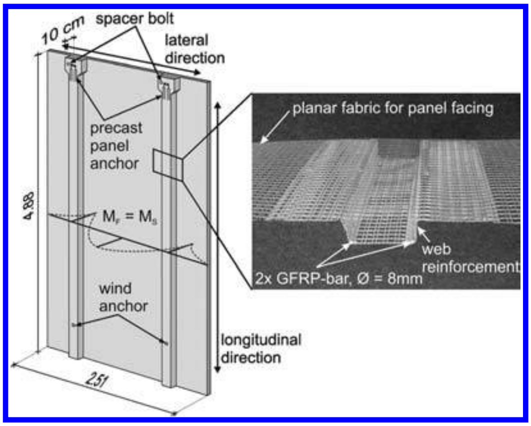

Figure 2. Façade panel and spatial web reinforcement.

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Modeling compressive strength of high-performance concrete with multiple regression

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ABSTRACT

Predicting the compressive strength of high-performance concrete is an important issue in the building practice, and therefore has been an active area of research. Recent studies in the subject indicated that compressive strength of high-performance concrete is determined not only by the water-to-cement ratio (W/C), but is influenced by the content of other concrete ingredients.

The objective of this work is to establish a mathematical model for the prediction of compressive strength of high-performance concrete. The basic concept of this model is to produce a reliable relationship between compressive strength, W/C ratio, concrete's components and age. To differentiate from previous studies, we use a modified multiple lineal regression. Many authors observed that compressive strength of concrete grows very slowly and it is well represented by a logarithmic function of time. We claim that it grows even slower and propose O(ln(ln(t))) class of functions as a good approximation of this relationship.

To test our ideas the experimental work was designed. Table 1 shows the content of components.

The compressive strength of high-performance concrete was determined with ten concrete mixtures and testing was performed after 2, 7, 28, 56 and 365 days of curing.

Table 1. Content of concrete mixtures' components (per m^3).

Component	Content, in kg/m^3
Portland cement CEM I 52.5R (C)	450–550
Tap water (W)	143–162
Coarse aggregate:	
Grit basalt 8–16 mm (CA_1)	535–570
Grit basalt 5–8 mm (CA_2)	468–652
Grit basalt 2–5 mm (CA_3)	201–659
Sand 0–2 mm (SA)	550–627
Silica fume (SF)	19.9–41.2
Superplasticizer (PC)	4.1–22.0

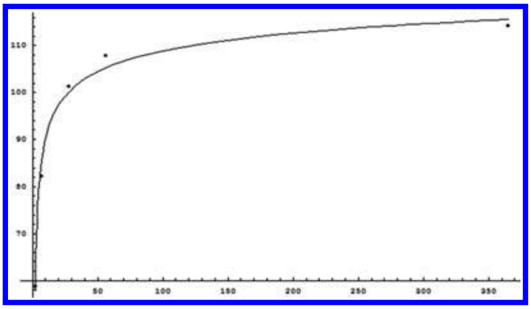

Figure 1. Relationship between compressive strength (f_c) and ln(ln(t)) for concrete C3.

Using Mathematica 5.0 program we obtain the equation for our model:

$$f_c = [0.22\ln(\ln(t)) + 0.60]\cdot(-190.66\frac{W}{C} - 1.47PC + 0.44SF - 6.56CA_1 - 4.02CA_2 - 6.06CA_3 + 11.50SA) \quad (1)$$

Figure 1 shows the exemplary relationship investigated.

We demonstrate the better performance of iterated logarithm as compared to the single. The proposed model provides the practical opportunity to predict the compressive strength as the function of time under existing concrete's components conditions.

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12. Durability, creep, shrinkage and transport processes in concrete

Advances and Trends in Structural Engineering, Mechanics and Computation – Zingoni (Ed.)
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Effects of creep and shrinkage of concrete on composite structures

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ABSTRACT

The paper deals with a research project, which was initiated to verify actual effects of creep and shrinkage of concrete (both normal-weight and light-weight aggregate concrete) on composite structures. For this reason, two nearly identical steel-concrete composite testing beams were prepared, varying only in the type of the used concrete. This paper describes the tests realized on the beam using normal-weight concrete. To ensure clear representation of results, a specimen with a steel basic beam is used, but the principles are valuable also for other types of similar structures, for example concrete–concrete composite structures.

Steel beam is made of material S355, composite slab is made of concrete C30/37. A welded steel beam was manufactured for the possibility to use relatively small thickness of steel sheets and so to obtain higher (and better measurable) values of stresses. The beam is 3,2 m long and is equipped with sensors for determination of strain and temperature.

Measured results are compared with values calculated according to several creep and shrinkage models. Figure 1 shows theoretical and measured values of normal stress in the middle of span, determinated for upper and bottom surface of the steel beam. Theoretical values were determinated according to the publication *Structural Concrete (fib* 2009) and according to the model B3 (Bažant 1995).

Measurements are supplemented with tests of basic material properties of the used concrete in particular time stages (compressive and tensile strength, modulus of elasticity, fracture energy, creep and shrinkage), as well as measurements of temperature and relative humidity of the surrounding environment.

Present results have approved a good compliance between theoretical and experimental values. The measurements are still running and they should take at least several years. In further time, other more detailed calculations will be also carried out to analyze some special questions, and other types of beams will be tested in an analogous manner as well.

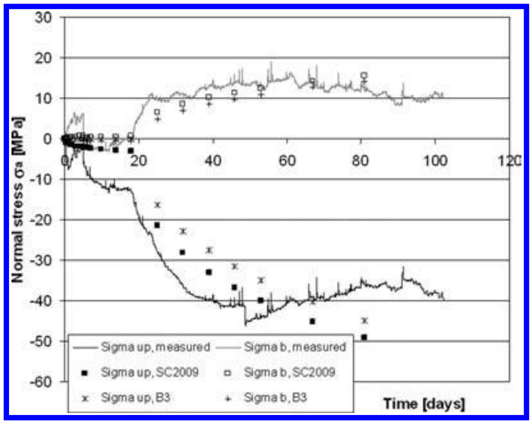

Figure 1. Comparison of measured and calculated values of normal stresses at midspan (upper and bottom surface of steel beam).

ACKNOWLEDGEMENT

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Effect of shrinkage on the time-dependent deflection of reinforced concrete slabs

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ABSTRACT

Laboratory tests have been undertaken on a number of identical reinforced concrete slab specimens with different load histories and different levels of drying shrinkage in order to quantify the time-dependent nature of tension stiffening in reinforced concrete flexural members. This paper discusses the factors affecting the change in tension stiffening with time. The experimental program is described and the results are presented and evaluated.

The instantaneous moment-deflection response of a typical reinforced concrete slab deflection is shown in Figure 1, together with the response at time *t* after a period of sustained loading with and without shrinkage.

Four identical slabs were cast from the same batch of concrete and subjected to different loading and drying histories. The specimens were tested in four point bending and two different test set-ups were employed (as shown in Figure 2) to assess the immediate response to load after differing levels of drying and the long-term response under sustained loads.

The effect of early shrinkage on the magnitude of the tension stiffening deflection is reported and the change in tension stiffening with time is determined. The measured forces in a steel bar determined from strain gauge measurements in one of the test specimens is plotted in Figure 3. The increase in steel stress away from the cracks (and the resulting reduction of tension in the tensile concrete between the cracks) in the first 15 days under load is evident.

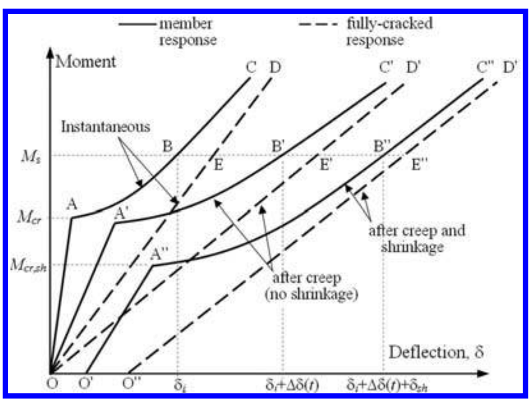

Figure 1. Long-term moment vs deflection relationship.

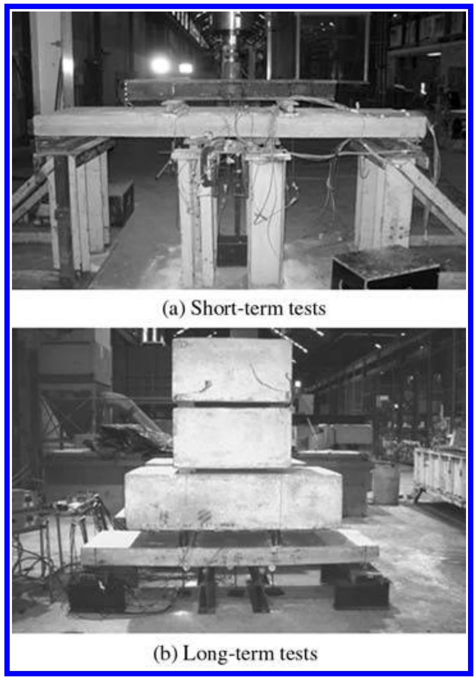

Figure 2. Experimental test set-ups.

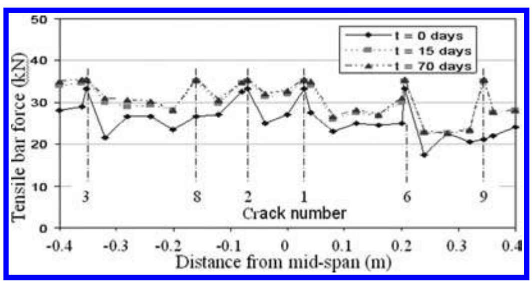

Figure 3. Tensile steel force in Slab S4 (Long-term tests).

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Creep and shrinkage prediction models for concrete water retaining structures in South Africa

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ABSTRACT

Concrete water retaining structures (WRS) in South Africa are under scrutiny due to the numerous durability problems that they have experienced lately. At the heart of these problems are the creep and shrinkage phenomena, for which extensive research has been conducted to properly identify their effects and predict their evolutions. This paper reports the progress in this research, in the form of development of pioneer South African data bases for creep and shrinkage, and their respective internal classifications as well as the investigation of three analytical prediction models, the GL 2000, CEB-FIP 1990 and EN 1992:2004-1-1.

The relationship between the CEB-FIP 1990 and the EN 1992:2004-1-1 is noticeable in their similar general forms. This means that for a given set of known concrete parameters, the calculated creep/shrinkage strains of these two models are mostly in close agreement. However some differences exist, most notably in the terms incorporating compressive strength dependence.

South African measured creep and shrinkage results were used to evaluate the analytical models. Data sets were carefully selected from a range of countrywide South African available experiments with criteria of data completeness for model parameter characterisation, as well as data and laboratory quality. The data was further selected to represent the concrete of WRS, through an elimination process involving typical concrete design mixtures used in the WRS construction industry. After the selection process, the experiments were classified into groups (with the same cement type, and compressive strength of concrete tested), then in clusters (of the same curing time and loading age used) and finally in data-sets (of similar relative humidity, temperature, volume/surface area and elastic modulus). In total, eleven (11) sets of data were created, and the comparison of calculated to measured strains over time could finally take place (see Figure 1).

The GL 2000 model is overly conservative in shrinkage prediction compared to South African data, while both CEB-FIP and the EN1992-1-1 models are accurate and in a few instances conservative. In terms of creep prediction, it appears at this stage that the

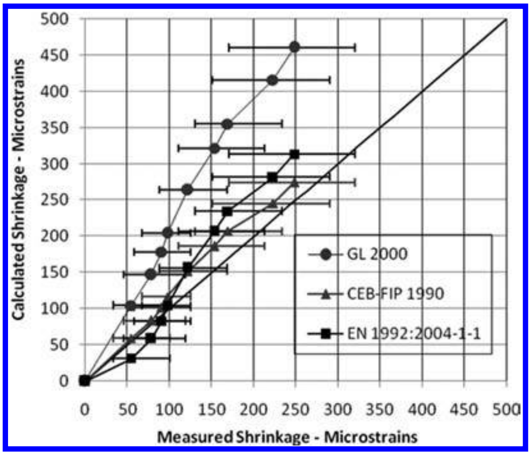

Figure 1. Typical comparison of prediction models (dataset B).

Table 1. Average error percentages of the analytical models.

Average %	GL 2000	CEB-FIP1990	EN 1992-1-1
Shrinkage	76	24	28
Creep	42	21	22

CEB-FIP and EN 1992-1-1 models are more accurate than GL 2000. However, in several instances South African creep data are under predicted by all three models, which must be addressed in future work. See Table 1 for overall results.

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Investigation of plastic shrinkage cracking in concrete

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ABSTRACT

Plastic shrinkage is the early age volume reduction of concrete due to the loss of water through evaporation. This mechanism can result in plastic shrinkage cracking if the concrete is restrained or if the evaporation causes a strain gradient in the material. These cracks not only result in an aesthetically unpleasing surface, but have serious durability issues due to the possibility of corroding agents infiltrating the concrete through the cracks. Plastic shrinkage cracking depends on various mechanisms, including the rate of evaporation and bleeding, particle size and distribution in the concrete. All these aforementioned mechanisms do not only vary for every condition and concrete mixture, but also varies with time for a specific condition. This paper presents the results of the preliminary tests on plastic shrinkage cracking.

The experiments were conducted in a climate chamber which allows constant environmental conditions with regard to wind speed, relative humidity and temperature. The climate chamber is shown in Figure 1. Each concrete mix is cast into two rectangular moulds used for crack width measurement and one square mould for concrete water evaporation measurement. The initial setting times of each concrete paste is determined with the Vicat needle apparatus (SANS 50196-3:2006). The bleeding is measured outside the climate chamber in cylinders with the same height as the moulds used for the concrete specimens. The bleed water is then extracted with a pipette and measured with an electronic scale. This is an similar setup to Josserand & de Larrard (2004).

Two different mixes were used in this study, namely a mix that showed significant bleeding and one that was almost without bleeding. The mixes differ only in sand type while the sand volume remained constant. Two climate conditions were chosen that would give different water evaporation rates. Both Climate 1 and Climate 2 had a constant temperature (45°C) and relative humidity (20%). The wind speed was 1.0 m/s for Climate 1 and 7.5 m/s for Climate 2 which gave

Figure 1. Climate chamber showing two rectangular moulds and one square mould for concrete as well as one pan for water.

average pan evaporation rates of 0.5 and 1.3 kg/m^2/h respectively.

The following conclusions could be made from these preliminary tests:

- For environmental conditions of high evaporation rates it seems that the amount of bleeding of the concrete does not influence the plastic shrinkage cracking of concrete. This is however deduced only from a limited test series and will be further investigated.
- It has been found from the limited tests that the plastic shrinkage cracking only occurs once the initial setting of concrete has taken place. This will however have to be verified in the future.
- The wind speed has a significant effect on the evaporation rate of the water in the concrete before setting.

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Numerical time-dependent analysis of prestress losses affected by rheology phenomena and nonlinear structural performance

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ABSTRACT: Design of prestressed concrete elements and structures is based on the determination of prestressing force (including losses and their time development) to satisfy stress as well as deformation limits. The intention of the paper is to describe especially long term prestressing losses affected by creep, shrinkage and warping.

1 INTRODUCTION

Prestressed concrete bridges are very sensitive to long-term structural performance. This phenomenon has paramount importance for serviceability, durability and long-time reliability of such bridges. This is why a reliable prediction of behaviour of bridges during their construction as well as during their service life is extremely important.

Long-term structure deformations (deflections and rotations) and stress distribution are extremely sensitive to prestressing losses, because the final value is a difference of great numbers (the effect of dead load and the opposite effect of equivalent loading due to prestressing, which depends on the real and actual value of prestressing force in each point of the structure). So it is evident that small changes in supposed values have significant effect on the result. Usually used computational routines and methods neglect or wrongly describe real effects and results obtained by analysis based on these methods could be wrong – great differences from results obtained by measurements on real structures.

One of the most important factors which provide perfect function of prestressed concrete structures (serviceability, loading capacity, durability) is real and correct assessment of prestressing – i.e. development of prestressing losses. It is possible to divide prestressing losses to two basic groups. The first one – short-term (technological) losses (e.g. losses due to friction, movement of tendons in anchorages) appear between time of prestressing and time of anchoring. The second group – long-term prestressing losses – appears between time of anchoring and the analysed time, and the final time of the operation life (e.g. losses due to creep, shrinkage, steel relaxation). It is evident that due to long-term character of this type of losses, their correct prediction has to be based on the right mathematical models for description of rheological properties of materials – concrete (creep and shrinkage) and steel (relaxation). For the analytical description of prestressing losses affected by cross section warping was used simple structural arrangement. A set of formulations was established to describe this effect by an analytical method.

For demonstration of relevance of the effect mentioned above the real simple structure element was analysed – slab beam with rectangular cross section (width $b = 3$ m, height $h = 0{,}6$ m) with the length $L = 10$ m. At time $t_p = 7$ days is prestressed by tendon (initial stress $\sigma_{p,0} = 1488$ MPa). For the calculation of rheology data of concrete, model B3 was used – modulus of elasticity at time t_p is $E_c(t_p) = 21748$ MPa, deformation due to shrinkage is ε_{shr} $(36500{,}7) = -430{,}074 \cdot 10^{-6}$, creep coefficient φ $(36500{,}7) = 2{,}49$. Total prestressing loss due to creep and shrinkage (including effect of cross section warping) is **14%**, without cross section warping it is **9%**. So the effect of warping plays significant role in prestressing losses calculation – increasing of the losses due to creep and shrinkage is approximately $14/9 \cdot 100 = \mathbf{156\%}$. Presented analysis confirmed common knowledge about importance of prestressing losses affected by creep and cross section warping.

Studies about prestressing losses play significant role for the right (correct) design of prestressed concrete structures. Results confirm consequence (especially for unevenly distributed prestressing tendons in cross section) of the effect of creep, shrinkage and cross section warping for prestressing losses, so it will be useful to established new type of prestressing loss – **prestressing losses affected by cross section warping increased by creep effect**.

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Performance characteristics of concrete wall panels under long-term loads

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ABSTRACT: The main objective relates to prestressed concrete load bearing wall panels, subjected to static long-term service loads. For structural and crack predictions, virtual work principles was used to estimate (a) transient strains due to the concrete creep and shrinkage, (b) the resulting time-dependent stress redistribution, as well as (c) displacement variations in the panels and finally (d) prestressing losses in the high yield tendons. The concrete shear stresses have been evaluated by the principle of Juravskiy.

The need to validate the numerical results, required detailed planning of a series of controlled experiments supporting the evaluations at (a) to (d), in addition to (e) calibrating the parameters of the prestressing losses, (f) predicting the panels sections' stiffness and strength by determining the flexural and nonlinear creep capacity of the panels' sections and hence, (g) devising a definition for structural durability and integrity, with regards to concrete stress-strain relationship under long-term loads.

Keywords: concrete creep, shrinkage, life safety, wall panels, strength, reliability, durability, prestressing tendons, prestressing losses

1 INTRODUCTION

It may be of grate importance to allow for factors influencing on the structural performance characteristics of the panels by averaging maximum and minimum estimated effects, and provided that this is done and there are a number of calculation methods available which will give reasonable results.

2 THEORETICAL ANALYSIS – CONCEPTUAL ACTING FORCES AND STRESSES

The numerical part of this study is based on the well-known concepts by Yasinskiy, Timoshenko, Vlasov, Drozdov, …etc and many other Authors, on the problem of strength, reliability, fatigue, life safety and stability of the panels. This study is also based on simplified equations, using the boundary conditions of the theory of elasticity concerning thin wall elements with small eccentricities.

3 EXPERIMENTAL ANALYSIS AND SOME SIMILAR TEST RESULTS

The experimental program consisted of casting of some specific subgroups (two wall panels per sub-group) of twelve $180 \times 180 \times 5$ cm pre-tensioned concrete wall panels in both directions with specified characteristic material properties of 40 MPa for the concrete and $E_{SP} = 195$ GPa for the high-yield steel tendons with nominal diameter of 10 mm which will be contained in frames made of 5 mm bent mild steel mesh with $E_S = 200$ GPa.

For the evaluation of concrete properties, specific subgroups (three cylinders and three cubes per sub-group) of 300×150Ø mm cylinders and 150 mm cubes will be tested periodically.

4 CONCLUSIONS

It is hoped to confirm at the end of the study that nonlinear concrete creep is able to contribute to the redistribution of stresses between the concrete and the steel reinforcement with long duration loads influences on structural panels. This said redistribution is able to provoke the accumulation of natural initial stresses in the steel reinforcement tendons and in the concrete, which are equivalent to the artificial initial stresses due to the prestressing efforts in the steel reinforcement tendons, and which could reduce losses and, consequently, able to increase the durability, the reliability and the exponents of the panels' limit states.

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Durability and long-term performance of concrete railway sleepers: Structural analysis and practice

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ABSTRACT

Twin-block concrete sleepers laid on railway tracks in operation in Greece presented extended cracking. The same type of sleepers were used in the French railway network with no problems. The existing, at that era, international bibliography cited many cases of appearance of cracks in monoblock sleepers of prestressed concrete as well. The extensive cracking led to a shortening of their durability and reduced their long-term performance, increasing the costs of maintenance. Experiments that were performed in the French railway network labs confirmed that these sleepers could safely carry the service and design load for which they were designed, in accordance with the specifications and regulations of the era. For the U2/U3 concrete twin-block sleepers, there are three regions of "strength": R1 region begins at 125 ÷ 130 kN, R2 region (cracking threshold) at 140 kN and the R3 region (failure threshold) between 140 and 175 kN.

An innovative formula (Giannakos et al., 2009) that was finally proposed, gives results that interpret the phenomenon of systematic appearance of cracks observed in a high percentage of sleepers. It is shown that if fastenings with very soft pads were used the appearance of cracks on the ties would have been avoided. The application of the above findings in the structural analysis and design leads to much better durability and long-term performance of the concrete sleepers in practice. Figure 2 depicts the results of the formulas cited in French, German bibliography compared to the new method. Measures are proposed for a better durability of the sleepers and an extension of their long-term performance on track.

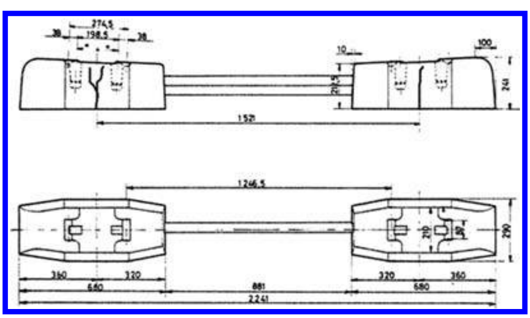

Figure 1. Cracked Twin-block concrete sleeper.

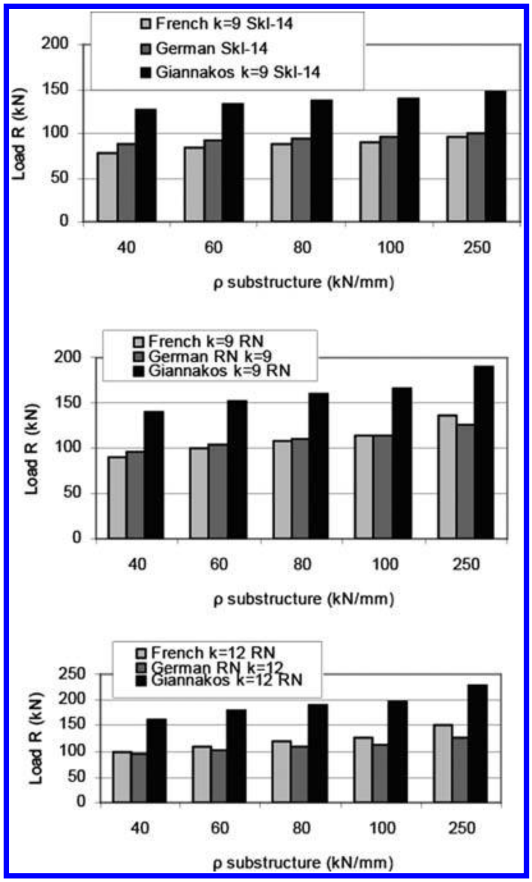

Figure 2. Actions on sleepers according to the French, the German, the Giannakos (2004) method, for concrete sleepers with fastening (a) W14, (b) RN and k = 9 and (c) RN and k = 12.

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The experimental research and analyses on usability of sulphur polymer composite for corrosion protection of reinforcing steel in concrete

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ABSTRACT

The degradation of reinforced concrete may be caused by the corrosion of the reinforcing steel or the concrete or by the simultaneous corrosion of the reinforcing steel and the concrete. The considerable porosity of concrete and cracks in or damage to the concrete cover contribute to the diffusion, absorption and adsorption of gases and to the diffusion of the substances dissolved in the pore liquid deep into the concrete. All kinds of aggressive substances from the surrounding environment diffuse into the inside of concrete and directly or indirectly cause the corrosion of the reinforcing steel, which usually ends in the loss of adhesion of the concrete to the steel, manifesting itself in the fracturing, loosening and spalling of the concrete cover.

Surface protection of the reinforcing steel, in the form of a hermetic protective coating, considerably reduces or prevents the access of the surrounding gas or water environment to the reinforcing steel. Various materials, e.g. polymer epoxy resins, inhibiting agents (inhibitors), noble metal admixtures, or cathodic protection are used for this purpose.

It seems that such protection can be provided by coating rebars with a polymer sulphur composite composed of a sulphuric binder, fillers and proper additives. Even though sulphur binders show: resistance to many aggressive water solutions, low absorbability, surface hydrophobicity and quite high (tangent and normal) adhesion to the surface of many materials (including metallic surfaces), they have not been used for this purpose before.

In order to demonstrate the suitability of sulphur polymer composites for the surface protection of concrete steel experimental research was carried out in the Institute of Building Engineering at Wrocław University of Technology. The research included: the experimental determination of sulphur polymer composite composition and manufacturing conditions, tests of the composite's selected physical, chemical and mechanical properties, tests of its tangent and normal adhesion to plain and ribbed reinforcing bars and to standard cement mortar and concrete, the determination of the mass decrement resulting from storage in aqueous solutions of acids, hydroxides and salts and in water and the polarization investigation of rebars subjected to tension in a solution modelling the pore liquid in carbonated concrete contaminated with chlorine ions.

In paper presents investigation results of corrosion rate for steel reinforcement bars that have been covered with polymer coating and have been exposed to tensile stresses in a solution simulating pore-liquid of concrete. Experimental investigation of tendencies that occur during corrosion process of reinforcing steel covered with polymer and exposed to tensile stress has been attempted. To determine an effect of tensile stress on corrosion rate for St3S steel that has been covered with sulphuric coating and exposed to aqueous environment that was to simulate pore-liquid of concrete contaminated with chloride ions was an aim of the investigation. The samples underwent loading in an one-axial state of the stress including varied values of tensile stress, at the same time corrosion rate was determined potentiostaticaly. Potentiostatic investigation has been carried out in order to determine parameters describing corrosion rate of samples tested. Corrosion rate for the steel has decreased by orders of magnitude when covered with protective coating even though this latest became unseal at load exceeding. A small decrease of corrosion rate has been found for the steel that has not been covered with polymer coating when placed in model pore-liquid of concrete and exposed to tensile stress increasing. The aim of investigation that has been led was to evaluate tendencies of the corrosion process for St3S reinforcing steel when covered with polymer sulphuric coating and exposed to tensile stress. Steel samples were loaded in a way that their yield points were much exceeded; in the same time these samples were exposed to an action of the solution the composition of which is similar to that of pore-liquid of concrete is and additionally contaminated with chloride ions (pH = 9,14). The composition said was as follows: 0,015M $NaHCO_3$ + 0,005M Na_2CO_3 + 0,001M NaCl. Corrosion rate for the steel has decreased by 2–3 orders of magnitude when covered with protective coating even though this latest became unseal at load exceeding 88,5 MPa.

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Modeling University of Cape Town Chloride Conduction Test for Concrete

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ABSTRACT

The University of Cape Town based Chloride Conduction Test exudes a benchmark in its setup and resolve to index diffusion coefficient of chloride in concrete. This paper aims to simulate the electrochemical basis of the Chloride Conduction Test for Concrete (Streicher & Alexander 1995) with regard to assessing its underlying assumptions. Proposition of some governing equations is presented for evaluating the effect that changes occurring during the test could have in terms of deviations from steady state conditions upon which conductivity offered by the test are based.

The response of concrete to electrical field is underscored by a number of factors that determines the variation of current flowing across the specimen over a period of time. Because concrete conducts electricity as an electrolyte the steady state assumption, under which direct current measurements are considered to be made in the conduction test, is affected by electrochemical kinetics of the chlorides occurring at the anode electrode (Andrade et al. 1995). This effect is currently unknown and therefore remains to be quantified for changes in chloride field that may occur during the conduction test.

To investigate the influence of these changes on the steady state based conductivity test, the use of mass transport law for electrolytes based on Nernst-Planck equation is well suited to capture the process (Bockris & Reddy 1974). The solution of the Nernst-Planck equation when coupled to the Faraday law is used in COMSOL Multiphysics development environment (COMSOL 2009) to simulate the behavior governing the conductivity and migration activities of the test problem.

Analysis of the model test is used to assess the sensitivity of the diffusibility ratio offered by the test to impact of deviation from stationary initial condition of the chloride field. The evolution in time resulting in changes in the diffusibility ratio at various times is given by Figure 1. It is evident that the time to bring about any appreciable impact on the diffusibility field is well beyond the maximum one hour test period attributed to the plots. For the time equal one hour, the spatial average of the resulting diffusibility ratio profile is calculated to be 1.40%, a paltry 2.74% deviation relative to its initial value (1.37%) when computed without an analysis of the chloride changes that occur during the test procedure. Nonetheless, extending scope of such assessment to establish proper duration and sensitivity analysis of the test on test data being generated is a target of current work in progress.

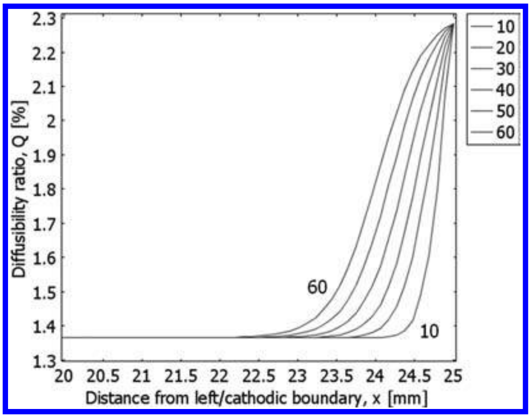

Figure 1. Diffusibility ratio profiles at different times, t[min].

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Conductivity of moisture in non stationary state

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ABSTRACT

Clearly, vapor condensation and high amounts of condensed water are problems that concern, first and foremost, roof constructions, namely flat roofs.

Liquid moisture that accumulates in a building construction as a result of water vapor condensation is carried into its surrounding.

If we create a function $w(x, \tau)$ that specifies the amount of water in $kg \cdot m^{-3}$ in relation to space (the *x-axis*) and time τ, then it is true that the change in the amount of water in a building construction over time equals to the negative value of the spatial change in the density of its mass flow.

From the viewpoint of physics, we may assume that in the partial time-space area in question, water will be distributed evenly, which may be expressed as follows:

$$\lim w(x,\tau) = konst. \tag{1}$$

Fig.1 shows this assumption was also verified using a numerical calculation

Fig.2 shows the changes in the amount of moisture per unit weight in the most critical spot within the building constructions.

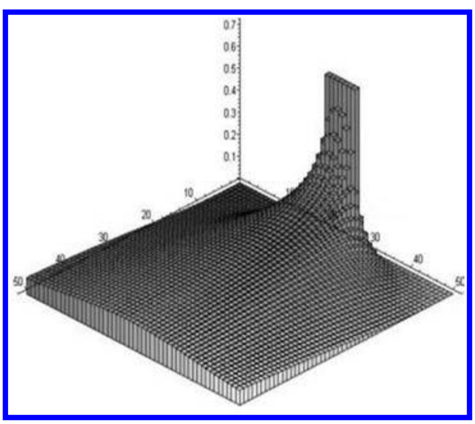

Figure 1. Function $w(x, \tau)$, verifying convergence for $\tau \rightarrow \infty$.

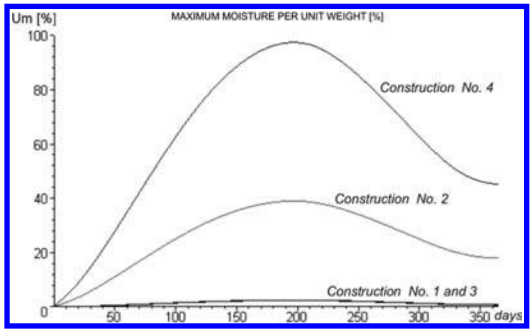

Figure 2. Changes in the values of moisture per unit weight in the most critical spot within the building construction over time.

It has been shown that capillary conductivity of liquid moisture has an effect on non-stationary balance calculations, especially in the case of flat roof constructions. In the porous materials used, it causes moisture values to change over time, which leads to changes in thermal conductivity of the respective materials.

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13. Construction materials, construction methods, building performance

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Appropriate constructions and materials for new lightweight building methods

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ABSTRACT

One of the first shelters of mankind is the tent, up to now it still belongs to the dwelling-form with the largest distribution all over the world. Today's modern membrane structures and other lightweight buildings have proved over the last decades to be durable and can be used for mobile and/or adaptable, however also as permanent buildings.

Due to the lightness of textile structures it is possible to obtain the largest spans without additional interior supports and to cover the hugest building volumes. They also achieve for certain construction tasks extremely short set-up and dismantling-times and low expenditures of costs, material and energy. Smaller tents are therefore indispensable as shelters in case of emergency. Large membrane structures are appropriate for shops, exhibitions and fairs, transportation buildings, cultural and religious events, leisure facilities, sports arenas and much more – last but not least for the soccer world cup in South Africa.

With membrane structures it is also possible to meet today's energy-saving thermal protection requirements by a number of special measures, for instance, with flexible insulating materials, with translucent insulation layers or with opaque, flexible mineral fibre insulations, reflecting intermediate layers and/or translucent insulation materials.

Acoustic requirements like sound-reflection or also noise proofing are fulfilled increasingly satisfying by the application of single-layer or multilayer membrane elements, for example by heavily coated or micro-perforated fabrics, or multi-layered coated fabrics or foils, filled with sand, granulates or mineral wool.

Modification and standardization of physical and mechanical properties of membrane materials and membrane elements arise according to their usability and durability mainly from the practical experience of the designers, engineers and users as a challenge to the manufacturers. Despite all evidence of their extraordinary abilities and obvious advantages lightweight structures made of textile materials and foils are still not used to the full range of their excellent properties. The reason for this is mostly a lack of dissemination of knowledge about the specifics of design principles of membrane structures, about the right evaluation of the mechanical and physical properties of the specific materials and their proper use, the workmanlike manufacturing and the correct construction and execution of details. Therefore this contribution deals with new materials such as foils, fibres, yarns and threads, coating and top coats of fabrics, including open meshed fabrics and nets. There will be presented furthermore experiments with special membrane materials and their applications in executed examples of membrane structures.

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Innovative concrete structures using fabric formwork

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ABSTRACT

Cement, 2.8×10^9t of which was produced worldwide in 2007, results in global concrete usage approaching $1.5\,m^3$ per person per year (USGS, 2008), making it our most widely used man made material. The constituent raw materials of cement are easily found and extracted, and are unlikely to be depleted in the foreseeable future. However, cement has a high embodied energy of approximately 4.6 MJ/kg (Hammond, 2008) and its manufacture is estimated to account for 3% of global CO_2 emissions (WRI, 2005).

This suggests that concrete should be cast in optimised structures that minimise material use and take advantage of the mouldability of concrete. This fluidity is rarely exploited and concrete structures tend to be cast in rigid, material intensive formwork systems. The optimisation of concrete structures such that they use less material, are structurally efficient and easy to construct would bring real innovation.

Research at the University of Bath has been looking at this issue for some time now. Using fabric formwork, it is possible to cast architecturally interesting, structurally optimised shapes based on simple design rules that are analogous to the growth of bone. When a bone is overstressed, it grows; when it is under stressed, it atrophies. In a similar way, fabric formwork can be used in the construction of concrete beams to place material only where it is required, and research has shown that this is both a predictable and practical approach that can achieve material reductions of up to 50% (Garbett, 2008) when compared to an equivalent orthogonally cast beam construction.

This paper explores the basis of fabric formed structures, beginning with work undertaken in the fields of offshore and geotechnical engineering in the early 20th Century. Architectural interest, which began with the Spanish architect Miguel Fisac and has since been growing around the world, is then discussed. Most notable is the work undertaken by Professor Mark West and his team at the Centre for Architectural Structures and Technology (CAST), at the University of Manitoba, where the design and construction of beams, trusses, columns, façade elements and shells has been developed.

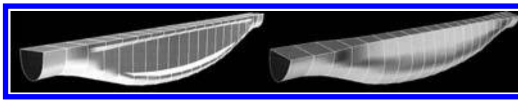

Figure 1. Fabric formed beams (Garbett, 2008).

Comparisons to conventional concrete construction are made, and the advantages of fabric formwork illustrated. Focusing on fabric formed beams, design methods used by previous researchers are outlined before optimisation processes are discussed. Four methods for the construction of fabric formed beams are then presented.

The limited available structural test data is outlined and areas of current research, including innovative anchorage methods for advanced composite longitudinal reinforcement, are discussed. The future of fabric formwork is then considered, focusing on the use of active reinforcement, advanced composites and participatory external formwork with additional thought given to improved design, optimisation and construction processes.

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A proposal of bamboo frameworks for low-cost constructions in seismic areas

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ABSTRACT

A low cost strategy for school buildings has been investigated with materials and techniques found in seismic area of Malawi. The basic structural material is bamboo, reinforced with timber, straw and mud, and masonry blocks. The seismic-resistant structure elements are fashioned of plywood and stainless-steel, screws easily assembled given suitable instructions. Experiments show the performance of the main joints (single or coupled Fig. 1–2).

The execution phases for joints are simple, low cost and easy to disseminate. The frame structure is able to sustain severe earthquake (more than 0,60*g* PGA) performing ductile mechanisms of collapse on the joints, in order to dissipate mechanical energy. The main results of tests are the following:

- the use of plywood to connect bamboo elements through steel bolts permit an appreciable ductility during collapse, due to the longitudinal deformation of the holes around bolts;
- the joint reinforcement offered by timber cylinders inside the bamboo extremity avoid the fragility consequent the longitudinal fracture of the bamboo fibers;

Figure 1. Single joint.

Figure 2. Coupled joint.

- in all tests the joint configuration permitted to apply relevant normal stress on bamboo elements, giving the possibility of high span rods, useful for trusses especially for roofs;
- to avoid fragile collapse is useful the addition of nails or glue for connecting the bamboo extremity to the internal wooden cylinder, or to execute external wraps for applying transverse load improving the stress state of bamboo.

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Testing shotcrete in accordance with new European standards

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ABSTRACT

Shotcrete is currently used mainly as material for primary lining of tunnels driven by means of a New Austrian tunneling method. It can also be used for temporary securing of a stope or an excavation, shotcrete is sometimes used even as the construction material of final lining. Therefore, the requirements for quality and particularly durability of shotcrete continuously grow.

One of the first documents dealing with testing shotcrete, and particularly testing so-called young shotcrete (i.e. concrete of age op to 24 hours from application on the structure) is the Austrian Directive for application and testing sprayed from March 1999. Application of shotcrete requires knowledge of development of strengths (maturing process) because of subsequent work. Procedures stated in the Directive completely serve the purpose, i.e. assessment of strength development of concrete in early stages of maturation.

New normative procedures for testing sprayed concrete were introduced in 2007; in particular requirement standards CSN EN 14487-4 (Sprayed concrete – Part 1: Definition, specification and conformity) and CSN EN 14487-2 (Sprayed concrete – Part 2: Implementing) and test standards series CSN EN 14488, in particular CSN EN 14481 – 1 (Testing of sprayed concrete – Part 1: Sampling fresh and hardened concrete) and CSN EN 14488-2 (Testing of sprayed concrete – Compressive strength of sprayed concrete). Above-mentioned standards follow up the Austrian directive for sprayed concrete, complement some parts and tighten requirements for test procedures and devices.

The main aim of the experimental work was to assess of basic properties of shotcrete, i.e. workability and estimate development of compressive strength. Test specimens preparation and execution of the tests followed procedures stated in CSN AN 14488-1 and CSN 1488-2. Since the Austrian Directive and new European standards are very similar, it was not necessary to compare individual test methods. Testing devices were selected in accordance with CSN EN 14488-2, i.e. very accurate apparatus was used. Following paragraph briefly describes simple methods used.

Technology of wet process of spraying concrete was used for manufacture of test specimens. At the first stage, so-called "zero" concrete was mixed, i.e. concrete with no accelerating admixture. Consistency of this concrete in fresh state was tested with slump test. Then, accelerators were added and testing panels were made. Testing panels were made by means of spraying concrete into boxes 600 × 6000 × 150 mm. These panels were subjected to testing of strength of young shotcrete with a penetration needle and by shooting and withdrawing nails. After finishing tests of young shotcrete, testing specimen with diameter 100 mm were drilled from the panels and subjected to test of compressive strength after 3, 7 and 28 days of maturing in laboratory conditions.

ACKNOWLEDGEMENTS

This outcome has been achieved with the financial support of the Ministry of Education, Youth and Sports of the Czech Republic, project No. 1M0579, within activities of the CIDEAS research centre and the project VG FAST of the Technical University in Brno, no. FAST-S-10-14 – Problems of determination calibration relationships for calculation strengths of sprayed concrete.

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Influence of selected hydrophobic agents on some properties of autoclaving cellular concrete used in building engineering

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ABSTRACT

This paper presents test results and evaluation of the impact of selected chemical agents used in the ACC on some of its strength parameters in terms of its application in Building Engineering. Enhanced strength parameters of the ACC with the addition of suitable hydrophobic agents will facilitate its more frequent application also in other sections of Civil Engineering, for example in bridge engineering, hydraulic engineering, etc.

The main goal of this paper was to determine the influence of selected chemical agents (*A*, *B* and *C* products type) on some properties of the ACC.

The research was directed to characterize and compare physical characteristics and strength parameters of two different types (500 and 600) of the ACC, which was exposed to hydrophobic process through the application of three agents selected from a wide range of products available on the European market that is *A*, *B* and *C* type. That was done in reference to the results of tests conducted with no use of hydrophobic agent.

The bending strength f_{cf} of the ACC specimens was calculated from the formula (1):

$$f_{cf} = \frac{1.5Fl}{bh^2}, \quad (1)$$

where F is maximal applied bending load, l is length of specimen, b and h are width and height of specimen, respectively.

The moisture absorption capacity was calculated from the Equation (2):

$$N_w = \frac{m_n - m_s}{m_s} \times 100\%, \quad (2)$$

where m_n and m_s stand, respectively, for the mass of the water saturated specimens and the mass of the specimens dried to a solid state.

The compressive strength f_c of the ACC specimens was received from formula (3):

$$f_c = \frac{F}{A_c}, \quad (3)$$

where: F is maximal failure load, A_c is cross-section area of specimen.

The influence the tested products exerted on the qualities of the two types of cellular concrete differed. The obtained test results varied depending on the type of the hydrophobic (sealing) agent that was administered and the variety of ACC. The results of the experimental research allow following conclusions:

1. A suitable application of sealing and hydrophobic agents to the ACC facilitates the modification of its given characteristics which, consequently, results in improving its durability and overall quality as well as widening the range of its application in both building engineering and maybe in bridge engineering, but it will be subject from next studies.
2. The sealing and hydrophobic products used for the ACC, that were analyzed in this paper, that is the *A* and *B* products type, effectively change several material properties at the same time, such as bending and compressive strengths. They have an especially significant impact on decreasing moisture absorption capacity, which proves crucial in building engineering.
3. Different behavior of both ACC types is rather related to their volumetric density, size of pores, humidity degree, and penetration depths of applied hydrophobic agents, but it still requires conducting of the additional research.
4. The intended modifications of the ACC parameters due to the application of sealing and hydrophobic agents are guaranteed only when the general application criteria specified by the producer if the appropriate company documents are followed. It applies to the amount of the product to be used, the method of its application to the concrete and proper surface preparation.

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Impact of indoor materials to indoor air quality

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Summary: *The measure for indoor odor pollution decreasing is the effective source control. In order to guarantee acceptable perceived air quality the identification of all important indoor odor sources is required. In this study, emissions and odours from different common indoor surface materials (waste wooden producs, vinyl, carpet, gypsum board, paint, wall coating materials etc.) and their combination were investigated in test chambe. The chemical measurements and sensory assessments were done by the standardized and stabilized conditions (23°C, 50%). The impact of individual and combined materials on the perceived air quality and odour intensity will be discussed.*

Keywords: building materials, odor, and volatile organic compounds

1 INTRODUCTION

The traditional way of material selection for building design has been primarily based on factors such as cost, aesthetic values, availability and durability. Many of the materials used in buildings, either as structural materials or as furnishings, are the main sources of indoor air pollution. The source control is often the most practical approach to improve the indoor air quality. The building materials are considered to be the principal sources of indoor air quality in addition to those caused by humans and their activities and HVAC systems. Nowadays the Environmental Engineering Institute focuses its research activities to the observation of indoor pollutants sources impact to perceived environmental quality. The percentage of dissatisfied was not higher with additional surface finishing odour sources as it was be expected [1]. Interior surfaces are generally accepted as source of VOCs emissions. Even the result perceived air quality can be affected by their interactions and sorption effects. The experimental investigation results of the various building materials and their combination effects on perceived air quality will be presented in the paper. The final aim of the running research project is to recommend proper building materials in order to guarantee acceptable perceived indoor air quality.

2 METHODOLOGY

The background perceived air quality and starting pollutant concentration before material testing was established. The measurements were realized under the same conditions. The building materials testing in this study are typical surface materials of the offices. The chemical measurements and sensory assessments were done by the standardized and stabilized conditions (23°C, 50%). The area specific airflow rate in the test chamber was adjusted by adjusting the test specimen area and the supply airflow rate. Non testing surfaces were covered with aluminum foil. Active sampling of VOCs was performed by using a pump with air flow rate of 500 ml/min on charcoal tubes during 24 hours. The absorbed VOCs were analyzed by gas chromatography after extraction into CS_2. An untrained sensory panel of 20 subjects assessed odour intensity and perceived air quality.

3 RESULTS

The highest TVOC value as well as dissatisfaction with perceived air quality and odour intensity of flooring materials caused the laminated floor. In case of walls and ceiling surface materials the PVC wallpaper/glued was the most unacceptable. The results were influced by adsorption/desorption processes indicated by combination of building materials. The impact of indoor odor sources on perceived air quality and odor intensity will be discussed in the paper.

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Analysis of durability-based design performance of coarse aggregates used for RC structures in corrosive environments

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ABSTRACT

The optimal structural design of Reinforced Concrete (RC) structures in corrosive environments is often *mainly* controlled by durability-based constraints that are invariably imposed by rate of corrosion of reinforcing steel bar. The rate of corrosion of reinforcing steel bars is also controlled by a host of other design parameters that play a major role in the design of the concrete matrix. To address this multi-dimensional design problem in a typical corrosive environments (here taken as the environmental conditions prevalent in eastern Saudi Arabia) an attempt has been made to investigate the performance of two types of coarse aggregates (AH-aggregates and TF-aggregates), obtained from two geographically distant regions in the Kingdom, when used in producing concrete. For this purpose, 27 concrete mixtures were considered for each type of coarse aggregates by varying key mixture parameters with *eight percent* of cement content replaced by *silica fume*. Key features of a selected sample of initial results obtained for compressive strength f_c' and reinforcement corrosion penetration rate P_r are presented and discussed within the context of structural design-methodology and objective to establish design criteria for durable RC structures in corrosive environments.

The results obtained indicate that the type of coarse aggregates used combine with other characteristics of concrete mix, structural design variables and exposure conditions to form the required design-criteria for an optimally durable RC structure in corrosive environments.

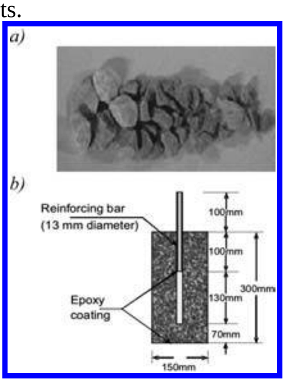

Figure 1. *a*) Coarse aggregates (AH), and *b*) Schematic diagram of test-speciemn prepared and used for corrosion rate monitoring of steel bar within a concrete matrix.

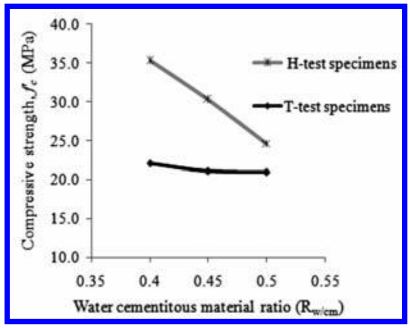

Figure 2. f_c' versus $R_{w/cm}$ curves for AH-test specimen and TF-test specimens at $R_{FA/TA} = 0.35$ and $Q_C = 350\,kg/m^3$.

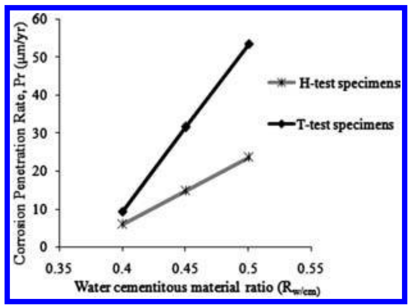

Figure 3. P_r versus $R_{w/cm}$ curves for AH-test and TF-test specimens at $Q_C = 400\,kg/m^3$, $R_{FA/TA} = 0.45$, $C_{CHL} = 3\%$ and $T_{CV} = 25\,mm$.

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Comparison of the properties of Portland cement and Portland-limestone cement

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ABSTRACT

A study was made in one of the cement factories in Dar es Salaam, Tanzania, where Ordinary Portland Cement (CEM I 42.5N) and Portland limestone cement (PLC) (CEM II/A-L/32.5R) are produced and conforming to the Tanzania Standard TZS 727 (Part1): 2002, which is equivalent to EN 197 published by the committee for European normalization (CEN). A comparison was made between the two types of cements in terms of physical, chemical and mechanical properties. It was found out that they all complied with the standards, that there was no significant difference in their setting times and that the Portland cement had higher strengths than the PLC. Also it was observed that there was a slightly lower water demand for the same consistency when compared to OPC and hence there is an improvement of the cohesiveness of a concrete mix when PLC is used. It was concluded, however, that the two cements are different and that using the two cements interchangeably as is done in Tanzania is wrong because they do not have equivalent strengths and therefore equivalent performance since the PLC is not optimized.

Some of the results of tests carried out on samples from the two types of cements are plotted on Fig. 1 to Fig. 4.

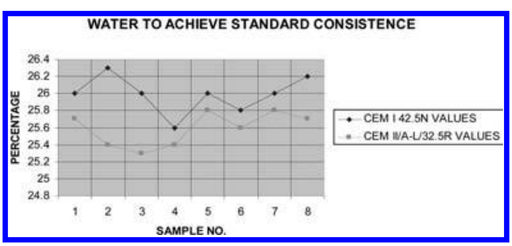

Figure 1. Water to achieve standard consistence.

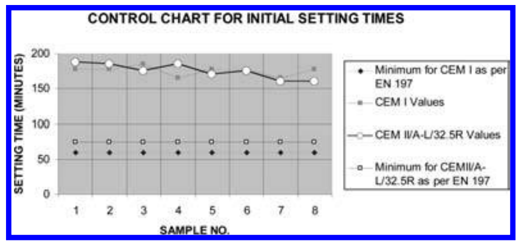

Figure 2. Control chart for initial setting times.

Table 1. Mechanical Properties of the cements tested.

Cement Type	Early strength (2 days)	Standard strength (28 days)	Requirements according to EN 197-1
Portland cement (CEM I 42.5N)	16.95 MPa	47.13 MPa	>10 MPa at 2days and between 42.5 and 62.5 MPa at 28 days
Portland-limestone cement (CEM II/A-L/32.5R	13.14 MPa	41.98 MPa	>10 MPa at 2days and between 32.5 and 52.5 MPa at 28 days

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Influence of crushed rock powder and fly ash on strength characteristics of concrete

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ABSTRACT

The purpose of this research is to study the properties of fresh and hardened states of M35 grade of concrete, using Crushed ROCK Powder (CRP) as fine aggregate to replace partial or full amount of sand with constant addition of 35% Fly ash to existing cement content. This paper investigates quantitatively the strength of concrete mix at different ages. The concrete contains 0%, 20%, 40%, 60%, 80% and 100% of crushed Rock Powder as part of fine aggregate. The overall test results revealed that in concrete mixtures, Crushed Rock Powder can be fully substituted as an alternative material for natural sand (fine aggregate) in presence of fly ash. The concurrent use of two byproducts will lead to a range of economic and environmental benefits.

The most widely used fine aggregate for the making of concrete is natural sand, mined from the river beds. However, the availability of river sand for the preparation of concrete is becoming scarce due to the excessive nonscientific methods of mining. Apart from this, issues like lowering of water table, sinking of bridge piers, etc. are becoming common threats. The present scenario demands identification of substitute materials for river sand for making of concrete. The main objective of this paper is to study the strength characteristics of plain concrete developed by replacing sand with Crushed Rock Powder (CRP) at 0%, 20%, 40%, 60%, 80% and 100% along with constant addition of 35% addition of fly ash. In the present investigation, Fly ash is considered as an additive to cement in concrete, so that the loss in strength due to CRP may be partially negated by the improvement in workability and strength imparted by the inclusion of Fly ash. The study is planned to determine whether such benefits could be obtained by the use of these two materials together, and to quantify such benefits, if any. Positive results will lead to the possibility of using two byproducts in large quantities, while reducing the dependency on chemical admixtures.

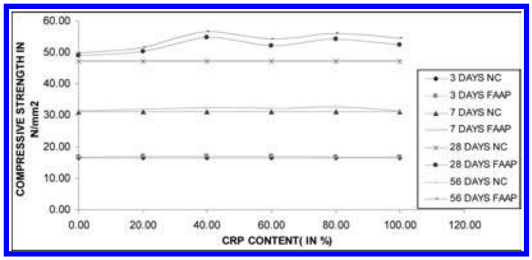

Figure 1. Comparison of Compressive Strength of FAAP Concrete with normal Concrete.

The experimental investigation include the study of compressive strength, split tensile strength and flexural strength of concrete at different periods of curing. The results of these observations indicates that the strength property of concrete with CRP and fly ash are higher than that of normal concrete. Fig 1 shows variation of compressive strength of fly ash added CRP concrete (FAAP concrete) with normal concrete for different levels of CRP replacement

The results of the investigations indicates that when flyash is used as an extra cementing material, the natural sand can be even totally replaced by CRP resulting in higher strength. Fly ash added CRP concrete gives higher strength at all replacement levels of sand by CRP.

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14. Timber structures, masonry structures, glass structures

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Analysis and modelling of the mechanical behaviour of structural steel and timber joints

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In steel and timber structures, the joints play a major role in the global behavior of the structures including the load carrying capacity, the stiffness and the ductility. Due to their geometrical configurations, the complex behavior cannot be encircled on exclusively experimental base which remains incomplete and expensive. For the structural analysis, the behavior of the joints can be characterized by the moment-rotation relation that can be determined on the basis of tests, analytical or numerical models.

This study presents numerical approaches used for the analysis of the bolted and doweled steel and timber joints. Thus, 3D finite element models are developed to predict the mechanical behaviour of steel-to-steel or steel-to-timber joints loaded in bending or in tension parallel to grain. The model used to characterize the behaviour of the materials is based on the isotropic behaviour of steel and the orthotropic behaviour of timber. In order to take into account the asymmetric behaviour of timber in tension and compression, a failure criterion is associated with the Hill criterion to control the plastic yielding combined with the damage evolution of timber. In addition to the elastic-plastic analysis with large displacements, the model takes into account the unilateral contact at the interface between the components of the joints. The results obtained by the numerical simulations are evaluated and compared with the available experimental results for timber joints in tension parallel to grain and bending, and for steel joints in bending.

The proposed FEM model shows the capacity to describe the behaviour of timber joints made with dowel type fasteners or bolts. This is confirmed by the comparison of experimental and numerical load-displacement curves in tension parallel to grain or bending. In timber joints, the failure modes, observed at the ultimate stage of the test, seem to be due to a combination of tension perpendicular to grain and shear. At this stage the steel dowels exhibit a more or less large plastic deformation in bending with large deformation of wood in embedment under the dowels. In tension parallel to grain, the block shear due to the group effect is mobilised in some cases. In bending, each fastener has a load with a different intensity and direction due to the anisotropic character of timber and the position of the centre of rotation. In the example presented in the paper, the failure occurred by a large plastic bending of the majority of the dowels followed by a brittle failure in the direction perpendicular to grain at one of the dowel rows.

In steel joints, the strengthening of the column flanges is sometimes necessary to increase the stiffness or the strength of the beam-to-column joints. The column flange strengthened in the tension zone by backing-plates is modeled. A 3D model is used to analyze the behavior of a whole joint considering the beam, the column, the end-plate, the backing-plate, the bolts (head, shank, washer and nut) and the contact zones between the various parts of the joint.

The modelling of a common joint, with three bolt-rows, is calibrated on the basis of the experimental results such as the moment-rotation curves, the evolution of the tension forces in the bolts and the deformed shape of the joints. The joint analyzed is with a thin column flange strengthened or not by backing-plates. Thus, the main sources of deformation are those of the flange in bending. So, the deformations of the column web and the transverse stiffener are neglected compared with those of the column flanges and the end-plate.

The column flange is thinner than the end-plate and thus it pilots the behavior of the joint. The evolution of the bolt forces in the joint shows that the bolt rows close to the compression zone play a significant role, at the final stage of behavior, for the joint with thin flange. For the studied thin column flanges without stiffeners in the tension zone, the flange which becomes flexible follows the deformation of the end-plate. The thin column flanges support a large part of the deformation due to the bi-axial bending of the plate supported by the column web.

The 3D-FEM models developed are common useful tools to investigate timber and steel joints with multiple fasteners. This leads to a reference solution to be used in a more extensive study on structural joints to determine their initial stiffness, resistance and rotation capacity. These validated models can be used to generate data concerning various geometrical and material configurations. The goal of the joints numerical database is to improve the available analytical methods available in the design codes to cover a larger variety of joints.

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Design of interconnected timber-concrete constructions

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ABSTRACT

The most difficult problem in the field of carrying wooden construction is the solution of connection between each parts. The utilized features of interconnected timber-concrete constructions are high rigidity (upright to plane and also in plane of construction) and reduced plastic flow influence of interconnected structure as a whole instead of separate elements (timber, concrete). The advantage of timber-concrete constructions is the increased fire resistance, which leads to new possibilities of spreading the timber buildings. The interconnected timber-concrete structures are requested mainly for their bearing resistance, life span, preciseness and difficulty of implementation, aesthetic value and resultant costs.

The article is focused on the construction of present and new designed interconnecting components used for connection of timber and concrete. Further speaking about design, implementation and evaluating the result of the tests of interconnecting components. In the article there is also a part which pays attention to the preparation of the test of timber-concrete interconnected structures.

This paper shows a complex view to the design and testing of the interconnected timber-concrete constructions. The different variants of interconnected connection are described in more detail here and mainly the connection with steel perforated moulding which is nowadays developed on VŠB Technical University Ostrava, Faculty of Civil Engineering.

The 3D numeric model was processed for the experiment, on which tension in element during stressing was analysed. Software ANYSYS ver. 11.0 was used for 3D analysis. Real elements show bigger time consumption of numeration and have more demands to engineering equipment too. To be able to analyze testing elements in detail, with relative fair time of numerical calculation, a demand of equivalently enlarge a finite elements has to be satisfied. Because of axis centricity of elements is modeled only one quarter of it, where already is possible to split model on a small finite elements which indicating more exact results.

Figure 1. Output of analyses software – shear (xy).

Cyclical stressing of test sample concluded into vertical and horizontal deformation. Vertical deformation shows shift of Glued laminated timber against concrete panels. Its value was during cyclical stressing approximately in hundredths of millimeter.

From study is obvious that simulation of testing element in software ANSYS ver. 11.0, really stating about behavior of sample under destructive test. From this results can be assumed that shear connection of steel, glue, timber and concrete is suitable option for use and next research.

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Numerical simulation of the ductile failure behavior of wood

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ABSTRACT

To improve or reinforce complex engineered structural details in wood, one needs to investigate the stress distribution and the failure behavior. For this, Finite Elements (FE) methods are very useful, but one needs to know the complete elastic-plastic strength behavior (also after the yielding) of wood under compression. Insufficient information on the compression strength values and the compression behavior for Radiata Pine lumber and laminated veneer lumber (LVL) from New Zealand is available in the literature. Compression tests were carried out to gather the necessary information about the failure behavior under compression and their parameters to describe the behavior. Furthermore, the FE methods or models need to have useful material laws to simulate the ductile failure behavior of the orthotropic material wood.

To determine the material parameters of New Zealand Radiata Pine which are needed to describe the material behavior in FE simulations, compression tests with different load to grain angles are carried out and simulated. The paper describes the test program, the standards and evaluation methods used and presents a possibility to compare simulation results with experimental results using photogrammetric measuring methods. These systems allow a more complex measurement of strains than conventional measuring techniques and an easy comparison with numerical results. Further details about the test program and the photogrammetric measuring system can be found in Franke (2008) and Franke & Quenneville (2010) respectively.

For the simulation process, the recent existing material laws of the program ANSYS and the three dimensional elasto-platic material model, developed by Grosse (2005), are described, compared and the results of these models evaluated. Regarding to the orthotropic material wood, both the extended Hill criterion (ANISO) and the three dimensional constitutive law from Grosse are used to simulate the compression behavior of wood in different directions. Depending on the stress situation, the simulation models allow a more or less good agreement with the experimental results, see Figure 1.

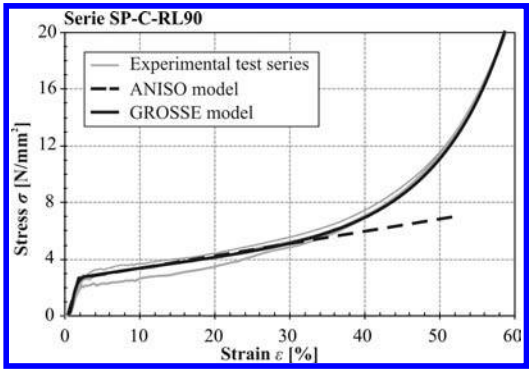

Figure 1. Comparison of the experimental and numerical stress-strain relationship of European spruce perpendicular to grain.

For the simulation of the compression behavior perpendicular to the grain, the simple ANISO model can be used up to 30% strains realistically, whereas for other load to grain directions the more complex 3-dimensional elastic-plastic material model from Grosse (2005) leads to better results, but is also more difficult to use. It shows that there is still a need for better material models which include the different failure behaviors and are easy to use.

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Numerical modeling of the brittle failure behavior of wood

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ABSTRACT

In addition to the conventional static analysis and design of structures, the finite element analysis is a very helpful procedure to analyze structures, details or connections. Sectional weakening or changing, notches, cracks or connections can lead to concentrated stresses in structural members. The simulation of those stress singularities or crack propagation in wood is very complex according to the cellular structure of wood. Furthermore, the stress strain behavior under tension perpendicular to the grain is characterized by a very brittle failure mechanism, which already occurs with small loads applied.

For wooden material, few steps are known, which one allows to simulate stress singularities or the brittle failure under tension or shear stress. The paper presents the numerical model which adopts the contact technology for the successful simulation of the brittle failure of wood under tension perpendicular to grain and shear stress. The built in contact elements open according to the cohesive zone material implemented along defined crack paths. The crack growth direction for wood loaded perpendicular to grain can be estimated very well, because of the cellular structure. An intelligent use of predefined crack paths shows that there are no restrictions for the simulation of the failure process. The cohesive zone material used considers the single fracture mode I or II as well as the mixed mode.

A summary of numerical results obtained for typical structures or connections under a combined stress of fracture mode I and II are presented. An example of an experimental test series with end notched beams, 80 mm in thickness and 200 mm in depth with a notch over the half beam depth and 150 mm in length was simulated. The stress plots presented in Figure 1 are obtained using one predefined crack path at the notch base throughout the complete length of the beam. The stress distribution is shown at maximum load with the crack initiation and two different crack propagations.

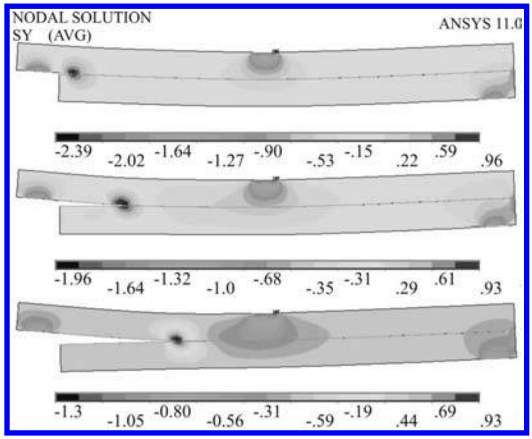

Figure 1. Transverse stress plots for the end notched beam at different load steps.

Connections in timber construction are used very often. Joints loaded perpendicular to grain fail either in ductile manner, which is well understood, or in brittle manner. For this reason, dowel-type connections were simulated using the contact technology. Crack paths along the grain are implemented at the centre of each dowel. In further numerical solutions, the material settings were probabilistically distributed to reflect the natural variability of the wood. A mean relative deviation of about 11% of 15 different test series confirms a close range between the experiment and numerical model.

The paper shows, that it is possible to simulate the load capacity of typical timber members or connections under tension perpendicular to the grain realistically. The load behavior of wood under tension perpendicular to the grain could be simulated independent of measuring values using the contact technology with a cohesive zone material. This technique allows the extension of experimental test series with new geometry parameters or different layouts of the connection for the estimation of the load carrying capacity and the development of design equations efficiently.

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Analysis and modelling of the thermo-mechanical behaviour of dowelled and bolted steel-to-timber joints

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In order to ensure the fire performance of timber structures, the development of accurate numerical models and optimal thermo-mechanical design rules is necessary. In fact, the number of experimental data and sophisticated numerical models available in the literature is relatively limited. Furthermore, among various structural components, the joints exhibit the more sensitive and complex thermo-mechanical behaviour.

The design procedure given in Eurocode 5 (EN 1995-1-2) for timber joints is based on a limited number of experimental and numerical results. In order to compensate the lack of data, a numerical model is developed and validated on the basis of the available experimental results. In this paper, the experimental results used for the validation of the numerical models are presented. They concern double shear steel-to-timber doweled and bolted joints.

Thus, a 3D finite element thermo-mechanical model is developed in two stages to simulate the behaviour of these joints.

The first stage simulates the heat transfer inside the joint components using the evolutions of the thermophysical properties of materials depending on the temperature. The thermal characteristics used in the numerical simulations are chosen on the basis of a sensitivity study. For the thermal analysis, a 3D-FEM model is developed using the MSC.MARC software. To insure the thermal continuity between the various members of the joints, a perfect contact, with continuous meshing, is considered at all the interfaces between timber, steel plates and fasteners. The evolutions of the calculated and measured temperatures for different components of the connections are compared and a good agreement is obtained. The heat transfer model used is validated on the basis of these results.

The second stage of the modelling represents the non linear mechanical behaviour of the connections in cold as in fire situations. For the material modelling, the timber behaviour is considered with transverse isotropy assuming identical properties in the radial and in the tangential directions. The Hill yield criterion is used to manage the plastic yielding in the timber components. The interaction between the different components of the joints is modelled using deformable contact elements at each interface (timber-steel plate, timber-fasteners and fasteners-steel plate). Thus, the meshing of the mechanical model is discontinuous and differs from that of the heat transfer. The 3D-FEM mechanical model is validated on the basis of the load-slip curves of timber joint tested in normal conditions. The numerical model predicts well the initial stiffness of the joint and gives the same tendency, as the tests, for the plastic part of the curve but with higher values of strength.

The thermo-mechanical analysis of the joints is based on the same 3D-FEM as for the mechanical modelling. The temperature-dependent mechanical properties of the materials are taken into account using the reduction factors of the mechanical properties given by Eurocode 3-1-2 for steel and by Eurocode 5-1-2 for timber.

To perform the thermo-mechanical analysis of the connection, the field of temperature inside the connection calculated using the thermal model is stored for each increment of temperature. A procedure was developed to make the transition between the stored field of temperature and the thermo-mechanical model considering the correspondence between the nodes of the two different meshes.

The results of the thermo-mechanical model are evaluated on the basis of the predicted time of fire resistance of the connections. These periods of fire resistance are defined using the time-slip curves. The acceleration of displacement, representing the failure of the connection, is well predicted by the model. The failure times of the connections calculated using the model are compared with the experimental values. The calculated values are always lower than the experimental values. The thermo-mechanical model predicts well the first stage of the failure characterized by a significant acceleration of the local displacement.

The fire resistance times given by the Eurocode formulae are compared to the experimental ones. It appears that the fire resistances given by Eurocode formulae are not valid for fire resistance times higher than 30 minutes. The numerical approach developed in this study will allow extending the application domain of the Eurocode formulae.

The model is validated for the connections in tension parallel to grain. However, as a general tool, it will be extended to various types of connections loaded in tension perpendicular to grain, at 45° or in bending. In all cases, the numerical results will be used to calibrate a simplified model to be used by the engineers in practice.

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Pull-out capacity of steel bars bonded parallel to grain of timber

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ABSTRACT

Bonded-in rod connections, comprising rods such as steel or fibre reinforced plastics (FRP'-s), housed in pre-drilled holes or slots machined into timber members are an efficient method of connecting timber members. This paper reports an experimental programme that investigates the performance of bonded-in rod joints loaded parallel to the grain of timber elements. Though joints made in this way are a preferred option for the design of timber structures current knowledge of their behaviour is not sufficient so that performance criteria differ for the countries that employ the technology. The main aim of this research was to evaluate the structural capacity of bonded-in steel connections using pull-out tests. The tests investigated the influence of bonded length, bar diameter and bar type (deformed or threaded) on the structural performance of the joints.

Figure 1 shows the configuration of pull-out specimen used. Two holes of diameters 20 mm and 16 mm (for 16 mm and 12 mm high tensile steel bars respectively), with varying bonded lengths were drilled – parallel to the grain – through the entire length of each block. The edge and end distances for each sample were 35 mm and 45 mm respectively. The surface of the – 12 mm and 16 mm – bars was deformed or threaded in order to enhance mechanical interaction between the resin and the bars. Glueline thickness for all joint configurations was 2 mm as this gives at least the best performance for epoxy joints (Ansell and Harvey, 2001).

After curing for days, the specimens were tested by conducting pull-out tests in a standard tensile test machine fitted with a 100 kN load cell. The load was applied at a constant rate of 2 mm/min and the failure mode for each joint configuration was observed and recorded. The experimental results were also used to validate some established theoretical models. Results showed that, capacity of the joint was significantly affected by the bonded length, bar diameter and bar type.

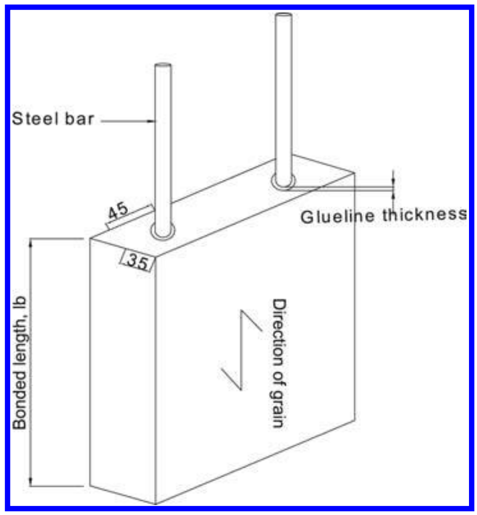

Figure 1. Pull-out test specimen.

The deformed bars (12 mm and 16 mm) recorded higher splitting failures than the corresponding threaded bars. This was due to the curve of the surface deformations of the deformed bars producing more prying and wedging action (Buchanan, 2002). The best structural performance was obtained with 16 mm threaded bar bonded 300 mm into the timber elements. The tests also showed that, in practice, misalignment of the steel bars in the holes did not influence joint performance. Comparison of the tests results with the theoretical predictions showed some inconsistent correlations.

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Reliability analysis of a cylindrical timber shell roof system

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ABSTRACT

There has been a gradual decline in the use of thin concrete shell roofs over the past few decades. This decline has been due to the high cost of construction and removal of temporary formwork and associated falsework for concrete casting of a thin concrete shell. This paper reports on the utilization and the estimated reliability level of a cylindrical timber shell roof composed of plywood membrane and supported on laminated veneer lumber (LVL) structural beams. Under different loading conditions, a finite element analysis (FEA) of the chosen shell roof form was conducted. Compared to other structural materials, such as steel and concrete, structural timber has a considerably higher variability of the strength properties both within and between members. Therefore considering loading, material and geometrical properties as random variables, the solution of FEA and the basic design requirements of Eurocode 5 (2001) were subjected to reliability analysis using a well-coded algorithm based on the first-order reliability method (FORM). Preliminary results reveal that timber roof systems can take the shape of a cylinder provided the ratios of dead to live and wind to live loads are moderate and appropriate thicknesses are used. Also, the ultimate limit state design requirements of Eurocode (2001) are seemingly conservative for high thicknesses while the ratio of dead to live load must be properly controlled to avoid unpleasant structural behavior.

Keywords: Structural timber, cylindrical shell roof, finite element analysis, reliability, variability, design requirements, first-order reliability method

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Numerical modelling of masonry towers: The case study of the "Rognosa" tower in San Gimignano

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ABSTRACT

Accurate modelling the mechanical behaviour of masonry towers, which constitute a very important part of the worldwide ancient architectural heritage, still poses a number of challenging problems. Firstly, the response of these structures to dynamic actions is strongly nonlinear. Moreover, they are characterized, even in the static case, by high compressive stresses that can cause additional damage in correspondence to openings or other geometrical irregularities. Even though some general aspects can be highlighted via simplified models, either linear (Selby & Wilson 2001, Carpinteri at al. 2005, Ivorra & Pallarés 2006) or nonlinear (Casolo 1998, Lucchesi & Pintucchi 2007), carefully definition of the geometry as well as the use of more refined models are of great importance.

This paper presents a preliminary study concerning the static and seismic vulnerability of the "Rognosa" tower in San Gimignano, Italy, added to the Unesco World Heritage List in 1990. The structure is modelled by using the finite element code NOSA (Degl'Innocenti et al. 2006, Lucchesi et al. 2008), which assumes masonry to be a nonlinear elastic material with zero tensile strength and bounded compressive strength. The behaviour of the tower, subjected to its own weight and an accelerogram recorded during a real earthquake, is investigated.

Figure 1. The "Rognosa" tower in San Gimignano, general view (left), some vaults bearing down on the tower (top right), the interior of the tower (bottom right).

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The effect of horizontal and vertical reinforcement on shear capacity of masonry walls

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ABSTRACT

The aim of the paper is to present the influence of reinforcement directions on shear resistance of masonry walls. Experiments dealt with solid clay units, reinforcement placed in mortar infill and joints.

The developed new type of bonding of bricks allows the application of the same solid bricks for all walls and placing of reinforcement between bricks. The test series consist of testing solid plain, vertically, horizontally and in both directions reinforced masonry walls. Altogether, aspect ratio, applied vertical force, amount, shape, type and layout of reinforcement in one direction were the same. Compressive strength of mortar and direction of reinforcement were changed. Walls were subjected to monotonic in-plain shear during constant vertical compression. Tables show the load at appearing of the first cracks, maximal load that could be reached during experiments, the maximal displacement and the incidental residual force. The forces of the plain masonry with weak mortar are considered as 100%.

The favorable influence of mixed reinforcement is clearly shown. The crack pattern modification effect and the shear capacity enhancement of reinforcement are unambiguously proved. Altogether, the new bond wall with reinforcement in both directions enhances the deformation capacity particularly and achieves a high increase in shear capacity. It is capable to decrease crack width. Further details about the experiments can be found in Fódi & Bódi (2010).

A discrete element model was developed being able to follow the crack pattern of plain masonry. There is a good agreement between the crack pattern of the experimental wall and the crack pattern of the model. The line of the principal compressive stresses from the model corresponds to the line of the cracks. It gives adequate distribution and measure of displacements and stresses. On the grounds of the experiments the numerical model can be developed on.

If the in-plain and the out of plain behavior of the walls were known one of the practical application of the developed bonded rein-forced masonry would be building some-storey constructions without using reinforced concrete frames. The biggest advantage compared to other masonry structures is that the compressive strength of the wall is much higher than in case of other masonry structures because of the failing of the big volume of the holes. Hence, this construction method can be applied in blasting or earthquake hazarded areas, next to gappy grounds or in case of earth pressure loaded walls with a bigger efficiency than plain masonry.

Table 1. The results of the series with weak mortar.

Type	First crack	Maximal load	Maximal displacement (residual force)
Plain masonry	100% (145 kN)	100% (154 kN)	26 mm (129 kN, 100%)
Vertically reinforced masonry	83% (121 kN)	105% (161 kN)	60 mm (161 kN, 125%)
Horizontally reinforced	119% (173 kN)	117% (180 kN)	67 mm (163 kN, 126%)
Vertically and horizontally	121% (176 kN)	164% (252 kN)	60 mm (252 kN, 195%)

Table 2. The results of the series with strong mortar.

Type	First crack	Maximal load	Maximal displacement (residual force)
Plain masonry	124% (180 kN)	133% (193 kN)	31 mm (143 kN, 111%)
Vertically reinforced masonry	121% (176 kN)	150% (231 kN)	42 mm (231 kN, 180%)
Horizontally reinforced	132% (192 kN)	125% (193 kN)	38 mm (171 kN, 130%)
Vertically and horizontally	138% (200 kN)	215% (331 kN)	32 mm (331 kN, 257%)

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Influence of sulphate attack on elasticity of fired clay brick masonry wall

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ABSTRACT

The service life of masonry walls is significantly influenced by material properties and the physical and environmental factors associated with the durability of the building material. Durability is very important for most building materials generally and sometimes may need to be evaluated to ensure the integrity and service life of building material. It is no doubt that soluble salt which transport by the moisture from natural condition or environment which normally sulphates could cause deterioration or decay on the performance of masonry walls especially in sub-tropical and tropical climates.

The study to investigate the influence of sulfate attack on elasticity of fired clay brick masonry wall was done in Heavy Structure and Concrete Laboratory, University Sains Malaysia, Nibong Tebal, Pulau Pinang, Malaysia. Masonry wall (see Figure 1) were constructed using one type of fired clay brick that supplied by SHL Sdn Bhd, Selangor Malaysia. The properties are shown in Table 1.

Table 1. Properties of bricks used in the tests.

Parameter	Strength (MPa)	Water absorption using 24 hours immersion (%)	IRA kg/m^2/min
Mean	34	13.60	1.60
Standard deviation	1.91	1.45	0.15

There were three severe sodium sulphate conditions which were prepared by weight volume with concentrations 5%, 10% and 15%. All specimens were prepared and cured under polythene sheet for 14 days in environmental controlled room with $80 \pm 5\%$ relative humidity and $25 \pm 2°C$. The specimens were exposed to the sodium sulfate after curing before testing and elasticity was measured and monitored at 14, 28, 56, and 180 days.

Figure 1. Sample of brickwork.

Efflorescence or salt crystal appeared on the surface of the samples after evaporation through chemical reaction. However efflorescence could not deteriorate the masonry material. The deterioration occurs due to the crystallization pressure and strongly influenced by pore structure and strength properties. Result from this study identified the reduction in elastic modulus of fired clay brickwork. This situation clearly showing that the masonry materials are not resistant to attack by sodium sulfate solution. The rate of brickwork deterioration increase when the sulfate concentrations increase. These phenomena become more serious after 180 day exposure for all concentration whereby at this age the mortars become soft and brittle and later resulting the reduction in elasticity. This reduction could verify that the sodium sulfate is very damaging salt and provides more extreme condition.

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Confined masonry as a reliable structural system for low income housing in earthquake prone zones

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ABSTRACT

Masonry buildings still represent a great part of both residential and public buildings because they are not expensive and they are suitable for people on low incomes. Confined masonry construction consists of masonry walls and horizontal and vertical RC confining members built on four sides of each masonry wall panel. This structural system offers an alternative to both unreinforced masonry and RC frame construction over the last 100 years. Important savings in cost and execution time can be counted as major advantages of this type of masonry construction. This system can be a conventional form for constructing new buildings as well as an alternative for post disaster reconstructing of buildings in many underdeveloped countries.

Low-rise confined masonry buildings have performed well in past earthquakes. Unfortunately, many seismic codes do not recommend confined masonry as a construction system as a structural system.

Previous studies revealed that lateral resistance and ductility of a confined masonry wall is greater than that of wall panel because of interaction between the masonry wall and confining elements. So it is essential that resistance of confined masonry walls be estimated by considering the participation of both structural elements, i.e. the brick panel and the confining elements and their interaction.

In this paper performance of confined masonry buildings in past earthquakes are reviewed then some advantages of confined masonry systems over conventional structural systems such as steel structures and RC structures are discussed. After that a few recent studies on confined masonry structures in experimental, numerical and analytical fields are presented.

The experimental results of confined masonry walls tested under lateral loads are verified with analytical results and equations recommended by the codes for masonry structures. Then an attempt is made to estimate seismic capacity of this kind of buildings. It is concluded that:

1. Results obtained from comparison of different relations with experimental result summarized in Table 1 reveal that we can predict shear resistance of a well constructed confined masonry wall efficiently.
2. From FE analysis and its agreement with experimental result, it can be concluded that shear resistance of confined masonry walls estimated accurately. Hence, having a robust methodology for estimation of seismic capacity is viable.
3. Considering that confined masonry system is cheaper than almost all other structural systems, they are good alternatives for any construction planning for low income families.

Table 1. Comparison of theoretical and experimental values for shear capacity.

Exp. No.	V (ton)	V_{dowel} (ton)	V_{total} (ton)	$V_{exp.}$ (ton)	Diff. %
1	5.3	1.5	6.8	14	51%
2	5.4	1.5	6.9	14	51%
3	6.1	1.5	7.6	14	46%
4	4.7	1.5	6.2	14	56%
5	5.4	1.5	6.9	14	51%
6	5.5	1.5	7	14	50%
7	8.9	1.5	10.4	14	26%
8	11.5	1.5	13	14	7%

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Derivation of buckling curves of pane-like mono section glass columns of heat strengthened and tempered glass

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ABSTRACT

The requirements of maximum transparency demands load-bearing elements, such as pane-like glass columns. To achieve sufficient robustness laminated cross-section will be necessary. The design of such load bearing glass structures requires the knowledge about the stability behaviour and appropriate technical rules. In this direction only a few investigations have been performed so far, predominantly at monolithic sections and sporadically at cross sections with laminated glass. However, before laminated glass is investigated, the buckling curves of monolithic glass columns needs to be resolved.

Therefore the aim of the investigations was to derive simple and consistent design rules for glass columns with monolithic sections of tempered and heat strengthened glass under axial loading with regard to the European buckling curves for steel columns based on the analytical methods of [Maquoi & Rondal, 1978]. The proposed design rules were to be verified by existing buckling tests [Holberndt, 2006, Liess, 2001, Luible, 2004] and by experimental tests and numerical simulations [Feldmann & Langosch, 2009] of the RWTH Aachen University.

The attained research results are the following:

– On the basis of the 2nd order theory, buckling curves for glass columns could be derived from the stress equation, being transferred into the format of European buckling curves. The comparison of the proposed analytic buckling curves to experimental buckling tests as well as to the numerical calculations shows good compliance, a fact that is also represented by low partial safety factors γ_M.
– For the effective imperfections the following values could be suggested: $e_0 = L/400$ for TG and $e_0 = L/300$ for HSG. However, in practice, installation tolerances have always to be considered. These have conservatively been estimated by a value of 3,0 mm for glass columns with a thickness of 12 mm. Considering this, effective imperfection values of $e_0^{TG} = L/400 + 3{,}0$ mm or $e_0^{HSG} = L/300 + 3{,}0$ mm respectively come out.

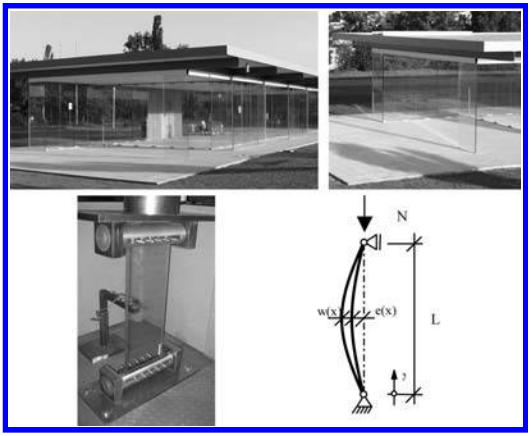

Figure 1. Above: Application of pane-like glass columns: Glaspavillon Rheinbach, Germany; below: experimental set-up for buckling of glass-panes according to [Luible, 2004].

– The research results also provide the foundation for further research on pane-like columns of laminated glass, buckling or the lateral torsional buckling of glass beams.

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The path to a bonded glass enclosure: Testing from small to large scale

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ABSTRACT

Modern architecture shows a growing demand for transparent building envelopes and components. Therefore glass is a widely used building material. To achieve even higher transparency, nearly invisible connections for the brittle material have to be designed. Specific adhesives such as UV- and light-curing acrylates permit transparent joints with high optical quality. Adhesives offer the unique possibility to join glass as substance-to-substance bond, which allow a nearly homogenous load transfer going along with a reduction of local stress peaks – in comparison to clamped or bolted connections.

Joining glass beams to whole transparent frames has been identified as a promising field of application for acrylate adhesives. To assess the strength, rigidity, durability and other material parameters of the selected adhesive system intensive testing was performed. Finally, a bonded frame for an all glass enclosure was developed from vision to final solution in close cooperation with industrial partners. As part of an all-glass enclosure for the helium liquefaction unit of the Leibniz Institute for Solid State and Materials Research in Dresden, a glued frame corner without additional metal fasteners has been produced for the first time in Germany (Fig. 1).

Four glass frames support the transparent enclosure, with the posts and rails of each frame connected at the corners by means of a transparent radiation-cured acrylate system in double shear. The panes of glass forming the enclosure are bonded to the frame members over their full length with a structural silicone adhesive. The frame members are made from four-ply laminated safety glass. At the corners the outer plies of the posts overlap with the inner plies of the rails to form what woodworkers would call a "corner bridle joint" (Fig. 2).

Material properties such as Young's modulus and Poison's ratio were determined by testing specimens which consist only of the bulk material. Further investigations comprise a test sequence of small bonded glass-glass-specimens which were shear-loaded at different temperature levels. Additionally, their shear strength was investigated after appropriate artificial ageing scenarios. Attention has also to be paid to the manufacturing technology. Studies on relevant curing parameters and the bonding process performed on life-size frame corners showed promising and repeatable results.

Figure 1. All glass enclosure after manufacture.

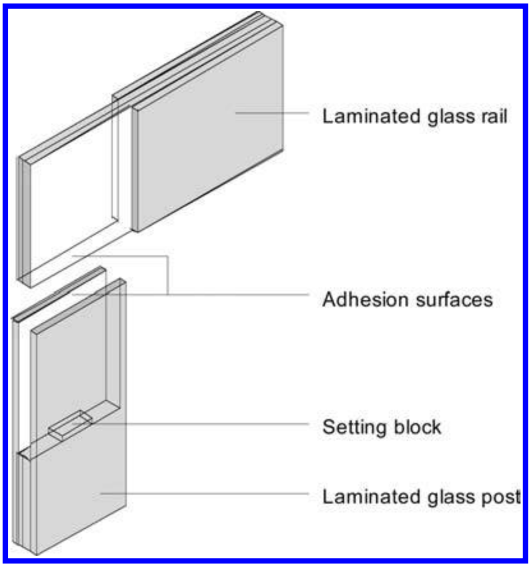

Figure 2. The principle of the "corner bridle joint" between the laminated safety glass frame members.

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Self-supporting glass roof as transparent space grid structure

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During the reconstruction of the former palace Reichstagspräsidentenpalais the Institute of Building Construction of Technische Universität Dresden, Germany and the Berlin architect Winfried Brenne Architekten took part in the realization of the glass roof above the inner courtyard of the palace. This glass roof is the first project realization of the new developed transparent space grid structures with load bearing glazing. These structures base on steel double layer space grid structures, at which the steel bars of the compression layer are replaced by load bearing glazing. Main objective of the roof design is lightness and high transparency for user, an elegant geometry from outside view and the avoiding of horizontal reaction forces due to vertical loads.

The inner courtyard has a trapezoidal geometry with 14.0 m/11.6 m width and 21.0 m length. The roof possesses a single curved geometry and consists of three parts: the two gables and the middle part. Both the gables are traditional steel elements with linear supported glazing. The middle part is a transparent space grid structure of Half-Vierendeel-beams with moment stiff connected lower chord and posts. It is 16.2 m long and consists of 9 segments á 1.8 m.

At the head of the posts cut stainless steel knots are bolted. They support the in-plane loaded IGU (build-up: 8 mm FT/16 mm spacing/19 mm heat-soaked FT/3.04 PVB/8 mm HS). The glazing possesses a regular size of 1800 mm × 1260 mm. Only the 19 mm heat soaked FT ply transfers the axial load. The structure is constraint-free supported in order to prevent horizontal reaction forces. The load application construction in the knots consists of three parts. A plastic block made of glass fibre reinforced poly-acetate is hinge fixed in a middle part made of aluminium. The third part, an aluminium wedge allows the tolerance adjustment. The load application via the block material only occurs into the edge of the 19 mm heat-soaked FT ply.

Basis for the realisation of the glass roof design is an issued individual approval of the Building Authorities with comprehensive testing. The testing included small specimen tests of the local load application into the glass edge, the long term behaviour of the plastic

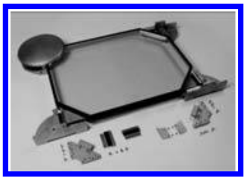

Figure 1. Glazing with knots.

Figure 2. Buckling test.

Figure 3. Load bearing with a total load of 13 tons.

Figure 4. Assembling of an arch segment.

blocks, testing of the buckling behaviour, walk-on and post-breakage robustness tests and finally a life-size mock-up of the largest arch with an applied total load of 13 tons.

For the realisation of the project only a few small time windows existed. Therefore all nine segments and the two gables were prefabricated at the steel contractor MBM Metallbau Dresden. The assembling on site was finished within a week. Daily two or three segments were transported to the site, lifted up, adjusted and bolted. The high exactness of the steel construction prevented any on-site drilling. After assembling the glazing joints were sealed with silicon and the knot huts fixed on the knots and sealed to the glazing, too. The glazing of the gables, the finishes and the drainage completed the work.

The authors thank the client Bundesamt für Bauwesen und Raumordnung, the architect Winfried Brenne Architekten, the steel contractor MBM Metallbau Dresden, the Building Authority Senatsverwaltung Berlin and as consultant Prof. Dr.-Ing. Jens Schneider for the excellent co-operation.

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Transparent mullions and transoms in facades

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ABSTRACT

Steel and aluminium are the main materials for transoms and mullions of contemporary curtain wall systems. The request of more transparency in façade constructions can not always be solved satisfactory in this way. Therefore, glass is increasingly used as load-bearing and stiffening element.

Currently there are no regulations for the structural design of such load-bearing glazing elements. Therefore, those constructions will always cause in special considerations related to technical issues and building law. Especially the brittleness of glass leads to unprofitable oversizing of structural elements. That increases the ultimate limit state of bearing capacity and serviceability. But apart from it the post-breakage behaviour after glass breakage must be taken into account, too. Recent researches demonstrate that – independently of the type and combination of glass used – this performance of glass beams is not guaranteed.

A hybrid glass beam consisting of laminated glass and steel is able to combine the transparent characteristic of glass and the ductility as well as the rigidity of steel. The research project HybridGlasSt studies the use of three different hybrid glass beams – as shown in figure 1 – for an application as mullion and transom in vertical facades for interior areas. The connection of the materials is achieved by linear bonding with a transparent acrylate adhesive, which is distinctive of having a short curing time in comparison to many other adhesives. Secondly, the linearly bonded joints enable continuous load distributions which avoid local stress concentrations. In conventional applications those concentrations often cause damages in glass. For the use the adhesive shall possess a high strength to distribute loads and a sufficient elasticity to compensate elongations due to temperature alterations.

Within a modified four-point bending test beams were tested about the strong axis regarding to their load bearing and post-breakage behaviour. The results show that the bending stiffness of the reinforced hybrid glass-steel beams regarding the first loading is about 20% higher than of the glass beams.

A difference between the hybrids and the glass beams is indicated to their post-breakage behaviour. A cross section like S3 can reach 80% of the bending stiffness as in first loading on undamaged specimens. In comparison to glass beams which lose almost all their stiffness after breakage the hybrid beams show a significant well-load-transfering behaviour.

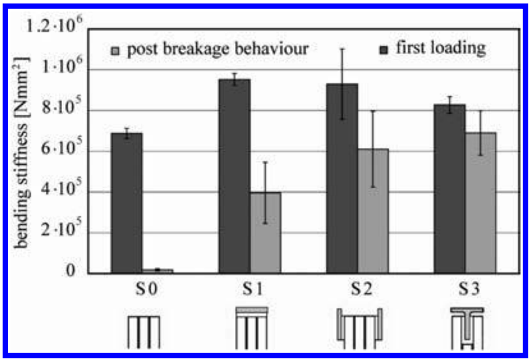

Figure 1. Bending stiffness of hybrid beams compared with glass beams.

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Polymers transfer significant compression forces into glass edges

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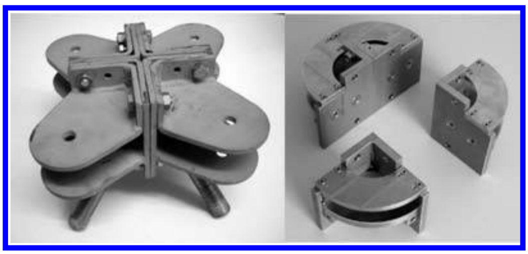
In a recently finished research project the Institute of Building Construction of Technische Universität Dresden was decisively involved in the development of transparent space grid structures. These structures base on steel double layer space grid structures, at which the steel bars of the compression layer are replaced by the load bearing glazing. The load application into the glass panes is realized by plastic blocks within the knots at the pane corners.

Different solid materials which could serve for the load application into the glass edge were tested. To ensure the transfer solid block materials have to possess certain mechanical, sometimes contradictory properties. The principal behaviour of the glass-block contact was investigated in preliminary tests for aluminium, poly-acetate POM-C and zinc cast.

A comprehensive testing program was executed at aluminium, glass fibre strengthened poly-acetate POM-C GF25 and poly-acetate POM-C specimens with dimensions of 80 mm × 16 mm.



Figures 1a and 1b. Different knot stages.

Figures 2a and 2b. Testing facility for block materials and failed glazing with aluminium blocks.

Polymers possess time depended visco-elastic respectively relaxating deformation behaviour. Polymers act with a certain time shift, the relaxation time τ on applied stress. The burger model is made of parallel and series connected spring and damper elements and allows the representation of the deformation behaviour.

$$\varepsilon_{ges}(t) = \left(\frac{1}{E_0} + \frac{1}{\eta_0} + \frac{1}{E_{rel}} \left(1 - e^{-t/\tau}\right) \right) \cdot \sigma_0 \qquad (1)$$

Another material law was developed by Kunz 2007.

$$E_c(t, \vartheta) \approx E(t_0, \vartheta_0) \cdot a_0^{\left(\frac{\vartheta}{\vartheta_0} - 1\right)} \cdot \left[1 - \frac{1}{3}\left(1 - c_c\right) \cdot \log_{10}\left(\frac{t}{t_0}\right) \right] \qquad (2)$$

The 1000 h long-time experiments were conducted in two groups at room temperature and at 45°C. A compressive stress of 27.5 MPa was applied with a linear spring.

The two described material models are compared with the measured creeping behaviour. The necessary start values for the use of the models are derived by the measured creeping data.

At the comparison with the measured creeping curve the Kunz material law possesses only certain suitability to represent the creeping behaviour within a time up to 4000 hour. The Burger-model exactly represents the creeping behaviour up to 50000 hours.

Figure 3. Specimens and testing facilities for creeping tests.

15. Structural safety, structural reliability, risk assessment

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An inverse reliability analysis based on stochastic simulation and artificial neural network

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ABSTRACT

An inverse reliability problem is the problem to find design parameters corresponding to specified reliability levels expressed by reliability index or by theoretical failure probability (Der Kiureghian et al., 1994). Design parameters can be deterministic or they can be associated to random variables described by statistical moments. The aim is to solve generally not only the single design parameter case but also the multiple parameter problems with given multiple reliability constraints.

A new general approach of inverse reliability analysis is proposed. The inverse analysis is based on the coupling of a stochastic simulation of Monte Carlo type and an artificial neural network. A novelty of the approach is the utilization of the efficient small-sample simulation method Latin Hypercube Sampling used for the stochastic preparation of the training set. The procedure has been tested and applied by authors for material parameters identification (Novák & Lehký, 2006) and damage detection (Lehký & Novák, 2009). It is now extended and applied for inverse reliability problems too. The validity and efficiency of the approach is shown using numerical examples with single as well as with multiple reliability constraints.

Inverse reliability problem can be formulated as follows. Suppose the safety margin Z and the limit state function $g(.)$ in the original basic space of random variables $\mathbf{X}$ is:

$$Z = g(\mathbf{X}). \tag{1}$$

Theoretical failure probability p_f is expressed as:

$$p_f = P(Z \leq 0). \tag{2}$$

In inverse reliability problem design variables can be deterministic or random ones. Therefore we include additionally to the vector of basic random variables $\mathbf{X} = X_1, \ldots, X_i, \ldots, X_n$ also the vector of design deterministic parameters $\mathbf{d} = d_1, \ldots, d_k, \ldots, d_p$ and the vector of design parameters of random variables $\mathbf{r} = r_1, \ldots, r_l, \ldots, r_q$. Design parameters of random variable are their statistical moments.

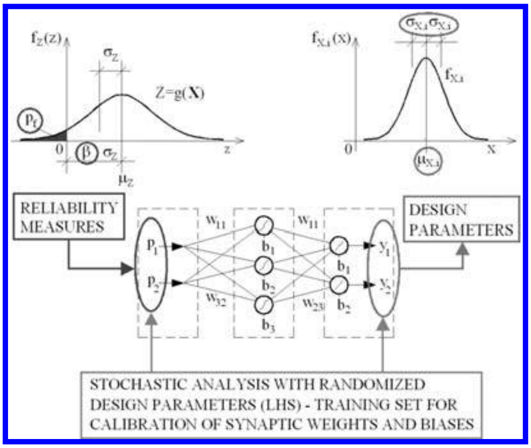

Figure 1. A scheme of stochastic training of the artificial neural network for inverse reliability analysis.

In case of multiple limit states we have several safety margins Z_j and target failure probabilities $p_{f,j}$, where $j = 1, 2, \ldots, m$. The inverse problem can be stated generally as:

$$\begin{aligned} &\text{Given: } p_{f,j} \\ &\text{Find: } \mathbf{d} \text{ or/and } \mathbf{r} \\ &\text{Subject to: } Z_j = g(\mathbf{X}, \mathbf{d}, \mathbf{r})_j = 0 \text{ for } j = 1, 2, \ldots, m. \end{aligned} \tag{3}$$

The procedure is sketched in Figure 1.

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Reliability differentiation in the Eurocodes

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ABSTRACT

Eurocodes are a series of ten European Standards providing a common and coherent approach to all aspects of structural design of buildings and civil engineering works. A major innovation provided by the Eurocodes is the implementation of modern reliability methodologies in the design of members and structures. Thereby acceptability limits (or targets) are taken into account for probabilities of failure, potential losses and the amount of investment necessary to improve reliability.

Target reliability criteria provided in the Eurocodes (EN 1990) are reviewed in this contribution. The basic reliability background is summarized first. The reliability acceptance criteria and the associated values are discussed then. Examples are provided.

Risk acceptance criteria are introduced in the Eurocodes in terms of target and acceptable (i.e. design) failure probabilities p_d and associated reliability indices β_d. The values have been derived through long studies by combining various approaches including optimization and calibration. They reflect the possible failure consequences, the reference time period and represent reasonable minimum requirements. It is emphasized that p_d and β_d are formal conventional quantities only and may not correspond to actual frequency of failures.

Table 1 shows classification of target reliability levels provided in EN 1990. Reliability indexes β_d are given for two reference periods T (1 year and 50 years) only, without any explicit link to the design working life T_d.

Typical example structures of reliability class 1 (RC1) are greenhouses and agricultural structures with limited risk to human life. Example structures of reliability class 2 (RC2) are residential and office buildings. Structures of reliability class 3 (RC3) are stadiums, public buildings and bridges.

The acceptable reliability indices of Table 1 are associated to failure of components. Global failure conditions are taken into account through the robustness requirements given in the Eurocodes.

Table 1. Reliability classification in accordance with EN.

Reliability classes	Consequences of failure	Reliability index β: β_a for $T = 1$ year	β_d for $T = 50$ years
RC3 – high	High	5.2	4.3
RC2 – normal	Medium	4.7	3.8
RC1 – low	Low	4.2	3.3

The reliability level is implemented in practical design through the partial factors. Based on reliability analysis considerations the partial factor is defined as a function of the reliability index. Consequently different partial factors can be derived for different structures as classified in the reliability classes.

The reliability differentiation procedure implemented in the Eurocodes is a step towards to optimal design.

ACKNOWLEDGEMENTS

The authors thank the Czech National Agency of the Leonardo da Vinci Programme (LdV NA) for supporting the presented work out of project CZ/08/LLP-LdV/TOI/134020 "Transfer of Innovations provided in the Eurocodes".

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Towards sound assessment and improvement of robustness of structures: Treatment of structural robustness in European standards

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ABSTRACT

The provision of adequate safety in structures can be set out in the form of requirements that comprise:

- a standard safety format that incorporates models of material and structural behaviour and the underlying uncertainties with the use of appropriate factors of safety as stipulated in modern design and assessment codes, and
- provisions to ensure adequate robustness.

This is illustrated in Figure 1.

However, a well defined separation does not presently exist between the standard safety format and the provisions for robustness. A starting ground for establishing sound assessment procedures and achieving improvement of robustness of structures would be to clearly isolate the requirements under the two categories (Faber 2009). This demands an effective understanding of the treatment of robustness in modern structural codes.

Towards this end, a scheme for systematic categorisation of the robustness related provisions is proposed in this paper. The following fields used in this scheme can be related to different aspects of risk management in structures.

- Approach to risk treatment (structural measures, avoidance, protection or sacrifice)
- Nature of risk control (active or passive)
- Relationship with event/exposure (independent or specific)
- Manner of reducing risk (probability of event occurrence, probability of local damage, probability of system failure or consequences)
- Applicability in life cycle of structure (planning & design, execution, operation or maintenance)

Next, an identification of major factors influencing the performance of structures as designed according to the standard safety format needs to be carried out. For these identified factors, the required/optimal provisions to ensure sufficient robustness would need to be identified. Assuming that the standard safety format is already optimised, the optimisation of the identified robustness provisions can be separately carried out based on considerations of overall/lifetime cost efficiency and investments into life safety.

In the second part of this paper, the results from a review study of European standards dealing with the design, execution, material aspects and maintenance of concrete and steel structures carried out to identify and categorize the various robustness related provisions in these standards are discussed (Narasimhan & Faber 2010). The categorisation provides a systematic and differentiated mapping of the treatment of robustness in European standards. This enables the identification of possible areas where i) redundant provisions concerning robustness exist and ii) the code provisions are presently inadequate. Also a viable platform for the establishment of improved or optimal provisions for robustness in structures is made available through this categorization.

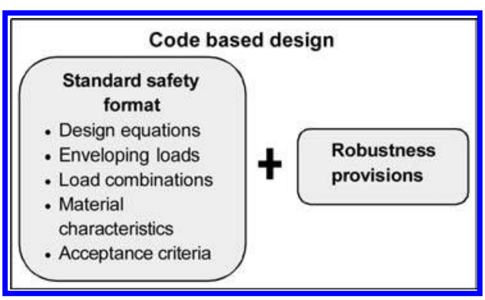

Figure 1. Illustration of the main components of requirements in design and assessment codes.

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Review of the ISO 2394 reliability basis of structural design

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ABSTRACT

The International Standard ISO 2394:1998 played a leading role in advancing the application of structural reliability as basis for structural design over many decades and through several editions. This paper surveys the way in which its present version should be reviewed to further enhance its role and function in harmonised international practice.

A number of related standards and pre-normative model codes have been issued since publication of ISO 2394 in 1998, including the Probabilistic Model Code (JCSS-PMC 2001) and Eurocode EN 1990:2002, which should serve as review reference. Other standards from ISO TC98 elaborate on topics that are incorporated in ISO 2394, such as risk considerations (ISO 13824:2009), existing structures (ISO 13822:2002), durability (ISO 13823:2008) and factored limit states design (ISO 22111:2007). Related pre-normative standardised design procedures include the JCSS risk assessment procedure (2008) and the *fib* Model Code 2010 (draft) (FIB 2009).

The revised standard should not only benefit from the advances presented in these standards, but should also satisfy the requirement to present a common *basis of design* to them, similar to the role and function of EN 1990.

The systematic development of the principles of reliability from the basis of risk management should play a central role in the review process, providing a coherent scheme of provisions for:

- **Structural performance:** Requirements for structural performance, which are treated in the verification procedures in terms of reliability, conforming to risk principles and procedures.
- **Reliability differentiation:** Develop and apply a scheme of reliability differentiation based on risk optimization, linked to risk based treatment of strategic and operational management of safety.
- **Design situations:** Define a comprehensive scheme of design situations which is related to the respective performance and reliability requirements, with special reference to transient and extraordinary (accidental) conditions and requirements for structural integrity and robustness.
- **Limit states:** Stipulate a comprehensive scheme of limit states, such as provision for durability and fatigue; the various classes of serviceability and associated criteria:
  - Relate the limit states to design situations; and
  - The associated levels of reliability.
- **Reliability calibration:** Procedures for calibration of partial factors and for design by testing.
  - Presentation of basic variables as random variables, processes and fields; including indicative models for classes of basic variables.

Following the experience with the development of Eurocode, clearer guidance should be provided for operational design procedures for conventional structures based on partial factor limit states design.

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Structural reliability and the basis of design for concrete structures

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ABSTRACT

In this paper the essential performance requirements for structural concrete of Eurocode EN 1992-1-1:2004 are reviewed to assess (i) its reliability performance (ii) its suitability for adoption as a South African standard.

Figure 1 presents graphs based on resistance reliability models of the *Eurocode 2 Commentary* (ECP 2008), indicating the partial factors for steel and concrete $\{\gamma_s; \gamma_c\}$ as a function of the target reliability (β_T) for two cases: (i) standard practice; (ii) improved quality control (QC) measures. Adjustment required for other reliability classes, notably the important RC3 structures, is also shown. Improved quality control measures are clearly effective.

The equivalent results for South African standards, in accordance with SANS 10160:2010 are shown in figure 2, using four reliability classes, with RC2 the reference class, RC3 for # stories > four.

Partial factors of {1.10; 1.4} are sufficient for all reliability classes, however requiring improved QC measures for RC3 & RC4.

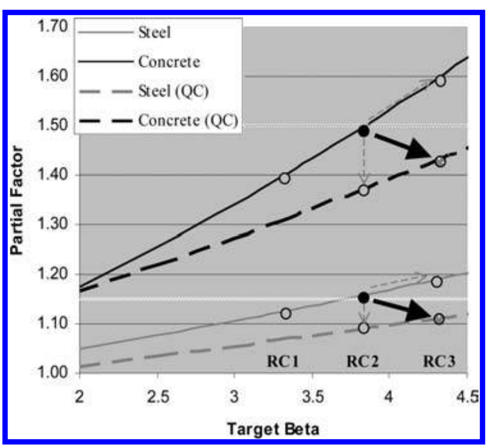

Figure 1. Partial factors for steel and concrete based on ECP model; showing three reliability classes RC1, RC2 & RC3.

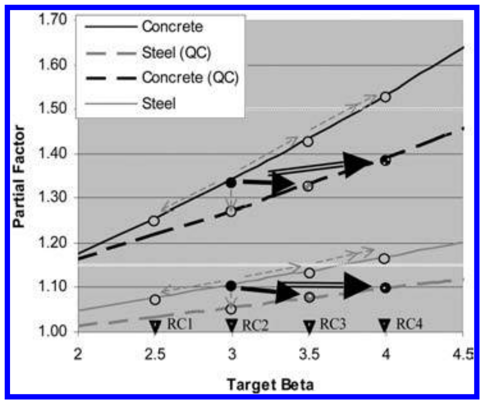

Figure 2. Partial factors for steel and concrete for SANS 10160 (RC1 – RC4) based on ECP model parameters.

The elaborate Eurocode scheme of design situations and limit states from EN 1990 should be contrasted to the rather simple scheme of partial factors employed in EN 1992-1-1 to cover a very broad scope of design procedures for concrete resistance. A scheme of model factors should effectively remedy this simplification. From a reliability perspective, EN 1992-1-1 is evidently suitable for use under South African conditions.

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Risk analysis of construction projects: From risk identification to contingency timetable

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ABSTRACT

The first part of the paper presents a review of problems related to the risk analysis in construction projects implementation: from the risk estimation understood as reliability of construction processors (including redundancy methods and inertia) to complex assessment of risk management.

The evolution of methods of risk analysis is presented. Earlier analyses, based on reliability and probability of keeping the project deadline, turned out to be insufficient. Therefore a special model/simulator has been prepared. As final effect, two distributions can be achieved: proper work of the system and breakdowns of the system.

Risk analysis needed to be strengthened using other methods, and economic concepts has to be utilised. An example which can be shown here are discounting methods, in this NPV. Besides, Value at Risk (VaR) is an interesting measure of project risks.

Another interesting current in development of these methods originates from analysis of network models, and ends at Earned Value (EV) and Earned Schedule (ES) methods. Detecting potential risks for the plan is the essence of those methods.

Experience shows that, from the point of view of usefulness, contingency of time and contingency of costs are proper forms of risk measure. Those measures influence the so called risk management cycle, including: Risk identification, Quantification, Mitigation of risk, Reduce of risk, Contingency plan, Contingency budge, Timetable. In consequence, we use two notions: contingency amount and contingency time.

In the second part, three practical examples are analysed as case study. Case 1 regards to methods based on a list of risk factors and hierarchy of risk level. Several methods are quoted (COMPASS, RAMP, ICRAM and MOCRA). Generally, the analysis is bases on three levels of risk and factors: macro level, market level, project level. A characteristic feature of these methods is risk factor list and correlations between factors.

Case 2 regards the allocation risk in timetable on example of gallery-shopping centre in Poznan. Two alternative timetables were developed. The first alternative timetable was developed for the latest time of conclusion of the project arrived at as a result of the simulation. The second alternative timetable was useful for the construction management in controlling the progress of work on the daily basis. Pertmaster v7.81 computer software was used for the purpose of simulation.

Case 3 presents a risk analysis in a project based on example of building football stadiums in Poland before EURO-2012 Cup, and preparation of cities hosting the football matches. The assessments of readiness status of the four football stadiums and an infrastructure are presented in the light of UEFA reports. Risk of infrastructure in cites hosting is define in a five level scale. Three level scale is designed for estimate of stadiums realization. UEFA reports had intensified the organisation of work. If the risk levels in specific towns do not change, those towns may be faced with the danger of withdrawing the right to organise the championships.

Risk should be regarded as a state in which there is a possibility of loss. The paper shows that the best notion describing risk is dispersion of expected and real results. Until recently, risk analysis of construction products was discussed more often as a scientific and research issue, and less often as a practical issue. Requirements of international contracts, including UEFA requirements shed new light on this problem.

The experiences presented in this paper may prove useful in the implementation of other construction projects.

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A new approach for time dependent risk assessment of coastal structures

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ABSTRACT

This paper is concerned with the risk assessment of coastal concrete structures subjected to the combined wave overtopping and strength deterioration. As is well known, rising sea levels and increased sea storminess have resulted in higher frequency of wave overtopping and greater magnitude of hydrodynamic action on coastal defenses. The situation has been exacerbated for reinforced concrete coastal structures due to the seawater induced corrosion of reinforcing steel in concrete which reduces the load carrying capacity of the structure. This combined effect has posed higher risk to the public and necessitated a thorough assessment for coastal structures.

In this paper a time-dependent systems reliability method is presented to predict the risk of failures due to the increased wave overtopping and reduced structural capacity. A stochastic process is proposed to model the time-variant and random nature of severe waves as shown in Figure 1. Also proposed is a structural deterioration model to allow for seawater induced steel corrosion in concrete. An example is provided to illustrate the proposed method whereby important factors that affect the risk are also studied. Some typical results are shown in Figures 2 and 3.

It is found that the rising sea levels and increased sea storminess reduce the safety and serviceability of coastal defenses. The method presented in the paper can provide useful information for structural engineers, operators and asset managers in developing a risk-informed and cost-effective management scheme for coastal defenses.

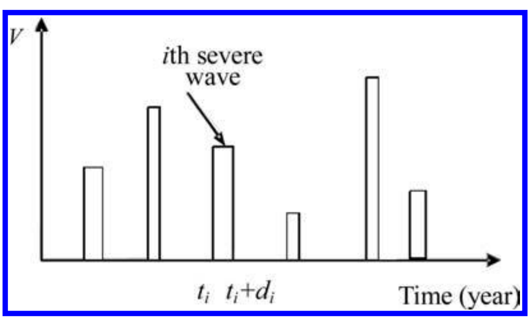

Figure 1. Poisson pulse process for severe waves.

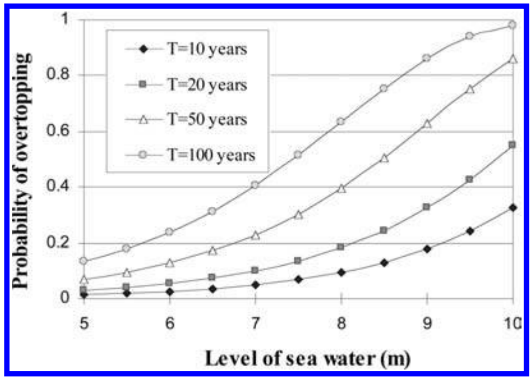

Figure 2. Risk of overtopping versus rising sea level for different service life.

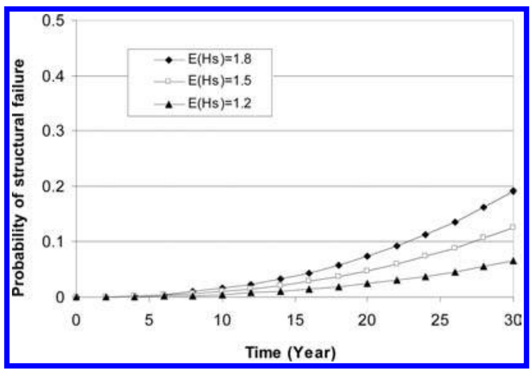

Figure 3. Risk of structural failure for different height of severe wave, i.e., overturning moment.

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Risk-based bridge inspection

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ABSTRACT

In Germany, there are 38,006 Bridges with a total length of 1,996 km and a combined bridge deck area of 28.97 million m^2 (as of 31.12.2007) within federal responsibility. The bridge inspection is currently regulated by DIN 1076 and is separated into major inspections (every six years), minor inspections (3 years after each major inspection) and special inspections. The major inspection has to be carried out within touching distance of all elements while having access to all parts and with a fixed scope of examination. The scope of examination for minor inspections has to be determined by the inspection staff based on the results of the latest major inspection. Thus, determining the inspection scope becomes very subjective. The condition of the bridge and predictions of the development of its condition are not considered with adequate accuracy.

Within a research project of *Technische Universität München* instructed by the *Bundesanstalt für Straßenwesen (Federal Highway Research Institute)* a risk-based bridge inspection model was developed.

Aim of this model is the determination of the inspection scope based on academic risk analyses.

Central point of the risk-based inspection model is the subdivision of the structures into its components and the definition of deterioration models for all potential defects of each component. Using all of these deterioration models it is possible to calculate the probability of a defect occurring over time. A comparison of the time-variant probability with a critical probability may then result in more accurate scheduling of the next inspection.

Using different critical probabilities it is possible to weigh the defects and then take the safety and economic relevance of the defect into account. Also, the possibility of cheaper and earlier repair of a defect due to earlier maintenance may lead to a lower critical probability.

A combination of all relevant deterioration models with the corresponding probability analyses leads to many different testing dates. To make the inspection cost-effective it is necessary to combine several testing dates. With inspection dates within fixed time intervals, but variable scope of examination, all components could be tested contemporarily to the critical defect with passable effort.

Defects which can be confirmed at the inspection have to be considered in maintenance scheduling. After maintenance, the new condition of the component then has to be incorporated into the deterioration model. If there is no maintenance up to the next inspection date the components have to be checked again, because the critical probability is still exceeded. If the predicted defect cannot be confirmed at the inspection the deterioration models have to be adapted to the results of the inspection. With this updating of the theoretical models the accuracy of the deterioration models can be improved (more detailed testing methods → more accurate deterioration models → decreasing number of inspections).

With this model the funds available for bridge inspections can be utilized more cost effective while simultaneously increasing structural safety because of the improved inspection intervals for the weaker parts of the structure. Furthermore it enables the authorities to consider inspection priorities in their budget planning.

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16. Structural optimization

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Optimum design of structures: Design, fabrication and economy

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ABSTRACT

Structural optimization is a design system for searching better solutions, which better fulfil engineering requirements. The main requirements of a modern load-carrying structure are the safety, fitness for production and economy. The safety and producibility are guaranteed by design and fabrication constraints, and economy can be achieved by minimization of a cost function. A lot of structural versions fulfil the design and fabrication constraints and designers should select from these possibilities the best ones. A suitable cost function helps this selection, since a modern structure should be not only safe and fit for production but also economic. This paper shows the elements of this system, describing optimization methods, cost calculations and an example. In the structural optimization process for an engineer it is important to know the behaviour of the structure well, the stresses, deformations, stability, eigenfrequency, damping, etc. It is as important to have a reliable optimization technique to find the optimum.

In our practice on structural optimization we have used several techniques in the last decades. We have published them in our books and gave several examples as engineering applications (Farkas & Jármai 2003, 2008). Most of the techniques were modified to be a good engineering tool in this work.

We have shown the Particle Swarm Method of global optimization which is one of such methods. A swarm of birds searches for food. Every member of the swarm searches for the best in its locality - learns from its own experience and their immediate neighbours and the ideal performers. The Particle Swarm method of optimization mimics this behaviour.

The other method IOSO is an advanced semi-stochastic algorithm for constrained multi-objective optimization (Egorov 1998) incorporating certain aspects of a selective search on a continuously updated multi-dimensional response surface. Both weighted linear combination of several objectives and true multi-objective formulation options creating Pareto fronts are incorporated in the algorithm. The main benefits of this algorithm are its outstanding reliability in avoiding local minima and its computational speed.

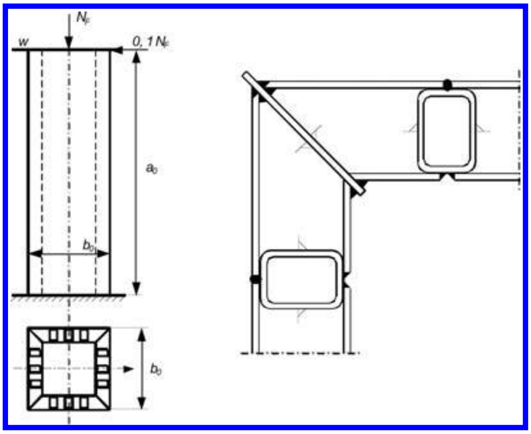

Figure 1. Stiffened box column with RHS stiffeners.

When we consider the interaction of design and technology, we should not forget the cost of the structure as the third leg of the system. These three together help us to find the best solution. These cost calculations are founded on material costs and those fabrication costs, which have direct effect on the sizes, dimensions or shape of the structure.

An example is shown (Figure 1).

This example shows that rectangular hollow section (RHS) stiffeners can be applied in welded cellular plates from which steel structures of advantageous characteristics can be constructed.

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Ant colony algorithms for nonlinear analyses, simultaneous analysis-design, and optimal design of structures

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ABSTRACT

In this paper nonlinear analysis of structures considering material and geometric nonlinearities are performed using the force method and ant colony optimization (ACO) algorithms. For this purpose, the complementary energy of the structure is minimized using ACO algorithms. A new formulation of the complementary energy of truss and frame structures for nonlinear analysis is presented. Considering the energy term next to the weight of the structure, optimal design of structures is performed. Finally simultaneous analysis-design of structures is formulated and applied to truss and frame structures. Each method is accompanied by simple examples for clear illustration and showing the efficiency. The comparison between ACO and other optimization algorithms shows that the ACO leads in better results in less computational time and iterations.

For this purpose an optimization algorithm is needed for minimizing the complementary energy function and the other goal functions. Considering the advantages, high performance and increasing applications of ACO, this optimization algorithm is selected as an efficient tool.

In this study some new goal functions are introduced for optimal simultaneous analysis and design of structures considering material and geometric nonlinearity which can be expressed as:

Linear analysis goal function:

$$\boldsymbol{F_U} = \mathrm{norm}(\{\mathbf{H}_{qp}\}\{\mathbf{p}\} + [\mathbf{H}_{qq}]\{\mathbf{q}\})$$

Nonlinear analysis goal function considering material nonlinearity:

$$\mathbf{G} = \mathrm{norm}(\mathbf{B}_1{}^t\mathbf{F})$$

Nonlinear analysis goal function considering geometric nonlinearity:

$$\mathbf{V} = \int_V e(\varepsilon)\mathrm{d}V - \sum_1^n \mathrm{P}_i \mathrm{u}_i$$

Simultaneous analysis-design goal function using specified stress ratios:

$$\boxed{\boldsymbol{F_U} = \{\mathbf{r}\}^t \begin{Bmatrix} \frac{\mathrm{L}_1 c_1 \acute{o}_{a1}}{\mathrm{E}_1} & 0 & 0 \\ 0 & \ddots & 0 \\ 0 & 0 & \frac{\mathrm{L}_m c_m \acute{o}_{am}}{\mathrm{E}_m} \end{Bmatrix} \begin{Bmatrix} \ddot{e}_1 \\ \vdots \\ \ddot{e}_m \end{Bmatrix}}$$

Optimal design of structures goal function which leads in the minimum weight of the structure:

$$\boxed{\boldsymbol{F}(\mathbf{q}, A) = \mathbf{W}(A)(1 + \alpha\, norm([\mathbf{H}_{qp}]\{\mathbf{p}\} + [\mathbf{H}_{qq}]\{\mathbf{q}\})) + \sum_{m=1}^{nc} \max[0, \mathbf{g}_m(A)]}$$

The examples studied in this paper for analysis, analysis-design and optimal design illustrate the capability and the accuracy of the present methods compared to those of the existing methods. Convergence histories of ant colony optimization algorithms for truss and frame structures show the high performance of ACO.

Formulation in terms of energy concepts permits the efficient application of ACO in optimization. The present method can easily be adopted for more general structural problems; therefore Beams, frames, plates can be treated in a similar way.

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An improved lower bound limit state optimisation algorithm

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ABSTRACT

Limit State analysis has been used in manual design methods for decades e.g. the yield line theory for concrete slabs, (Johansen 1972). Analysis of elastic structures was around 1960 revolutionised by the introduction of computers and the Finite Element concept. Soon after the first attempts to solve Limit State problems by computers were implemented, see (Grierson and Gladwell 1971). The methods did not penetrate into practice in the same impressive way as the linear Finite Element analysis did. The field of Computerized Limit State analysis did grow and extended the applications from frame and slabs also to include geotechnical problems, see e.g. (Sloan 1988) and reinforced plates, see e.g. (Ashour and Morley 1994). New applications of Limit State analysis in form of material optimisation were introduced, see e.g. (Damkilde and Høyer 1993) who dealt with optimisation of reinforcement in frames and concrete slabs. In the last decade the main developments have been in the optimisation procedure, where the interior point method in various formulation has increased the performance considerably, see e.g. (Mehrotra 1992).

In this work only lower bound Limit State analysis is considered, which gives great advantages for more complex yield criteria and in optimisation of material layout almost is compulsory. The lower-bound formulation results in a non-linear convex optimisation problem. The variables will be the stress state in the elements and either a load parameter or design parameters such as volume of reinforcement. The object function will in this context be the load carrying capacity. The restrictions will be linear equilibrium equations and non-linear convex yield criteria.

In order to have a more efficient implementation two remedies are used. The first is to eliminate the equality constrains a priori. This gives a considerably reduction in the number of variables. The method has in previous studies shown its capability, see (Damkilde and Høyer 1993). The second is to deal with the non-linear yield criteria directly and in this respect avoiding the large number of linear inequalities.

In the present paper the method is improved in terms of computational efficiency and improvements on the optimisation algorithm. In the earlier work, (Krabbenhoft and Damkilde 2003) a non-linear equation system is solved by Newton's method in order to obtain the solution of the optimisation problem incrementally. In the present work, the non-linear equations are solved more directly by elimination of equations.

As a test case, a slotted block in plane strain is considered, using the von Mises yield criteria and triangular elements with a linear stress variation. The results are identical to results obtained in earlier work using different optimisation algorithms.

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Multicriterion approach to minimum compliance topology optimization of trusses with comparison to stress-constrained minimum weight design

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ABSTRACT

In this work, the minimum compliance problem of trusses under multiple loading conditions is studied. This problem is typically treated in the literature by minimizing a weighted sum or the maximum of the compliances of the different loading conditions. In this study, a formulation based on multicriterion optimization is proposed.

In the proposed approach, the criteria of the vectorvalued objective function are the compliances of the loading conditions. This definition of the objective function is derived from the view, that the compliances of the separate loading conditions are usually conflicting and should be minimized simultaneously. Since generally this cannot be achieved by a single structure, a compromise in the performance with respect to the different compliances is sought. This can be accomplished mathematically soundly using the concept of Pareto optimality, which regards a structure optimal, if none of the criteria can be improved without deteriorating at least one other criterion. It follows, that multicriterion optimization problems have a set of Pareto optimal solutions. In the topology optimization of trusses, the Pareto optima may have different topologies. Based on the information obtained from these Pareto optimal topologies, the designer can choose those, which have most appealing properties for example with respect to cost or manufacturability.

The purpose of this study is to examine the proposed multicriterion formulation and the types Pareto optimal topologies which can be obtained. Another important aspect is the comparison of the minimum compliance topologies with stress-constrained minimum weight design.

The work is based on thorough numerical experiments on a test problem. First, the stress-constrained material volume minimization problem is solved for each statically feasible topology. Member buckling is handled in a simple way by setting the compressive stress limit to 15% of the maximum allowed stress in tension. The obtained minimum of the material volume is used as an upper bound for the allowable material in the minimum compliance problem in order to compare the results of the two problems. Then, a large number of Pareto optimal points is generated for the multicriterion compliance minimization problem in order to capture all Pareto optimal topologies.

The results verify that the multicriterion compliance problem has several Pareto optimal topologies. For the test problem, the minimal set in the criterion space is a continuous curve consisting of segments, each corresponding to a certain Pareto optimal topology. Along the curve, the transition from one optimal topology to another seems to be smooth. Thus, in several distinct locations, two or more different topologies produce identical optimal criterion values.

As the Pareto optimal topologies are compared with the minimum weight design, it can be observed, that for the test problem, the minimum weight topology is not included in the Pareto optimal solutions (see Fig. 1 for minimumweight design and two Pareto optimal trusses). Thus, the stress-constrained minimum weight design cannot be obtained by performing sizing optimization for the Pareto optimal solutions of the compliance problem, without altering the topology. Therefore, no benefit for finding the minimum weight design is gained through solving the multicriterion compliance minimization problem. On the other hand, for the compliance minimization problem itself, the multicriterion formulation provides more information about the optimal solutions than the traditional weighted sum and minmax approaches.

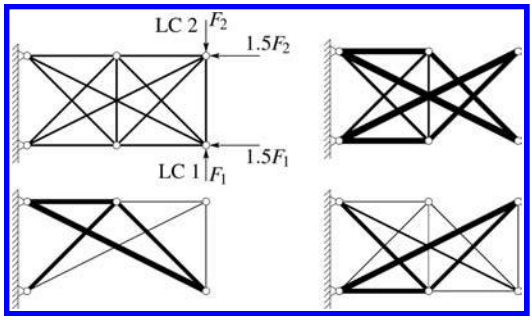

Figure 1. Ground structure with two loading conditions (LC 1 and LC 2) and results. Upper right truss is the minimum weight design. Two lower trusses are Pareto optimal for the minimum compliance problem.

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Optimum design of tubular trusses for displacement constraint

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ABSTRACT

A method is worked out to design for minimum volume of a truss made of circular hollow sections (CHS) for a displacement constraint. These cross-sections are larger than those required to prevent overall buckling of rods. The developed method is applied to truss columns of parallel and non-parallel chords. The investigated cantilever columns are loaded on the top by a horizontal concentrated force and the horizontal displacement of the top is limited. In the case of the parallel-chord-truss the distance of the chords and the CHS profiles are optimized. In the case of non-parallel chords the slope angle of chords and the CHS profiles are optimized for minimum structural volume. The comparison of the two optimized trusses shows that the truss with non-parallel chords has smaller volume.

Furthermore the method is applied to a tubular truss tower of a wind turbine. Since for such tower the eigenfrequency should be larger than the rotor eigenfrequency, this limitation is used to prescribe a displacement constraint for the tower top.

As a numerical problem a 45 m high 1 MW capacity wind turbine tower is investigated. The loads on the tower top according to Lavassas et al (2003) are as follows: a horizontal force 282 kN, a bending moment 997 kNm and the head weight according to Spera (1994) 940 kN.

The tower is divided to three parts. The upper and middle part (each 14 m high) should have a prescribed chord distance of 2.5 m because of the rotating blades of 27 m radius. Only the bottom part of 17 m high is designed with non-parallel chords and they inclination angle is optimized (inclination angle is 67.4^0, the tower width at the bottom is 18 m).

The allowable horizontal displacement on the top according to Baseos et al (2002) is H/100 = 450 mm. The required CHS profiles for chords and diagonals are calculated so that the displacement caused by the elastic deformation of rods from the upper truss is 34 mm, from the middle part 48 mm and from the bottom part 365 mm. The total weight of the tower is 650 kN. The eigenfrequency is approximately 0.8 Hz.

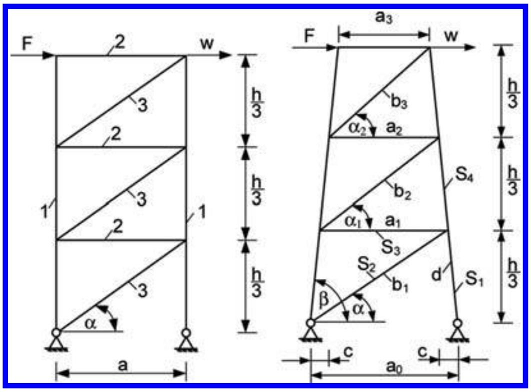

Figure 1. Tubular trusses with parallel and non-parallel chords.

The following comparison shows that the developed method can give realistic solutions and can produce competitive structural versions.

Hau (2003) gives the following data for a trussed tower: The tower height is 46.6 m, head mass 1800 kN, the truss widths on the top and on bottom are 3.5 m and 11.6 m respectively. The tower weight is 1100 kN, the eigenfrequency is 0.60 Hz.

It can be concluded that the tower constructed with the developed method has lower weight and larger eigenfrequency than that given by Hau, despite the fact that our tower has smaller cord distance in the upper and middle part.

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Shakedown design of structures under dynamic loading

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ABSTRACT

Since the first years of the second half of last century the optimal design problem has been studied by several authors (see, e.g., Gallagher and Zienkiewicz (1973), Save and Prager (1985), Rozvany (1989), Giambanco et al. (2002). Today, many of the fundamental features of the behaviour of an optimal structure under different loading combinations and limit conditions are sufficiently known and these results together with the technological progress led to the gradual upgrading of the international codes.

In particular, the Italian code prescribes that a multicriterion structural design be effected taking into account different combinations of static, cyclic and dynamic loadings, respecting the serviceability conditions for a combination of fixed and low seismic actions and not reaching the instantaneous collapse condition for combinations of fixed and high seismic load or fixed loads and wind actions. In Benfratello et al. (2009) the same authors provided a formulation devoted to the search of the optimal design as prescribed by the Italian Code.

Unfortunately, by solving the proposed formulation it is not possible to have information on the optimal structure behaviour slightly below the instantaneous collapse: actually, the structure could be subjected to ratcheting (and suffer a fast incremental collapse) or it can behaves in a plastic shakedown manner (possessing the capability for resisting to several load cycles before collapsing). Always in Benfratello et al. (2009), on the grounds of the obtained Bree diagrams, it is showed that the optimal structure obtained by solving the standard problem always approaches the limit state in conditions of incremental collapse. On the contrary, it is preferable to have a structure that under high intensity loads doesn't exceed the alternating plasticity exhibiting small plastic deformations.

Therefore, in the present paper the minimum volume design problem is performed imposing an elastic shakedown behaviour in serviceability conditions and, simultaneously, a plastic shakedown one for high intensity loads.

The sensitivity of the structural response has been investigated on the ground of the determination of the Bree diagrams of the obtained structures. In all the examined cases it has been observed that by solving the standard design problem (namely strictly following the Code) the optimal structure exhibits an incremental collapse behaviour even for loads not very close the instantaneous collapse ones. On the contrary, by solving the (elastic/plastic) shakedown design the optimal structure exhibits an alternating plasticity behaviour even for load combinations very close to the limit ones. Furthermore, the shakedown designs are characterized by just a very modest cost increment with respect to the safety improvement related to the plastic shakedown behaviour.

The examination of the Bree diagrams related to the optimal shakedown structures shows that often (depending on the frame typology) even such a safe structures are characterized by low safety factors against the fixed load. In order to definitely improve the optimal structure behaviour, a search problem is formulated imposing appropriate constraints on the actual and fictitious plastic deformations of suitably chosen elements. The related Bree diagrams confirm the expected improvement.

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Advanced high performance vehicle frame design by means of topology optimization

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ABSTRACT

Automotive chassis design is often based on company know-how and designer experience, and it develops mainly by trial-and-error. This fact implies that the result of the design process is likely to tend towards consolidated solutions that are poorly innovative and not necessarily altogether efficient.

Collaborative design techniques make it possible to avoid this drawback. Over the last decades many optimization algorithms have been developed and refined. These algorithms are powerful means for systematic design. Optimization techniques exhibit a wide range of applicability and can be employed in different scientific and technological fields of investigation.

The present study proposes a methodology for automotive chassis design based on topology optimization coupled with FEM analyses. In particular, the methodology is applied to the design of a chassis suitable for a rear-central engine high performance car.

At first, a 3D elements FEM model was made. In the model the suspension links are kept in fixed position, and room for the main vehicle compartments and for the passengers has been taken into consideration. The model was conceived in order to leave maximum freedom to the optimization algorithm in choosing the best design during the optimization process.

The objective of the optimization process is the minimization of the chassis mass in fulfillment of given constraints in terms of bending and torsional stiffness of the component. In a second moment, constraints on the local stiffness of the suspension, engine and gearbox links were added.

In the end, more constraints were added step by step, gradually increasing the complexity of the processes. In particular, a constraint on the modal response and on the behaviour of the structure in the event of crash. A suitable simplification was adopted, due to the complexity of the task, and to the fact that the optimization software employed does not allow non-linear analyses to be included in the topology optimization process: the dynamic non-linear simulations required by the crashworthiness assessment were brought back to an equivalent linear static problem so that it was possible to incorporate it into the optimization process.

The result of a topology optimization does not represent in itself a final design for the chassis. Topology optimization is important to define a suitable preliminar architecture for the structure, from which the chassis has to be re-drawn and further developed for the next stages. After the topology of the structure was established, the results were interpreted in view of creating 2D shell models to be used in the subsequent steps. In particular, topometry and size optimizations were used in cascade to the topology optimization.

Using topometry optimization the final layout of the chassis was checked and defined in more detail, and the designer had the possibility to save some weight by removing parts having almost no relevance from a structural point of view. In the end, when the final layout was assessed, size optimization was used to attain the optimal shell thicknesses.

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Sport car space-frame chassis design in view of weight reduction

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ABSTRACT

Over the last few years road vehicle homologation has become more and more challenging due to the introduction of stringent regulations regarding pollutant emissions. In order to fulfil these requirements two different design approaches can be pursued. In recent years, the focus in automotive industry mainly settled on the improvement of the engine performance in terms of fuel consumption. Another design approach aims at the vehicle lightening, yet in the respect of the safety standards.

In the present work a methodology for preliminary space-frame chassis design for rear-central engine sport cars is proposed. The purpose of the methodology is to design a chassis able to attain given goals of static torsional and bending structure stiffness. At a later stage targets regarding the NVH analysis were also included.

An elastic isotropic material (aluminum) is considered for the chassis so that it is possible to match with existing solutions which allow a relatively high productivity rate with a low manufacturing cost.

The chassis layout proposed are made in fulfilment of given geometrical constraints concerning the components overall dimensions (passengers, accessories, engine), and keeping the vehicle wheelbase, the track, and the suspensions layout fixed.

The design problem is faced using finite element analysis. The chassis numerical model is split into three sub-models: front, central, and rear. For each of them, a suitable interface constraint and loading condition have been conceived, by adopting superelements. In this way it is possible to parallelize the work of different teams working on a sub-model each.

Moreover, the idea of splitting the chassis in three parts allows to focus on different goals for each submodel. For instance, for the front and the rear part of the chassis designers are most interested in verifying the behaviour of the structure in event of crash and in ensuring enough stiffness on suspensions joints. Whereas, the design of the central part is more limited by geometrical constrains to house the passengers, but should also avoid stiffness drop granting a good linkage between the other two sub-models.

At first the method has been validated on the FEM model of the Ferrari 430 chassis to ensure that the finite element analysis of the sub-parts and of the full chassis hold equivalent results.

At a later stage, several sub-models have been built and tested, and the most promising ones have been presented in this paper. At first, the performance of the structures have been evaluated in terms of their torsional and bending stiffness. The loading condition, the constraints, and the method adopted for evaluating the structure stiffness adhered to Ferrari SpA internal regulations. The work also allowed the general applicability of these regulations to be critically analysed.

The sub-models proposed were mainly dictated by the designer experience aided by the application of optimization techniques to the sub-models, and were composed by shell finite elements. On each submodel, in fact, topometry and size optimizations were performed in order to assess the optimal shell thickness distribution throughout the structure.

At a later stage, the sub-models designed have been tested to reach NVH targets.

It is found that the methodology proposed, being based on experience and optimization techniques, is reliable and effective, and makes it possible to save weight yet in fulfilment of the targets imposed. The procedure is in constant development and the next step will be to extend the number of performance targets included, *e.g.* fatigue and crash, to fulfil all the required tests for chassis design.

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Optimum design of a rack and pinion gear pair using the Taguchi method

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ABSTRACT

In the case of a seatbelt retractor using gunpowder used in passenger vehicles, the seatbelt is retracted by operating the retractor using the gas pressure which is generated directly after a crash. As gas pressure pushes a rack and the pinion gear connected with the rack is rotated, the seatbelt is retracted. While the retracted length of the seatbelt retractor using gunpowder is at most 60∼70 mm, the safety regulation requires the length to be 120 mm. If the explosion pressure is increased to improve the retracted length, a problem in the structural safety of the retractor occurs. In addition, since the tension of the seatbelt is increased, a passenger may be harmed.

In this study, the geometry of a rack & pinion gear is optimized to increase the retracted length while the tension of the seatbelt is kept under 3 KN (this is the value that most automobile companies require). A design program for a rack & pinion gear is developed using MATLAB. To construct the profile of a spur gear, involute curve (Kuang *et al*) is used. Since involute curve has good manufacturability and compatibility and the angular velocity is constant regardless of center distance, it is widely used as the profile of a power transmitting gear (Marita). Figure 1 shows the profile of the rack & pinion gear designed using the spur gear design program developed in this study.

Increasing the retracted length of a seatbelt is set as an objective function and the number of teeth, module, pressure angle and tip radius of a tool are set as deign variables. The maximum surface stress and bending stress is considered as the smaller the better characteristic and the SN ratio is produced. For efficient experiments, $4 \times 2 \times 2$ orthogonal array is

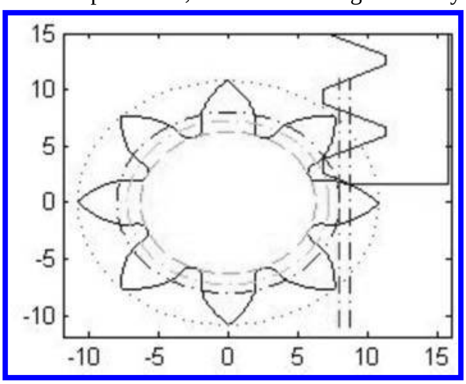

Figure 1. The profile of a rack & pinion gear.

Table 1. Input values for experiments.

Factors	Level	Value
Number of teeth * module (N*m)	0	8*1.75
	1	8*2
	2	9*1.75
	3	10*1.5
Pressure angle (ϕ)	0	20°
	1	25°
Tip radius (γ)	0	0.25 mm
	1	0.3 mm

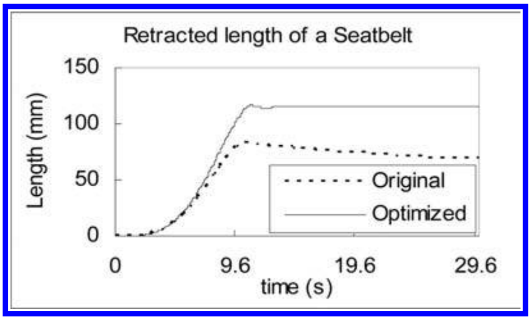

Figure 2. Result comparison between original & optimized gear.

created. On the basis of the dynamic simulation results about 8 gear types as shown in Table 1, the number of teeth and module of the gear, whose power transmission rate is the best, is selected. The pressure angle and tip radius, whose surface and bending stress is minimum, is chosen. As a result of experiments, when the number of teeth is 8 and module is 2, the efficiency of power transmission is the best. Also, in the case that the pressure angle is 25° and tip radius is 0.25 mm, both surface and bending stress becomes the minimum.

Figure 2 compare results acquired by applying original and optimized gear to the dynamic model. The slope of the retracted length is increased and also the amount of the retracted length is improved; the power transmission is concluded to be transferred better.

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Optimization of stiffened plates for steel bridges based on Eurocode 3 Part 1-5 using genetic algorithms

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ABSTRACT

Bridges are an important part of the infrastructure of states. Welded stiffened plates are widely used as a box girder for bridges. Therefore this paper deals with the design optimization problem for a single steel box with longitudinal web stiffeners.

The main aim of this study is to show how the provisions consigned in the CEN (2006) Eurocode 3 Part 1-1 and 1-5 may provide a rigorous mathematical formulation for an optimization task.

The second aim of this study is to check how all design rules of EC 3 Part 1-5 refer to each other to conduct a rational design.

Especially ultimate strength, buckling and serviceability provisions are taken into account to formulate the optimization problem. Width to thickness ratio (c/t-ratio) and a special requirement consigned in EC 3 Part 1-5 to guarantee the torsional buckling stiffness of longitudinal stiffeners are also considered in the formulation of the problem.

Optimization means solving problems in which one seeks to minimize or maximize a function by systematically choosing the values within an allowed set.

The optimization problem can be written in the canonical form as:

$$\begin{array}{llll} \text{minimize} & f(x) & x_i \in R^N & i = 1,\dots N \\ \text{subject to} & & & \\ & h_j(x) = 0 & j = 1,\dots M & \\ & g_k(x) \leq 0 & k = 1,\dots P & \\ \text{and} & & x_{LB} \leq x \leq x_{UB} & \end{array}$$

To solve this problem genetic algorithms are selected. Genetic algorithms are nature-inspired; thinking in terms of real life evolution helps to understand genetic algorithms. The genetic operators, selection, crossover and mutation contribute to improve the quality of the population. The solutions are comparatively analogue to the chromosomes. They work with possible solutions rather than deterministic ones.

Genetic algorithms are promising tools to assist design engineers to select optimal designs. Results from figure 1 and table 1 show their good quality.

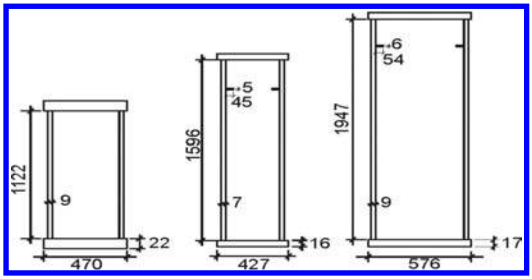

Figure 1. Possible solutions for the optimization problem.

Table 1. Solutions for a box girder with stiffener.

Method	b_f	t_f	h_w	t_w	t_s
Genetic algorithm	427	16	1596	7	5
	473	15	1567	7	5
	486	14	1602	7	4
	347	15	1774	8	6
Backtrack in Jármai (2001)	470	17	1440	6	5
	490	16	1440	6	5

all dimensions in mm.

Longitudinal stiffeners attached to a web of a box girder and subjected to compression are special cases of the problem of torsional buckling. In this respect the proposed provision in EC 3 Part 1-5 needs to be explained, thus an alternative provision in DIN 18800 Part 3 was used.

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Optimum design of plate girders by genetic algorithm

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ABSTRACT

The major obstacle in using raditional optimization techniques them is their mathematical complexity and only certain types of structural problems can satisfy their rigid requirements. On the other hand, soft computing techniques such as the GA were found to behave efficiently over a wide range of applications, even when employing numerous design variables and constraints. Goldberg & Samtani (1986) appeared to be the first that employed the GA for structural optimization. A 10 bar truss problem was utilized to study the GA's role in structural optimization. It has also been used to optimize framed structures (Jenkins 1991). Fu et al. (2005) used the GA for optimum design of composite girder bridges.

This paper presents a genetic algorithm-based optimization procedure for the optimum design of steel plate girders. The plate girder is composed of a top flange plate, a bottom flange plate, a web plate and transverse stiffeners, as shown in Figure 1.

The example presented is a four-equal-span continues plate girder (PG) to be designed and optimized. The plate girder has been subjected to ten load conditions with different space lengths. The results are presented in Figure 2 and compared with the results obtained with those found in the literature. Results indicate that the proposed approach found better designs than neural network application and traditional approach.

In plate girder design, web slenderness ratio (h/t_w) and flange rigidity ratio (b/t_f) are important variables which often control the design. A parametric study on these ratios will certainly reveal the behavior of the design variables and can offer some guidance to engineers for future design. For plate girders a study was made to investigate the optimum design for (h/t_w) from 260 to 340 in 10 increments and for (b/t_f) from 10 to 32 in 1 increments.

The minimum weight designs of girders are plotted in Figure 3. Observing these figures enables one to conclude that for flange rigidity over 14, when the web slenderness ratio increases, the weight saving increases.

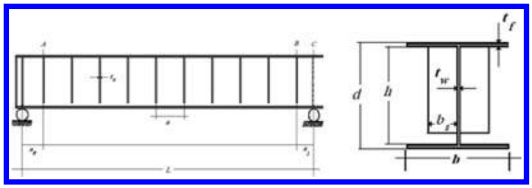

Figure 1. Details of plate girders.

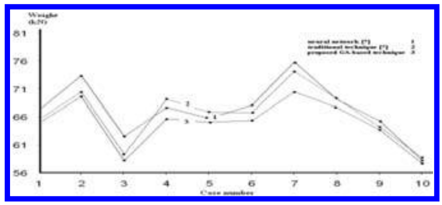

Figure 2. Comparison of the proposed GA-based optimization results with the results obtained from the literature for PG.

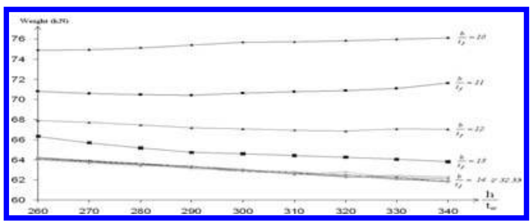

Figure 3. Minimum weight versus web slenderness ratio (h/t_w) and flange rigidity ratio (b/t_f) for PG3.

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The virtual work optimization method applied to structures: An investigation into cellular beams versus trusses

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ABSTRACT

The Virtual Work Optimization (VWO) method used in this paper is an automated and iterative method to minimize a given structure's mass. The method selects members' sections from a discrete database in such a way as to meet all strength and stiffness criteria. In the optimization process multiple load cases and deflection points are considered simultaneously. Within each iteration the efficiency of each section change is computed, and the section that produces the lowest mass increase per deflection decrease is chosen. How the efficiency of each member is compared is new in this paper. The process continues until all the deflection criteria, as stipulated by the user or by the building code, are met. The power of the method is demonstrated by optimizing two warehouses and comparing a truss to a cellular beam roof system.

Each structure has dead, live, wind and crane loads applied to it. Deflection limits are considered for the roof in each bay, and for the laced columns. Both structures must satisfy the South African steel code (SANS 10162:2005) using grade 300W steel.

The computed distribution of mass in the optimized warehouses with the truss and cellular beam roof system are shown in Figure 1 and 2. Members which are stiffened to limit deflections are shown in black. Members governed by strength requirements are shown in grey. The thickness of the lines is proportional to the mass per unit length of the member.

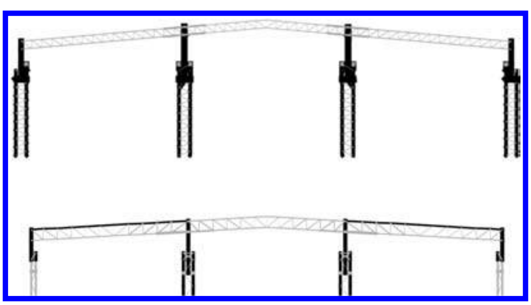

Figure 1. Distribution of mass with a truss roof system. Black members are deflection dependent, grey members are strength dependent.

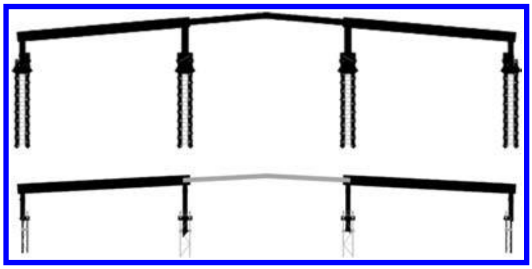

Figure 2. Distribution of mass with a cellular beam roof system.

Table 1. Comparison of structural masses for the warehouses.

Roof system	Warehouse 1 Mass (tons)	Warehouse 2 Mass (tons)
Truss	15.6	3.94
Cellular Beams	18.7	4.58
% Difference	19.9%	16.2%

The optimized masses of the structural configurations are summarized in Table 1. The computational time required to design the case studies is 76 seconds for Warehouse 1 and 20 seconds for Warehouse 2

As expected, the warehouses with cellular beams are heavier than those with trusses (by 19.9% and 16.2%). This material saving may be offset by fabrication, erection and painting costs. Costing information regarding material and manufacture, specific to each solution, should be included to determine which structural system is most economical.

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Ant colony system for the formation of sparse flexibility matrices: Tetrahedron elements

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ABSTRACT

The force method of structural analysis is appealing to engineers, since the properties of members of a structure most often depend on the member forces rather than joint displacements. This method of analysis requires the formation of a maximal set of independent self-equilibrating stress systems, known as a null basis (*SES*s).

The main problem in the application of the force method is the formation of a self- equilibrating stress matrix, B_1, corresponding to sparse flexibility matrix $G = B_1^t F_m B_1$, where F_m contains the flexibility matrices of the individual members of the structure in a block diagonal form.

For a general structure, the combinatorial force methods have not yet been properly defined, and further research is needed. On the other hand, the algebraic force methods need the large storage requirements and the higher number of operations.

Heuristic algorithms, such as ant colony algorithms, have found many applications in optimization problems in the last decade. The essence of these algorithms lies in the fact that their capability to converge to a good solution. In this paper, the ant colony system (*ACS*) which is a variation of the ant colony optimization (*ACO*) is applied to the formation of null bases of tetrahedron finite element models corresponding to highly sparse and banded flexibility matrices.

A mathematical modeling is developed for this optimization problem. In this model, for satisfying the independency of selected null vector, the vector is normalized based on one specific entry, which this entry shall be zero in other next null vector. This specific entry is called *generator* of the null vector. Since the different choice of *generator*s can alter the sparsity of the null basis matrix B_1, the ant colony system is applied to choose the *generator*s. To improve the run time of the proposed ant algorithm, first of all, the topological property of a *FEM* is transferred to the connectivity of a graph by the *interface graph*, and then the process of finding proper *generator*s are executed in two steps. In the first step the *generator*s are selected from multiple members of *interface graph*.

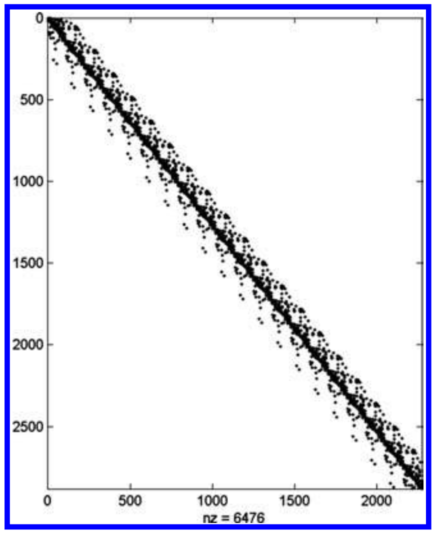

Figure 1. The sparse null basis matrix resulted from the proposed algorithm for a thick beam-type structure consisting of 480 tetrahedron elements.

After omitting the *generator*s in each multiple members, the resulted *interface graph* will be changed to a simple graph as a space truss. Then, in second step, the ant colony system is applied to choose the other required *SES*s from this simple graph such that leads to null basis with maximal sparsity.

The efficiency of the present method is illustrated through examples. The sparse null basis matrix resulted by the proposed algorithm, for a thick beam-type structure consists of 480 tetrahedron elements is shown in Figure 1.

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A search algorithm for optimizing the grouping of members

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ABSTRACT

In this paper, an algorithm for grouping discrete members in a structure with a given and fixed geometric topology is presented. The algorithm determines how members can be efficiently grouped together into a pre-specified number of groups. The method relies on first optimizing the structure assuming that each member can have an unique cross-section. While the solution produced at this step is the lightest, it is impractical and uneconomical to construct.

The Virtual Work Optimization (VWO) method (Walls and Elvin 2009) is employed in the optimization function. However, any method could be used. The VWO method is an automated, iterative method which selects discrete sections for structures with fixed geometries. Strength and deflection criteria are satisfied. The method is capable of handling a large number of design variables, which is vital when addressing ungrouped structures.

Next, grouping is performed based on the members' mass per unit length. All possible member group configurations are investigated and the resulting structure's mass is estimated. The number of permutations is given by the binomial coefficient. The computational cost is low since no structural analysis is performed. The grouping permutation that estimates the lightest structure is chosen.

The resulting structure with the grouped members is optimized one last time. The final structure satisfies all strength and stiffness conditions as well as having its members grouped.

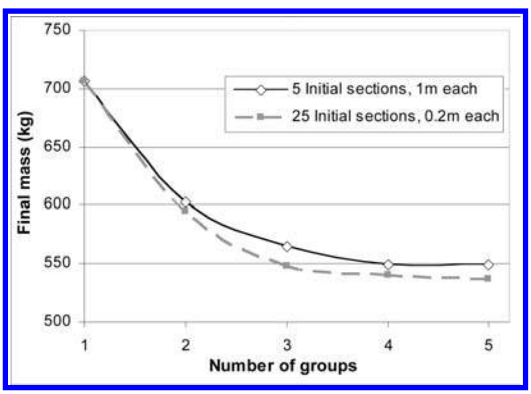

Figure 1. Comparison of masses for the cantilever grouped starting with 5 (solid line) and 25 (dashed line) sections.

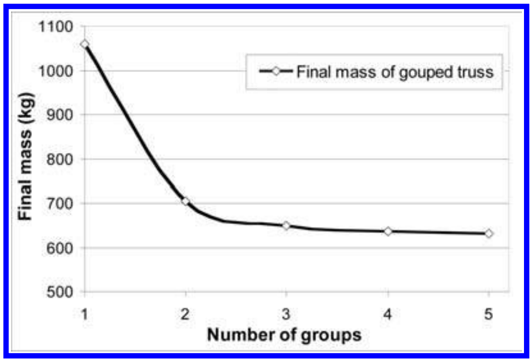

Figure 2. Masses of the truss when 1 to 5 groups are created.

Figure 3. Mass distribution in truss with 3 groups created.

Two case studies are investigated to study the effectiveness of the grouping method: (a) a stepped cantilever, and (b) a truss.

The stepped cantilever is discretized initially into 5 and 25 equal segments. Figure 1 shows how the mass of the cantilever varies with number of groups created. Figure 2 shows how the mass of a truss varies with the number of groups. Figure 3 shows the mass distribution in the truss with only 3 groups.

As the number of groups in the structure increases, the mass decreases *asymptotically*. Thus, increasing the number of groups beyond a specific point produces negligible savings in mass.

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"SampleRecycling": An adaptive DOE for response surfaces

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ABSTRACT

In many engineering problems solving responses of a complex model can take a lot of computational capacity and time. Since these are limited, in many cases it becomes impossible to accomplish parameter studies like optimization or reliability analysis. Response surface techniques are widely used to reduce this computational effort. The major challenge therefore is to find an representing, optimal but minimal support point set. Some adaptive DOE techniques where presented in the past. But most of these refer solely on one kind of metamodel and one single overlying algorithm (e.g. optimization). They show a lack of flexibility in their usage. Classical approaches of Robust Design Optimization (RDO) use a double loop to optimize a system with respect to uncertainties. Therefore every design in deterministic space needs a statistical analysis. Several approaches to reduce the resulting effort can be found in literature. Because these methods are strictly related to a hard defined selection of combined algorithms there is no flexibility in usage. Different measures of dispersion than reliability analysis are hardly addressed in these approaches. The same has to be said about the combination of different measures.

A new method will be presented that builds adaptive DOE independently from exclusive metamodel or investigation method. While running the investigation an algorithm decides whether it is necessary to solve the complex engineering model, or if its possible to recycle the result from an underlying response surface. Therefore the new proposed algorithm SampleRecycling decides whether it is possible to recycle the result from an underlying response surface. If the quality of the approximation is not as good as necessary the complex engineering model has to be solved. By adding all obtained results of real evaluations to the support point set, the approximation quality crows only in a region of interest. All further investigations can

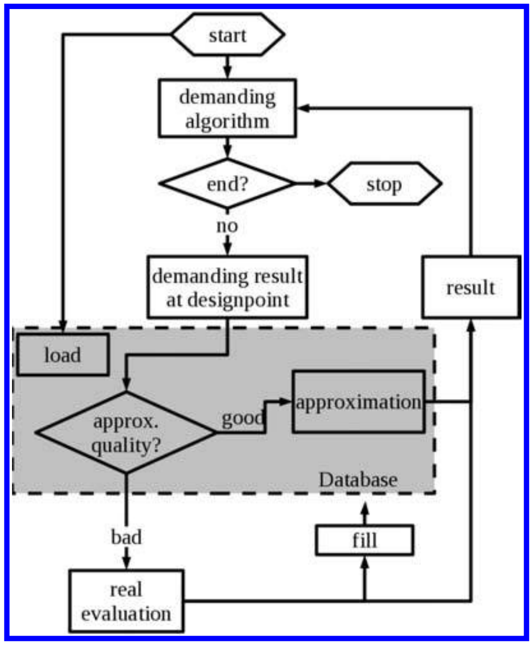

Figure 1. Principle of proposed method.

access to once made information. So no intelligence will be lost during a design process. Coupled methods like RDO or RBDO can be accomplished with high efficiency. Decision criteria, whether recycling is possible or not, will be discussed. Some examples will be presented to proof the benefit and usability of the proposed method in different fields of application. Through the described strategy it is also possible to improve the approximation quality iteratively. Coupled methods like robust design optimization can be accomplished with high efficiency.

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Application of numerical optimization in geotechnics

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ABSTRACT

Geotechnical problems are sophisticated tasks due to the complexity in soil behavior and non-linear soil-structure interaction. Depending on his/her possibilities and skills the geotechnical engineer normally undertakes a more or less innovative, creative and heuristic search for a defined objective under given boundary conditions. Wholistic approaches which involve numerical optimization do not exist until now. Abstract problem formulation is not state of the art. Typically, parameter studies are performed whereby the extend is restricted by the available computer capacity. Effective optimization algorithms in geotechnical calculation software are inexistent. Therefore, potential gains with regard to effectiveness are neglected.

In abstract form, the solved problem can be formulated as optimization tasks according Equation (1). In that $\boldsymbol{x}$ represents the parameters of the problem while f stands for the objective function(s). Depending on the dimension of f the optimization task is called mono- or multi-objective.

$$f(\boldsymbol{x}) = \min! \qquad (1)$$

In this paper, systematic approaches for the comprehensive use of optimization methods are presented for the processing of selected geotechnical problems. Applications in optimizing a design process, in the determination of soil properties and optimized finite element analyses are shown. Different optimization paradigms like the mono- and the multi-objective optimization are demonstrated and their use for more effectiveness is shown. The optimization is enforced by using Evolutionary Algorithms whose basic principles are explained. Also their applicability for geotechnical real world problems including nonlinearities, discontinuities and multi-modalities is shown. The routines are adapted to common problems and coupled with analysis procedures.

The introduction of new approaches for the use of numerical optimization in geotechnical engineering shows potentials is every day engineering work. The considered examples of inverse parameter identification, design optimization and topology optimization show the great extend of possible applications in geotechnical engineering. The obtained results provide a comprehensive data base to determine the optimal problem solution. To establish the shown methods in practice they will have to be used in real project design.

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17. Structural health monitoring, damage detection

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Use of optical fibre technology to measure structural performance

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ABSTRACT

A major concern in civil engineering structures such as bridges, offshore structures and dams is the assessment of a structure to quantify structural integrity, overall performance and load carrying capacity in the aftermath of damage or aging/deterioration. Similarly, determining the effect of construction processes such as tunnelling on nearby structures, may be a necessary requirement.

Optical fibre methods of sensing offer many advantages. Fibres can be installed in a structure during construction or introduced at a later time. They are relatively inexpensive, physically and electrically stable, and have a minimal effect within the measured environment. Structural monitoring using optical fibre technology may be undertaken to establish the long-term behaviour of structures, components and materials of construction. Condition monitoring may be used to establish the effectiveness of repairs and structural strengthening. Fibres may be used to determine the behaviour of concrete specimens in which small changes in porosity or permeability are important in understanding the mechanisms and effects of chemical attack, by chlorides, for example.

Fibre systems have been deployed in steel, concrete, and composite steel concrete and polymer composite structures. These distributed, multiplexed sensor systems provide a considerable amount of data, potentially over large distances.The grating in a Bragg sensor acts as a wavelength specific mirror, whilst allowing other light to pass in order to interrogate other gratings in a multiplexed system. If the grating is subjected to a strain, then its optical characteristics change and can be detected. Strain can be induced by structural loading, thermal, moisture and chemical effects.

A sensor system for temperature monitoring has been developed. It allows the measurement of temperature and would provide a mechanism to compensate changes in the strain measurements within a structure caused by changes in temperature. The method uses small temperature-sensitive elements of doped fluorescent fibre. This technique is sensitive over the whole range of temperatures to be measured in a structure up to +350°C. The durability of optical fibre sensors exposed to high temperature has been investigated and it has been possible to measure the durability and fatigue properties of concrete specimens.

The measurement of moisture absorption in concrete may be achieved using a humidity sensor. The sensor Bragg grating was coated with a moisture sensitive polymer. Strain is induced in the grating through the swelling of the polymer coating.

In order to prove the reliability of a sensor system in the field and to measure the performance of a bridge under controlled loading conditions, the five span continuous composite box Mjosundet Bridge in Norway has been monitored using a 100 sensor system. The individual field trials consisted of a number of static and dynamic tests where the structure was subjected to loads from parked or moving vehicles, respectively. The static tests consisted of three loading states where the structure was subjected to maximum sagging and hogging moments and maximum shear forces up to the design load levels. A series of discrete load and no-load events allowed data to be recorded continuously for the test and easily processed afterwards.

Determining the effects of construction processes is a further to use sensors for monitoring. In a recent example, sensors were installed in an existing tunnel in Tokyo to determine the changes in cross section and tunnel displacements during the advance of the Joban New Line tunnel boring machine Measurements proved to be reliable not only for actual monitoring, but for the verification of appropriate geotechnical models for the prediction of soil stress and pore water pressures.

The examples and applications, which have been discussed, have demonstrated the potential of optical fibre monitoring to improve the knowledge and understanding of material and structural behaviour in both the short and long term.

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Optical fiber grating sensors

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ABSTRACT

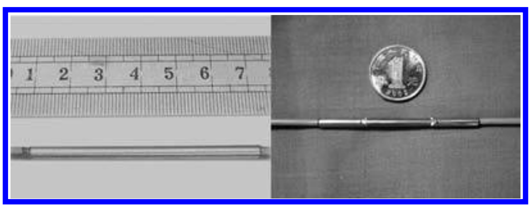
The past decade has seen rapid advances in research, development, and application of optical fiber Bragg grating technology. Some advantages, like small in size, inert and corrosion resistant, immune to electromagnetic interference (EMI), made fiber Bragg grating sensors promising candidates for structural health monitoring (Baldwin 2001 & Kersey 1997).

This paper introduces some principles of strain transferring of FBG sensors, which is the basis of FBG sensors application. The strain transfer rate along an optical fiber, and also the result by Ansari (Ansari & Yuan 1998) for comparison is shown in Figure 1, which demonstrates that the strain sensed by the fiber at the midpoint does not equals to the strain in the host material in this instance.

Novel FBG temperature sensor and strain sensor are designed, shown in Figure 1. In the design of the FBG temperature sensor, a novel way was put forward to reduce the adverse effect of strain availablity and improve the thermal sensitivity. On the other hand, the strain characteristics of sensor were studied by using universal material testing machine, the results of which showed that the linearity of the sensor's wavelength to bare FBG wavelength was well.

Afterwards, FBG strain sensors were applied in strain monitoring of oil production offshore platform No.CB271 (Fig. 2), located in Yellow Sea. An FBG temperature sensor was also placed close to those strain sensors for temperature compensation. A strain course induced by an impaction of ship with hundreds tons weight was recorded by FBG sensors. The power spectrum density function of one FBG strain sensor, excited by boat collision, was also analyzed.

The paper presents a review of resent research and development activities in structural health monitoring of civil structures using Fiber Bragg Grating (FBG). One of the exciting fields wherein fiber optic sensors and health monitoring is expected to play a significant role is smart structures and intelligent systems. In smart structure applications, composite materials, fiber optic sensing systems, piezoelectric actuators and microprocessor based control schemes seem to offer the best advantages in the future.



(a) FBG temperature sensor (b) FBG strain sensor

Figure 1. Novel FBG sensors.

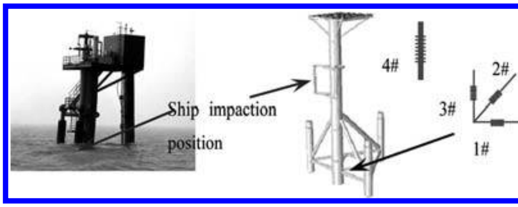


Figure 2. The platform picture and sensors position.

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A multi-channel wireless transmission system for structural monitoring

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ABSTRACT

A permanent Structural Health Monitoring system is conceived to facilitate the maintenance and to reduce the relevant cost while improving the structural safety. From a technological point of view, the main difficulties encountered during in-field applications are related to the cabling system, due to the high cost of the cables, their difficulty of installation and exposure to mechanical damage. For these reasons, the adoption of wireless connections is regarded as a fundamental aspect for the spread of permanent monitoring solutions and much effort has been focus on this area (Lynch and Loh, 2006; Casciati et al., 2005; Faravelli and Chen, 2009).

In this paper, a newly designed wireless transmission system is proposed. It mainly consists of a wireless sensing unit, a base station unit and a computer (Figures 1 and 2). Its main features include the capability of real-time and multi-channel data transmission, a high compatibility to different types of sensors, a highly efficient power-supply and low-cost. Instead of adopting the most commonly used commercial wireless modems, the wireless communication is pursued by implementing an optimized and customized solution based on a recent System on Chip wireless transceiver. The Frequency Division Multiplexing method is applied in order to ensure the real-time feature of the multi-channel data transmission. A simple and practical point-to-point topology is pursued. The Cyclic Redundancy Check and the retries-acknowledgement mechanism are employed to ensure a reliable wireless communication. The usage of switching regulators which feature low quiescent current, a highly efficient power conversion, an adjustable output voltage, and a high output power make this platform suitable for both low-power and non low-power structural monitoring applications involving different types of sensors. In order to validate the prototyped platform, a laboratory test is performed. The measurements of the acceleration response of a reduced scale, three-storey structure mounted on a shaking table are acquired from both a wired data acquisition system and the developed wireless platform. The data comparison enables to validate the efficacy of the real-time, multi-channel wireless transmission.

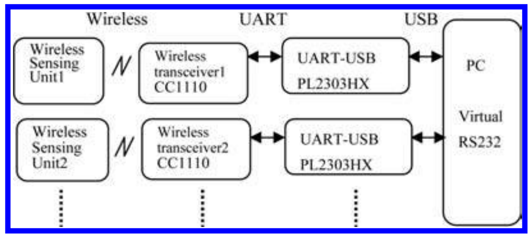

Figure 1. Block diagram of the wireless transmission system.

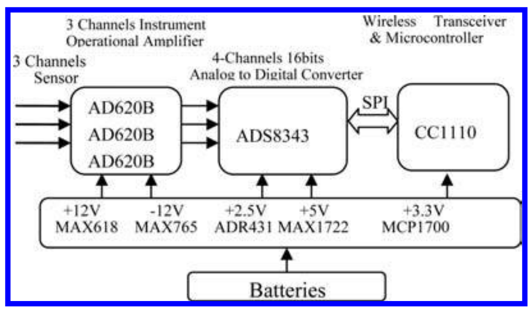

Figure 2. Block diagram of the wireless sensing unit.

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Application of FBG sensors in monitoring curing process of carbon fiber composite

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ABSTRACT

Carbon fibre composite is a material widely applied in aerospace engineering. One of the popular composite structure is advanced grid stiffened (AGS) structure (Steven et al. 2002). It is essential to determine a proper cure cycle for AGS co-curing process by monitoring the variation of temperature field and strain field (Guo et al. 2004).

Fibre Bragg grating (FBG) sensors have been established as a major leading technology over other competing fibre-optic sensor technologies in structural health monitoring (Li et al. 2004). At present, there are numerous successful cases of embedding FBG sensors in a composite structure for curing process monitoring or continuous structural health checking (Kang et al. 2002).

In this paper, basic principle and merit of FBG are firstly introduced. A novelty strain sensor and temperature sensor on FBG technology is also presented, as shown in Figure 1.

Figure 1. A novel FBG sensor.

The theoretical and experimental analyses of FBG sensor are implemented. Using universal material testing machine and water-bath heater to calibrate strain sensitivity coefficient and temperature sensitivity coefficient, respectively. The linearity of strain to wavelength is excellent, which means the capsulation technique does not decrease the measurement accuracy.

The manufacturing process is detailed and the cure cycle is introduced. 4 FBG strain sensors (S1∼S4) and 4 FBG temperature sensors (T1∼T4) are embedded along the rib or skin inside the composite material, as shown in Figure 2.

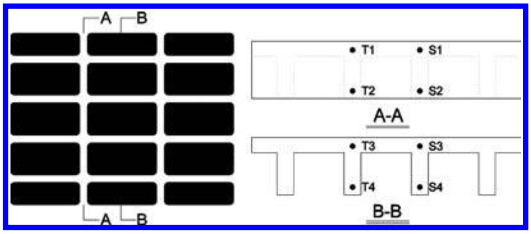

Figure 2. Location of the embedded FBG sensors.

The temperature and shrink strain of carbon fibre composite have been successfully tested. The temperature difference between skin and ribs top is discussed. When analyzing the strain variation, wavelength drift caused by temperature variation must be eliminated to get the accurate strain variation. The strain variation of FBG sensors embedded into the skin and ribs are also different at different time stages.

The experiment resultes indicate that application of FBG sensor is a suitable method in monitoring the composite in curing process.

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Study on the effect of different construction materials on GPS carrier phase signals

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ABSTRACT

The GPS satellite positioning technology has been successfully applied in structural health monitoring domain of large-scale civil engineering, due to its high positioning accuracy, all-weather work, simplified operation and so on, it. However, the GPS satellite signals are susceptible to many kinds of disturbance errors, such as ionospheric and tropospheric errors, orbital errors, clock errors and multipath effect and so on. Although the public errors can be eliminated through different signals due to the short distance between GPS reference station and rover station, the multipath effect is also difficult to be removed. Therefore, it has become the biggest restrictive factor of high accuracy positioning (Yi et al. 2009).

In order to study the multipath signal rule caused by the different construction materials, a set of GPS multipath signal simitation and monitoring system was designed. The system was established on the roof of 1st test building in Dalian University of Technology, where can be considered as a multipath-free site since it has a very good observing environment (see Fig.1).

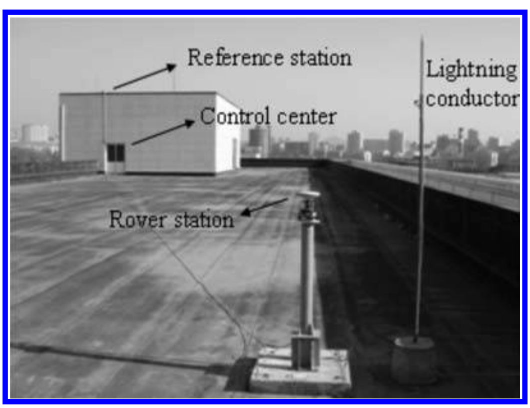

Figure 1. Photo of monitoring system.

The experiment includes three parts: 1) Do not lay aside any reflecting objects nearby the rover receiver antenna, and one continuous day-long static observation is carried on; 2) In order to obtain the receiver's system noise, do not lay the reflected objects around the receiver antenna in 50 m, and carry on continuously three-day-long real-time dynamic data gathering; 3) For the purpose of studying the multipath error's regularity when different construction materials (wood board, toughened glass, aluminum plate) were in the different distance, and reflected objects were placed at the distance of 1 m, 4 m and 7 m away from rover receiver antenna. Continuously two-day-long real-time dynamic observation was carried on at each distance, and each material was 6 days, the total of three materials (wood board, toughened glass and aluminum plate) was 18 days.

By comparison, it's shown that the multipath errors caused by reflecting objects at different distances have certain regularity, some meaningful conclusions are obtained:

(1) The amplitude of the multipath signals caused by the non-metal materials increased with the material dielectric constant.
(2) The standard deviation of the multipath signals caused by the non-metal materials increased with the material dielectric constant g.
(3) The amplitude and the standard deviation of the multipath signals caused by the metal material is bigger than those by the non-metal material.
(4) The relevance between multipath signals caused by the non-metal material is bigger than that between the non-metal and metal materials.

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Laser measurements of deformations of structure components

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ABSTRACT

Contemporary geodetic equipment allows for a static projection of the geometric state of the examined objects or structures. By their very nature, the results are filtered, i.e. error analysis, averaging, data processing by the finite-element method etc. The truth about the state of the object, however, can be captured by the remote sensing measurement in real time. Such projections, applying dual-pulse laser scanning, and the result being an interference image of an object, allow for the assessment of its conditions, the dynamics of "life" or the destruction, and eventually the object state at a given moment of time. All that is feasible with a low-cost upgrade of the currently manufactured, GPS receiver equipped tachometers.

The principle of measurement applied in holographic camera system or raman lidar system should be applied in the currently manufactured laser scanners. Then, against the background of the object being scanned, by generation of two pulses (or pulse packets) with a duration time forced by the accuracy of the measurement (nanoseconds), one can read the geometric and thermal (also mechanical) "life" of its surface. Modernization of existing scanners with respect to dual-pulse broadcasting channel and receiver sensor matrix is possible. That would involved the ability to receive the signal over the period comprising duration of the original pulse, the time interval required for the registration and duration of the second pulse, which creates an interferogram with the original one. It is due to the inertia of the sensor matrixes, internally and externally referenced to the terrain (a site spot) and its projection.

The research discussed in the article is a basis for implementation of a new version of laser scanners, which, apart from the geometry of the object will be capable of measuring its thermal, mechanical (i.e. deformation, destruction) and chemical (i.e. chemical composition) state.

The authors believe that the laser scanner implementation expansion is going to be towards the topography of buildings, land and environmental pollution (raman scanner).

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LMI based fault tolerant control of building structures

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ABSTRACT

Fault tolerant schemes in engineering systems provide early warnings of faulty sensors, actuators, or system components. A component fault refers to a change in the operating behavior of a component such that the new behavior differs significantly from what is defined as normal behavior for that component. Common examples of such faults include bias errors in the output of a sensing device and loss of function for an actuating device. Health monitoring systems are needed to provide early notification of faults before they lead to catastrophic failure so that remedial actions can be carried out to retain the system stability and performance. Consequently, the fault detection and isolation (FDI) and fault tolerant control (FTC) problems have received considerable attention in control systems literature. Most applications of fault tolerant control schemes are based or partially based on hardware redundancy. The use of this kind of hardware redundancy is common, but it carries with it the problem of additional equipment, maintenance cost, and space requirements. In some engineering environments, extra space for redundant sensors and actuators are hard to come by. Recently developed analytical redundancy techniques use residue signals to monitor the health of systems. The term residue is defined as a signal that is zero when the system functions properly and nonzero when some abnormal behavior is observed. This residual signal indicates faulty information in the system and can be used not only for fault detection, but also for fault identification. Further more, it also provides basic information for the fault tolerant control purpose of use. It is a natural way to cope with the FTC problem by employing fault diagnosis information online.

Under foreground of the fault detection and isolation skill using in civil engineering, this paper presents the design of an H∞ fault detection and isolation (FDI) filter. State observation methods design FDI system. The linear matrix inequality (LMI) methods work out the FDI filter based on H∞ robust control theory. Simulation results for a three-story building are used to demonstrate the necessity of fault detection and the effectiveness of FDI. The controller minimizes the control objective selected in the presence of disturbances and faults. The residuals obtained from the filter through simulation clearly identified the fault signals. The simulation results of the proposed FTC controller confirm its effectiveness for vibration suppression of the faulty control system.

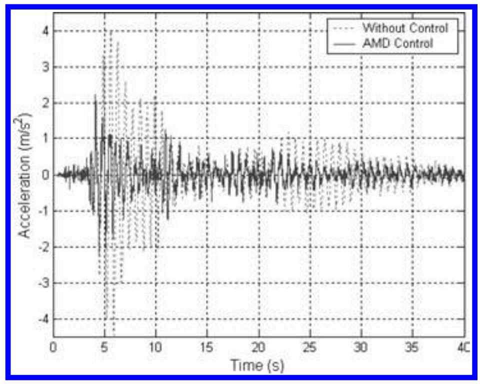

Figure 1. Acceleration History of the Second Story.

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Urban disaster management system development focusing on predicting performance

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ABSTRACT

From the 60th's year last century, GIS started to be applied for spatial data management, analysis and transmission[1]. With the fast development and maturation itself as well as the commercialization of software develop platform, GIS which has been used in many fields is taken as a modern integrated knowledge dealing with spatial data. There has been more than 10 years for GIS to be taken into disaster-defend and China has gotten lots of improvement in this field. According to the advantages of GIS for displaying directly, presenting and connecting of diversiform geo-spatial and property database, this paper set up the system database aim at assisting the quick decision making after or during disaster. Furthermore, lifeline engineering which belongs to the urban infrastructure is the "driving power" of reaction of urban disaster emergency so that this paper also added function modules for disaster prediction into urban disaster management system to try to make sure the vulnerable places before disaster. The system transfers the control object Super map Objects through VB integrating develop circumstance to carry out managing workspace, editing data resource and objects, map-manage, map-query and map-Statute as well as network analysis etc. The two sub-systems both conclude modules of reliability calculating and connecting calculating of the network units which could be used for quick estimation on disaster-defend capability. They also program in VB using the existing prediction theories of network and take GIS as the main interface to calculate and display. The 3-dimension emulational scenes display directly the environment to describe the key points of disaster-defend and disaster-mitigation clearly.

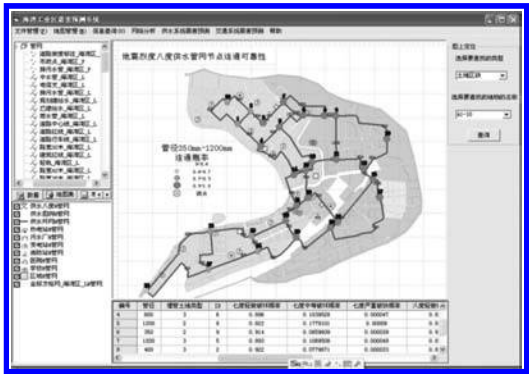

Figure 1. Special map of pipelines network connectivity under earthquake.

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Structural health monitoring (SHM): The state of the art

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ABSTRACT

The term 'Structural Health Monitoring (SHM)' refers to continuous monitoring of a structure in order to track the changes in its dynamic characteristics and detect damage. In Civil/Structural Engineering, the majority of SHM applications are directed towards studying the response and damage from natural hazards, such as earthquakes and strong winds. The monitoring typically involves measuring continuously the vibrations of the structure by acceleration sensors. Some recent applications have also included GPS sensors, which provide superior accuracy for measuring displacements. Although a significant number of structures are now installed with SHM systems, the utilization of data for practical applications are still lacking. Some of the new findings resulting from SHM include the significant influence of environment on structural frequencies and damping, strong dependency of damping on amplitude and frequency, exponential decay modal damping values with increasing building height, and the prevalence of 3D modes and non-proportional damping. A critical need in SHM is the simple tools and techniques for real-time data analysis and interpretation. Since data come continuously, the analysis cannot be done in batch mode; it should be done in real-time. This paper summarizes the latest developments in SHM, with emphasis on data analysis and damage detection. The topics discussed include real-time analysis techniques, noise reduction in ambient vibration data, utilization of wave propagation approach as an alternative to spectral analysis, inadequacy of modal parameters for damage detection, applications of *Seismic Interferometry* for data analysis, and identification and damage detection for historical structures.

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Monitoring aspects in a rotating tower

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ABSTRACT

The response of structural systems under extreme hazard events, such as earthquakes and typhoons, could be improved by adding control devices. However, these events occur randomly in time and space, and many years could pass before the structure under study undergoes one of them. Hence, the control systems should preserve their performance capabilities regardless of how many years have passed since their last activation. This achievement shall require a periodic maintenance of the systems to ensure their serving ability at any time, but it implies an undesired additional cost for the building owners,. Therefore, it was suggested to use the control system not only for infrequent events, but also for routine events that are related to the daily life of a building (Kobori, 2003). In this case, the building itself has a maintenance schedule time scale which also includes the control system.

A fertile field to which this concept can be applied is represented by the class of very tall buildings (Pelli et al., 1997). In particular, the rotating tower by Arch. Fisher (Fisher, 2008) represents a new innovative building based on Dynamic Architecture, and it is planned to be constructed in Dubai, in the near future. This project is characterized by three fundamental aspects (Casciati-AlSaleh, 2008; Casciati et al. 2009):

1. each floor will have the ability to rotate independently each of the other; this rotation will infer to the tower a continuously changing shape;
2. energy will be produced from the sun, by solar panels and from the wind, by wind-turbine;
3. the construction process of the tower will adopt a new technology based on prefabricated modules which are assembled on site, apart from the central concrete core which is built on site using traditional techniques; 90% of the building will be produced in an industrial park, transported to the site and finally connected to the concrete core.

In this paper the task of monitoring such a structural system is approach. For this purpose, GPS receivers (Casciati-Fuggini, 2009) and wireless sensor networks are regarded as practicable solutions for the SHM of the rotating tower.

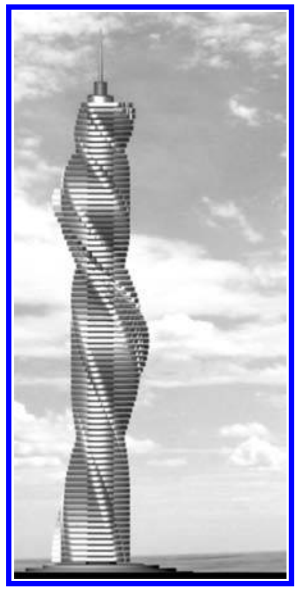

Figure 1. Example of rotating tower design (Fisher 2008).

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Probabilistic monitoring aspects and optimisation of a jointless bridge

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ABSTRACT

In recent years the pressure for efficient design has been significantly increased especially in the light of the economic crisis. This situation led to a promotion of new design principles, materials and building technologies with the goal of reducing overall life cycle costs mainly by limiting necessary maintenance actions. Jointless bridges, which are characterized by integral abutments and their lack of bearings and expansion joints, represent a construction type that is less susceptible to degeneration and thus is currently being pushed by designers in Austria. However, as there is little experience regarding this design principle especially for longer structures a research project was set up aiming at determining acting loads and structural response at a probabilistic basis. This provides the data basis for the verification of current design principles and codes as well as the optimization of structural details.

In course of the construction of a new Danube crossing several foreshore bridges had to be designed and construction, one of which as Marktwasser Bridge S33.24. Marktwasser Bridge is a joint less three span reinforced concrete plate structure situated in Lower Austria, Austria. The span lengths are 19.50 m 28.05 m and 19.50 m. One especially critical detail is the transition area between bridge deck and earth dam because changes in elongation of the jointless structure lead to significant strains in the pavement and dilation area.

In Austria there is currently little experience dealing with jointless frame bridges of this total length. Consequently the structures operator commissioned an integrative monitoring system with the goal to capture structural response and its change over time especially considering the effects of time dependent processes such as construction stages, creep and shrinkage and temperature loads on the performance of (a) the slab detail and (b) the bridge itself. In Figure 1 the finally installed monitoring system for the bridge deck is laid out.

Additionally four different sensor systems were installed behind the abutment including 2 extensometers, 20 electrical strain gages, 10 fibre-optic strain

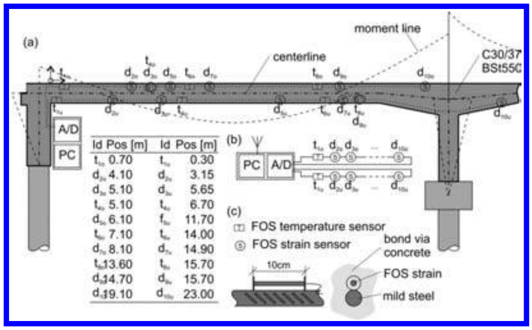

Figure 1. Sensor placement in bridge deck.

sensors and 3 vertical inclinometers. The extensometers aim at detecting the movements of the abutment and the earth dam behind it. All 30 strains sensor are divided between 4 layers of additional geotextile reinforcement and should be able to accurately capture the strain field in the earth body above and well behind the slab. The three inclinometers provide a time discrete absolute verification of the strain and displacement measurements.

The two previously mentioned sensor networks provide a large amount of data which is to be checked, properly prepared and finally evaluated. Considering the amount of data recorded by a permanent monitoring system it becomes clear that efficient and meaningful data analysis and interpretation necessitates an appropriate methodology. Time series analysis in combination with extreme value statistics allows data evaluation on a probabilistic basis. From the huge amount of data firstly extreme values (maximum and minimum) as well as mean value and standard deviation are determined, e.g. for 10-minute spans. Subsequently the properties of the extreme value distributions for longer time spans, e.g. 7 to 14 days, are derived which leads to a quantification of the time dependent processes.

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Managing the structural health of concrete dams

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ABSTRACT

Many infrastructures are approaching or exceeding their design life. As a result of economic issues, these civil engineering structures are still in use despite of ageing and the associated damage build up. Dams are not an exception. Nevertheless, safety is one of the priorities of dam engineering, namely due to the high potential risk associated with these engineering works. Many dams, even though showing deterioration signs, are expected to continue in operation significantly beyond the lifetime estimated in the design.

Therefore, the ability to monitor the health of these structures is becoming increasingly important.

Dam safety management is a complex task and requires knowledge on a wide range of subjects and the enrolment of experts from many fields. The activity is supported by a comprehensive collection of data derived from a set of instruments strategically placed thought out the structure in order to monitor the structure health. It is not difficult to guess that data can grow tremendously through the structure lifetime. Moreover, safety standards have been consistently improving and information and communication technologies are themselves still facing great developments and challenges. All these issues call for the development of intelligent tools to efficiently support the management of the dam structural health.

The present work refers to the development of a powerful and innovative IT-based framework to support dam engineers in the complex task of dam safety management – the gestBarragens system (Fig. 1). The framework has an open and flexible architecture able to integrate current and future research in the field of structural health monitoring of a large portfolio of dams. Despite the development focus in concrete dams, the developed framework can be easily extended to other type of dam structures.

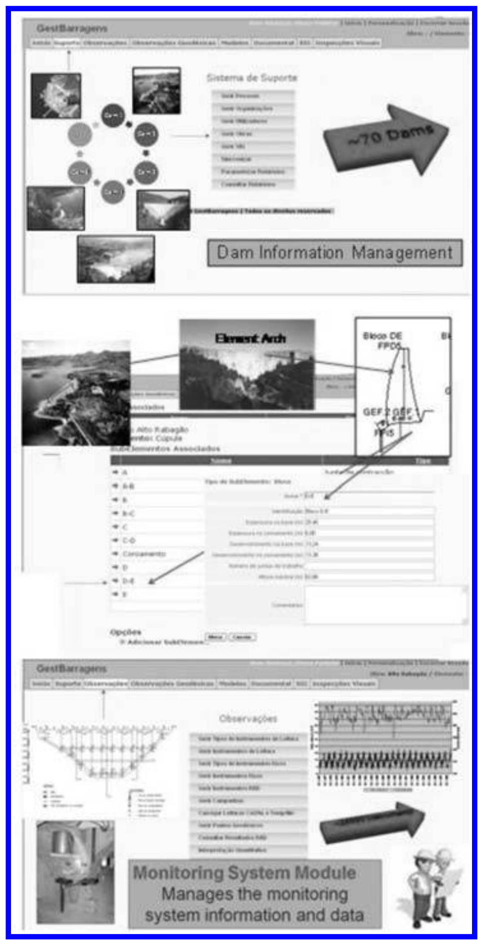

Figure 1. gestBarragens system.

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Infrared thermography and ultrasound techniques for detecting FRP – concrete adhesion problems

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ABSTRACT

Externally bonded Fiber Reinforced Composites (FRP) are increasingly being used for strengthening, repairing and retrofitting of existing structures due to their excellent mechanical and physical properties. However, the structural efficiency of the reinforcement techniques is dependent on a proper application. Indeed, a perfect adhesion between FRP materials and concrete substrates must be obtained so to assure the efficacy of FRP reinforcements. For this reason an important role is played by the quality control of FRP application by using Non Destructive Techniques (NDT).

The present paper reports the preliminary results of an experimental campaign aimed at verifying the reliability of the Infrared Thermography (IRT) and the Ultrasound Technique (UT) in detecting FRP – concrete adhesion problems and defects. For this purpose, twelve small concrete beams have been prepared. The beams have been strengthened using different reinforcements types and technologies, e.g. FRP sheets and FRP laminates. Some artificial adhesion defects have been settled in the interface between the reinforcement and the concrete substrate, e.g. Teflon, Plastic Button, Cut of some FRP fibers, Collect Glue. Moreover, the beams have been sorted into groups having different concrete compressive strength and different structural anomalies, e.g. cavity, area filled with repairing mortar, in order to simulate some real structural cases. Active IRT and UT have been carried out on each specimen before and after the reinforcement application, in order to verify concrete properties and to evaluate NDT reliability in detecting and classifying defects and adhesion problems. For the IRT test the samples have been oven-heated for one hour at a temperature of 45°C, then the exposed surfaces have been monitored for 30 minutes in the cooling stage with a thermal infrared camera. Thermograms have been acquired every minute (Fig. 1).

Achieved results led to the conclusion that both IRT and UT seems to be promising tools for detecting FRP – concrete adhesion problems and for highlighting the presence of some structural anomalies. However, further research – e.g. increasing specimens number, analyzing different cases of anomalies and defects, using more UT features beside velocity, modifying IRT heat source – is needed for investigating the reliability of both NDT. Moreover, performing some tests on full scale reinforced elements might be useful to confirm the effectiveness of the methods.

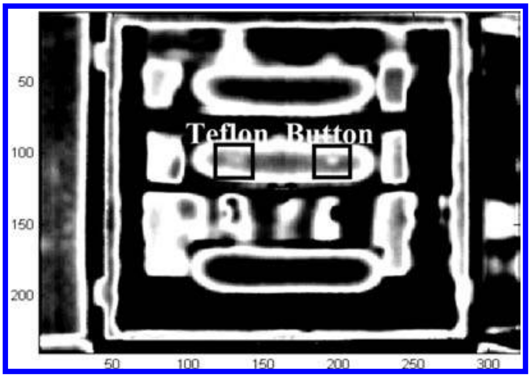

Figure 1. Example of IRT thermogram.

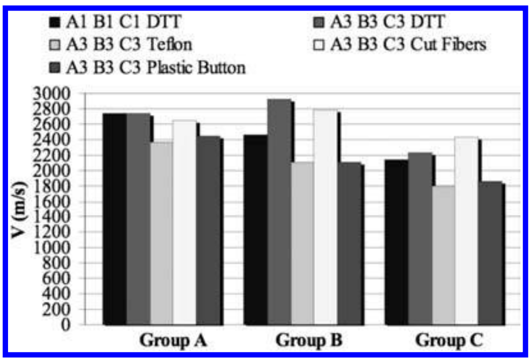

Figure 2. Example of UT velocity distribuition.

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Compressive damage detection in concrete by a nonlinear ultrasonic testing procedure

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ABSTRACT

The presence of cracks or damage phenomena in concrete elements is strictly correlated to the occurrence of nonlinear effects in their elastic response to an ultrasonic excitation. The Fourier analysis is often inadequate to pinpoint such effects, since the signal-to-noise ratio of higher order harmonics is usually very low.

In order to overcome this drawback, we suggest an alternative procedure, denoted as Scaling Subtraction Method (SSM), to extract nonlinearity indicators from the recorded ultrasonic signals, based on the dependence of the response of the system on the excitation amplitude. The SSM is first described and then used to analyze the evolution of nonlinearity due to progressive crack formation induced by quasi-static compressive loads in concrete core-drilled cylinders.

Our approach allows to distinguish the initial micro-damage formation from the semi-stable progression and the pre-rupture phases.

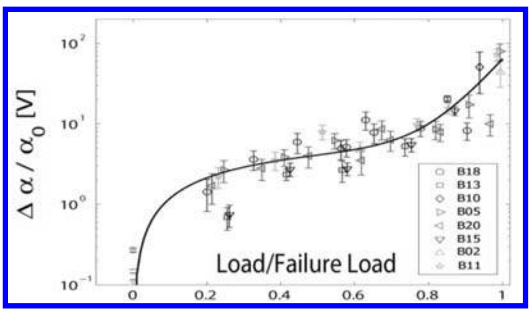

Figure 1. Evolution of the relative SSM indicator as a function of normalized compressive load.

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18. Structural failures, damage assessment, repair, strengthening, retrofitting

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Progressive debonding in RC beams shear-strengthened with FRP side strips

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ABSTRACT

Almost all reinforced concrete (RC) beams shear-strengthened with externally bonded FRP strips on their sides fail in shear due to debonding of the FRP strips. The bond behaviour between the FRP strips and the concrete substrate therefore plays a crucial role in the failure process of these beams. Despite extensive research over the past decade, there is still a lack of rigorous modelling and understanding of how the debonding of the FRP strips in such a beam propagates and how the debonding process affects its shear behaviour.

This paper presents an analytical study on the progressive debonding of FRP strips in such strengthened beams. The complete debonding process is modelled and the contribution of the FRP to the shear capacity of the beam is quantified using a closed-form solution, based on an analytical solution for the full-range behaviour of FRP-to-concrete bonded joints (Chen et al. 2007) (Fig. 1). More importantly, the solution is a full-range solution for the development of FRP shear contribution with shear crack width, which can thus be directly employed in investigating the effect of shear interaction in RC beams shear-strengthened with FRP (Chen 2010). The closed-formed solution also provides valuable insight into the debonding failure process of RC beams shear-strengthened with FRP side strips (Fig. 2). The validity of the solution is demonstrated by comparing its predictions with finite element (FE) predictions (Fig. 3).

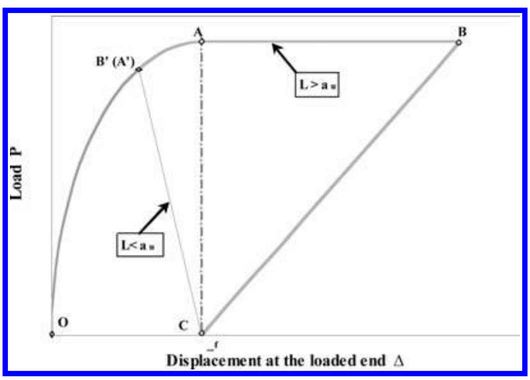

Figure 1. Full-range behaviour of an FRP-to-concrete bonded joint.

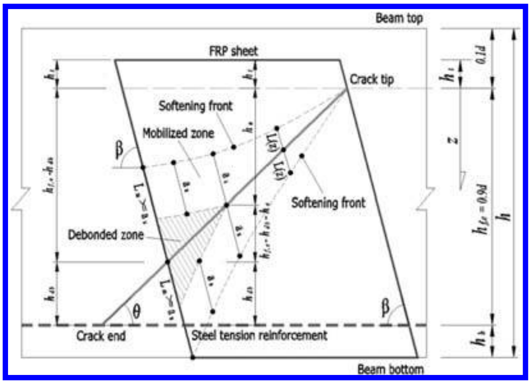

Figure 2. Debonding process of an FRP side sheet ($L_m \geq a_u$).

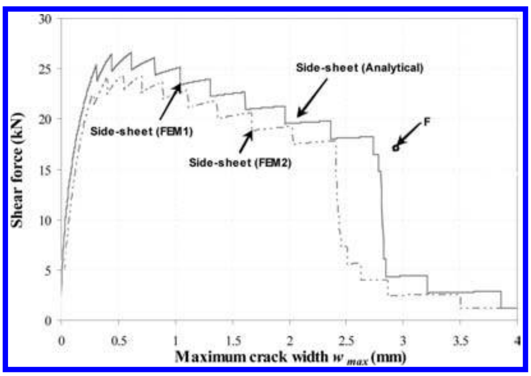

Figure 3. Analytical solution versus FE predictions.

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Structural failures and forensics

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ABSTRACT

Forensic engineering may be defined as the investigation and reporting of the causes of engineering failures in clear, concise and understandable language suitable for a court of law. In deliberations relating to structural failures, it is the duty of the retained forensic structural engineer to serve as consultant and often as an expert witness and in so doing to strive to reach the truth as explained in simple language understandable by all concerned. The root cause of structural failures varies and can be related to inadequacies of design, materials, construction, and unforeseen or unexpected loads or material deterioration with time. Various structural failures are discussed and the causes are explained along with suggestions for future preventative measures.

Engineers have been investigating and analyzing failures of structures for many years but it was not until a few years ago that the term Forensic Engineering had become more widely applied to these services. Computers are often used to analyze structures and to back-trace causes of failure for structures that had been built long before computers had been commonly available to structural engineers. Although computers may be used for this purpose, design and practical experience of the engineer is much more important in pin-pointing and defining causes of failure. Once determined, it is often the task of the investigating engineer to impart the acquired knowledge to uninitiated lawyers, judges, and lay people in ways that are easily understood. Forensic engineering sometimes involves expert testimony before a court of law or other judicial forum when required. While it is the duty of the engineer to strive to find the truth of the matter, it is also the engineer's duty to declare that he or she is unqualified to perform a forensic investigation if it lies outside of his or her particular area of expertise. Furthermore, the engineer should be ready to fully cooperate with other experts who may specialize in associated areas.

Figure 1. Bridge Pier Cap Failure.

Steel Formwork Failure

Figure 1 shows a finished pier cap on Pier 8 which was cast in place before the pier cap pour for Pier 7 was attempted. At full cast, the formwork failed and split apart at a seam directly above the through-bolt support on the pier as shown in Figure 5. The inspector responsible for approving the pour was inexperienced and failed to notice that the heavy-duty moment bolts used to hold this seam together had been left out. It was later discovered that, while the seam-stitching quick-bolts were in place, the moment bolts at the top rail of the form had been omited

Unfortunately two workers standing on the formwork drowned when their clothing became entangled in the reinforcing steel which dragged them to the bottom of the river.

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Computational aspects associated with the integrative lifetime assessment of engineering structures

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ABSTRACT

The determination and maintenance of a certain safety level at serviceability and at bearing capacity is an expensive, problematic and questionable task during the design, construction and maintenance phases of a structure. Quality of planning and design, the appropriate choice of building materials, the quality of production and construction have significant influence on the safety level. As changes take place in material properties over time, caused on the one hand by natural and man-made environmental impacts (e.g. de-icing salts) and on the other hand by changes in the applied actions (e.g. change of the user's behaviour), the safety level is subjected to a temporal development.

The determination of the safety level during the phase of maintenance nowadays in most countries is based on (visual) inspection programs. Based on the subjective rating of the evident situation these strategies have some weaknesses because the results are related only indirectly with the internal mechanical loading capacity of the structure.

In addition the classical approach usually does not contain any measurements (e.g. of chloride concentration) and therefore the objective database is often very poor.

Therefore these inspection-programs are not fully optimized and lead to some uncertainties in the assessment of the remaining lifetime and in the assignment of required strengthening works.

In this paper the software shell "SARA studio" is presented. This tool combines a series of specialized stand-alone software tools. A finite-element program based on nonlinear fracture mechanics (ATENA) can be used to assess identified damage on a deterministic level or obtain stochastic material parameters from a probabilistic software (FREET) in order to take uncertainties into account. Furthermore the deterministic input parameters can be determined by neural network analysis (DLLNNET) based on available monitoring data. Another possibility is to calculate the bearing capacity from a structure subjected to degradation by combining ATENA with the degradation software FREETD and chloride ingress simulation by CATES.

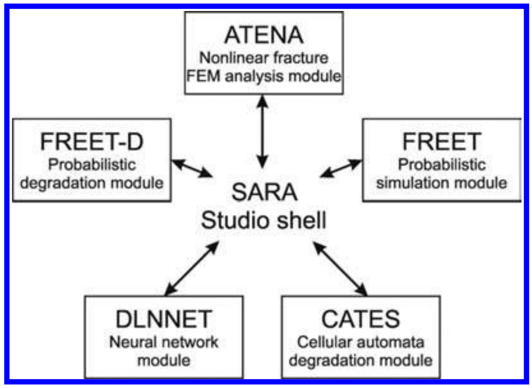

Figure 1. SARA software shell.

The presented tools can generally improve the decision basis for maintenance planning compared to traditional visual inspection only without endangering structural safety. This is especially valueable in case an engineering structure shows damage or the end of initially predicted service lifetime is approached. Furthermore the intervals between the inspections can be planned according to the actual condition of the structure.

The proposed methodology is applied to a case study subjected to severe chloride induced deterioration showing the potential for cost savings in maintenance while ensuring a sufficiently high safety level (Strauss et al. 2010).

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Displacement-based seismic assessment procedure for multi-span reinforced concrete bridges

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ABSTRACT

Some bridges, especially the viaducts on the highways, are of great importance after an earthquake for allowing the civil protection interventions and first aid organizations. In Italy, as well as in other seismic countries, many of these "strategic" bridges have been built without antiseismic criteria. Therefore, they are needed to be assessed against seismic risk. In this extent, the development of a seismic assessment procedure which gives reliable results and, at the same time, is sufficiently simple to be applied on a large population of bridges in a short time is very useful.

In this work, a displacement-based seismic assessment procedure, satisfying these requirements, is presented particularly for reinforced concrete multi-span bridges in transverse response. The assessment procedure, based on the principles of Direct Displacement–Based Design (Priestley et al. 2007) and hence named Direct Displacement–Based Assessment (DDBA), requires the following steps: 1) acquisition of the structural information; 2) derivation of the bridge displacement shape; 3) idealization of the Multi Degree of Freedom (MDOF) structure as an equivalent Single Degree of Freedom (SDOF) system; 4) determination of the risk in terms of displacement. In particular, the risk for the given seismic input is checked with a pass or fail method: the displacement capacity of the structure $\Delta_{\text{cap-el}}$ is compared with the demand displacement $\Delta_{\text{dem-el}}$, figured out using the elastic demand displacement spectrum.

The methodology was applied to geometrically regular and irregular hypothetical bridge configurations as well as to existing bridges. In particular, in this paper DDBA will be applied to two Turkish existing bridges: the Sadabad viaduct, a connector highway bridge in conjunction of Okmeydanı and Hasdal on Kınalı-Sakarya motorway route, and the Gedik Ahmet Pasa (GAP) viaduct, a connector bridge on TEM highway in Istanbul. Both the bridges were assessed for Significant Damage Limit State.

The results of the procedure in terms of equivalent SDOF system properties and the seismic risk defined by Capacity/Demand ratio are reported in Table 1.

Table 1. DDBA equivalent SDOF properties and Capacity/Demand ratios of the bridges.

Viaduct	Δ_{cap} m	ξ_{sys} –	$\Delta_{\text{cap-el}}$ m	K_e kN/m	T_e s	C/D –
Sadabad	0.296	5.0%	0.296	84,130	3.29	0.42
GPA	0.533	9.4%	0.680	29,512	2.60	1.11

Table 2. IDA results.

	IDA C/D	Standard deviation	Coefficient of variation
Sadabad	0.43	0.08	18.31%
GPA	1.09	0.13	11.51%

DDBA was verified using Incremental Dynamic Analysis (IDA). Five spectrum compatible accelerograms was incrementally scaled till one of the pier tops first reached the ultimate displacement capacity: at this time, the corresponding scale factor was recorded as Capacity/Demand ratio. The mean value of Capacity/Demand ratios for the five considered accelerograms was accepted as global Capacity/Demand ratio for the structure being assessed. The dispersion in the IDA results was investigated by means of standard deviation σ and coefficient of variation c_v. The results are summarized in Table 2. The errors in DDBA for Sadabad and GPA viaducts results are equal to 5.4% and −7.4%, respectively. The results show a good accuracy of the DDBA method to estimate the risk when compared with IDA results.

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Mean damage ratios for reinforced concrete and masonry buildings constructed in Turkey

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ABSTRACT

Earthquake damage is estimated based on the observed damage statistics compiled from past earthquakes for low to medium rise buildings. Considering the various aleatoric and epistemic uncertainties involved both in seismic demand and capacity, assessment and estimation of earthquake damage should be carried out based on probabilistic and statistical techniques. Here the damage probability matrix (DPM), which tabulates the discrete probability distribution of damage that will be experienced by structures designed according to some particular set of requirements, during earthquakes of various intensities, is used for this purpose. However, because of space limitation it was not possible to display the resulting DPM's, instead the outputs of the study are presented in terms of mean damage ratios (MDR) conditional on seismic zone-intensity level pairs. The degree of damage is quantified in terms of the damage ratio (DR), which is defined as the ratio of the cost of repairing the earthquake damage to the replacement cost of the building.

Here, DPM's are obtained by using only the empirical results. Observational data bases are enriched and improved with the damage distribution statistics of recent earthquakes that occurred in Turkey (belonging to approximately 120 000 buildings). The discrimination of the damage distributions of reinforced concrete and masonry buildings was poor in the available damage data bases. However, by using the building census reports of the State Statistics Institute of Turkey, the percentage of masonry buildings in different cities for which the earthquake damage data were compiled are obtained. This ratio is termed as the masonry ratio and is used while separating the MDR's of reinforced concrete and masonry buildings. From the available domestic and international studies, the relative vulnerability coefficient, which is defined as the ratio of the MDR of masonry buildings to the MDR of reinforced concrete buildings, is computed as 1.83. Using this value of the relative vulnerability coefficient, together with the masonry ratios compiled, empirical MDR's of reinforced concrete and masonry buildings are separated. The resulting empirical mean damage ratios for reinforced concrete and masonry buildings are presented in Tables 1a, b and c, for the seismic zones I, II and III of the current seismic zoning map of Turkey (Code 1997), respectively.

Table 1a. Empirical mean damage ratios of reinforced concrete and masonry buildings for seismic Zone I.

Intensity (EMS-98)	VI	VII	VIII	IX	X
MDR (%) (Reinforced concrete)	7.9	8.6	11.8	17.9	19.6
MDR (%) (Masonry)	14.4	15.8	21.6	32.8	35.8

Table 1b. Empirical mean damage ratios of reinforced concrete and masonry buildings for seismic Zone II.

Intensity (EMS-98)	VI	VII	VIII	IX
MDR (%) (Reinforced concrete)	3.2	7.6	7.2	15.7
MDR (%) (Masonry)	5.9	13.9	13.2	28.7

Table 1c. Empirical mean damage ratios of reinforced concrete and masonry buildings for seismic Zone III.

Intensity (EMS-98)	V	VI	VII
MDR (%) (Reinforced concrete)	0.6	2.4	11.4
MDR (%) (Masonry)	1.1	4.4	20.9

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Causes of structural damage to low-rise buildings in Gauteng (South Africa)

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ABSTRACT

The subject of bad building construction is not new. It is widely reported in daily press, TV or in specialist literature, especially if fatal accidents happened. In this paper specific types of damages are presented, namely caused by mistakes in the designs or approval of poor quality building plans and finally due to construction of buildings by ignorant builders. This damages affect not only buildings but also finances of unfortunate house owners.

Most serious damages in Gauteng usually happen in buildings situated on dolomitic areas, where formation of sinkholes is quite common. The buildings situated on heaving clays are also subjected to damages, if proper designs procedures were not followed. Recently however, the new type of damages can be observed. Namely damages caused by construction of buildings without proper knowledge and against all Codes of Practice, Standards or Regulations.

This paper presents random examples of damages to buildings in the result of mistakes in approved plans or by construction of buildings with absolute ignorance. They are not supported by any scientific or statistical investigations. They shows only deterioration of quality of building construction in some places in Gauteng.

The reason of terrible damages in the first example was probably lack of special foundations or positioning of the building in the wrong place. This type of design mistakes hopefully decreased now, due to strict approval of foundation plans by NHBRC for buildings situated on dolomitic areas. The second case deals with unfortunate remedial measures to a building situated on heaving clays. The company specialized in building repairs cut "expansion joints" in walls of the building with disastrous results. After the rains the clays started to swell and the "expansion joints" in walls opened about 10 cm wide. This building is beyond repair now, with great financial loses to the owner. The next three examples deal with construction of RC beams by ignorant contractors. One can understand that due to difficult economic situation many individuals claim that they can build houses. But what about financial loses incurred by unsuspecting home owners appointing such Home Builders? In the first case the RC ring beams required a lot of remedial measures before final approval. In the second presented case the beams supporting slab were cast in inadequate shuttering and with poor quality of concrete. Again the owner had to pay for extra remedial work and delays in construction. In the third case the builder filled the structural beam with paper and rubble to save on concrete (?) The next example shows one of the worst cases of building construction. It is unfortunately not uncommon. Again some contractor started the works and disappeared or went bankrupt. The spiral stairs constructed with very thin flight slab paradoxically serve well and are used by workers carrying building materials to the first floor. The next example shows leaking flat light-weight roofs constructed as per approved designs.

Finally the current ways of certification of buildings before occupation is discussed. Another issue, beyond the scope of this paper, is the question how structural engineer can protect himself against eventual claims resulting from poor quality of works done by Home Builder. This Builder, who actually appointed him as a Competent Person in accordance with NHBRC requirements?

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Parametric study on seismic performance of an existing RC building

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ABSTRACT

The recent earthquakes revealed the fact that existing buildings in many earthquake prone countries have various deficiencies regarding their seismic performance; therefore many of them experienced heavy damage and total collapse causing irreparable life and economic losses. Seismic authorities working on loss assessment studies also emphasize the expected seismic risk around Istanbul in the near future, indicating the existing large number of seismic vulnerable building stocks in the region. For that reason, some immediate measures should be taken into account to minimize the future probable losses. There are several vulnerability assessment procedures including code-based detailed analysis methods as well as rapid assessment techniques which are based on quick inspection and experience to identify the safety level of buildings.

The current earthquake code of Turkey (TEC'07) issued in 2007 includes a new chapter about the rules and constraints on seismic assessments of existing buildings as well as retrofitting methods which can be considered as quite modern and compatible one as compared with the contemporary codes. On the other hand the most of those code-based 3D static pushover and dynamic approaches are of different level of complexity and time consuming work, thus it is sometime more practical to give precedence to apply a reliable and a practical preliminary assessment approach for the identification of the collapse vulnerable buildings to minimize the life loss and total collapse. Considering also the economical constraints, various preliminary assessment techniques have been developed for this purpose.

The present study outlines the main steps of the code-based linear and nonlinear seismic assessment procedure for RC buildings and P25 Preliminary Assessment Technique that recently suggested method for screening collapse vulnerable structures. Then, a case study has been carried out on an existing RC school building which has many structural deficiencies such as poor concrete quality, various structural irregularities, inadequate detailing, corrosion, etc.

The three-story RC framed building has been investigated for the required immediate occupancy (IO) and life safety performance (LS) levels. First, damage levels of the structural components and the seismic performance level of the structure has been determined using linear and non-linear approaches. Computations performed revealed the fact that the school building is far from satisfying the required seismic performance levels. As a second step, P25 Preliminary Assessment Method is applied to the same structure and the results are compared. The structural and material deficiencies of the building are also discussed. Finally, in order to examine the effect of each inadequacy on the whole seismic behavior of the building, a parametric study has been performed by decreasing or increasing the level of different deficiencies and analyzing the system again. The change of the maximum base shear values for each case has then been compared, considering their capacity curves.

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Structural behavior of deteriorated mining steel structures

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ABSTRACT

The behavior and load bearing capacity of new structures can be analytically predicted based on a number of assumptions, such as: the structural members are of a known geometry, section and material properties have been accurately determined and the loading conditions are known. With the passage of time however, structures undergo deterioration and the level of degradation depends, among other things, on the environment and loading systems to which the structure is subjected. Mining structures are often exposed to unpredictable loading conditions leading to member deformations as well as to aggressive environments leading to the corrosion f steel members. Members of structures on mines are also often deliberately altered by technical and production personnel in order to accommodate operational processes. These alterations could be cutting out parts of member cross-section in order to accommodate ventilation, water or other service pipes. Some alterations could be as drastic as the removal of entire members (e.g. beams, columns, bracing members) in order to accommodate equipment. The deformations due to impact loading have the effect negating the original assumptions made that the structural members have a known geometrical form, while corrosion damage often reduces the cross-sectional dimensions of the member and thus weakens the structure.

Figure 1. Member Cut-out to accommodate Handrail.

Figure 2. Model of Cut-out.

In order therefore to assess the effect of cut-outs on the strength of members an investigation using finite element models was conducted. The models simulate this loss of cross-section and its effect on the buckling load of the members.

The buckling loads were determined and then compared with those of intact (members without cut-outs). Channels and angles of different cross-sections were investigated and their comparative buckling loads determined.

It is evident that the load bearing capacity of such members is severely compromised by cut-outs. Depending on the position of the cut-out and it's magnitude the reduction in capacity could be as high as 60%. The next phase of this investigation would be to quantify the effects of cut-outs as well as their interaction with other deterioration mechanisms in influencing the behavior, not just of individual members but of entire structural systems.

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Coordinating and directing whole building collapse investigations

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ABSTRACT

When faced with coordinating or directing a whole building collapse investigation, one must focus not only on the collecting and sampling of evidence but the development of protocols to address the interests of all stakeholders as well as how to quickly and efficiently map and collect evidence. The presentation utilizes an actual six story building collapse investigation to illustrate the process of coordinating and directing a whole building collapse investigation.

This paper begins with a discussion on how the Authority having jurisdiction addresses the initial response and the impact that response can have on the ensuing forensic investigation. Developing an understanding of this response and its activities can be critical to interpreting or understanding the disposition of the onsite evidence. This paper then goes on to discuss and provide examples on how to establish a clear and concise investigative protocol that addresses the interest of all stakeholders. Stakeholders are defined as those individuals or parties that have an interest in the outcome of the investigation. Identification of these individuals along with their respective interests is critical to avoiding future pitfalls.

This paper discusses the need to identify the basic constraints governing the forensic investigation. By fully identifying and prioritizing the constraints one is better able to establish priorities and set realistic timelines. This paper discusses and illustrates the necessity of focusing collection and sampling efforts based on reverse engineering of the collapse. By reverse engineering the collapse, the investigator is better able to focus the efforts of the investigation. The presentation of the six story building collapse is used to demonstrate how an understanding of materials and a reading of component dispositions better enables the investigator to focus their efforts.

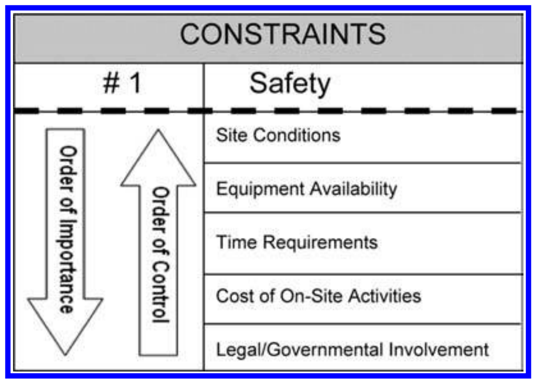

Figure 1. Constraints Table.

This paper concludes with a brief discussion on the importance of mapping, collecting and removing debris in an efficient and timely fashion. Given that time is a luxury the investigator generally does not have, the need for a well developed protocol is essential. Along with the need for efficiency is the need for clear and effective communication with all of the stakeholders.

Utilizing clearly established protocols have proven invaluable in the wake of construction collapses and other catastrophic events when one is faced with conducting a whole building collapse investigation.

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Collapse of a multi-storey building at the final stages of construction

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ABSTRACT

This paper is based on investigations into possible causes of failure of a three-storey office building, which was under construction. Half of the nearly completed office block was destroyed when the top two storeys collapsed on to the bottom storey. The building collapsed suddenly on 16 October 2008, tragically killing two construction workers, trapping another and injuring 14 others.

The collapse took place in the West Rand area of Johannesburg, Roodeport. The basic structural form of the building consisted of a ground floor surface bed, two suspended floors supported by a rectangular column grid and a timber truss ceiling system. The trusses were fixed to a series of masonry walls that in some instances extended to a height of 5.9 m. Vertical structure was consistent throughout the building height. Post collapse measurements of rubble and photographic evidence show columns to be 400 mm × 350 mm. Columns were arranged on a grid that on plan varied between 4.6 m to 7.4 m horizontally and 5.8 m to 6.1 m vertically. Horizontal structure was made up of flat slabs without drops or beams.

Samples of both vertical and horizontal structure were acquired for laboratory testing. A total of nine cores were sampled and tested for compressive strength. The average compressive strength achieved was just over 26 MPa.

The as-built structure was analyzed using a combination of beam elements (forming a 3-dimensional rigid frame) and parabolic (8-noded) finite elements. Flexural, shear and punching shear requirements were determined for the flat slabs. Column and foundation designs were also done in accordance with local building codes. Foundations were found to be extremeley undersized exerting a bearing pressures exceeding allowable limits for the in-situ ground conditions. Columns were checked for the most critical combination of bi-axial bending and axial load. A moment-axial interaction graph was resolved based on the actual column geometric and reinforcing properties.

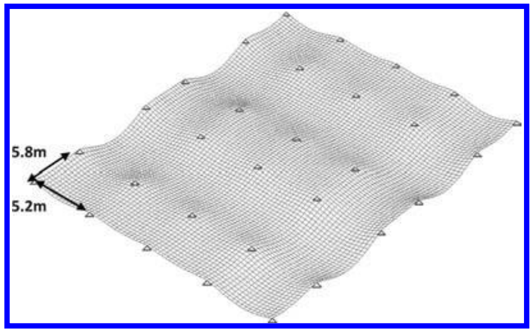

Figure 1. Finite element mesh of suspended slab used for collapse sequencing.

The results from the analysis were then superimposed on the chart to determine adequacy. Columns were found to be suitably sized and sufficiently reinforced.

In order to obtain a more accurate depiction of the stress flow and moment distribution within the flat slab, and to determine a collapse sequence, the finite element method was adopted. A convergence study was undertaken using 8-noded quadratic elements and suitable convergence was found to occur within an internal panel consisting of a grid of 24 × 16 elements. The mesh used for the study and the deflected shape under serviceability loading conditions is shown in the figure 1.

Punching shear was found to be the trigger mechanism for collapse. The behavior of the columns punching through the slab was modeling by removing the appropriate supports, reanalyzing, checking the subsequent moment distribution and shear force. This process was then repeated until a complete collapse sequence could be developed. This sequence was determined to have been initiated by four internal columns punching through the slab, a subsequent redistribution of moments and reaction forces, two other internal columns punching through and then finally flexural collapse of the slab.

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Investigation of partial collapse of a cylindrical roof

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ABSTRACT

Cylindrical roofs constructed by continues cold forming, mostly use of skin structures, must be precisely analyzed not only for symmetric buckling but also for anti-symmetric buckling as well during displacement and stress calculations. The overall buckling mode is symmetric in the circumferential direction when the shell is shallow but becomes asymmetrical when the shell becomes deeper. On the otherhand, due to anti-symmetric loading (mostly due to snow or sand accumulation), anti-symmetric buckling failure occurs.

Djoudi & Bahai (2003) proposed a cylindrical strain based shallow shell finite element which is developed for linear and geometrically non-linear analysis of cylindrical shells. Numerical and experimental investigation related to the buckling and post-buckling behavior of thin-walled cylindrical steel shells with varying thickness subjected to uniform external pressure was conducted by Aghajari et al. (2006). For the post-buckling analysis of these structures, material and geometric nonlinear collapse analysis were carried out.

In this study, the reasons of partial collapse of roof structure having cylindrical shells after an extreme snowfall were investigated. Three dimensional finite element model of the shell roof was prepared by using general finite element analysis software COSMOS/M (2002). By using prepared computer model, material and geometric nonlinear analysis were carried out.

Geometrical nonlinear analysis of the thin walled shell roof, in which large deformations are involved, was carried out by using finite element model. For finite element model, the material nonlinearity was modelled as elasto-plastic material law with von Mises yield criterion, associated flow-rule and small linear hardening ($E_t = 0.001E$). The end connections of cold-formed profiles were simulated by using four-node shell elements to represent the 1 mm thick plates that are connected to roof elements with the help of 4 M16 metric bolts.

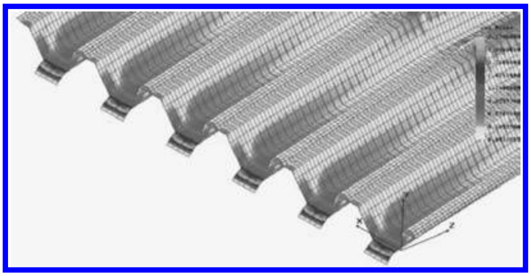

Figure 1. Stresses at end points of the roof.

National Building Code of Canada (1985) was employed to investigate the partial collapse of the roof. Based on this standard, snow load profiles were determined and were used as imposed loads during finite element analyses. Resulting stresses are given below in Figure 1.

Results have shown that although the rest of the structure behaved as required, end connections were subject to high stress concentrations well above the elastic limits and failure of these critical locations have resulted in collapse of the roof structure.

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Shear strengthening of RC beams with PBO-FRCM composites

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ABSTRACT

Common design approaches for the shear strengthening of reinforced concrete beams with fiber reinforced polymers (FRP) materials involve the concrete/fibers bond properties. In this paper the provisions of the CNR (2004) are applied to a fiber reinforced cementitious matrix (FRCM) composite considering the bond-slip relation proposed in a previous study of the authors. The results are compared with those obtained using a CFRP (Carbon FRP) material. The considered FRCM strengthening material is made of PBO fibers. For this material the bond-slip relation presented in (D'Ambrisi & Focacci 2010) is considered. For the CFRP material the bond-slip relation suggested by CNR (2004) is considered. With the same geometrical arrangement of FRP and FRCM strengthening materials applied on the lateral surfaces of a beam, the effectiveness of the shear strengthening depends on the force per unit width that the strengthening material can hold:

$$F_{fv} = \phi(L_{eff}) \cdot t_f E_f \varepsilon_{dbu} = \phi(L_{eff}) \cdot F_{dbu} \qquad (1)$$

where t_f is the fibers thickness, E_f is the Young modulus, ε_{dbu} is the fibers strain at debonding for the fibers with an anchorage length greater than the effective anchorage length L_{eff} and $\phi(L_{eff})$ is an effectiveness factor taking into account that not all fibers have an anchorage length greater than L_{eff}. The parameters involved in (1) are compared in Figs. 1-3 for the cases of the considered materials. The comparison shows that: (a) the larger effective transfer length of the FRCM material implies a larger reduction of the force (1) with respect to the case of the FRP material; (b) the debonding force per unit width of the considered materials is similar when the same fibers thickness is considered; (c) the shear strengthening effectiveness of the FRP material is larger than that of the FRCM material when the same number of strengthening layers is considered; this is due to the larger thickness of the fibers in the FRP sheet with respect to that of the fibers in the PBO net.

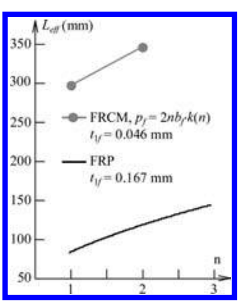

Figure 1. Effective anchorage length vs. number of layers.

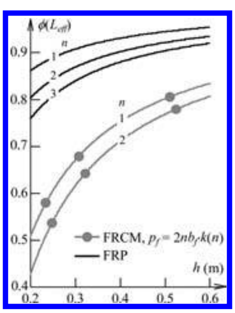

Figure 2. Effectiveness factor vs. *h*.

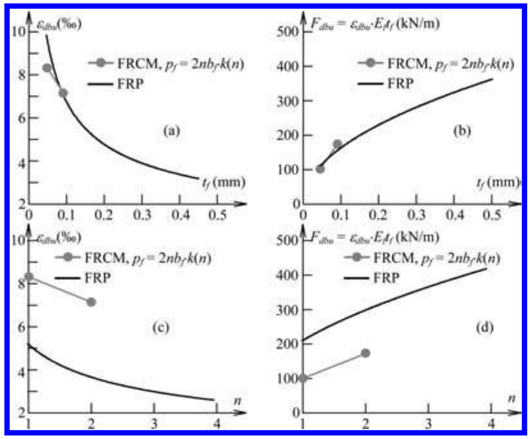

Figure 3. (a) Maximum debonding strain and (b) debonding force per unit width vs. the thickness; (c) maximum debonding strain and (d) debonding force per unit width vs. the number of layers.

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Rehabilitation or new structures: Potentials of concrete-to-concrete bond

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ABSTRACT

Modern codes such as Eurocode 2 or the corresponding german national code DIN 1045-1 generally differentiate between checks for the serviceability limit state and the ultimate limit state. However, this principle is not followed where subsequently added (shear) joints are concerned as the problem of concrete-to-concrete bonds is usually considered too complex. Current design practice assumes a simplified situation where the added component acts as a monolithic one and that displacements of the bond surfaces to one another are either small or do not occur at all. The loads shared between adhesion, friction and reinforcement are thus simply added to one another without considering any joint displacement and thus differ from the real behaviour of the load-bearing factors.

The component resistance is reduced to three factors, the roughness of the surface, the tensile strength of the added concrete derived from the concrete compressive strength, and finally the position and the amount of tie bars. The applied shear force is calculated as a portion of the internal shear force being transferred to the added concrete. Interactions between the shear force and other corresponding internal forces do not need to be considered.

Due to the different ages of the concrete layers but also because of differing material properties (such as compressive strength, elastic modulus, degree of shrinkage) constraints will act in the joint that may disrupt the bond or even destroy it completely.

The test series presented here show that with appropriate treatment of existing concrete surface and carefully selected concrete mixtures a nearly monolithic bond can be achieved. The current normative approach used to determine the adhesive resistance of subsequently added concrete elements may be easily applied but is not necessarily sensible from a mechanical or economic point of view. Two individual parameters (c, f_{ctd} according to DIN 1045-1) are not sufficient to safely describe the adhesion resistance's potential.

The test series also demonstrate the immense influence of imposed stresses on the concrete-to-concrete bond. Should the adhesion's role in overall resistance be increased due to more concrete knowledge of the bond mechanism then constraints must also be considered as part of the loading.

Concrete-to-concrete bonds depend on multiple factors and the interaction of these factors. The most important parameters in qualitative order would be roughness, moisture gradient, and interaction of superplasticising and bonding agents.

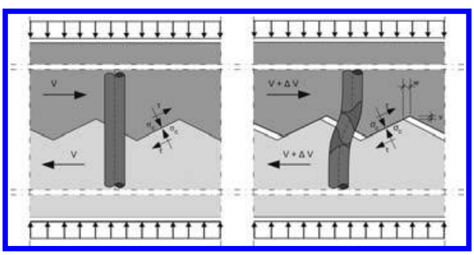

Figure 1. Uncracked, cracked joint.

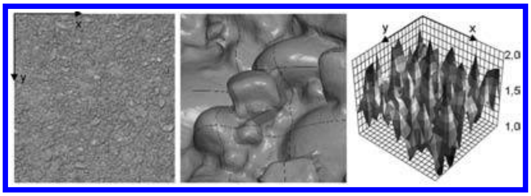

Figure 2. Digitized, high pressure water jetted surface (on the left), enlargement with grid lines [a = 6.25 mm] (center), Wenzel's quotients for individual grid fields (on the right).

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Local bond-slip relations of PBO-FRCM composites for strengthening RC members

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ABSTRACT

In recent years innovative composites made of cementitious matrix reinforced by continuous fibers (FRCM) have been introduced for strengthening reinforced concrete members. The effectiveness of an externally bonded strengthening material strongly depends on the bond between fibers and concrete and on the mechanical properties of the concrete cover. Recent experimental analyses show that the bond between FRCM materials and concrete have different peculiarities with respect to the bond between FRP materials and concrete. In this paper the results of bond tests (Figure 1) on a PBO-FRCM material bonded to concrete specimens are presented. One net layer and two net layers and bond length from 50 to 250 mm have been considered in the experimental analysis.

The considered strengthening material is made of a poliparafenilenbenzobisoxazole (PBO) fiber unbalanced net. The nominal thickness in the two fiber directions is 0.046 mm and 0.011 mm, respectively. The obtained experimental results allow the calibration of local bond-slip relation for the considered strengthening material. Figure 2 shows the bond-slip relation $\tau(s)$ calibrated starting from the experimental results with different calibration criteria. Figure 2 also shows the comparison between the analytical and the experimental results in terms of $P_{db}(L)$, that is the force in the fibers at debonding P_{db} vs. the bond length L. The analytical results have been determined adopting the calibrated $\tau(s)$ relation.

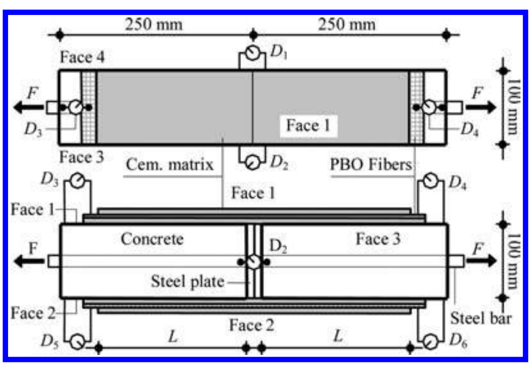

Figure 1. Test setup.

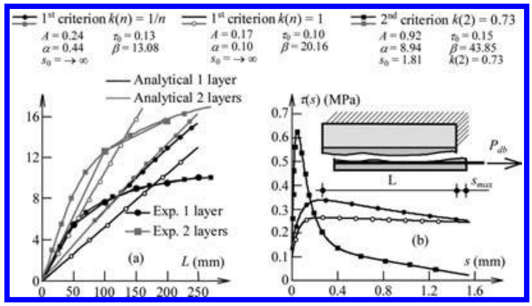

Figure 2. (a) Comparison between the $P_{db}(L)$ curves obtained with the bond-slip relations (6) and the experimental results; bond-slip relations (6) for the different calibration criteria.

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Hybrid advisory system for the process of industrial flooring repairs

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ABSTRACT

Industrial flooring is an essential element of a building from which the existence of the building depends. Depending on the character of a building it comes under various influences which cause damage. The mechanism of destruction is very often similar. The occurrence of "minor" damage in the form of scratches and cracks creates further possibility of degradation via the penetration of various aggressive agents connected with technological processes. The advanced degradation may even cause the exclusion of the building. Damage is also caused by various factors, in most cases by: point overloads, breaks of the edges of expansion joint cuts, corrosion, maintenance neglect. The repair of industrial flooring consists of many stages. Part of these stages is unique – the analysis of symptoms and cause of damage. A repeatable stage consists in searching for material and technological solution under known conditions and limitation of application. At this stage it is worth using support tools in order to relieve a decision-maker.

Hybrid advisory system is one of possible tools supporting technical services in the selection of material and technological solution for the repair. The structure of the advisory system is similar to a typical expert system and consists of such modules as: knowledge base, database, inference engine, dialogue interface. The knowledge base includes expert knowledge – during the development of the system, at the stage of knowledge acquisition, the role of an expert was assigned to an engineer with long experience in industrial flooring (construction and repairs). Gathered knowledge includes a set of premises and conclusions concerning the possibilities and conditions of applying certain material and technological solutions. The knowledge has been formalized in the form of fuzzy rules. The fuzziness of the knowledge base was the effect of natural language included in the knowledge base and as a result the majority of variable values (premises) were represented in a qualitative description. The knowledge base included information related to available material and technological solutions such as: costs, prescriptions, limitations and technological regimes as well as repair algorithms. Inference engine is an essential element of the system. The role of this module was played by artificial neural networks. The uniqueness of the advisory system is based on its hybridity which consists in combining various intelligent tools within one system, being also intelligent. The combination of these tools within one system may be categorized as so called "immersing" and the model of the system as a loosely coupled one.

Modus operandi of the advisory system is simple and user-friendly. After performing the analysis concerning symptoms and possible cause of damage, it is possible to start using the system. Based on a dialogue with a user the system obtains information concerning the type/depth of damage, the size of mechanical influences, the size of thermal influences, aggressive environment, the time and character of the influences, the time necessary to complete the repair, the temperature of application. Based on the knowledge base and available data, the system, in the process of reasoning, is searching for possible repair solutions. From the solutions generated by the system the user chooses the final one which complies with his/her preferences (e.g. costs). The operation of the system was checked based on several real cases of damage of industrial flooring. It was found out that the system generates proper answers.

This system does not resolve the issue of repair in a comprehensive way. However, it serves as the basis for further development of support tools. The use of these tools should constantly increase. The intended system will be responsible for supporting a decision-maker in the so-called partial decisions concerning the management of industrial surfaces – technical management and economic management.

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Experimental research on prestressed concrete main beams of road bridge strengthened by CFRP strips under static loads at different repair stages

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ABSTRACT

This paper presents the results of the research on a five-span prestressed concrete road bridge under static load performed at different stages of its repair. The examined bridge was built from 1957 to 1959. Particular effective spans are 18.00 m. All bridge spans become reinforced by the strips made of carbon fibers CFRP (3 for every beam) glued on bottom flanges of main beams, and external flat steel stirrups of 5×50 mm section, with axial base by every 0.35 m, as well as overlaid a new concrete deck layer of variable thickness 0.12–0.185 m (Figure 1).

The main aim of the conducted bridge repair was to increase its load capacity to 300 kN (class C) according to the Polish Bridge Loads Standard (PN-85/S-10030) and to determine the behavior of the chosen spans structures subjected to considerable static loads at various indirect phases of the performed bridge repair. The results obtained during the research at different stages of repairs were conducive to determining the behavior of the analyzed span structure under static loads, which allowed for an assessment of the efficiency of the strengthening, as well as establishment of guidelines for future reference concerning this type of strengthening in the engineering practice.

The research allowed to find out on which elements of the load-carrying structure of span the biggest forces were exerted during the progress of repair works on one half of the bridge. It is quite an important problem not only in terms of construction works safety but also as the safety of vehicles and people, using the non-repaired part of the bridge are concerned. Furthermore, there is also a possibility of overloading some structure elements, like e.g. main beams.

The measurements were performed at eleven stages (Stages I–XI) of bridge structure repairs. At Stages I–V and VII of the experimental research, the outer span I and II of the bridge was loaded on closed for traffic (left) half of the bridge by the truck. On this half of the roadway only repair works were performed there at that time. At Stages VI and VIII–XI the second half of the bridge spans I, II and V was loaded during research.

The final results of the bridge acceptance inspection, conducted after the complete repair under the trial static and dynamic load, allowed a comprehensive evaluation of the efficiency of the main beams strengthening by applying CFRP strips. Moreover, it enabled a comprehensive evaluation of the change of the spans structures behavior under the same load during different stages of repair works, simultaneously with the standard traffic running on the second, then non-repaired half of the bridge.

The results of the measurements of the main beams deflections and strains on the bottom flanges caused by static loads were of elastic character.

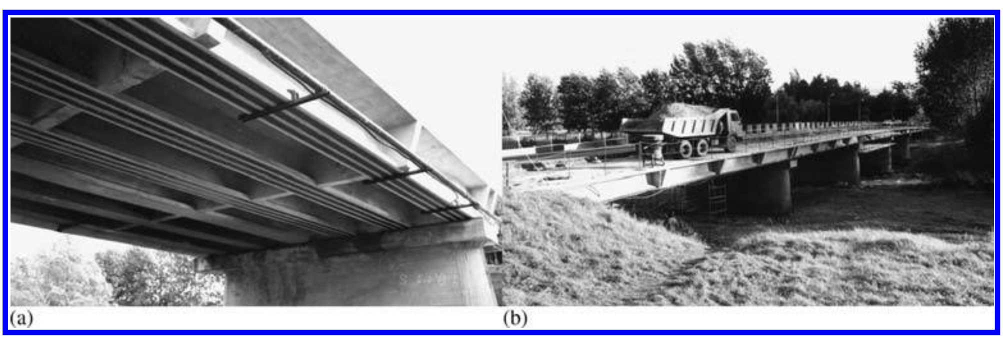

Figure 1. A general view on bridge: (a) bottom view on span I after gluing of strengthening strips on bottom surfaces of three main beams from headwater and (b) side view during II Stage of experimental research (visible loading truck set up on span I).

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Analysis of post-tensioned concrete road bridge beams strengthened by CFRP strips using FEM and field tests under static load

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ABSTRACT

The paper is presented the method and results of the FEM analysis and research, and also the main conclusions of a one-span post-tensioned concrete road bridge over the Nysa Klodzka river (Fig. 1) situated in Klodzko (Upper Silesia, Poland) damaged by 1997 big flood performed under static field load before and after of its repair. There was a conception to make reinforcing by the CFRP strips made of carbon fibers CFRP SikaDur M1214 type glued on the bottom flanges of main beams or by overlaying a new concrete deck layer. The bridge load capacity before its repair, determined in expertise, was classified as class E, that is 150 kN, mainly due to a very poor technical condition of the load-carrying structure of bridge span, resulting mostly from transverse cracks in the main beams. The main aim of the conducted bridge repair was to increase its load capacity to 300 kN (class C) according to the Polish Bridge Loads Standard (PN-85/S-10030). The results obtained for four different static load schemes to determining the behavior of the analyzed span structure, which allowed for an assessment of the efficiency of the strengthening, as well as establishment of guidelines for future reference concerning this type of maintenance in the engineering practice. The conclusions were drawn from the passed analysis and tests can be helpful mostly for the assessment of behavior of such type of the bridge strengthening system by the FRP strips.

The program COSMOS/M was used for computation. Finite element analysis was used to model the behavior numerically to as to provide a valuable supplement to the field investigations, particularly in parametric studies.

Some results of the deflections at midspan of four beams B1–B4 (Fig. 2) obtained from the research and calculation before and after reinforcement are shown in Table 1. The final results of the bridge acceptance inspection, conducted after the complete repair under the trial static and dynamic load, allowed for a comprehensive evaluation of the efficiency of the main beams strengthening by applying the CFRP strips.

Figure 1. Terrain level side view on the post-tensioned concrete road bridge span and pier in Klodzko after modernization.

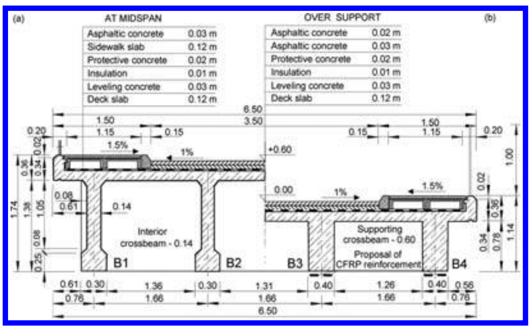

Figure 2. Cross-sections of post-tensioned concrete span at: (a) midspan (before repair), (b) support (after its renovation).

Table 1. Results of the deflections at midspan of four beams B1–B4 obtained from the research and calculation (10^{-3}m) before and after reinforcement.

Test stage	Result nature	B1	B2	B3	B4
Before repair	measured	5.82	6.37	6.86	7.39
	calculated	8.86	8.86	8.86	8.86
With CFRP	calculated	8.69	8.69	8.69	8.69
After repair	measured	5.16	5.19	5.48	5.55
	calculated	6.83	6.85	6.85	6.83

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Torsional behaviour of concrete T-beams retrofitted with steel plates

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ABSTRACT

Steel plate bonding technique using epoxy resin or bolting is a well established concept, used to provide additional resistance to bending, reduce deflection, and is particularly useful as a retrospective method.The behaviour and performance of reinforced concrete T-beams strengthened with externally bonded steel plates subjected to pure torsion is presented. The parameters considered in the experimental study include the length and position of steel plates adhered on soffit of the beam or web or both web and soffit of the beam. Experimental results revealed that the externally bonded steel plates significantly increase the stiffness of the beam and the failure due to torsion is sudden, without much warning.

Current literature reveals that strengthening using steel plates, provides a substantial increase in post cracking stiffness and ultimate load carrying capacity of the members subjected to flexure and shear (Reference 2). Research related to the strengthening of torsional members with plate bonding is limited and meagre data or design guidelines are available in the literature.

Sixteen RC T-beams have been tested externally, bonded by steel plates of various lengths placed at different positions and tested to predict the strain behaviour, torque and observation of cracks at different positions of the beam.

Comparison of the three sets of retrofitted beams with various lengths of steel plates suggest that the cracking resistance is between 12 to 58% with least resistance for plates with length 2d bonded to only soffit of the beam and largest resistance is for the beam with 4.5d steel plate bonded to soffit and web of the beam.

For beams with soffit retrofit, plate with 4.5d appears to better which resists about 19% of torsion as compared to the non retrofit beam. Similarly, for the beams with web retrofitting and web and soffit retrofit, as shown in figure 1, 4.5d is a better choice with about

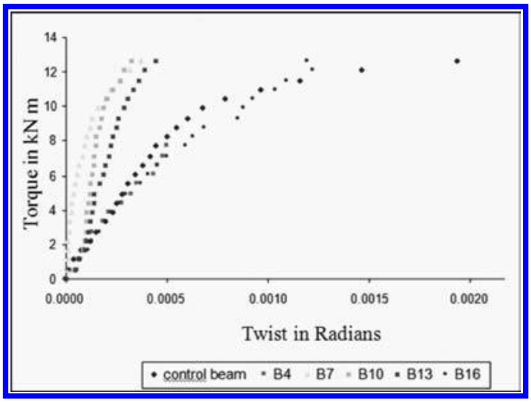

Figure 1. Torque vs. Twist of Control beam B1 and web and soffit retrofitted beams.

44% of increase in strength and 70% of torsional stiffness though about 46% of increase in strength and 72% of increase inn torsion has been observed for full length retrofit.

Plastic strength can be observed in the case of non steel bonded beams, but the same is not pronounced in the case of bonded beams.

Though, stiffness is more in the case of web and soffit, retrofit by about 5%, it is suggested that web retrofit is better suited to resist torsion.

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19. Sustainable construction, preservation, reconstruction

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Hybrid or composite constructions: A way to economic and sustainable structures

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ABSTRACT

It is a well known fact, that physical and mechanical properties of building materials differ quite a lot. Thus it would be a very efficient concept to use different materials for different parts of a structure or cross-section depending on the stressing of the different parts. The basic idea of composite or generally speaking hybrid structures is to combine different building materials in order to take profit from the advantages of the respective material. At the same time specific disadvantages of each building material can be avoided.

But due to traditional thinking and specialized education at the universities till today many engineers tend to think in only one material – concrete or steel or wood – when designing new structures.

Nevertheless built structures like for example the cable stayed Normandy bridge but also theoretical and experimental research clearly show that the use of different materials in one construction may lead to more economic, more sustainable and technically advanced structures.

This paper deals with two examples which clearly demonstrate the value of composite or hybrid constructions.

The first example is a new composite slab made of timber and concrete. The new idea is that the timber parts for the tension zones and the concrete parts for the compression zones of the slab are completely prefabricated. Thus a very short construction period without bringing humidity into the building is possible. Especially in countries with great timber resources this new slab construction delivers economic but also ecological advantages in comparison to traditional monolithic concrete slabs.

As second example the development and first use world wide of a hybrid girder for high speed maglev trains will be presented. This girder consists of prestressed concrete, steel function units and connecting parts made of cast iron.

In comparison to former girder types made of only one material – either concrete or steel – this innovative construction is cheaper while at the same time delivering technical advantages like e.g. lower tolerances, lower noise emissions and a higher resistance against vibrations.

This kind of girder was used for the first commercial maglev train track built in Shanghai and proofed the advantages of the hybrid concept.

Summing up composite or hybrid constructions can deliver technical, economical and environmental advantages in comparison to traditional constructions consisting only of one certain material.

Thus in the future this kind of constructions will gain more and more importance.

Figure 1. Test set up for dynamic and static tests of timber – concrete slabs.

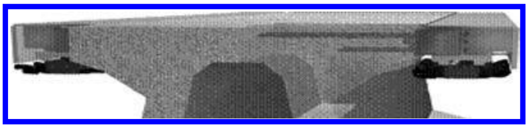

Figure 2. Hybrid girder for the Transrapid guideway consisting of prestressed concrete, structural steel and cast iron elements.

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On modeling of projects with utility assessment

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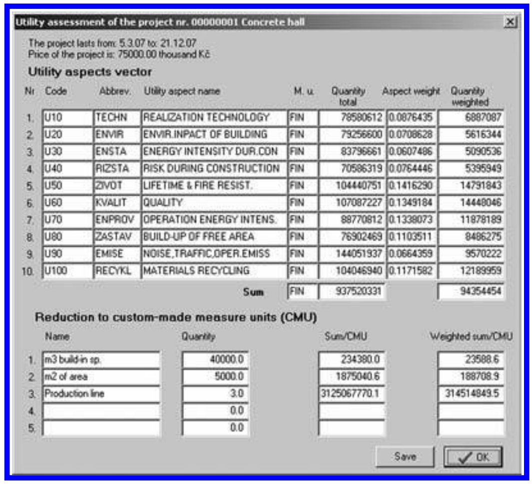
ABSTRACT

The paper describes the methodology of utility assessment of building and projects by use of computer modelling of the project and of the building process. The main principles of the integrated cost estimation, project management and quality assurance system developed in last years are shortly mentioned. This expert system is based on quick modelling of the building process of different buildings and structures by use of typical network diagrams, which are prepared in advance, created by an original construction technology network diagram method. Thanks to the database of construction processes and to the typical network diagrams the model of the building process including the cost assessment can be made about 50 times quicker than by use of classical cost estimation or project management systems. Models of the building process created in this way can be used easily for utility assessment of building and projects.

As it is known that about 80% of costs and labour consumption are influenced by only 25–30% of construction processes, the cost estimation can be done very quickly and precisely. This method can simultaneously create and update quality assurance checklists (ISO 9001), environmental plans (ISO 14001) and safety work plans (OHSAS 18001) in direct linkage to network diagrams and time schedules as parts of the construction technology design.

The system creates cost estimations, project management documents, quality assurance checklists, environmental plans and safety-at-work plans on the base of the construction technology, time and resource analysis. Resulting documents answer not only the question of price and costs, but also the optimum construction processes flow, their linking and terms, cash and resource flow during the time of erection. According to more particular data available about the structure in the course of planning of the project the model can be stage by stage defined with more precision or easily updated according to the actual flow of the building or reconstruction progress.



Utility assessment of the project nr. 00000001 Concrete hall

The project lasts from: 5.3.07 to: 21.12.07
Price of the project is: 75000.00 thousand Kč

Utility aspects vector

Nr	Code	Abbrev.	Utility aspect name	M. u.	Quantity total	Aspect weight	Quantity weighted
1.	U10	TECHN	REALIZATION TECHNOLOGY	FIN	78580612	0.0876435	6887087
2.	U20	ENVIR	ENVIR.INPACT OF BUILDING	FIN	79256600	0.0708628	5616344
3.	U30	ENSTA	ENERGY INTENSITY DUR.CON	FIN	83796661	0.0607486	5090536
4.	U40	RIZSTA	RISK DURING CONSTRUCTION	FIN	70586319	0.0764446	5395949
5.	U50	ZIVOT	LIFETIME & FIRE RESIST.	FIN	104440751	0.1416290	14791843
6.	U60	KVALIT	QUALITY	FIN	107087227	0.1349184	14448046
7.	U70	ENPROV	OPERATION ENERGY INTENS.	FIN	88770812	0.1338073	11878189
8.	U80	ZASTAV	BUILD-UP OF FREE AREA	FIN	76902469	0.1103511	8486275
9.	U90	EMISE	NOISE,TRAFFIC,OPER.EMISS	FIN	144051937	0.0664359	9570222
10.	U100	RECYKL	MATERIALS RECYCLING	FIN	104046940	0.1171582	12189959
			Sum	FIN	937520331		94354454

Reduction to custom-made measure units (CMU)

	Name	Quantity	Sum/CMU	Weighted sum/CMU
1.	m3 build-in sp.	40000.0	234380.0	23588.6
2.	m2 of area	5000.0	1875040.6	188708.9
3.	Production line	3.0	3125067770.1	314514849.5
4.		0.0		
5.		0.0		

Figure 1. Calculation of the project utility assessment.

A vector of 10 main aspects for utility assessment was created with a same measure unit. The aspects were given a certain level of significance each. A database of construction processes was created including the aspects for utility assessment. On the base of the methodology of the construction technology design a method for computer utility assessment including the program was developed. Figure 1 shows the result of the utility assessment of a certain project on the computer screen.

A few examples of use of the mentioned methodology for preparation and management of significant projects are discussed in the final part of the paper.

The paper is presented as part of a CTU in Prague, Faculty of Civil Engineering research project on Management of sustainable development of the life cycle of buildings, building enterprises and territo-ries (MSM 6840770006).

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Adaptive floor slab systems as a contribution to sustainability

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ABSTRACT

At present, buildings in urban areas are often broken down or elaborately rebuilt before they reach their economical and technical lifetime. The most common reasons are the inflexibility of the construction to allow for changes in utilization (e.g. from residential to office building or v.v.) and the realization of current architectural requirements or new technical innovations. In order to use a building for a longer period more efficiently, one has to aim at designing more flexible structural systems, which also enable economical and ecological recycling.

For the estimated lifetime of a showcase building unit three utilization phases has been defined. The aim is to compare adaptive building concepts with hitherto commonly used structures. An alteration of the occupant of an office building leads to rebuilding and rearrangement of the floor plan. Modern office buildings usually allow for those changes in utilization. But a change from office to residential utilization is a different story which leads to a demolition of the building. In contrast a flexible structure according to Figure 1 allows for changes between residential and office utilization (Hegger 2007). Thus, the high service lifetime of the concrete structure can be used effectively.

Only the life cylce assessment (LCA) for the structural system is considered, which means that façade systems as well as building services systems and extension sytems are neglected. The principles of sustainability assessment of concrete structures are derived in (Graubner 2007).

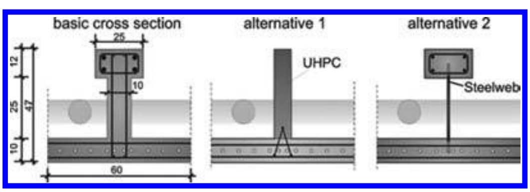

Figure 1. Cross section of floor slab systems with integrated building services systems.

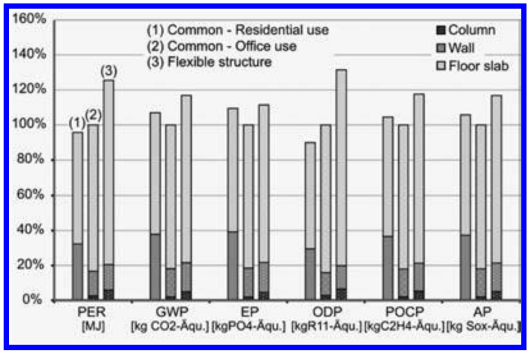

Figure 2. Environmental impacts for steel and concrete.

Figure 2 presents the results of the LCA for the structural systems. The flexible structure leads to 10–40% higher environmental impacts compared to hitherto commonly used structures. The impacts of the three investigated structural systems lay in between ± 20%. One could easily imagine, that the higher environmental impacts due to the erection of the structure can be equalized during the hole life cycle because of lower impact for rebuilding. Most of the impact is caused by the floors. Thus the floor slab systems are of high interest for the ecological assessment as well as the planning of structures for flexible utilization.

Due to the higher service life of flexible structures less environmental impacts can be achieved. Thus, structures allowing flexible utilization can contribute a sustainable progress.

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Numerical modeling of behavior of an insulation block from recycled polymers

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ABSTRACT: The article is focused in particular on recycled polyethylene application in products designed for construction industry, especially for passive houses. Currently certain building details of passive houses are not perfect or their solution results in higher economic demands related to house purchase and its further use. For the purpose of this thesis detail of an elimination of thermal bridges in wall footing has been chosen. Product was subject to mathematic modelling of thermal technique and statics. The executed mathematic models documented that product is fully functional and that the suggested product successfully eliminate insufficiencies of some currently applied solutions.

1 INTRODUCTION

Using of waste and waste material is a frequent and actual topic, which corresponds to sustainable development

An important subgroup of waste is formed by polymers (plastic materials) in various forms, for example PP, PE, PET, PVC, PUR etc. One of the groups of polymers are thermoplasts.

For use in civil engineering it is necessary to select such polymers that have suitable thermal technical and mechanical properties and it is possible to modify their final properties and behavior. The example of polymers meeting the requirements is the waste polypropylene PP and waste polyethylene PP in low-density LDPE or high density HDPE form.

2 INSULATION BLOCK FROM RECYCLED POLYMERS

Current trend of energy savings is the proposal and construction of low-energy and passive houses. This new concept is associated with the arisen need in these types of houses to solve the originated details.

The detail which can constitute problematic place in designing and construction is a wall footing detail. This detail form thermal bridge with power loss if it is performed incorrectly.

For solution of a wall footing detail was designed insulation block from recycled polymer in PP, and in HDPE version (or to advantage in composite plastic material), see Figure 1. This block directly interrupts the thermal bridge and meets the requirements for functionality, absorbability, mechanical resistance.

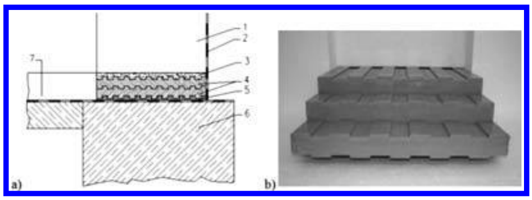

Figure 1. Insulation block: a) cross-section; b) product of an insulation block composite by 3 boards.

The design and assessment was carried out using FEA analyses carried out by ANSYS system. In the mathematic modeling the installation frame was assessed in terms of statics and thermal technology.

For verification of design was realized long-term experimental measuring on wall with an insulation block in 1:1 scale.

3 CONCLUSION

Waste plastic materials in general may form an important element in selection of building materials in the future. The article deals only with plastic material PP, HDPE used for design of an insulation block and its numerical modeling and experimental testing.

ACKNOWLEDGMENT

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Sustainable building experience

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ABSTRACT: Most University projects are of a theoretical nature and do not offer future professionals the opportunity to link theoretical knowledge to personal experience. Building a new kindergarten for underprivileged children in South Africa became such an exceptional experience for a group of students of Civil Engineering and Architecture from Munich, who first designed the structure and then built it with their own hands. The design makes use of simple building techniques and a large share of biogenic materials like sun-dried bricks and timber. In addition, a solar heating system supplies energy for under floor room heating. The student team thus came up with a prototype sustainable building adopting basic technologies for the use of regenerative energy and ecological materials.

In this interdisciplinary project, students worked together in groups of one architect and two civil engineers. The assignment asked for a kindergarten with four playrooms for up to 80 children. Each member of the design teams had to contribute results from his or her special field to come up with a collective proposal. In a second, practical stage of the project, the participants were given the opportunity to actually build the structure within the very short construction time of nine weeks. The challenge of full-scale-projects is to design a building of convincing artistic quality, which can be built with adequate means and using sustainable materials.

The designated construction site for new kindergarten was next to an existing primary school in Raithby, a village near Stellenbosch.

Figure 1. View of the finished building from the Southeast.

On a concrete slab of $50 \times 9{,}5$ m the rooms were arranged along a covered veranda on the South side of the plan. The construction consists of a circumferential adobe brick wall and a timber structure carrying the roof.

Two strategies were pursued in an attempt to make this project a prototype for similar buildings of its kind in the region: The first was to make use of the abundant supply of solar radiation for a sustainable system of space heating: The building was equipped with a solar heating system, supplying each of the four playrooms with under floor heating. The second was an effective passive energy-concept, which includes the layout and orientation of the building, size and location of openings and the design of a well-insulated outer building skin.

Design-build projects provide a much deeper understanding of a number of relevant issues than the theoretical approach of university education can offer. A spontaneous planning process is part of the concept. These projects include a consideration of the social and humanitarian needs of users and try to give an example of a reproducible approach to sustainable architecture.

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Societal considerations in the design of sustainable concrete structures

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ABSTRACT

The design and construction of concrete structures has been associated with unsustainable practices including massive resource consumption, pollution and impacts on bio-diversity and people's health. Sustainable design is meant to encourage the development of concrete structures that are environmentally, socially and economically sustainable. The environmental dimension of sustainability has received the greatest literature coverage among the main design methods in use. Integrated design tools should include the technical, economic, health and environmental aspects into design in various engineering disciplines such structural engineering. The objective of this paper was to show the dominating elements of social sustainability that need to be considered in the design of concrete structures. For this reason the paper sought to answer the following questions:

1. What are societal impacts of concrete infrastructure and how should they be accounted for in design?
2. What are the measures of social sustainability and how well developed are they in literature?

By conducting a literature search it was concluded that current focus on sustainable construction is on ecological aspects and more or less ignores societal sustainability issues. In terms of civil structures, social sustainability refers to the non-technical issues (soft issues) that the structure impacts on the society. The paper identified the societal impacts of civil infrastructure and highlighted that societal impacts need to be accounted for in design. The societal impacts of concrete infrastructure that were identified included:

- Safety issues
- Environmental externalities generated from the structure over the life cycle of the structure works
- Life quality through:
  - Human satisfaction
  - Minimal environmental impact
- Aesthetics, efficiency and durability

For purposes of quantification, the societal aspects were further subdivided into monetary and non-monetary impacts. *Monetary impacts* deal with direct and indirect economic losses, such as damage, repair, loss of assets, loss of revenue, loss of appearance, loss of service, user delay or inconvenience, and impact on growth and employment.

Non-monetary impacts represent losses suffered by individuals or groups of individuals and to which a monetory value cannot be placed. They include death, maiming, injury, loss of long-term income, emotional distress and social disruptions. The “costs” associated with these impacts were found harder to quantify using conventional estimation methods. Several tools such as (i) Hedonic pricing (Gilchrist & Allouche, 2005). (ii) Contingent valuation technique (iii) Use of social indicators such as Life-Quality Index (Nathwani et al., 1997 and Pandey et al., 2006) were identified as means of quantifying non-monetory impacts.

In conclusion, it was emphasised that a balance is required between the different dimensions of sustainability to ensure that social sustainability does not come at the expense of economic or ecological sustainability.

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Revitalizing health and safety in the concrete construction for oil and gas industry in Libya

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ABSTRACT

This paper presents an innovative approach to revitalize health safety and environmental protection consideration for increasing profitability by water and energy saving related to concrete production, transportation, placement and compaction especially for oil industry. It demonstrates that how development of an innovative Coastal States Concrete code with enhanced HSE and economy can result from water and energy saving potential in line with AU/NEPAD supported African Productive Capacity Initiative (APCI). Framework proposed has the potential to enhance productive capacity of concrete production for oil producing coastal states like Libya. It is based on a series of innovative bio science and self organizing concrete material based case studies approaches to develop environment friendly green concrete code. The approach advocated has potential to help achieve ultimate goal to design and construct serviceable and safe concrete structures in the coastal states in harmony with nature and their links to Millennium Development Goals to eradicate poverty in Africa.

The Libyan initiative is based upon a series of innovative concepts initiated to document successful practices for evolving oil producing coastal states concrete code using the unified field of all the laws of nature. It utilizes Meisnner effect of propelling and dispelling all internal and external impediments to obtain deterioration free orderly, durable, economical, serviceable, safe and coherent concrete with outstanding performance. It is being achieved by establishing a network of Public Private Partnership Committees focusing on competitiveness and employment with the objective to build a regional observatory on competitiveness in construction sector through quality concrete export market in line with WTO initiatives. It provides a road map in line AU/NEPAD strategic plan supported by: 1. national concrete code action plans, 2. specific public private partnership programs and projects and 3. Peers review committees focusing on competitiveness and employment through quality concrete products. It is capable of assuring sustainable concrete production using global production network similar to the problem free and prevention oriented administration of our ever-expanding galactic universe. It utilizes regular consultation between coastal states and non-state actors/institution to mobilize the resources and promote advocacy.

It suggests the need to organize subsequent meetings in developing initiatives at national, regional, continental and global levels to add and refine the suggested initiatives at each level. It advocates the need to adopt a value chain approach to establish oil producing coastal states concrete code with enhanced HSE and economy using water and energy saving potential in line with UNIDO approach. Fourteen steps from vision to action as proposed by Conference of African Ministers of Industries (CAMI) during Nov 2003 at Vienna are taken as a common framework for implementing the APCI.

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Interface model for the nonlinear analysis of blocky structures of ancient Greek temples

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ABSTRACT

A frequent problem for structural analysis is the study of structures where are present singularity sufaces in the displacement field. Strong discontinuities are present in old masonry structures where dry joints connect the blocks or the mortar ageing suggests to neglect the adhesion properties. To fully analyze the behaviour of such structures is necessary to takein account the deformation modes of the joints. The different ways in which these deformation modes are modelled characterize different approaches.

Two wide classes of approaches could be identified: *Continuum* approach and *Discrete* approach. The first one tries to model the structure considering an homeogeneous *equivalent* continuum where the deformation modes of the joint are incorporated in the constitutive relations. Typical examples are *notension* material (Fuschi et al. 1995) where the opening mode of the joint is modelled, Cosserat continuum theory (Cerrolaza et al. 1995) where additional degrees of freedom are considered and leads to the definition of additional material parameters, nonsmooth multisurface plasticity (Mistler et al. 2006) where specific failure and damage mechanisms are simulated introducing separate softening functions for each strength parameter.

In the *Discrete* approach the structure is considered as an assembly of blocks connected by contact joints. In this way the deformation modes of the joints could be modelled by apposite interface laws. This detailed approach is usually more time consuming but for many real world cases is the only approach capable to consider the several mechanical aspects the structural problem presents.

The *Discrete* approach is followed in the present work, using interface laws derived from the *double asperity* interface model (Mróz and Giambanco 1996). The classical interface model assumes that the contact surface between two bodies Ω^1 and Ω^2 could be assumed as a as a contact layer of thickness h. In the double asperity model the planar joint is modelled by spherical asperities of different radius.

After a brief description of the interface model adopted, the discrete interface laws, suitable for the adoption in a finite element code are illustrated.

The numerical applications regard the analysis of a couple of greek temples of Agrigento in Italy. The temples of *Giunone Lacinia* and *Concordia* are old monumental structures that belong to the ancient greek city of *Akragas*, examples of the extraordinary monumental complex of *Valle dei Templi di Agrigento* inserted, from 1997, in the world heritage list by Unesco. For the temple of *Giunone Lacinia* it was modelled a structural element composed by two columns and the architrave, *trilite*, subjected to dynamic analysis and a time-history of acceleration, obtained by a response spectrum defined according to European Standard EC 8. For the temple of *Concordia* it was modelled the west front composed by six columns, the architrave and the *timpano* subjected to a pseudostatic analysis with increasing horizontal forces till the collapse.

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Brunelleschi's dome in Florence: The masterpiece of a genius

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Gratefully dedicated to the memory of our former Teacher, Prof. Andrea Chiarugi, for his pioneering contributions.

ABSTRACT

The paper presents some recent results concerning the static and seismic behaviour of the Brunelleschi's Dome of Santa Maria del Fiore (Fig. 1).

After a brief description of both the main geometric characteristics and the relevant constructive aspects ideated by Brunelleschi, the present cracking pattern on the Dome is sketched and a finite element model built to assess the static and seismic behaviour of the Monument is described. The FE technique has been employed and an ad hoc non-linear procedure to reproduce the non linear behaviour of masonry has been proposed, to evaluate both the internal stress and the cracking pattern in the Dome.

The non-linear FEM, performed with respect to a series of in-situ measures has allowed to reproduce the behaviour of the structure and the static problems in the area of the damaged webs. It made also possible to identify the most significant aspects on the structural behaviour of the monument by proposing a reliable and likely time evolution of main cracks. Eventually, a first assessment of the seismic vulnerability of the Monument has been proposed.

All results offer a first identification of the buildings behaviour under seismic loading: while showing how advanced numerical analyses provide useful information to evaluate both the existing damage and the structural behaviour, the paper offers a contribution in the assessment of the safety and vulnerability of one of the most emblematic masonry domes all over the world.

Figure 1. The Brunelleschi's Dome.

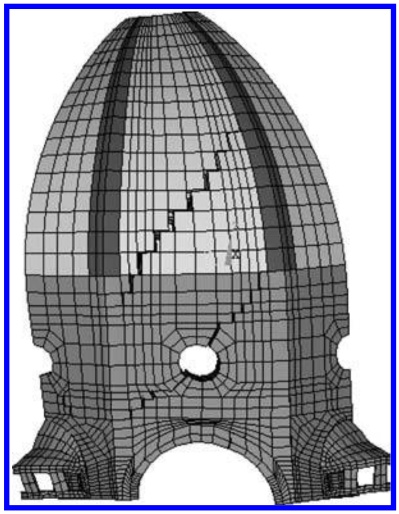

Figure 2. Model cracks.

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Preservation of historical buildings in connection with underground constructions

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ABSTRACT

A new metro line (Metro 4) is currently being built in Budapest. The 7.4 km long section located between Kelenföldi and Eastern railway stations will be a deep section. Tunnelling shields are used to construct two parallel tunnels that run under live waters, mountains and urban environment. 10 stations will be constructed along the line of Metro 4, mostly by diaphragm wall technology and box station design. The metro terminal and the reversing facility at Eastern railway station are located very close to 100 year old residential buildings and to the central building of the railway station (Fig. 1). These traditional masonry structures are highly sensitive to both vertical and horizontal ground motions.

Our tasks were, on one hand, to determine the expected vertical and horizontal ground motions due to underground construction works, and, on the other hand, to predict the expected damages of the buildings caused by ground settlements. Deformations of the surrounding soil were calculated by 2D and 3D finite element analysis considering the different stages of the construction.

In-situ investigation of these buildings was carried out including geometrical surveying and non-destructive material tests. The effect of the expected ground motions on the surrounding buildings was investigated. The aim of the numerical analysis was to predict the development of cracks, the expected rotation of the building and the overloading rate of load bearing walls. Slipping-down of abutments of arches due to horizontal ground movements was also checked. The method for strengthening and renovation had to be proposed in necessary cases.

Due to its complex architectural built-up and high importance as a public building and national monument, the main entrance region of the Eastern railway station was also examined by more detailed finite element analysis. According to results of this investigation, the main building of Eastern railway station is subjected to insignificant damage only.

Figure 1. The main entrance of the Eastern railway station.

Appropriate settlements limits were specified for each affected building separately. These limits were entered into an automated monitoring system that is able to provide immediate feedback if the specified settlement limits are approached, reached or passed.

We believe that this work contributes to the preservation of the surrounding historical buildings, as well as to the quality of the Metro 4 project.

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Reconstruction of the "Berolinahaus" in Berlin: Use-dominated structural design for construction in existing buildings

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ABSTRACT

The Berolinahaus built in 1932 by the famous architect Peter Behrens located at the Alexanderplatz in the center of Berlin is one of the first reinforced concrete skeleton-frame structures ever and is therefore a listed building. In order to fulfill the new demands on a shopping center, an innovative design is presented, which shows the doubling of the support grid in the lower floors, while the upper four floors and the foundation construction remain preserved in their entirety. In addition it is shown, how it was possible to preserve the existing structural framework with its, compared to the state of the art, inferior concrete qualities by making use of system reserves, load reserves and material reserves without instigating extensive strengthening measures. The complexity of the design and its application, while minimizing deformations, time and costs, is shown by ambitious constructions based on a structural analysis in special construction phases.

The Berolinahaus was used to demonstrate that the historic building structure to be integrated is capable of fulfilling today's requirements and thus constitutes an economic alternative to a new building, utilizing present day knowledge of material properties and activating system reserves.

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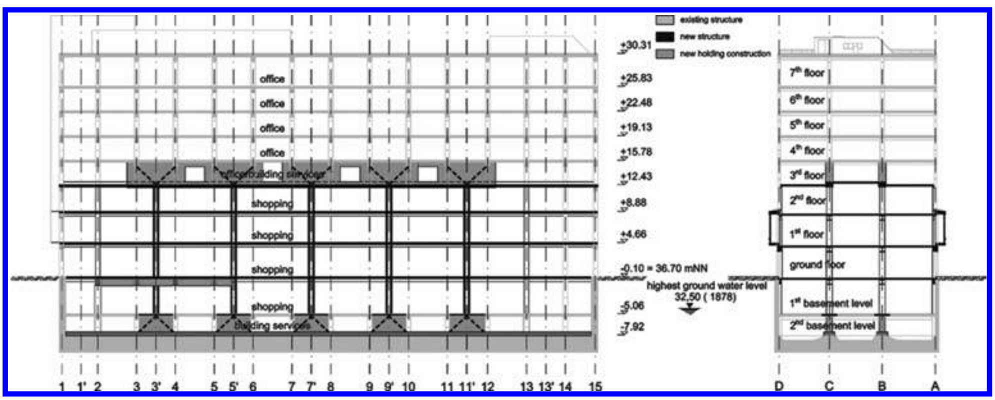

Figure 1. Structural design of the reconstruction.

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Structural assessment and consolidation of an historical cupola

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ABSTRACT

This paper deals with the structural assessment of the main cupola of the 17th century church *Beata Vergine di Monserrato*, in the center of Barisardo, a small town in the eastern coast of Sardinia.

In 2004 the whole structure was subjected to a conservative intervention, including a restoration of the dome, affected by an extended state of damages. During this intervention the dome's masonry was found to include a set of wooden ties whose function was not definitely identified. These ties were reconditioned and masonry consolidating injections were carried out. Nevertheless soon after the dome required some new interventions giving the opportunity of a more comprehensive structural assessment.

In this paper the safety level of the dome and an interpretation of the damages have been tackled by means of a two-way approach: Finite Element modelling and Limit Analysis calculations.

The former has been carried out by means of a commercial code providing a non linear constitutive model capable of represent the masonry damaging behaviour due to onset of cracks. An attempt has been made to reproduce the complexity of the historical masonry with respect to texture and material heterogeneity. The results of this analysis under the effect of dead loads, have shown a low stress state and a crack pattern similar to that observed during 2004 interventions. The deformed shape gave a suggestion of the possible collapse mechanism. Limit Analysis calculations have been preformed according to Heyman's theories on masonry Plastic Analysis (Heyman, 1967, 1995). Presumable collapse mechanism has been identified by dividing the dome into slices and tracing the possible thrusting lines by means of graphical statics tools. The analysis gave a presumable collapse mechanism in accordance to the FEA results and the correspondent thrust value. The presence of the wooden ties aroused some doubts about stability especially in case of interventions on the covering.

The former results led to a reinforcing proposal by means of FRP sheets glued on the external surface of the masonry structure, with the aim of providing a flexural strength to the dome segments and a hooping effect preventing the arising of the described collapse mechanism.

Figure 1. A view of a church.

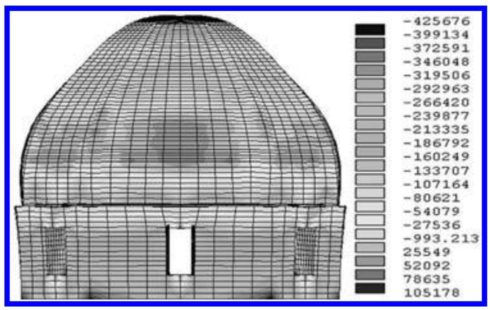

Figure 2. Vertical stresses (Pa) plotted on the deformed shape.

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20. Soil-structure interaction, tunnels, underground structures, foundations

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Effect of soil structure interaction on internal forces in integral bridges

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ABSTRACT

Integral bridge decks have no expansion joints. Together with girders they are monolithically connected to abutments. In a full integral construction abutments are rigidly connected to piles. Consequently, an in integral bridge is a single structural unit, which accommodates thermal loading together with the surrounding soil. Elimination of deck joints brings multiple sustainability and performance benefits among which are lower construction and maintenance costs, increased durability and resilience, lower impact and noise produced by vehicular traffic, and easier future widening and replacement.

While Cross (1932) laid foundations for structural analysis of integral bridges, a complex soil structure interaction still remains purely understood. Thus, design of integral bridges remains mostly experientially rather than scientifically based. The present study was undertaken to advance the knowledge of soil-structure interaction in integral bridges. To this end an actual integral bridge located in Fitchburg, Massachusetts (Faraji et al 2001) was selected for computational modeling. It is a three span bridge, whose superstructure comprises concrete deck and seven W 36×135 steel girders. It is monolithically connected to stub abutments, each of which is rigidly connected to a single row of seven vertical steel HP12 $\times$ 74 piles. To facilitate accommodation of thermal load their strong axis of bending of piles is oriented parallel to the longitudinal direction of the bridge. Only half of the bridge is modeled due to symmetries in geometry, loading and soil conditions. The temperature of superstructure is increased, while the temperature of substructure is held constant. A portion of abutment, which is exposed to the atmosphere, is also heated up, thus resulting in the internal thermal gradient within the abutment. Thermal load is applied together with gravity load. The loading scenarios are devised to simulate the presence of different soils behind abutment. The soil adjacent to piles was dense sand for all loading scenarios.

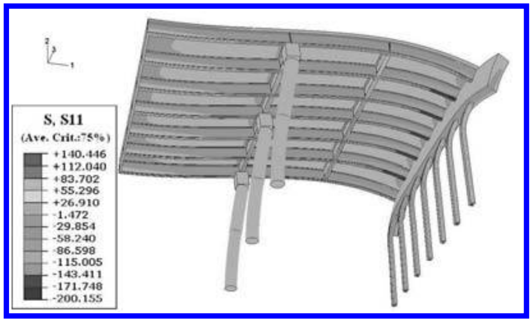

Figure 1. Longitudinal stresses (MPa) in the bridge in the presence of dense sand behind abutment and temperature change of 55.5°C (deformation scale factor is 130).

The results of finite element analyses indicate that the bridge superstructure responds to an increased temperature with a combined extension and bending (Fig. 1). The denser the sand behind the abutment, the more significant is the bending of the superstructure. In addition, relatively large compressive stresses build up in bottom flange of girders in the vicinity of the abutment as a consequence of a restrained extension in the presence of dense sands. Composite bending moments of the superstructure, which were obtained by numerical integration of the first moment of stresses about the center of gravity of the bridge superstructure indicate that the influence of soil compaction level extends all the way to the bridge centerline, although it is more pronounced in the end spans.

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Parametric study of influence of subbase height to the contact stress distribution in buried vaults

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ABSTRACT: Buried vaults are at present mainly built as environmental structures. They belong to group of structures called green bridges. These structures are mostly designed as a thin-walled reinforced concrete arches. The paper describes the contact stress distribution within these reinforced concrete structures from traffic in case of different subbase height. Determination of the value of contact stress and its distribution in the contact between subbase and supporting structure and between subbase layers is done using computer mathematical modeling. The value of maximal contact stress and the distance of action is described as a function of subbase height.

1 INTRODUCTION

Buried vaults are mostly designed as a thin-walled reinforced concrete arches. The shape of arch in a form of 2nd degree of parabola or in a form of hyperbolic cosine function.

Design of buried vaults is influenced by many factors, including load, choice of the shape of the load bearing construction, transverse layout, etc.

Within article is with help of computer mathematical modeling using parametrical model analyzed influence of height of subbase on size, distribution and length of acting of vertical (contact) stress on load bearing structure of buried vaults from traffic load.

Buried vaults in the Czech Republic are designed in accordance with national design code ČSN 73 6203 in 2005 which defines loading patterns. One of the loading scheme is 4 axle vehicle acting with intensity 4×200 kN (one wheel of 100 kN). This type of load was taken into account to find an influence of subbase height on size, distribution and acting length of vertical (contact) stress on load bearing structure of buried vaults from traffic load.

2 PARAMETRICAL MODEL

The analytical model is created in the ANSYS program. The FEM 2D model is defined as fully parametrical, created by means of eight nodes finite element. The geometry of the analyzed structure respects all structural layers including interconnection between structural and pavement parts. Model geometry is defined using 9 independent parameters. The ideally elastic material model is used to model mechanical properties of materials in layers of pavement structure, subbase and supporting structure.

The analytical model also respects all interconnections between pavement layers and between subbase and supporting structure.

3 CONCLUSION

Using a parametric study was determined functions in 3rd degree polynomial shapes for dependence of length of distribution of contact stress L and maximum contact stress V on subbase height h:

$$L = 0.0286h^3 - 0.3225h^2 + 3.1913h + 2.8929 \tag{1}$$

$$V = \left(-0.0123h^3 + 0.2194h^2 - 1.2866h + 3.9459\right) \cdot 10^4 \tag{2}$$

Presented results are usable as essential inputs for designers of buried vaults and ecoducts, since the classical method expects equal distribution of contact stress, which is determined by spread angles in each layer, as shown for example in national design codes. For complex description is appropriate inside parametrical studies take into account uncertainties in input variables and also the statistical and sensitivity analysis should be performed.

ACKNOWLEDGMENT

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The influence of the installation method on soil plugging in open-ended piles

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ABSTRACT

Open-ended steel piles are often used for offshore and onshore constructions. During the design stage of these piles it is necessary to predict the driveability of the profiles and the bearing capacity of the piles. Both parameters are predominantly influenced by the well known effect of soil plugging.

In this contribution numerical analyses concerning the installation of open-ended steel piles are presented using finite element method. The soil in the analyses is discretised using a hypoplastic constitutive law, which is well suited to reproduce typical soil behaviour during pile installation like compaction and dilatancy. In several numerical analyses the installation of different piles into the subsoil is simulated using different installation methods (pile jacking, vibratory pile driving and impact driving).

Out of these calculations the influence of the installation method on the tendency of forming a soil plug inside the pile is demonstrated. It can be found out that pile jacking more likely leads to the formation of a soil plug compared to the dynamic installation methods.

Furthermore, it can be concluded that smaller pile diameter and higher soil density significantly increases the tendency of soil plugging.

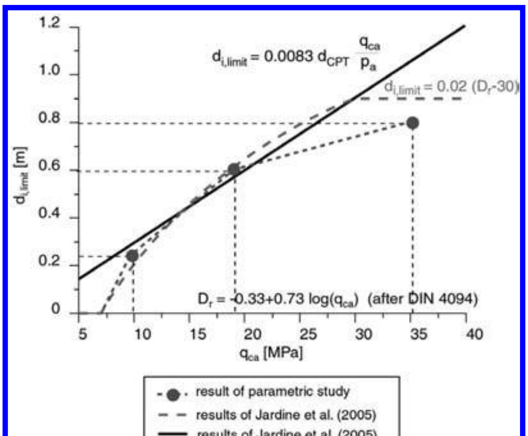

Figure 1. Comparison of the numerical results with in-situ measurements of Jardine et al. (2005).

The results of the parametric study are evaluated using the theoretical approach of the active length of the soil plug after Randolph et al. (1991) and are compared to in-situ measurement results of Jardine et al. (2005), see Fig. 1.

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Comparative analysis of simulation and field monitoring on the metro pit constructing safety

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ABSTRACT

Construction of metro in many cities of China is in the ascendant. But frequently, many accidents happened when the pits are excavated, and also, the ground deformation caused by excavating would influence the near building. Verified with the pit excavating of a city metro, both FLAC3D numerical simulating and field monitoring are used to analyze the construction safety. Further, the results of two methods are compared to validate the veracity of numerical simulation and the applicability of the Mohr-Coulomb rule. And then the method can be used in other subway projects, so as to provide a important basis for the metro designing and construction.

Open excavation is one of the most common construction methods for metro, while pit construction safety in the open excavation method is related with many factors, such as neighboring environmental safety, envelope stability, ground deformation and other factors (Figs 1–3, 6–7, 9). Numerical simulation has been widely used and applied in engineering in ground deformation and stress distribution caused by the excavation, such contents has carried out extensive studies (Figs 4–5, 8). However, the combination of numerical simulation of pit constructing safety and field monitoring is relatively rare, this paper bases on the open excavation of metro pit, numerical simulation and field monitoring data are compared, and the application reliability of numerical simulation in constructing safety are demonstrated, which provide engineering reference for similar projects.

In order to comprehensively and systematically reflect force and deformation of surrounding rock as well as support during pit excavation, three-dimensional commutating model is established based on FLAC3D model for numerical simulation.

Seven months of monitoring has been carried out from May 2006 to January 2007, tracking other indicators changes during the excavation process, including ground deformation, the settlement of surrounding important structures, axial force of support, bottom rebound, monitoring data is recorded 1 time per day.

The field monitoring lasted more than seven months, monitoring frequency is once a day, and accumulated a large number of monitoring data, only a sample of monitoring data is presented in this paper.

In this paper, several aspects, such as ground deformation along the X direction, ground deformation along the Y direction, and the axial force in the support which affect the pit and the surrounding building safety, are comparatively analyzed. The result shows that calculated values is good consistent with measured values, although there is slightly different.

In a word, the numerical model can more accurately simulate the impact of construction process on the pit and safety of surrounding buildings. It also demonstrates that it is feasible to simulate the displacement and stress change during metro constructing using numerical simulation approach. Ground deformation trends around pit and stress state of the supporting structure, which calculated by numerical model, can provides a theoretical basis for pit support methods and selection of construction program.

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Bearing behaviour of embedded piles due to cyclic and dynamic lateral head loads

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Vertical piles carry off lateral loads at the pile head towards the surrounding soil by means of horizontal bedding stresses,for instance in offshore piling (cyclic loading with low frequency due to wave excitation) or cantilevers of noise load reduction walls as to high-speed railway tracks (dynamic loading due to higher frequencies of wind shock waves). In this process the pile's bearing performance is determined by the pile-soil-interaction. Despite numerous investigations throughout the past years, the actual knowledge in lateral pile-soil-interaction is still insufficient. In practice, the pile-soil-interaction is commonly disregarded or represented in extremely simplified forms like subgrade reaction models, which can lead to uneconomic or even defective design. The complexity of pile-soil-interaction is due to the non-linear behaviour of the soil, the contact phenomena developing in direct transition of pile and soil and – last but not least – to the spatial circumstances of the matter.

If there is a periodic or dynamic strain on the pile head, the level of complexity is still elevated by the extreme disparities of rigidity of the soil for charging and relieving as well as from the simultaneous change of the soil's density. Therefore only those techniques are suited for the examination of pilesoil-interaction, which give consideration to the complexity of the problem. Accordingly, small-scale simulation tests in a sand box as well as numerical investigations by means of the Finite Element Analysis under three-dimensional conditions were carried out.

In the physical model tests in sand of different initial densities, the single floating pile models were exposed to static, cyclic and real dynamical (up to 7 Hz) lateral loads, leading to results like the loaddisplacement-curves of the pile top and the distribution of the bending moments, which were counted back from strain measurements along the pile shaft. Furthermore, the distribution of bedding stresses along the pile shaft could be recalculated from these measured strain values by means of inverse methods. The geometrical scale in the physical modeling was taken to 1:12, representing a prototype reinforced concrete pile of 30 cm in diameter and 5 m in embedded length. The model piles are constructed from brass tubes surrounded with epoxy resin to assure a correct bending stiffness as well as the scaled geometric width of the pile. The brass pile shafts are equipped with triple sets of strain gauges in different depths. The inside of the tube is filled with lead balls to assure a suitable average density of the model pile according to dynamic modeling theory. The model soil is dry medium coarse grained quartz sand in different initial densities. The model piles are driven by means of a guided pushing process into the soil mass made from dry sand in a certain initial density. Due to this the installation process is taken into account. The applied pulser is able to perform vibration loading with frequencies up to 7 Hz and different waveforms. The tests were conducted with harmonic excitation. Fig. 1 shows the typical result of a dynamic loading test with displacement-controlled amplitude $y(t) = 2.5 \cdot \sin(2\pi \cdot f \cdot t)$ [mm] and 7 Hz frequency.

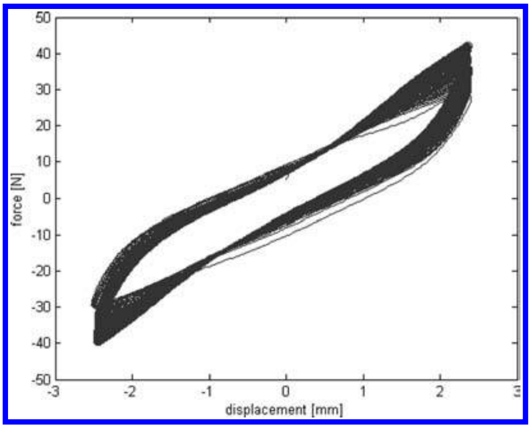

Figure 1. Load-displacement-curves for harmonic excitation f = 7 Hz (initially medium dense sand).

Contemporaneously, a Finite-Element-model of the surrounding soil, based upon the hypoplastic material law, was developed, which can be used for the recalculation of the small scale model tests leading to a calibration of the law parameters as well as for a variation of geometrical parameters in the prototype scale. This model is able to simulate the change of soil properties during the excitation and can predict the pile behaviour under cyclic and dynamic loading as observed in the model tests.

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1g model tests with foundations in sand

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ABSTRACT

The main purpose of the present project is to show, that by using a non linear Mohr-Coulomb failure criterion, which takes into account the fact that the friction angle is dependent upon the stress level one can obtain results from theoretical calculations, which are in very good accordance with the equivalent results from single gravity model tests. The variation of the triaxial friction angle with stress level is given in table 1.

To demonstrate this 1g model tests in axisymmetric and plane strain conditions were carried out. The sand used for the tests was Esbjerg sand (Krabbenhoft 2009), which is a medium grain sized quartz sand, and the tests were conducted at a relative density $D_r = 0.84$. A circular footing with diameter $B = 10$ cm was used for the axissymmetric case and for the plane strain case the dimensions of the foundation were: $B \times H \times L = 10\,\text{cm} \times 10\,\text{cm} \times 40\,\text{cm}$. For both foundations the base was covered with a rough material, to make it perfectly rough. In all tests the footing was resting on the surface, resulting in no overburden pressure.

The numerical (finite element) analysis were carried out applying the following yield function:

$$\sigma_1 = k_0\sigma_3 + s_{c0}\left(1 - \exp\left(-a\frac{\sigma_3}{s_{c0}}\right)\right) \tag{4}$$

which is a curved envelope, that passes through the origin and tends toward the asymptote:

$$\sigma_1 = k_0\sigma_3 + s_{c0} \quad \text{for} \quad \sigma_3 \to \infty \tag{5}$$

Table 1. Test results for Esbjerg sand, $D_r = 0.84$.

Confining pressure, kPa	Peak angle of friction φ_{peak}, degrees	Angle of dilation ψ_{max}, degrees	Modified peak angle of friction φ_{mod}
1.5	53.3	23.0*	47.0
5.3	48.6	18.3	43.0
20	46.1	14.8	40.1
50	42.4	15.5	38.7
100	41.3	13.7	37.6

* estimated value

Table 2. Parameters in nonlinear yield function.

State of stress	k_o	s_{co}, kPa	a
Axissymmetry	3.9355	19.7073	1.4330
Plane strain	4.876	29.0391	2.330

Table 3. Results from tests and FEM analysis.

Footing	circular	strip
Failure load from tests [kPa]	148	255
Failure load from FEM [kPa]	151	277

The values of k_0, s_{c0} and a are given in Table 2.

To overcome the numerical difficulties caused by nonassociative conditions a modified friction angle making it possible to deal with the material, as if it were obeying the normality condition. The modified friction angle φ_{mod} can be found from the equation:

$$\tan\varphi_{mod} = \frac{\sin\varphi\cos\psi}{1 - \sin\varphi\sin\psi} \tag{1}$$

The values of φ and ψ on the right hand side of the equation are given in column 2 and 3 in Table 1 and the values of φ_{mod} – used in the numerical calculations for axissymmetric conditions – are given in column 4 in Table 1. For plane strain conditions a friction angle $\varphi = 1.13\varphi_{tr}$ was used. (Wang & Lade 2001).

Table 3 shows the test values together with the results from the finite element calculations. The test values are taken as the average of ten tests for each type of foundation.

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Free slip plane analysis of a strip footing using a genetic algorithm

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ABSTRACT

A relatively simple problem in geotechnical engineering is the design of an infinitely long strip footing on a layer of homogeneous cohesive material. Lower bounds of this equilibrium system can be derived using an equilibrium system and results in a bearing capacity of minimal four times the cohesion.

Upper bounds for the failure load can be obtained by defining a slip surface and determining the driving and resisting forces along that surface. This displacement field can, for example, have the shape of half a circle, but Prandtl's solutions is the lowest upper bound. Prandtl's determination of the load of a half plane carrying a strip load is the most famous one with the lowest bearing capacity: $c \times (\pi + 2)$.

In the recent past, very effective search mechanisms are developed to find the stability factor using limit equilibrium methods on a free slip plane. This paper shows the representative slip plane with Spencer's limit equilibrium method for a strip footing. The representative slip plane is found using a genetic algorithm that searches for a truly free representative slip plane. Spencer's method can be seen as an upper bound just like Bishop's method.

The approach by which the genetic algorithm finds the free slip plane is represented in figure 1. An upperand lower bound is defined with the same number of points. In the case of figure 1, the number of points is 13. The upper and lower bounds are connected with lines.

The first and last ones are connected over the surface line. The free slip plane consists of points along these transversal lines. In figure 1, a completely random slip plane is drawn between the upper and lower bounds. The genetic algorithm moves the points along the transversal lines in order to minimize the safety factor. The calculation with the lowest safety factor is the representative slip plane with its associated safety

Spencer's limit equilibrium method converges to the slip plane given in figure 2. The safety factor that belongs to this mechanism under the assumption of c = p is exactly 4.00, the same as the lower limit.

This upper bound safety factor is identical to the lower bound derived using an equilibrium system.

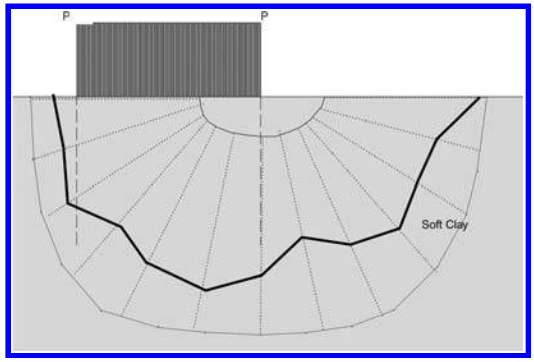

Figure 1. Free definition of Spencer's slip plane.

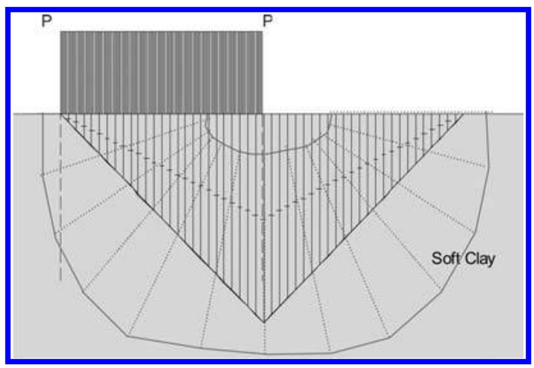

Figure 2. Upper bound by genetic algorithm with Spencer's method, f = 4,00; p = 4*c.

This exercise proves that Spencer is a good limit equilibrium method as it gives the lowest possible value for the safety factor. In combination with a genetic algorithm, it does not only find a global minimum safety factor of this simple strip footing, but it can do so for any load or embankment on any subsoil. This makes its applicability very broad.

One cannot prove that Spencer's method in combination with this search algorithm will always result in a safety factor that is equal to a theoretical lower bound, but this paper does show that this is a better approach than Bishop's method with circular slip planes or Prandtl's method.

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Diverse design approaches to slope stabilization

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ABSTRACT

Slope stability problem is complex and requires particular treatment at each time. This paper presents three different approaches to the design of slope stabilization and to the retaining structures. Calculation were conducted first for the traditional concrete cantilever wall according to polish standards (PN-B-03010:1983). Then the CAE methodology was introduced via well known geotechnical program called Plaxis 2D and the slope was stabilized by means of geosynthetics.

The main problem which is the lack of polish standardization in the subject of reinforced fill design was solved by using Building Research Institute (BRI) guidelines and Plaxis 2D main module. The first two method are consecutively compared with BRI approach within the paper. Results obtain from FEM program can not always be directly put side by side with standards or guidelines. Computational methods haven't found the proper place in Polish or European standardization yet.

The slope of relevant geometry, reinforced with geosynthetics is designed using Building Research Institute guidelines (BRI). The problem is modeled and solved in Plaxis 2D as plain strain and with soil – structure interaction taken into account. The Figure 1 shows deformed finite element mesh. During calculation all stages of construction are considered and the phi/c reduction at the and. The Factor of Safety is obtained for both structures. It is smaller for slope and concrete wall then for the reinforced fill. This means that geosynthetics play its part properly by crossing the slip plane. What's more ground reinforced structure is lighter then concrete wall and can be placed on weaker soils. Besides it is more environment friendly and has some aesthetic advantages. In the other hand long-term durability of such structures and geosynthetics susceptibility to damage are still unsolved problem. It causes also some difficulties with defining proper material parameter for calculations.

The paper presented the unusual approach in which authors tried to tied up the results from computational calculations with guidelines of standards and technical approvals.

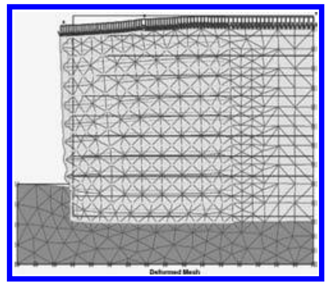

Figure 1. Deformed mesh with FEM discretisation.

Using CEA and visualizations of the results (here by means of Plaxis) one can better understand the behavior of soil and observe shear stresses' concentration. Sometimes it can't go along with engineering insight. In the paper it also proved that geosynthetics played its part properly (see Fig. 5). Although computational methods do not allow to ensure ULSes, they still are of the great help. For example we can estimate tensile strength by the magnitude of reinforcement axial force and the mean value of strains calculated by the program in its surrounding.

It also appears that polish Building Research Institute guidelines are quite restrictive and may cause over-dimensioning.

Summing up all approaches have its advantage and disadvantages so that in the designing process different criteria should be considered.

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Vibration of infinite medium which includes two cylindrical cavities when partially subjected to harmonic inner pressure

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ABSTRACT

In today's world, due to the urbanization and rising population, traffic volume is getting larger. Building upper ground roads is necessary, but many times it is impossible to find enough area for new roads in urban places. In these situations, building underground tunnels is the only way to maintain traffic flow. Because there are too many people using these tunnels for transportation, safety of tunnels is very important. Tunnel may be damaged because of forces that come outside, like earthquakes, or forces inside, like vehicle and train loads.

In this study, response of a homogeneous, isotropic and linear elastic infinite medium, which includes two cylindrical cavities, is investigated. First cavity is partially subjected to a harmonic inner pressure, between center angles α_1 and α_2. In this region, amplitude of pressure is increasing from zero to a constant value in the first part, constant (maximum amplitude) in the second part and decreasing from this constant value to zero in the third part. In this region, amplitude of pressure can be written analytically by using Heaviside theta function. Outside this region, surface is free. Also there is no inner pressure in second cavity.

Because geometry of the medium, material specifications and pressure don't change along the cavity axes, the problem can be considered as a plane strain state. Polar coordinates are used because of the geometry of the problem. Equations of motion that is written based on Newton's second law and then Navier's equations are derived using constitutive equations for homogeneous, isotropic and linear elastic medium. Coupled equations of motion are reduced into two wave equations by use of Helmholtz potentials in polar coordinates. These reduced wave equations are solved by using the multiplication series of the Bessel and the trigonometric functions. Propagations of waves from the first cavity are shown the potentials $\phi^{(1)}(r_1,\theta_1)$ and $\psi^{(1)}(r_1,\theta_1)$ and scattered waves from the second cavity $\phi^{(2)}(r_2,\theta_2)$ and $\psi^{(2)}(r_2,\theta_2)$. N terms are taken in each FourierBessel series, so there are 4N+4 unknown coefficients come from each cavity. Unknown coefficients are calculated by applying the stress type boundary conditions on the cavities.

For various values of maximum amplitude (P_0), forcing frequency (ω), interval of inner pressure (α_1 and α_2), distance between centers of cavities (D), λ and μ Lamé constants, stress and displacement can be calculated in anywhere. In numerical examples, it is shown that for increasing values of distance between cavities (D), effect of second cavity disappears and behavior of medium is similar to infinite medium with only one cavity. Same situation happens for decreasing values of radius of second cavity (a_2). Stress distribution along first cavities surface approaches symmetric situation for increasing values of D or decreasing values of a_2. All results are shown by graphics.

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Dynamic response of an infinite medium with two materials including twin cavities

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ABSTRACT

In the 21st century, necessity of having infrastructure has increased rapidly, with increasing urbanization. For big cities, line systems such as underground tunnels, lifelines, and petroleum, gas, energy and communication lines are considerably indispensable. The probable damages, which will occur at the infrastructure, have very big effects on the city life. Especially, dynamic effects like an earthquake, which is temporary but devastating, or a vehicle load, which is steady state, are unwanted situations at these systems. For this reason, underground cavities have been one of the attractive topics in recent years.

In this study, the response of an elastic fullspace with two materials which include twin circular cylindrical cavities is investigated when subjected to a harmonic load. The full space has been occurred two half-spaces having different Lamé constant. There is one cavity in the each medium. The cavities will be considered as tunnel or pipelines. Excitation comes from one cavity. In this study analytical method is used, because analytical methods are still valuable for providing explanation the physical nature of the problems.

The geometry, material properties of the medium and the excitation have not changed through the axis of cavities. Thus, the problem is considered the plane strain case. The cavities are treated in two local coordinate systems defined at the center of cavities in the each media. Using Helmholtz' potentials, governing equations are reduced into two wave equations in local polar coordinates for each cavities. Reduced wave equations are solved with respect to two local coordinates by using the FourierBessel series. Because infinite number of terms can not be taken at series, the solution series are cut in the finite number of N. Thus, totally $16N+16$ unknown coefficients appeared at the end of series solution in two different media. Written boundary conditions on the cavities $8N+8$ equations are obtained. Thus, boundary conditions from type of stresses on the each cavity surface are satisfied exactly. Moreover, the contact between the media having different mechanical properties is assumed to be welded at the interface. There exist both of displacements and stresses type boundary conditions at the interface. Satisfying these conditions coordinates transformation method is used. This method based on the half space boundary near the cavity is approximated by an almost flat circular boundary of the cavity. The wave potentials are transformed from the local coordinates of the cavity to the coordinates of the large curved surface. Graf's addition theorem is used at this transformation. $8N+8$ equations are obtained from the boundary conditions written contact surface. Thus, writing the number of unknown coefficients the solution has been completed.

The results for the displacements on the cavities versus the radius of the second cavity and ratio of the elasticity modules will be given in graphics.

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On the displacement of pipelines due to adjacent trench excavations

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ABSTRACT

The construction or rehabilitation of sewer pipes in urban environment requires the excavation of trenches with depths of up to (or even more than) 5 m. Due to the excavation measure, displacements occur in the adjoining ground, which can lead to damage of adjacent pipelines. A schematic sketch of the situation to be considered is given in Figure 1.

Observations concerning trench excavation effects on adjacent pipes have been reported from Crofts et al. (1977), Howe et al. (1980), Symons (1980), Rumsey & Cooper (1982) and Cooper & Rumsey (1983).

Rumsey & Cooper (1982) gave results of detailed ground movement measurements and also described the boundary conditions of the test sites thoroughly. These results were used for the calibration of a numerical model.

The Plaxis programme was used to simulate the process of trench excavation and refilling and its effect on adjacent pipes. The HS-small material law was used, which is an elastoplastic material law which takes stress- and strain-dependent stiffnesses into account. The process of construction of the trench and the refilling procedure was simulated thoroughly.

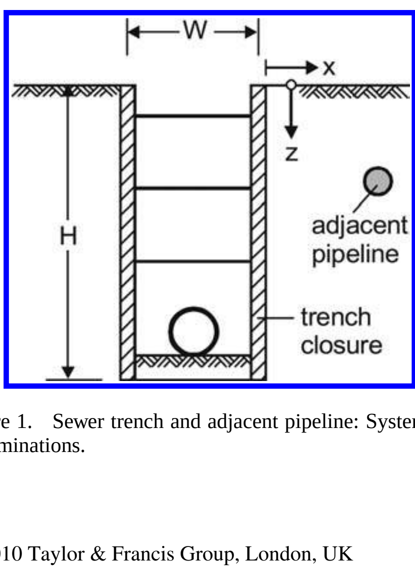

Figure 1. Sewer trench and adjacent pipeline: System and denominations.

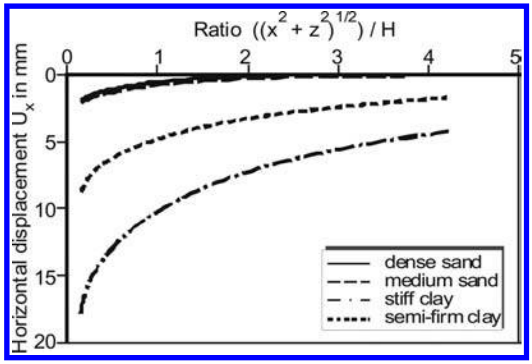

Figure 2. Suggested ground movement chart.

By comparison with the measurement results of Rumsey & Cooper, a calibration of the model parameters was done. Fairly good agreement of the numerical results with the measured results was obtained. Thus, the realistic estimation of pipe displacements with the numerical model was proved.

Based on a parametric study, charts were derived, which give horizontal and vertical pipe deflections dependent on soil conditions, trench depth and pipe location with respect to the trench closure (Fig. 2). These charts can be used for a rough estimation of pipe deflections which have to be expected.

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A review of basic soil-structure interaction models for geotechnical problems

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ABSTRACT

Behavior of soils is very complicated and it shows a great variation of behavior when subjected to different conditions. Various constitutive models have been proposed by several researchers to describe various aspects of soil behavior in details, and also to apply such for geotechnical engineering applications. First, a model of the soil-structure interaction has to be developed to determine the relationship between the soil and the substructure. Also needed in the analysis are the effects on substructure and, the nonlinear soil response. This phenomenon is even complex during earthquake. As waves from an earthquake reach a structure, they produce motions in the structure itself. These motions depend on the structure's vibration characteristics and the building or structural layout. To understand the seismic behavior of soil-structure system there are following parameters: Firstly soil response analysis based on realistic estimate of earthquake magnitude that structure will face, secondally dynamic response of super structure & effect on foundation.

To counter, this role of instrumentation in SSI evaluation is significant.

In the present study RonalB.J, Brinkgreve modeling method was also studied in detail. Experimental evaluation of the models with respect to their suitability to fit experimental data from a various available tests and the ease of the evaluating of the soil parameters from standard tests, detail are presented. All the present day constituent models with their adaptability in different applications w.r.t parameters were studied. The final criterion should be numerical and computational evaluation of the models with respect to the facility which they can be implemented in computer, all this will lead to site specific and performance based design.

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21. Structural engineering education

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Bologna Process: Friend or foe? Experience between change and tradition

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ABSTRACT

The European Bologna Process has been causing the most comprehensive change in the German Higher Education System since the reforms by Wilhelm von Humboldt 200 years ago. The starting point for this development was the Bologna Declaration in 1999, by which 29 European countries committed themselves to create a common European Higher Education Area and to support the Academic Globalization. Until the year 2010 German universities had to put in action, what the politics agreed on paper.

The classical German civil engineering education leading to the degree *Diplom-Ingenieur* had to be replaced by the new bachelor and master study courses. The difficulty was to modify the structure of the study courses without losing the quality and the reputation of the degree *Diplom-Ingenieur*, which is well known all over the world. And also money plays a major role. Bologna is an austerity program, which the politics want to make seem as a growth program.

Up to now the civil engineering education in Germany was leading to the degree *Diplom-Ingenieur*. At universities *Diplom-Ingenieur* programs normally had a regular period of study of 10 semesters, at universities of applied sciences 8 semesters. In the new bachelor and master programs the universities are able to decide independently about the regular duration of study. It has to range from minimum 6 to maximum 8 semesters in bachelor study courses and from minimum 2 to maximum 4 semesters in master study courses. In total students have to study 10 semesters and award 300 ECTSCredits (Credits in the European Credit Transfer and Accumulation System) to a master's degree.

But currently German universities are fighting against problems caused by the structural change of the study programs. It commits a virtually inconceivable amount of human and financial resources. Students and faculty are dissatisfied and students' mobility decreases. Students in bachelor programs are afraid of being second-class graduates compared to the *Diplom-Ingenieur* graduates.

The main problems in creating new bachelor curriculums were the reduced regular period of study and the increasing importance of interdisciplinary courses and general subjects such as presentation techniques, foreign languages and career guidance. Thus, time for the specific technical subjects and internships had to be reduced. To avoid disadvantages for the graduates many universities were overburdening their study programs. Nevertheless, the professional associations and the industry are very skeptical about the bachelor's qualification to work professionally as a civil engineer in the existing job market and to fulfill the same real job requirements as the *Diplom-Ingenieur* does.

The Bologna Process is in the crisis. Unfortunately instead of learning from other study systems they have been adopted without any critical reflection. Germany is one of the countries famous for the high quality in their civil engineering education, although this could be seen as true only in the past. In accordance with the proverb "never change a winning team": Was it the right decision to replace a well-established and successful system by a new one? The future will show if the new developments prove themselves and the vision of a unified European Higher Education Area will become a reality.

Whether you vote for the Bologna Process or not, the consequence will be at least the loss of the uniqueness of the German Civil Engineering Education System.

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SDC software for teaching Structural Mechanics

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ABSTRACT

The SDC program is the result of a long experience in the teaching area of Structural Mechanics started at the Polytechnic of Turin under the leadership of prof. Placido Cicala. After publishing the book in two volumes and the Exercise book on Strength of materials, we decided to add to the book this new instrument, that is an interactive software program made to create and to solve exercises on the different subjects of the book.

The software is already used by the students of the Politecnico di Torino, but it is not known in universities outside Italy. This is the first international presentation in an English version.

The program is very simple and fast to be used: it can help students in the first phases of learning to check and solve the exercises manually and, subsequently, to be used for exercises of great complexity with the possibility to vary any input data and to quickly verify the consequences.

It is a sufficiently practical program that can be used also by planners at least in the first phase of choices of structural schemes.

The program allows you to compile charts and to insert the data in files that can be saved, recalled and printed. All the exercises introduced in our book are loaded together with the SDC program.

Every file has a space where to point out the unities of measure adopted that are consequently used for all the measures, a space for comments, a possible indication for the choice of the orientation of the axis.

The calculation is performed clicking the third button of the toolbar. In the non resolvable cases because the data are incomplete or not compatible, the type of error made is signalled.

Every voice of the menu is devoted to a subject of the book. The subjects are:

1. Vectors
2. Geometry of the areas

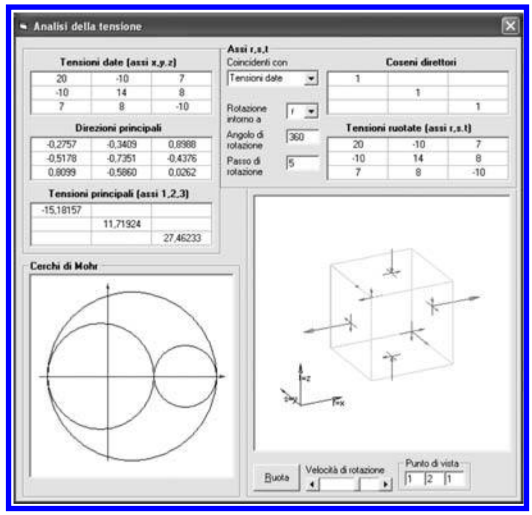

Figure 1. Mohr's circles. The tensions on the faces of an infinitesimal cube.

3. Stresses
4. Flat frames
5. Plane trusses
6. Spatial trusses
7. Diverted bending
8. Shear and torsion. Thin cross sections
9. Reinforced concrete

The program is made with the purpose of helping the students to see directly different solutions to different problems. They can try and see the results changing shapes, loads and all kinds of parameters. It is possible to change the input data at any moment and at any step of the program, see the diagrams and get the outputs in Word format.

Most of the programs allow you to see two windows: one for the insertion of the data and the other with the figure contextually created.