Life-Cycle and Sustainability of Civil Infrastructure Systems

Editors
Alfred Strauss, Dan M. Frangopol and Konrad Bergmeister

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LIFE-CYCLE AND SUSTAINABILITY OF CIVIL INFRASTRUCTURE SYSTEMS

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Life-Cycle and Sustainability of Civil Infrastructure Systems

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Preface

Civil engineering structures have to meet long-term availability and sustainability requirements, with particular emphasis placed on technical safety, efficiency and ecology. Life-cycle civil engineering relates to the design, inspection, monitoring, assessment, maintenance, and rehabilitation of civil engineering structures in order to effectively manage the function of these structures throughout their lifetime.

The objective of the International Association for Life-Cycle Civil Engineering (IALCCE) is to promote international cooperation in the fields of life-cycle civil engineering for the purpose of enhancing the welfare of society (http://www.ialcce.org). For this reason, it was deemed appropriate to bring together all the very best work that has been undertaken in the field of life-cycle civil engineering at the Third International Symposium on Life-Cycle Civil Engineering (IALCCE 2012) held in one of Vienna's most famous venues, the Hofburg Palace, October 3-6, 2012. The first International Symposium on Life-Cycle Civil Engineering (IALCCE 2008) was held in Varenna, Lake Como, Italy (June 10-14, 2008), and the Second International Symposium on Life-Cycle Civil Engineering (IALCCE 2010) was held in Taipei, Taiwan (October 27-30, 2010).

IALCCE 2012 has been organized on behalf of the IALCCE under the auspices of the University of Natural Resources and Life Sciences, Vienna (BOKU). This four-day symposium encompasses all aspects of life-cycle civil engineering. The interest of the international civil engineering community in fields covered by the IALCCE has been confirmed by the significant response to the IALCCE 2012 call for papers. In fact, over 600 abstracts from 53 countries were received by the Symposium Secretariat, and approximately 60% of them were selected for publication. Contributions presented at IALCCE 2012 deal with state-of-the-art as well as emerging applications related to the key aspects of the life-cycle civil engineering field.

All major aspects of life-cycle engineering are addressed, including aging of structures, deterioration modeling, durable materials, earthquake and accidental loadings, sustainability, fatigue and damage, structure-environment interaction, design for durability, failure analysis and risk prevention, lifetime structural optimization, long-term performance analysis, performance-based design, service life prediction, time–variant reliability, uncertainty modeling, damage identification, field testing, health monitoring, inspection and evaluation, maintenance strategies, rehabilitation techniques, strengthening and repair, structural integrity, decision making processes, human factors in life-cycle engineering, life-cycle cost models, project management, lifetime risk analysis and optimization, whole life costing, artificial intelligence methods, bridges and viaducts, high rise buildings, offshore structures, precast systems, runway and highway pavements, tunnels and underground structures.

Life-Cycle and Sustainability of Civil Infrastructure Systems contains the lectures and papers presented at the Third International Symposium on Life-Cycle Civil Engineering. It consists of a book of extended abstracts and a DVD with 344 full papers presented at IALCCE 2012, including the Fazlur R. Khan Lecture, 10 Keynote Lectures, and 333 Technical Papers from 52 countries.

The aim of the editors is to provide a valuable source for anyone interested in life-cycle and sustainability of civil infrastructure systems, including students, researchers and practitioners from all areas of engineering and industry.

Alfred Strauss, Dan M. Frangopol and Konrad Bergmeister
Chairs, IALCCE2012
Vienna, Austria and Bethlehem, Pennsylvania, USA, August 2012

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The Editors take this opportunity to thank all sponsors, whose support contributed to the success of this Symposium. A special acknowledgment must also be given to the University of Natural Resources and Life Sciences, Vienna (BOKU) for organizing and co-sponsoring the Symposium along with the International Association for Life–Cycle Civil Engineering (IALCCE), as well as BOKU's Department of Civil Engineering and Natural Hazards, whose support facilitated the Symposium's organization.

The IALCCE 2012 was conceived in close collaboration between the IALCCE and several members of the National Advisory Committee including Christian Bucher, Hans Georg Jodl, Oliver Englhardt, Peter Maydl, Peter Veit, Jürgen Feix, Günter Hofstetter and Martin Treberspurg.

The Editors would also like to thank several members of the IALCCE Executive Board including Alfredo H–S. Ang, Harald Budelmann, Hitoshi Furuta and Fabio Biondini and of the National Organizing Committee including Stefan L. Burtscher, Stefan Deix, Roman Geier, Markus Petschacher, Michael Reiterer and Helmut Wenzel.

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Finally, the Editors offer special thanks to Andrea Aglibut, Eveline Bruckmüller, Pauline Friedrich, Gregory Hili, Katharina Haider, Evelin Kamper, Alexander Krawtschuk, Janez Schellander, Klaus Voit, Oliver Zeman and the entire team of the Organizing Secretariat whose contributions were crucial for the successful organization of this Symposium.

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FAZLUR R. KHAN LECTURE

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Fazlur Khan's legacy: Towers of the future

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ABSTRACT: Dr. Khan's contributions to the design of tall buildings have had a profound impact on the profession. Kahn had a unique understanding of forces, materials, behavior, as well as art, literature, and architecture. Long before there was widespread focus on environmental issues, Kahn's designs promoted structural efficiency and minimizing the use of materials resulting in the least carbon emission impact on the environment. Kahn was interested in the performance of structural systems over an expected life; recognizing a building's life-cycle and issues of abnormal loading demands, he developed concepts to apply to severe wind environments as well as early concepts of seismic isolation of structures. These system ideas have led to the development of other concepts which have yielded buildings much taller than those considered by Kahn. His ideas have inspired others to expand the possibilities in tall building design, life-cycle engineering and the effects of the structures on the environment.

Dr. Khan's contributions to the design of tall buildings have had a profound impact on the profession. Kahn had a unique understanding of forces, materials, behavior, as well as art, literature, and architecture. Long before there was widespread focus on environmental issues, Kahn's designs promoted structural efficiency and minimizing the use of materials resulting in the least carbon emission impact on the environment. Kahn was interested in the performance of structural systems over an expected life; recognizing a building's life-cycle and issues of abnormal loading demands, he developed concepts to apply to severe wind environments as well as early concepts of seismic isolation of structures. These system ideas have led to the development of other concepts which have yielded buildings much taller than those considered by Kahn. His ideas have inspired others to expand the possibilities in tall building design, life-cycle engineering and the effects of the structures on the environment.

Inquisitive as a child, Fazlur Rahman Kahn was interested in form and how forms could be made. He created objects with mud, clay and sand, and was interested in mechanical objects, learning that some where fragile and could easily be broken if they did not have the required strength. He was passionate about literature and poetry and had a special interest in societies, particularly those related to his family's background and the people of Bangladesh. As a young student Kahn was interested in studying physics but his father encouraged him to focus on applying his mathematical skills to his practical and mechanical interests.

Several of Kahn's early works were focused on the design of bridges. After graduating with a civil engineering degree from the University of Dhaka previously known and the University of Dacca, East Pakistan, he worked as an engineer for the Design Division, Communications and Buildings Department of the Government of East Pakistan. After completing his graduate studies at the University of Illinois, he began his career at SOM, initially hired to design highway and railroad bridges under the direction of the U.S. Air Force Academy. In the late 1950's Kahn returned to his home country to be the Director of the country's Building Research Center. However, the offer was subsequently withdrawn and he went on to become an Executive Engineer with the Karachi Development Authority. Because he felt that his technical abilities where not fully utilized, Kahn returned to SOM in 1960 where he spent the rest of his career developing arguably some of the most important structural designs of the century.

It is interesting that Kahn's favorite poet was Rabindranath Tagore and that his favorite poem was the "Tagore Song," which begins with the Bengali lyric "This is your beginning and my end. The flow (of life) continues mixing both of us." So much of what is considered in recent developments of structural optimization, efficiency, and life-cycle considerations is based on flow—the flow of forces, material, and energy. The future of structures is to design for this flow and create structures that behave naturally without damage when subjected to extreme loading conditions.

Renewal and opportunity followed the Great Chicago Fire on October 9th, 1971, first through Chicago's World's Columbian Exposition schedule for completion in 1892 (four centuries after the discovery of the Americas by Columbus) but finished in 1893 with the development of new ideas from Daniel Burnham, William Holabird, Louis Sullivan, John Wellborn Root and others. Following the exposition, the first skyscraper age as well as the era of the First Chicago School emerged corresponding with the early use of structural steel and advances in structural engineering. The age ushered in the idea of structural frames clad with exterior wall systems and vertical transportation through passenger and freight elevators. The first skeletal form with a glass and structured façade was designed by William Le Baron Jenny, a civil engineer that practiced as an architect and considered the father of the First Chicago School, through the Leiter Building (1879), and later in the Home Insurance Building (1885), which is considered the world's first skyscraper. Jenny, Sullivan, and Root, among others, designed structures that where utilitarian, economical, and free of excessive ornamentation. Root's Monadnock Building completed in 1891 was the tallest in the world at the time with 16 stories of load bearing masonry. It is still the tallest load bearing masonry building in the world and an excellent example of a structural response to force flow through gradually increasing widths and depths of the masonry walls as the building meets its foundations.

The second skyscraper age was one that used steel construction to new heights while seeking aesthetic inspiration from classic historic models including style and ornamentation from Greek and Roman monuments. Especially in New York City, corporate owned skyscrapers became a symbol of strength and prosperity. The Chrysler, Empire State, and Rockefeller Center Buildings are important symbols of this period.

After a sharp decline in construction in the period leading up to and following World War II, construction of tall buildings began again in the late 1950's. The third skyscraper age and the formation of the Second Chicago School were dominated by European architects such as Mies van der Rohe and Le Corbusier. Heavier masonry facades were replaced with metal and glass, and art deco aesthetics were replaced with the International Style which emphasized the expression of structure by exposing it on the exterior of the building. Myron Goldsmith, a student of Mies became an important contributor to this modernist movement. Khan was holistically and conceptually driven by the modernist movement.

Many buildings designed in the late 1970's lacked a particular style and recalled ornamentation from earlier buildings designed during the second age of the skyscraper. The fourth age of the skyscraper ignored the environment and loaded structures with decorative elements and extravagant finishes. Sculptural imagery and monumental expression dominated architectural practices bored with modernism. Khan strongly opposed this approach to design and considered the designs to be whimsical rather than rational. Most importantly he considered the work to be a waste of precious natural resources.

Khan's scathing assessment of post-modernism was summarized in his written address to the Architecture Club of Chicago in 1982, accepting his unprecedented election to President of the Club. Khan died before the address could be given. Khan wrote "Today it seems the pendulum has swung back again towards architecture that is unrelated to technology and does not consciously represent the logic of structure. Nostalgia for the thirties and even earlier times has hit a large segment of the architectural profession; in many cases façade making has become the predominant occupation. It is apparent that postmodernism in architecture is very much the result of the architect's lack of interest in the reality of materials and structural possibilities: the logic of structure has become irrelevant once again. This attitude in architecture suits many engineers because of their overspecialization in engineering schools which treat the solution of the problem as the ultimate goal, and not the critical development of the problem itself."

Fazlur Khan is mostly known for his work in tall building structural systems and their integration with architecture. What is less known is his interest and work in the area of life-cycle engineering and ideas of protecting structures from abnormal loading, particularly strong earthquakes. Khan and Mark Fintel conceived ideas of dissipating energy through a shock-absorbing soft-story concept that would be introduced into a structure's first story above grade. This concept was a precursor to the widely accepted seismic isolation systems used today for the mitigation of the strong ground motion effects on structures.

The shock-absorbing concept introduced stability walls topped with neoprene pads and deformable steel cables into the first story of a structure to dissipate energy and isolate upper floors from any damage. Instead of designing the entire structure for high seismic forces, the lowest story above grade was designed to distort when subjected to an earthquake while filtering out imposed forces to upper levels. The primary premise of the idea was to design the upper stories to remain elastic, minimizing damage and thus increasing the structure's life. Columns in this system are designed for P-delta effects caused by gravity loads applied eccentrically based on relative movement. Steel cables played an important role in acting to self-center the structure following ground motions.

This concept is still very important today when considering continuous use following a major seismic

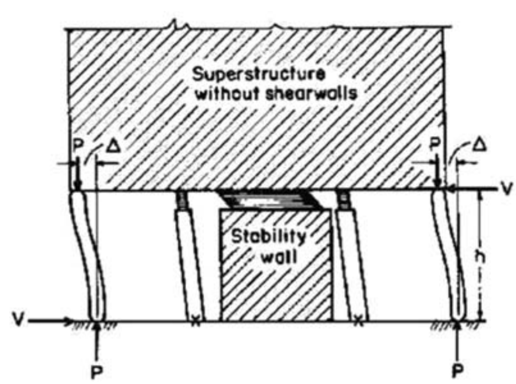

Figure 1. Sketch of a shock-absorbing first story during seismic motion. The columns and neoprene pads were expected to provide restoring forces (Drawing from Khan and Fintel, "Shock-Absorbing Soft Story," 1969).

event. The seismic isolation systems developed in the years following this idea have led to many sophisticated systems including isolators made of high-damping rubber, lead-rubber, and stainless steel among others.

Seismic isolation energy dissipation systems have increased in complexity. In many taller structures seismic isolation is used with displacements limited by dampers that range from viscous elastic devises to coil and offset linkages. Finally, friction pendulum butterfly bearings have been developed to resist uplift, and are particularly useful in taller buildings where gravity load can be overcome by uplift loads due to wind and seismic events.

Objects in nature are designed so that the least energy is expended when work is done by them. For structures to be most efficient, minimum strain energy will result in the minimum expenditure of energy to resist loads. In minimizing the energy, forces and deformations should be distributed as evenly as possible throughout the structure through a synergetic placement of material. Forces will flow through the easiest and shortest load path natural to the structures form. Khan sought to define natural load path to create systems demonstrating optimal performance.

Khan was empathetic to structural behavior and the dichotomy between emotional mysticism and scientific rationalism. Khan revealed that he often felt that he was himself the building when he was designing a building project.

Fazlur Khan and Bruce Graham developed three elegant designs that demonstrate proportioned structure with the flow of force. The exterior columns in the 52-story One Shell Plaza, Houston, Texas (1971), were proportioned to resist gravity loads and control relative creep deformations between the core and exterior columns. Reinforced concrete columns were enlarged toward the outside of the building rather than the inside. The reinforced concrete spandrel beams in the perimeter tubular frame were designed with variable widths in plan to match the column depths. Spandrel beam depths, also proportional, decreased over the height.

Instead of introducing a depth transfer girder system for some tower columns above the lobby space at the base of the 26-story Two Shell Plaza, Houston, Texas (1972), Khan and Graham created a natural load path of reinforced concrete arches where deep spandrel beams carry vertical loads through shear. Face dimensions of columns remain constant over the height of the building.

The final example of flow is illustrated in the 21-story Marine Midland Bank where reinforced concrete column sizes gradually increase as load is distributed to just a few columns at the lobby level. Perimeter tubular spandrel beams increase in depth in an upward manner in elevation. The columns and beams near areas of load transfer increase in dimension both in and out of plan. Column face dimensions are almost impossibly thin over the height of the building—particularly at the top of the building at the mechanical penthouse. Columns were removed from the corner. Graham conceived of the architecture as a demonstration of how a building should land on its base.

Khan's address to the Architectural Club in 1982 ended with signs of optimism. He concludes "but logic and reasoning are strong elements of human existence, always important when man must transcend into the next level of refinement. There are already some signs of that happening in architecture. New structural systems and forms are beginning to appear once again and with them new architectural forms and aesthetics. The pendulum of structural logic in architecture continues to swing."

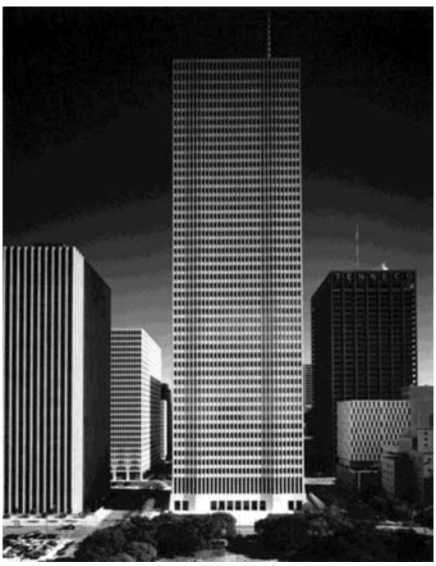

Figure 2. One Shell Plaza, Houston, TX.

Figure 3. Two Shell Plaza, Houston, TX.

Since Khan was deeply interested in technology and its effects on society, the developments of his work lead us to a more comprehensive consideration for structures that will define the next age of the skyscraper. The next era of skyscrapers will focus on the environment including performance of structures, types of materials, construction practices, absolute minimal use of materials/natural resources, embodied energy within the structures, and perhaps most importantly, a holistically integrated building systems approach which will shape this movement. These buildings will be designed to respond to any loading without damage, and regeneration rather than only consumption will begin to appear. LEED is only the start in raising awareness of responsible design approaches, and other ideas will develop with requirements specific to limits of embodied energy for buildings or financial incentives for those that reduce them.

The impetus behind the concept for the 232 meter-tall Jinao Tower in Nanjing, China, was an integrated idea where the perimeter of the structure forms that basis of an energy efficient double wall system, exterior wall enclosure support, and the primary lateral load supporting system. By simply incorporating an offset bracing system using a conventional steel pipe, 55% of the reinforced concrete was eliminated from the core, 40% of the concrete was eliminated from the perimeter frame with a net overall material savings of 20% of the entire structure. The perimeter steel pipe system attracts up to 60% of the shear due to the lateral load near the top of the building lateral load. One and a half days of construction time was saved on each of the 56 stories because of this reduction of material. For this 105,000 square meter structure, the total reduction of carbon emitted into the atmosphere by the structure only was reduced from 47,780 tons of CO_2 to 39,300 tons (28% reduction) or the equivalent of keeping 600 cars off of the road for one year.

Figure 4. Marine Midland Bank, Rochester, NY.

Slots in the folded exterior wall system takes advantage of positive and negative pressures created by applied wind loads. Air flows within the double wall cavity from windward to leeward sides draws moving warm air heated by the sun on one face to the other cooler face not exposed to the sun.

In considering a structure's life and its effects on the environment, structural solutions, building orientation and integrated systems are important considerations. To achieve a high level of sustainability, the structure must allow the entry natural light and natural ventilation, air distribution from below the floor, shaded outdoor spaces, green roofs and minimal material use. For example, shading provided by the bracing and buttress walls on the south facades significantly reduces the need for mechanical cooling of towers over their operational life.

For the Poly International Plaza, Guangzhou, China, the double line of diagonal braces on the south facades of the towers provide significant stiffness in the east-west direction of the towers (long direction), but, more importantly, act to anchor the structure in the north-south (short direction) of the towers functioning in tandem with the buttress walls as a "stressed skin" through the action of the outriggers. The stressed skin acts as a flange to resist compressive and tensile loads applied in the short direction.

Sustained gravity loads are used as ballast within the primary lateral load resisting elements achieved through long-span reinforced concrete framing where compressive loads are placed on both the north and south facade structural systems. Openings were introduced into the mid-height of the two towers to provide an area of refuge during an emergency and to provide an aperture for predominant winds to pass through the structure. The screen enclosures at the top of the towers consist of tilted individual panels, providing open paths for wind to pass through, reducing wind loads on the top of the towers where they would induce the highest demand on the structure. Through balancing load on the structure by using an efficient bracing system on the south facades and the openings that allowed for winds to pass through the structure, a 15% savings of structural materials was achieved when compared to conventional structures of the same height (even with slender forms).

The initial architectural concept for the Goldfield International Gardens, Beijing, China, included what appeared to be elevations containing a random spacing of transparent and opaque elements. Elevations of the towers (150 meter and 100 meter-tall) were conceived to control heat gain on the facades while providing the required lateral stiffness to resist wind and seismic loads. The elevations reveal patterns, albeit asymmetric, but repetitive.

The idea reflected on the large-scale super-frame is similar to Khan's 1980 proposal for the Chicago World Trade Center. The superframe was stiffened to provide enough lateral and gravity resistance using a concept of the infilled frames. The mega-frame is formed by a structural bay 9 meter-wide and three stories high (12 meter-high or three levels at 4 meter each). Each mega-panel is infilled with an asymmetrical frame only introduced to provide the appropriate lateral stiffness. The frame consists entirely of reinforced concrete with mega-frame section sizes of 1200 mm × 1300 mm for columns and 1200 mm × 700 mm for beams. The infill frames include 900 mm × 700 mm column and beam elements. The main tower is 150 meter-tall and incorporates the screen frame into two of the four facades with the other two facades regular, incorporating frames with columns typically spaced at 6 meter on-center. The second tower is 95 meter-tall and also incorporates the screen frame into two of the four facades.

Discontinuous diaphragms located in plan along the two infilled frame building facades on two-thirds of the floors were engineered to transfer seismic and wind loads between the interior reinforced concrete shear wall core and perimeter frames. Large composite girders where used to collect and transfer these loads from the interior floor diaphragms to the frames. The open slots at the discontinuous diaphragms allowed the building façade to pass behind the frames enabling light and shadow to interact with the screen frames. On every third floor, horizontal diaphragms were fully engaged with the perimeter screen frames.

These expressed asymmetrical frames incorporated calibrated stiffness to tune the structure so that frames on all sides had similar stiffness avoiding adverse torsional effects. The mega-frames and infill elements required the most advanced detailing and member sizing to ensure strong column-weak beam behavior and the required ductility to resist strong seismic loadings. Complex, non-linear pushover-analyses were performed to confirm the ductile behavior and it was found that the asymmetrical building frames behaved in a superior manner even when compared to the conventional frames.

Finally, the design introduced construction joints into the infill frames to keep any gravity loads from entering into the infill frames during construction as a result of creep, shrinkage, and elastic shortening of the mega-frames. During construction vertical in-fill columns were held out of the initial frame pours to achieve the same behavior as the construction joints.

The most efficient structure is one that emerges from individual elements absent of internal bending – the greatest resistance with least material. For instance, the natural behavior of a fixed-base cantilever subjected to lateral loads is to bend. If, however, this bending could be resolved into a mesh of individual axial "strings" capable of only resisting tension and compression, the most effective structure would emerge. At a small scale, these "strings" would transform larger forms into a smaller single element repeated throughout.

When compared to Khan's work with the conventional tube frame, far more efficiencies are achieved through virtually eliminating all bending. The total cantilevered displacements are essentially due to column shortening (compression) or elongation (tension).

The structure is one that is self-healing if violated naturally or unnaturally. Neighboring elements within the structure assume newly imposed loads if violated in a seismic event or a man-made explosive attack. The structure would remain stable through its inherent redundancy.

It is particularly useful to consider emergence theory concepts for structured forms with fluid definitions. A topographic strain density analysis can be performed with imposed boundary conditions and external loadings. Iterative computer analysis programs using finite element techniques result in a natural response by placing material density only where it is required.

The analysis is started by placing a uniform thickness of material over the entire structure. The analysis then looks for areas that material is not required and moves that material to other areas of higher demand. The solution is not obtained after the first analysis; most frequently the results do not converge on an optimal solution until several hundred analyses are performed. The internal structured forms that emerge are intuitive. The final response to the internal forces may be a discrete placement of material or variations of material thickness along the membrane form.

The premise that objects in the universe are continuously in motion encourages more advanced theories for the performance of our structures where moving parts are essential. These moving parts reduce if not eliminate damage when subjected to extreme load.

Significant advancements have been made in the design of structures in regions of high seismic risk, but most developments have been focused on life safety with modest focus on performance or long-term economic viability. Structures that naturally coexist with their site conditions produce the most efficient designs, the most cost-effective long-term solutions, and have the least impact on the environment.

A goal of the design community should be to create structures that behave elastically even when subjected to the most extreme seismic events. Structures would be designed based on natural behavior principles rather than conventional approach. Future building systems be designed to dissipate energy, deform elastically instead of plastically, and allow the building to be placed immediately back into service after an earthquake.

Allowing controlled movement with the dissipation of energy in structures during an earthquake is crucial. This movement can occur at its base or within the superstructure itself. Seismic isolation is an excellent solution to decoupling structures from strong ground motions—even with taller towers—provided that issues of uplift are controlled and there is enough benefit in period separation of the tower relative to the isolation system.

When structures are fixed to their foundations, this movement must be designed to occur within the joints of the superstructure. Pin-Fuse Seismic Systems are designed to maintain joint fixity throughout the typical service life of the structure. When a significant seismic event occurs, forces within the frame causes slip in the joints through friction-type connections. This slippage alters the characteristics of the structure, lengthens the structure's fundamental period, reduces the forces attracted from the ground, and provides energy dissipation without permanent deformation. Pin-Fuse systems can be introduced into moment-resisting frames, link beams or braced frames.

The design goal for the developer-led 375 foot-tall, 375,000 square foot 350 Mission Street Tower in San Francisco is to be the most innovative office building in America. LEED platinum is only a benchmark for design with the premise that the most advanced thinking would be incorporated into the sustainable design goals for the tower. An offset core allows for natural daylight on the three sides of the building that are not abutted against neighboring towers and enables fresh air intake for each floor. The urban design concept for this project blurs the boundary between public and private realms with the ability to completely open the lobby space to the street.

Social media and movable seating in the lobby activate the space. Interactive art will include metrics of energy use, public events, public transportation etc. Long, column-free spans use only flat formwork and allows for spaces that do not require finished ceilings. Daylight is maximized into the space with 13′-2″ floor-to-floor heights, 16″ raised floor, and a 10′-8″ clear height in the space. Originally conceived of assembling and inserting plastic water bottles capped to trap air into the structural floor framing system, the Sustainable Form Inclusion System (SFIS) displaces concrete not required for the structure. The system uses post-consumer, non-recyclable, light-weight waste products assembled and placed to reduce mass and keep this unusable waste out of the landfills. This reduces demands on column and wall systems as well as foundations, particularly important in regions of high seismicity.

With the use of SFIS for the 45 foot clear spans, the structure is built with a full structural depth of 14 foot and flat top and bottom slab construction. Considering the displaced concrete and the long-span flat slab system 35% less concrete is required when compared to conventional systems. This results in a carbon emission savings of 20% for the initial construction and the use of a significant amount of post-consumer plastics that would otherwise be placed in landfills.

In addition, if the performance based-design core-only lateral system were to incorporate a device such as the Link-Fuse Joints™ for the link beams that are used to reduce if not eliminate damage in seismic events, an additional 10% savings in carbon is realized over its 50-year service life.

The idea of a component-assembled form inclusion system evolved into the use of a product that could be mass-produced while diverting one of the most difficult wastes to manage from the waste stream. Ground waste polystyrene (Styrofoam) is placed in a lightweight mortar paste and formed into simple dimensional blocks. These blocks are place like tiles into the reinforced concrete floor system. Post-tensioning is used to further reduce the amount of concrete required for the structure.

The design premise for the 415 meter-tall Al Hamra Tower, Kuwait, is a fluid response to force flow while providing an integrated response to the harsh desert sun was the. Punched reinforced concrete engineered to only allow indirect sunlight into building spaces is used at the south façade while resisting both lateral and gravity loads. This hyperbolic parabola form is integrated into a regular closed form shear wall core to resist self-imposed twist due to the structure's own weight. Parametric modeling was used to define the shape of the walls and used directly for self-climbing formwork systems.

A regularly spaced structure frames the west, north, and east faces designed to accept maximum daylight and best views. A regular column grid is designed for a repetitive office module. The south façade was reinforced with complex varying geometries to allow force flow over the height of the tower while allowing for angular cuts in the wall system.

Parametric modeling was used to define the base of the tower where the structured flared outward from the straight face above and was designed to create a 24 meter-tall lobby space. The modeling was used to define the geometry for the non-linear buckling analysis of perimeter columns and was later used to define the geometry of the formwork system. The primary load bearing columns were braced three-dimensionally with flared bracing both in the plan of the façade and within the tower spanning from the exterior frame to the interior shear wall core. The lamella structure includes a combination of structural steel and concrete at the perimeter frame and all-reinforced concrete for the framing over the lobby space.

Exterior wall systems for structures represent the single greatest opportunity to consider flow and interaction between structure and building service systems.

Figure 5. Exterior view, Poly International Plaza, Beijing, China.

Hundreds of millions of square feet of occupied area are enclosed each year by a system that essentially provides protection from the elements and internal comfort. A closed loop structural system integrated with the exterior wall system that includes liquid-filled structural elements such as pipes could provide a thermal radiator that when heated during the day could be used for building service systems such as hot water supply or heat for occupied spaces, especially during the evening hours. A solar collection system could be integrated into the network and incorporated into double wall systems where it can be used to heat the internal cavity in cold climates.

Transparent photovoltaic cells could be introduced into the glass and spandrel areas to further capture the energy of the sun. When storing fluid in structural systems of great height, pressures within the networked vessel become very large. With this level of pressure, water supply systems to the structure or to neighboring structures of lesser height could easily be supplied without requiring additional energy to move the water. Constant low flow through these systems would prevent the liquid from freezing.

Liquid within the networked system could act to control motion with fluid flow acting to dampen the structure when subjected to lateral loads due wind and earthquake events. In addition, liquids at high pressure could add significantly to the axial stiffness and stability of structural members subjected to compression, increasing capacities without increasing structural material by creating capped compartments. Combining ultra-high strength tensile materials such as carbon fiber fabricated into closed circular forms where loads are primarily resisted by hoop stress with the liquids under ultra-high compressive stress would likely result in greatest efficiency in resisting applied load.

The concept of flow can be further developed into structures that are interactively monitored for movement. Through the measurement of imposed accelerations due to ground motions or wind, structures could respond by changing the state of the liquid within the system. For instance, the structure could use endothermic reactions to change liquids to solids within the closed network. Sensor devices could inform structural elements of imminent demand and initiate a state change in liquids that would be subjected to high compressive loads where buckling could occur. In the simplest sense, water within the system could be frozen for additional structural rigidity.

In a more sophisticated application, when imminent demand from ground motions is sensed, electromagnetic flow could be used to create a separation of the superstructure from its foundations. Temporary levitation created by electromagnetism provides frictionless seismic isolation.

In cases where base isolation is not practical, pneumatic dampers that incorporate flow of compressed air could be strategically placed within frames to increase damping and consequently reduce the forces attracted from the ground.

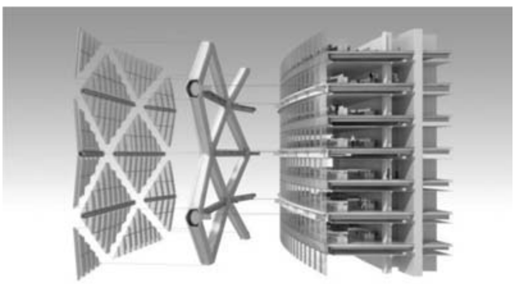

Figure 6. Exploded view of Integrated Systems.

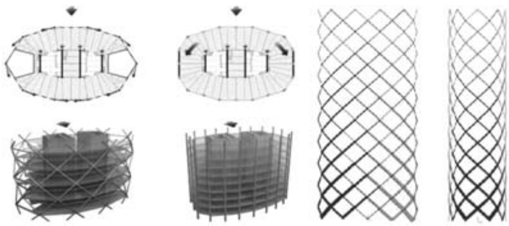

Figure 7. Force flow diagrams, Poly International Plaza, Beijing, China.

Figure 8. Interior column-free space.

Figure 9. Elevation at building base, Poly International Plaza, Beijing, China.

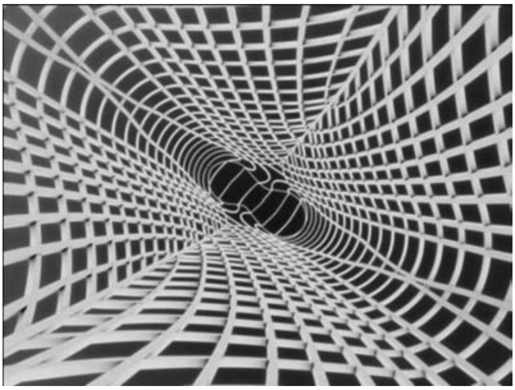

Figure 10. Fluid structural forms of the future.

The Poly International Plaza, Beijing, China, offers a unique insight into towers of the future. The structure makes significant steps in the direction of the most advanced technologies. Because of significant levels of seismicity and the concern for life-cycle seismic performance, concrete was used in the exterior tubular frame. In the future, however, this framed system could be utilized for a fully integrated approach where structure, architecture, and mechanical systems are completely synergetic.

The study of these emerging forms as they interact with the architecture (overtly or covertly) will only yield further opportunities to explore light, space, structure and a new relationship that combines them all in an ephemeral solution. The investigation into the flow of material that can be manipulated to adhere to a seismic, temperature or safety condition can only inform us of new ways to design and build. The combination of these two studies, emergence theory and flow can give us the basis for new structures that will no longer limit themselves by being static. They can organically emerge as a singular system that from the ground up provides efficiency in material, intelligence in response to unknown forces, and a form that is derived from nothing but the purest and most absolute function.

Khan's legacy for innovations in tall building design has led to new ideas in building design. The concepts that Khan developed in his career have seeded ideas for projects completed and many of those yet to be built. The environment perhaps is the next major design consideration where designers initiatives consider this as important as life-safety goals. The environmental design platform must be based on life-cycle engineering and superior performance.

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KEYNOTE LECTURES

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Lessons from the 2011 Great East Japan Earthquake: Emphasis on life-cycle structural performance

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ABSTRACT: This keynote paper presents the damage of structures and infrastructures in the Tohoku Region during the Great East Japan earthquake and tsunami of March 11, 2011. Field investigations after this earthquake proved the effectiveness of upgrades of seismic specifications and seismic retrofit. Meanwhile, some of existing bridges without seismic retrofit were severely damaged due to strong ground motions. In addition, many superstructures were completely washed away by the tsunami, and several substructures were overturned due to scour. Some of reinforced concrete components and steel bearings were severely deteriorated due to chloride-induced corrosion. Life-cycle reliability of bridges in Japan has to be estimated taking into consideration the seismic hazard, tsunami hazard, and hazard associated with airborne chlorides. Lessons from the 2011 Great East Japan earthquake with emphasis on life-cycle structural performance is the topic of this keynote paper.

1 INTRODUCTION

The 2011 Great East Japan earthquake (i.e. Off Pacific Coast of Tohoku Region, Japan earthquake) with the magnitude of 9.0 occurred at 2:46 p.m. (local time) on March 11, 2011. The fault zone extended 450 km and 200 km in the north-south and west-east directions, respectively (Kawashima 2012). The 2011 Great East Japan earthquake caused multiple disasters, including the damage and collapse of structures and infrastructures due to strong ground motions and/or liquefaction, the washout of structures and infrastructures due to the subsequent tsunami, fires, landslides, and the subsequent radiation crisis.

The transportation networks including bridges are one of the most critical civil infrastructure systems when a natural disaster occurs (Decò & Frangopol 2011, 2012, Frangopol & Bocchini 2012). Since the bridge transportation network plays a crucial role in the evacuation of affected people and the transportation of emergency goods and materials, the functionality of the network is necessary to be recovered as soon as possible (Unjoh 2012). A prompt restoration of the critical infrastructure facilities after an extreme event is always a goal of paramount importance (Bocchini & Frangopol 2012a, b). However, bridges may be susceptible to damage during an earthquake event, particularly if they were designed without adequate seismic detailing. Structures built using earlier specifications (i.e., without proper seismic detailing) often lack adequate flexural strength and ductility capacity, and/or shear strength. When subjected to strong ground motions, these structures have the potential to exhibit brittle failure. Several destructive earthquakes in Japan (e.g., 1978 Miyagiken-Oki earthquake, 1995 Hyogoken-Nanbu earthquake, 2003 Sanriku-Minami earthquake, and 2004 Niigataken-Chuetsu earthquake) inflicted various levels of damage on the structures and infrastructures. The investigation of these negative consequences gave rise to serious discussions about seismic design philosophy and to extensive research activity on the retrofit of as-built bridges. The seismic design methodology for new bridges has been also improved. Comparing the damage state of bridges before and after the seismic retrofits, or the performance of bridges designed according to old and latest seismic specifications during the 2011 Great East Japan earthquake, the effectiveness of seismic retrofit and upgrade of seismic specification against the strong ground motions could be investigated.

The giant tsunami due to the 2011 Great East Japan earthquake inflicted substantial damage to many coastal communities in Japan, including their critical port facilities, residential and commercial buildings, and infrastructures. Bridges and embankments on the transportation networks collapsed due to the resulting impulsive pressures of breaking waves and hydrodynamic pressures associated with water velocity. These structures are very vulnerable under tsunami hazards as reported in Saatcioglu et al. (2006) based on the 2004 Indian Ocean Tsunami. Since in the seismic design and retrofit specifications in Japan, the tsunami effects have not been taken into consideration, the code provisions against tsunami need to be established based on the failure mechanism gathered from the tsunami induced damage effects on bridges.

A team of structural and bridge engineers from the Japan Society of Civil Engineers (JSCE) visited the Tohoku Region shortly after the 2011 Great East Japan earthquake to investigate the performance of structures during the disaster (Kawashima et al. 2011). In Tohoku Region, where the effects of the seismic shocks and the tsunami waves due to the 2011 Great East Japan earthquake were felt with very high intensity, many structures and infrastructures were severely damaged or washed away. Based on the field investigation by JSCE, this paper presents ground motion induced and tsunami induced damages to bridges. In addition, life-cycle performance of the coastal structures under seismic and tsunami hazards, and hazard associated with environmental stressors are discussed. Field investigation confirmed that several concrete and steel structures and bridge bearings were seriously corroded. The lessons from the 2011 Great East Japan earthquake with emphasis on life-cycle structural performance is the topic of this keynote paper.

2 2011 GREAT EAST JAPAN EARTHQUAKE

2.1 *Ground motion induced damage*

After the 2004 Niigata-ken Chuetsu earthquake with a magnitude of 6.6, the first seismic retrofit program was initiated for the Tohoku and Joetsu Shinkansen viaducts. The objectives of the program were to prevent shear failure by ensuring that shear strength of steel-jacketed RC columns exceeds the shear corresponding to maximum flexural strength, and/or to prevent the damage to bridge piers which have the cut-off of the longitudinal rebars by improving the flexure and shear resistant using steel jacketing, RC jacketing, or carbon fiber reinforced polymer sheet. After retrofitting the as-built RC columns, these columns can be the prime source of energy dissipation responding to strong seismic attack. The program was completed in 2007 after retrofitting 12,500 columns. In 2009, the second retrofit program for enhancing the shear and flexural strengths, and ductility capacity of the RC columns was initiated and is still in progress.

Under the first retrofit program for Shinkansen viaducts, RC columns were retrofitted if the ratio γ of shear strength to shear force corresponding to maximum feasible flexural strength is less than threshold C_t. Even if $\gamma \geq C_t$, RC columns may exhibit brittle behavior because of the variability of concrete and reinforcement rebar strength, and uncertainties associated with the estimation of shear and flexure strength.

Figure 1 shows the damage to RC columns of Tohoku Shinkansen viaducts in Miyagi-ken and Iwate-ken during the 2011 Great East Japan earthquake. These viaducts were not retrofitted in the first seismic retrofit program. The investigation after the 2011 Great East Japan earthquake confirmed that RC bridge piers were damaged due to the insufficient anchorage length at the cut-off point of longitudinal rebars, or the RC columns of the single-story RC moment-resisting frame failed in shear. These failure modes of RC members observed in the 2011 Great East Japan earthquake are the same as those observed in the past earthquakes.

Figure 2 compares 5% damping response accelerations of the 2003 Sanriku-Minami earthquake with those of the 2011 Great East Japan earthquake. Both ground motions were measured at the same seismic station near the No. 3 Odaki viaducts. As shown in Figure 2, because the fundamental natural period of a single story RC rigid frame ranges between 0.4 sec and 0.6 sec depending on the soil condition and column height, it is reasonable to assume that the response acceleration of the No. 3 Odaki viaducts were nearly the same in the 2003 Sanriku-Minami earthquake and the 2011 Great East Japan earthquake.

Figure 2 also shows the damaged state of the No. 3 Odaki viaducts recorded after the 2003

Figure 1. Example of damages to Tohoku Shinkansen viaducts before retrofit during the 2011 Great East Japan earthquake (Left and right photos were provided by East Japan Railway Company and Dr. Takahashi, Kyoto University, respectively).

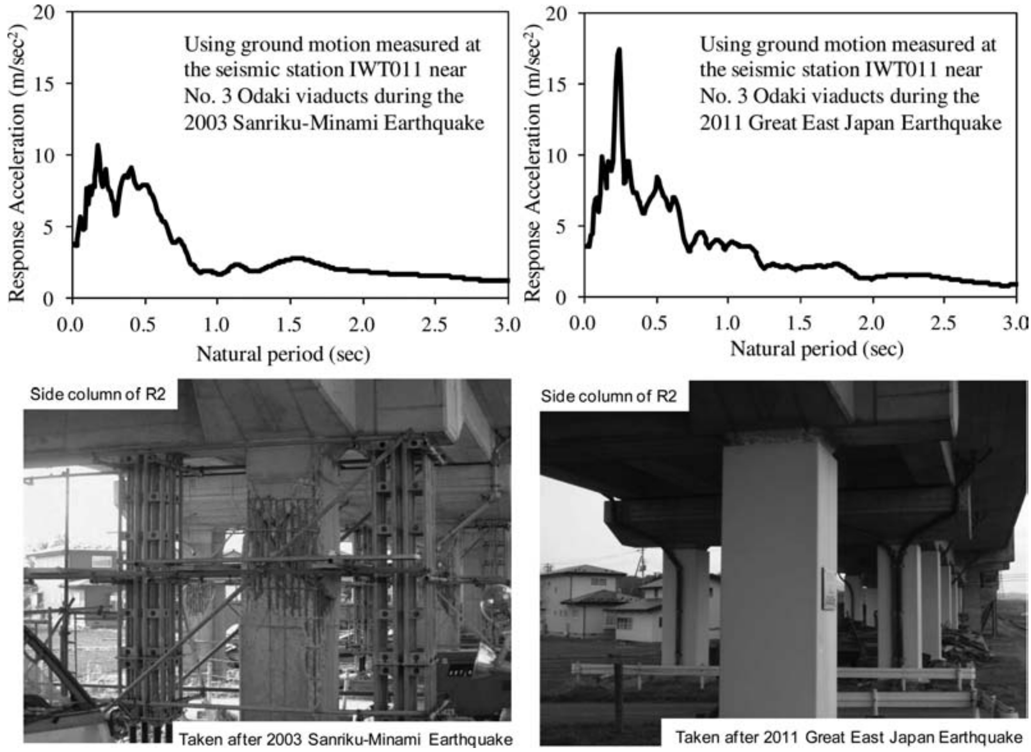

Figure 2. No. 3 Odaki viaduct taken after the 2003 Sanriku-Minami earthquake and the 2011 Great East Japan earthquake (Note: The viaducts were retrofitted before the 2011 Great East Japan earthquake).

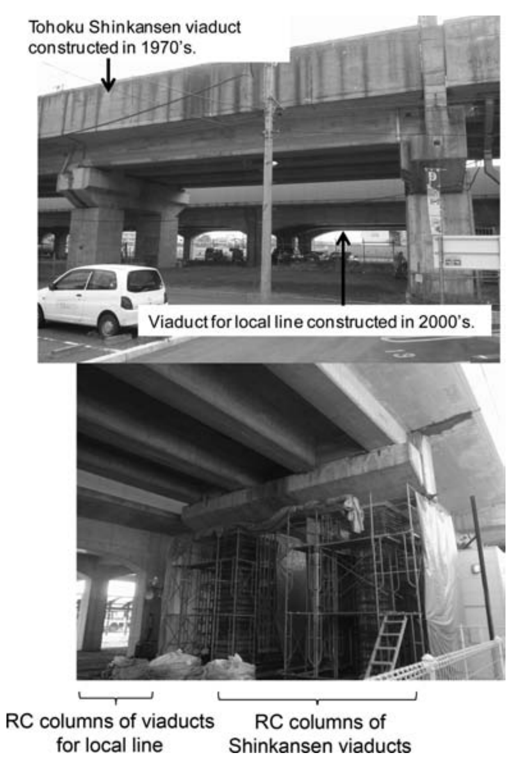

Figure 3. Comparison of damage states of RC columns constructed by 1970's and 2000's.

Sanriku-Minami earthquake and after the 2011 Great East Japan earthquake. Because of deficiencies in the number of ties used to prevent brittle failure, the RC columns of the No. 3 Odaki viaducts failed in shear during the 2003 Sanriku-Minami earthquake. Under the first seismic retrofit program, the viaducts were retrofitted by means of steel jackets for the RC columns so that they had sufficient shear capacity. The retrofitted columns of the viaducts performed well, with almost no damage during the 2011 Great East Japan earthquake. The effectiveness of seismic retrofitting using steel jacketing to prevent significant damage to the Shinkansen viaducts was proved.

In the Nagamachi Area, five kilometers away from the Sendai station, local trains run parallel to bullet trains of Tohoku Shinkansen as shown in Figure 3. The viaducts of both local line and Tohoku Shinkansen are single-story RC moment-resisting frames. However, the local line viaducts were designed according to the seismic specification revised after the 1995 Hyogoken-Nanbu earthquake. While Shinkansen viaducts without seismic retrofit were severely damaged and had large diagonal cracks as shown in Figure 3, local line viaducts had very minor flexural cracks. This proved that the revisions of seismic specifications contribute to improving the seismic performance of bridges.

Since March 11, 2011, there have been many aftershocks in the east Japan. Especially, on April 4, 2011, aftershock with magnitude of 7.2 caused damages of many structures and infrastructures in the Tohoku Region. Although Shinkansen viaducts had been repaired since the March 11, some of repaired RC columns were damaged again due to this aftershock. Figure 4 shows the RC bridge pier damaged due to the aftershock on April 7 under re-repair work, since RC bridge pier repaired after the 2011 Great East Japan earthquake had large diagonal cracks.

2.2 *Tsunami induced damage*

Many bridges were washed away by the tsunami waves due to the 2011 Great East Japan earthquake. Figure 5 shows the collapse of Mizushiri railway bridge in Kesen-numa Line of East Japan Railway Company (JR East) due to the tsunami. This bridge has three simple prestressed concrete (PC) girders with span of about 20 m to 25 m and two RC bridge piers with the height of about 10 m. The distances shown in Figure 5 were provided by laser ranger in-situ measurement. It should be noted that these distances vary considerably.

The middle PC girder (PC girder 2) was displaced more than 200 meters from the original position. Two RC bridge piers and the embankment behind the Abutment 2 were completely destroyed. Since Mizushiri bridge was constructed in 1971, its RC bridge piers have less amounts of longitudinal reinforcement and ties than required by the current code. There may be likelihood of some damages due to strong ground motions before the tsunami arrived. To evaluate the tsunami bridge risk, damages due to strong ground motions have to be considered.

Some of RC bridge piers in road network were retrofitted by means of RC jacket on column against the strong ground motions. Retrofitted RC bridge piers were not destroyed as shown in Figure 6, although

Figure 4. Damage to RC columns repaired after the 2011 Great East Japan earthquake on March 11 due to the large aftershock on April 7.

Figure 6. RC bridge pier with retrofit using RC jacketing which had no damage during the 2011 Great East Japan earthquake.

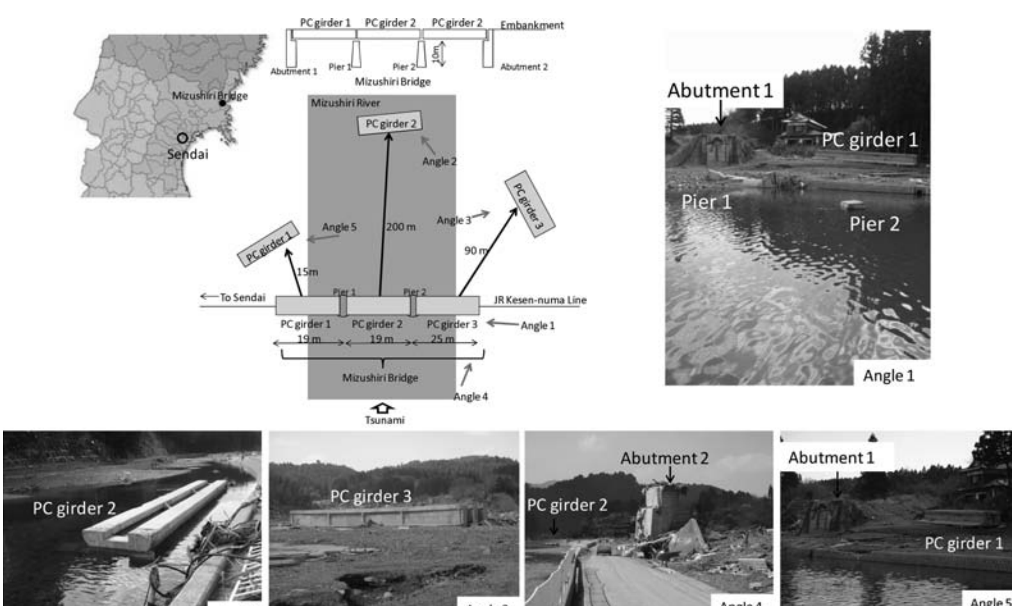

Figure 5. Damage to Mizushiri bridge by the tsunami due to the 2011 Great East Japan earthquake.

Figure 7. Bridges which were not washed away by tsunami

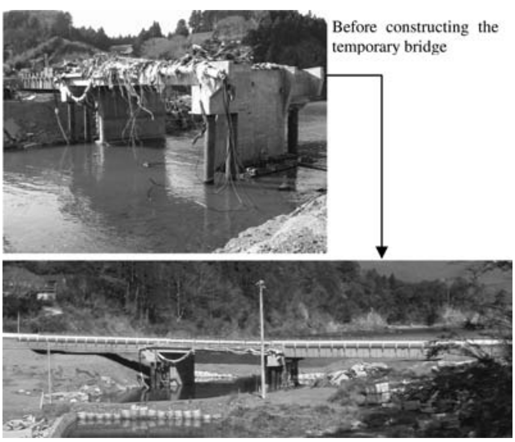

Figure 8. Temporary bridge using the original components

the superstructures were washed away. Retrofitting the RC bridge columns to prevent failure from the strong ground motions can also improve their capacity against the tsunami wave.

Figure 7 shows the bridges which were not washed away by the tsunami. These bridges were located near the coastline and were engulfed by the tsunami. They have common structural features; concrete superstructures with a wider width of the road and shallow girder height, or RC moment-resisting frames. Based on these structural features, the code provisions against tsunami effects need to be established.

Tsunami forces caused the collapse of a number of bridges that formed a vital link between towns in the Tohoku Region. Since this severely constrained post-earthquake disaster recovery, temporary bridges have been constructed at several locations. Abutment backfill of Niju-ichigahama bridge with simple span and steel pile foundation was completely destroyed under the tsunami wave. However, since the original bridge had minor damage due to the tsunami wave, a temporary bridge could be constructed within one month after the 2011 Great East Japan earthquake as shown in Figure 8. Temporary bridges were used for the evacuation of affected people and the transportation of emergency goods and materials. The code provisions against tsunami effects need to established by taking into consideration the recovery time to restore the functionality of the bridge.

3 LIFE-CYCLE PERFORMANCE OF STRUCTURES UNDER SEISMIC HAZARD, TSUNAMI HAZARDS, AND/OR HAZARD ASSOCIATED WITH ENVIRONMENTAL STRESSORS

Structures and critical infrastructure facilities with very high performance standards to prevent any damages and failures from a huge earthquake and/or giant tsunami would be too expensive and totally impractical. Risk assessment is useful in circumstances when an event is very rare yet its consequences are very severe. The tools to be used in risk-informed decision making for performance assessment and optimal life-cycle management of structures and infrastructures under low-probability high-consequence events are rapidly growing.

The performance-based methodology proposed by Cornell (2002) for seismic risk assessment has been widely adopted. The performance levels describe the desired level of structural behavior in terms of structural demand and capacity. The probabilistic distributions for demand, capacity and seismic intensity hazard are considered in the seismic risk assessment. When only the seismic hazard and a single limit state associated with structural damage are considered, the expected risk can be described as follows (Frangopol & Akiyama, 2011)

$$R = C(S) \times Pf[S] \quad (1)$$

$$Pf[S] = \int_{\alpha} \left[-\frac{dH(\alpha)}{d\alpha} \right] P[S|\Gamma = \alpha] d\alpha \quad (2)$$

where $C(S)$ is the consequence associated with the limit state S, $P[S|\Gamma = \alpha]$ is the conditional probability of occurrence of the limit state S given that ground motion intensity Γ (such as peak ground acceleration or velocity) is equal to α (seismic fragility curve), and $H(\alpha)$ is the probability that the ground motion intensity α is exceeded at least once during the time interval T (seismic hazard curve).

Using the fragility curves for bridges before and after seismic retrofit, a comparative seismic risk assessment could be presented. This information is essential for seismic risk management and decision making on retrofit and mitigation strategies. Due to the limited empirical data available, developing the fragility curves for retrofitted bridges based on damage investigation is impossible. Analytically derived fragility curves for retrofitted road bridges were developed by Kim & Shinozuka (2004) and Padgett & DesRoches (2008, 2009). As observed in the 2011 Great East Japan earthquake, the benefits of the seismic retrofit of existing bridges which might exhibit brittle failure were clearly evident in mitigating severe damage. Based on the comparison of risk among existing bridges, the high-priority bridges should be identified.

When estimating the expected risk due to the tsunami, the tsunami fragility curve could be used in Equation (1) as a proper loss estimation tool against a potential future tsunami. Based on the probabilistic seismic hazard analysis, a tsunami hazard curve can also be determined. This curve shows the relationship between hydrodynamic features and the probability of exceedance for a specified period. Annaka et al. (2007) proposed a logic-tree approach to construct the tsunami hazard curve representing the relationship between tsunami height and the probability of exceedance. The tsunami hazard curves are obtained by integration over the aleatory uncertainties, and numerous hazard curves are provided for different branches of logic-tree representing epistemic uncertainties.

Using the tsunami hazard and the fragility curves, the expected risk can be estimated by using Equations (1) and (2). In Equation (2), $P[S|\Gamma = \alpha]$ is the conditional probability of occurrence of the limit state S given that hydrodynamic features Γ (such as tsunami wave height and current velocity) is equal to α (tsunami fragility curve), and $H(\alpha)$ represents the probability that the hydrodynamic feature α is exceeded at least once during the time interval T.

Ellingwood (2006a, b) presented the annual probability of structural collapse in the case where multiple hazards and structural damages are considered

$$P[\text{Collapse}] = \sum_H \sum_D P[\text{Collapse}|D] P[D|H] \lambda_H \quad (3)$$

where λ_H = annual mean rate of occurrence of H, $P\ [D|H]$ = conditional probability of damage state D given H, and P [Collapse$|D$] = probability of disproportionate damage or collapse given damage state D.

Li & Ellingwood (2009) presented a framework for multihazard risk assessment using hurricane and earthquake hazards. Basu & Prasad (2012) estimated the seismic risk assessment of RC bridges in flood-prone regions. The regional multihazard scenario was characterized by combining scour resulted from regional flood events of different intensities with a suite of earthquake ground motions representing regional seismicity. While hurricane and earthquake, or flood and earthquake are quite distinct events, tsunami intensity correlates with the magnitude of oceanic earthquake. The effect of damage to bridge components under stronger excitation on the reduction of the capacity for tsunami wave load must be considered given higher tsunami wave height in the tsunami fragility analysis.

In addition, the effect of corrosion on the deterioration of the capacity of bridges under seismic and tsunami hazards has to be considered. Field investigation conducted after the 2011 Great East Japan earthquake confirmed that some of RC structures and steel bearings were severely deteriorated due to chloride-induced corrosion as shown in Figure 9.

Figure 9. Corrosion of reinforcing bars and bearings attacked by chloride.

Akiyama et al. (2011) presented a computational procedure to integrate the probabilistic hazard associated with airborne chlorides into life-cycle seismic reliability assessment of RC bridge piers. Details of hazard associated with airborne chloride were reported in Akiyama et al. (2010). They pointed out that the seismic demand depends on the results of seismic hazard assessment, whereas the deterioration of seismic capacity depends on the airborne chloride hazard. In seismic and tsunami risk assessment, it is important to take into consideration the effect of material corrosion on structural performance.

Based on the tsunami-induced damages to bridges during the 2011 Great East Japan earthquake, possible failure mode of bridge under tsunami hazard is estimated as described in Figure 10. The effect of damage due to strong excitation and tsunami wave load, corrosion, and scour on the behavior of bridge needs to be incorporated in the tsunami risk assessment. Future seismic specifications may require devices to prevent the superstructure collapse from giant tsunami. However, in that case, the piers are provided with additional lateral force from these devices during the tsunami. Even though bridge pier designed by the current seismic specification has larger lateral strength and ductility capacity, it may be necessary to provide additional lateral strength for the pier to prevent the collapse of superstructure due to tsunami. A capacity design procedure of bridge under tsunami hazard is needed to obtain the hierarchy of resistance of the various structural components and improved performance to avoid the catastrophic damage and ensure a prompt restoration (Priestley et al. 1995, Akiyama et al. 2010). Optimal code provisions against seismic and tsunami hazards must be established using a risk-based and a resilience-based life-cycle design perspective (Ang & De Leon 2005, Ellingwood 2005, 2006a, b, Frangopol & Liu 2007, Frangopol 2011, Decò & Frangopol 2011, 2012, Frangopol & Bocchini 2012, Bocchini & Frangopol 2012a, b).

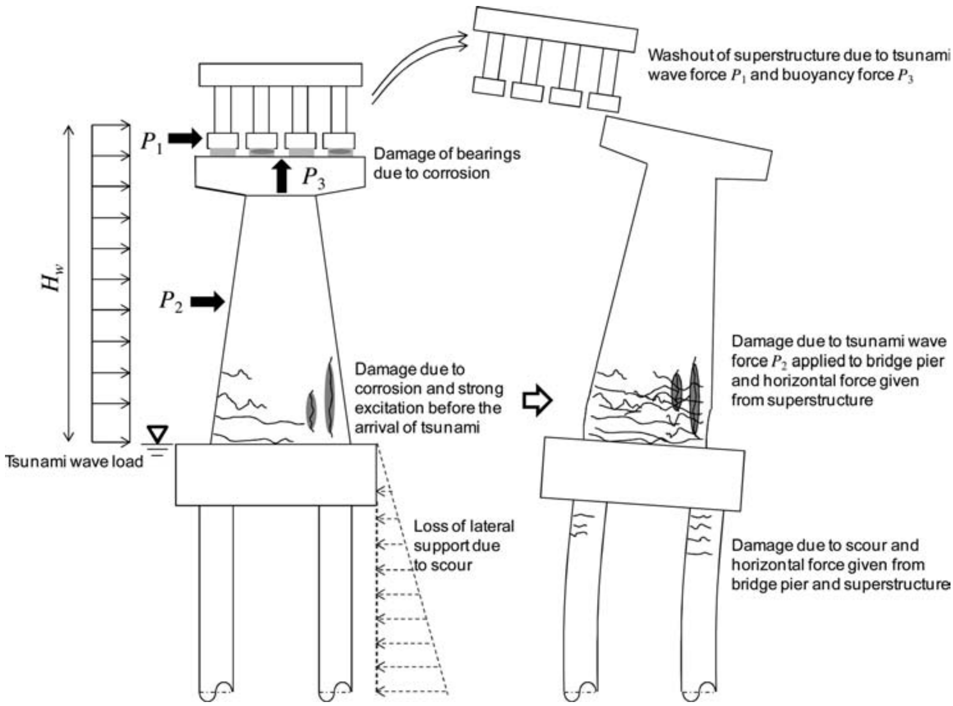

Figure 10. Possible failure mode of bridge under tsunami hazard.

4 CONCLUSIONS

1. Since the 1995 Hyogoken-Nanbu earthquake, seismic retrofit has been conducted for RC bridge piers which have insufficient shear reinforcement and/or have cut-offs of longitudinal rebars without adequate anchorage length. However, there were still a significant number of bridge piers which required retrofitting before the 2011 Great East Japan earthquake. As a result, some as-built bridges exhibited similar failure modes as observed in the past earthquakes.
2. Comparing the damage state of bridges before and after the seismic retrofits, or the performance of bridges designed according to old and latest seismic specifications during the 2011 Great East Japan earthquake, the effectiveness of seismic retrofit and the improved seismic specifications against the strong ground motions was proved.
3. It is necessary to establish a seismic retrofit strategy to improve the seismic performance of existing bridges in order to minimize the difference between their seismic performance and the performance required according to the latest seismic specifications. Seismic hazard, importance of bridge, failure mode of components, and seismic specifications used need to be considered when determining priorities for seismic retrofit.
4. Many superstructures were completely washed away by the tsunami and the substructures were overturned due to scour. In addition, there may be likelihood of structural damages due to strong ground motions before the tsunami arrived.
5. There were bridges which were not washed away even though they were engulfed by the tsunami. These bridges have several common structural features; concrete superstructures with a wider width of the road and shallow girder height, or RC moment-resisting frames. Based on these structural features, the code provisions against tsunami effects need to be established.
6. The effect of damage due to strong excitation, tsunami wave load, material corrosion, and scour on the performance of bridge needs to be incorporated in the seismic and tsunami risk assessment. A new design philosophy is needed to avoid catastrophic damage and to ensure a prompt restoration using a risk-based and resilience-based life-cycle design perspective.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Minimizing the effects of uncertainty in life-cycle engineering

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ABSTRACT: Reducing uncertainties is important in engineering; however, this is often be difficult or costly, and is seldom practically feasible. Minimizing the effects of uncertainties. may be more realistic and practically achievable. Proposed is a procedure for this latter purpose, with emphasis on life-cycle performance and minimum cost design of structures. By separating uncertainties into two types – those due to natural randomness or variability in observed data known as the *aleatory* type, and those associated with our inability to predict reality known as the *epistemic* type – the effect of randomness requires a probability measure (e.g., probability of failure or safety index); whereas in light of the epistemic uncertainty the correct probability becomes a random variable. On this basis, the effects of the underlying uncertainties, especially of the epistemic type, can be minimized in developing optimal designs of structures for life-cycle cost.

1 INTRODUCTION

The conventional approach to reliability analysis and design, as well as in risk assessment, is generally based on considering the total uncertainty; i.e., in particular, combining the two types of uncertainty as defined below in Section 2.

For example, the safety of a structure is based on modeling the maximum load over the life of the structure and of its capacity as random variables containing the respective total uncertainties. The resulting probability of failure, or safety index, of the structure is the mean value. The estimated mean safety measure (i.e., the failure probability or the safety index) does not convey its underlying uncertainty; it is a single value.

However, uncertainty in the estimated safety measure (or in the calculated risk), is equally important – this serves to show or define any confidence (or lack of confidence) in estimating the correct measure of safety or risk.

Proposed is a procedure to explicitly determine the uncertainty in the estimated safety or risk measure, and suggests how to establish conservative or high-confidence safety measure in formulating criteria for design with emphasis on developing optimal design for minimum life-cycle cost of structures.

2 ON ISSUES OF UNCERTAINTY

For practical purposes, uncertainties may be classified into two broad types; namely, the *aleatory* and the *epistemic* types (Ang and Tang, 2007).

- The aleatory type is the variability in the observed data (i.e., data-based), and represents the natural randomness in a physical phenomenon, which cannot be reduced – measure of its effect requires probability, e.g., the calculate probability of failure.
- The epistemic type is knowledge-based and represents the analyst's inability or lack of perfect knowledge to predict reality – due to this type, the calculated probability of failure becomes a random variable.

The epistemic type may be reduced, with improved knowledge or information; this is generally not easy. However, its effects can be reduced or minimized.

One way to reduce the epistemic uncertainty is through applying the Bayesian approach – in this regard, appropriate data is used to update the information and reduce the uncertainty (Ang & Tang, 2007).

2.1 *Estimation of uncertainties*

For the aleatory type, its estimation would normally be based on available data or information with its inherent variability.

For the epistemic type, its estimation must often rely on engineering judgments. The degree of uncertainty may be expressed in terms of a range of possibilities – i.e., a lower bound and an upper bound with a plausible distribution (e.g., the uniform distribution within the range). Judgmentally, estimating the range of possibilities, is more likely to be correct, than estimating a single value.

2.2 *Treatment of uncertainty and its significance*

If possible, uncertainties should be reduced especially the epistemic type. However, this is seldom practically or economically feasible. Reducing this type of uncertainty involves acquiring improved knowledge and therefore could be costly in terms of time and resources.

An alternative to reducing the uncertainty is to reduce its effects on the performance or cost estimation of an engineering system. When the two types of uncertainty are combined into the total uncertainty, its effect on the performance of an engineering system is measured by its mean value, such as the mean safety index. However, if the two types of uncertainty are separated, the effect of the aleatory variability is measured by the safety index (or failure probability), whereas the effect of the epistemic uncertainty leads the safety index to become a random variable with a range of possible values of the correct safety index. Presumably, it is reasonable to assume that the correct safety index is more likely to be within this range than that of a single value.

3 ON LIFE-CYCLE PERFORMANCE

Reliability-based performance measures over the life-cycle of a structure may be expressed in terms of the safety index. This must include its safety under the maximum load that can be expected over the life of the structure, as well as the performance reliability under repeated cyclic loading with associated maintenance.

3.1 *Numerical example of life-cycle safety*

The performance function of a structural element may often be represented by the linear function

$$g(\underline{X}) = R - D - L$$

in which,
$R =$ the strength of the structural element;
$D =$ the dead load effect on the element; and
$L =$ the maximum life-cycle live load effect on the element.

If the aleatory c.o.v.'s of R, D, and L are respectively as follows:

$$\delta_R = 0.11$$
$$\delta_D = 0.10$$
$$\delta_L = 0.25$$

and using the ACI provision for R/C beams

$$\mu_R \geq 1.56\mu_D + 1.72\mu_L$$

and with a live load to dead load ratio of

$$\mu_L / \mu_D = 0.75$$

$$\mu_R = 2.85\mu_D$$

then the safety index of the beam is

$$\beta = \frac{2.85\mu_D - \mu_D - 0.75\mu_D}{\sqrt{(0.11\times 2.85\mu_D)^2 + (0.1\mu_D)^2 + (0.25\times 0.75\mu_D)^2}} = 2.904$$

This is the safety index due to the aleatory uncertainty only; i.e., assumes no epistemic uncertainties. The estimated mean design parameters, however, are invariably subject to inaccuracies and thus uncertainties of the epistemic type.

For illustration, suppose the correct mean-values may range, respectively, as follows:

$$\mu_D = \overline{D} \pm 10\%;\quad \mu_L = \overline{L} \pm 20\%;\ \text{and}\ \mu_R = \overline{R} \pm 5\%.$$

where $\overline{D}$, $\overline{L}$, and $\overline{R}$ are the estimated mean values.

These ranges reflect the epistemic uncertainties associated with imperfections in estimating the mean loads and mean resistance. Uniform distributions within each of the ranges may be assumed.

The effects of these epistemic uncertainties will lead to uncertainty in the calculated failure probability and in the safety index. In this light, p_F and β become random variables. With the above ranges of possible mean values, the distribution of the failure probability p_F and of the corresponding safety index β can be obtained through Monte Carlo simulations; Fig. 1 shows the histogram of β with a mean value of 2.764. Observe that this mean-value of β is less than the value of 2.90 calculated earlier with FORM; this is because the earlier calculations did not include the epistemic uncertainties in the estimated mean parameters. Including these epistemic uncertainties with the following c.o.v.'s associated with the above ranges of the mean parameters: namely,

$$\Delta_{\mu_D} = 0.06;\quad \Delta_{\mu_L} = 0.12;\quad \Delta_{\mu_R} = 0.03$$

and combining these with the aleatory uncertainties (to obtain the total uncertainties) the safety index, by FORM, would be

$$\beta = \frac{2.85\mu_D - \mu_D - 0.75\mu_D}{\sqrt{(0.114\times 2.85\mu_D)^2 + (0.117\mu_D)^2 + (0.277\times 0.75\mu_D)^2}} = 2.73$$

which is close to the mean safety index of 2.76 of the Monte Carlo results.

This illustrates the fact that combining the aleatory and epistemic uncertainties leads to the mean failure probability or the corresponding mean safety index.

The histogram of β shown in Fig. 1 provides more complete information (than just the mean value) on the correct safety index of the structural element; from which specified percentile values of β can be

Obtained, such as the following:

mean $\beta = 2.76$
75% $\beta = 2.82$
90% $\beta = 2.86$

For safety in design, the histogram of the safety index allows selection of a high-confidence or conservative value; e.g., the 90% value of the safety index of 2.86, rather than the mean safety index of 2.76.

3.2 *Numerical example of safety factor for design*

A popular code format for the design of structural elements is the following:

$$\phi R_n \geq \gamma_D D_n + \gamma_L L_n$$

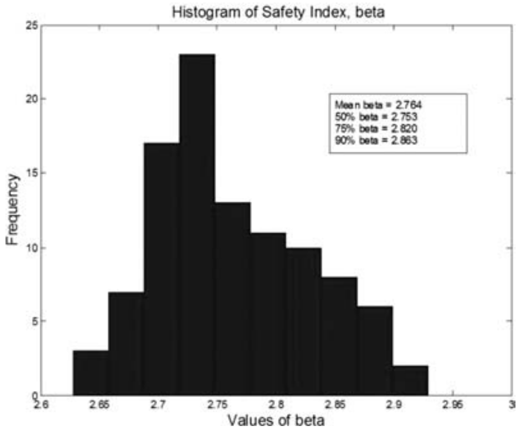

Figure 1. Histogram of Safety Index, β.

in which φ, γ_D, and γ_L re respectively the resistance factor, the dead load factor, and the live load factor. These factors can be determined to satisfy a prescribed safety index.

In this example, suppose a 90% safety index of $\beta = 2.5$ is required for the design. The total c.o.v.'s of the design parameters are

$$\Omega_R = 0.11; \qquad \Omega_D = 0.10; \qquad \Omega_L = 0.25$$

and the mean load ratio is $\mu_L/\mu_D = 2.0$.
Then,

$$\frac{\mu_R - \mu_D - \mu_L}{\sqrt{\sigma_R^2 + \sigma_D^2 + \sigma_L^2}} = \beta = 2.5$$

where μ_R, μ_D and μ_L are, respectively the mean values of the resistance, dead load, and live load. Converting the above c.o.v.'s into the corresponding standard deviations yields

$$\frac{\mu_R - \mu_D - 2\mu_D}{\sqrt{(0.11\mu_R)^2 + (0.1\mu_D)^2 + (0.5\mu_D)^2}} = 2.5$$

The required mean load and resistance factors can be shown to be,

$$\bar{\phi} = 1 - 0.722 \times 2.5 \times 0.11 = 0.80$$
$$\bar{\gamma}_D = 1 + 0.136 \times 2.5 \times 0.10 = 1.03$$
$$\bar{\gamma}_L = 1 + 0.678 \times 2.5 \times 0.25 = 1.42$$

The load and resistance factor design (LRFD) requirement, therefore, is

$$0.80\mu_R \geq 1.03\mu_D + 1.42\mu_L$$

These factors must be applied strictly to the respective mean design parameters. If other parameter values (e.g., nominal values) are used in the design process, the corresponding design factors can be derived from the mean design factors.

The above load and resistance factors can be translated into an equivalent safety factor. With the load ratio of $\mu_L/\mu_D = 2$ the mean safety factor against live load is evaluated as follows:

The mean design capacity is

$$\mu_R = \frac{(1.03 \times 0.5 + 1.42)\mu_L}{0.8}$$

Therefore, the mean safety factor against the live load L is

$$\mu_R / \mu_L = \frac{1.03 \times 0.5 + 1.42}{0.8} = 2.42$$

Whereas, against the total load, the safety factor would be

$$\bar{\theta} = \frac{(1.03\mu_D + 1.42\mu_L)}{0.8(\mu_D + \mu_L)} = \frac{(1.03 + 2 \times 1.42)\mu_D}{0.8(1+2)\mu_D} = 1.61$$

The above illustrates the fact that the load and resistance factors can always be translated into the equivalent safety factor for design.

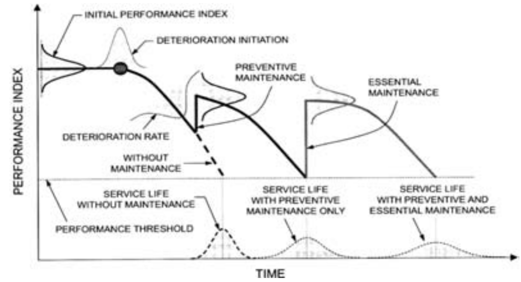

Figure 2. Life-cycle performance profile under uncertainty (after Frangopol, 2011).

3.3 *Life-cycle performance against fatigue*

The resistance, or load-carrying capacity, of a structure deteriorates with time; for example, due to fatigue damage under repeated or cyclic loadings. The deterioration process of civil infrastructure is complex. In the case of bridge structures, it is a function of the environment and vehicle loads among other factors. As these factors are highly variable, the deterioration of structural resistance contains significant uncertainty.

In order to ensure a threshold (or minimum) level of safety or reliability throughout its life-cycle, maintenance including periodic inspection and repair (as necessary) will be required. The deterioration process contains significant uncertainty, including the rate of deterioration. Following Frangopol (2011), the profile of the life-cycle performance of a structure, with or without maintenance, may be portrayed graphically as shown below in Fig. 2.

4 LIFE-CYCLE COST AND SAFETY OF REAL STRUCTURES

In formulating reliability-based design, a prescribe level of safety is necessary; this may be in terms of the safety index β. The specification of the required safety index for design is an important engineering decision.

For this purpose, observe that the failure probability due to the total uncertainty (i.e., combined aleatory and epistemic types) yields the mean failure probability (or mean safety index).

However, separating the aleatory and epistemic types would yield the distribution of the complete range of the calculated safety index. From this distribution of the safety index, a high-percentile value (e.g., 90% value) of the safety index may be specified for designs. This serves to minimize the effect of the epistemic uncertainty.

Illustrated below are two examples of real structures showing the respective safety indices underlying current design standards compared with the proposed 90-95% safety indices for ensuring safe designs.

4.1 *Design of cable-stayed bridges* – Example 1

Reliability evaluation of the cable-stayed bridge in Jindo, Korea was performed by Han & Ang (2008). The bridge (profile shown in Fig. 3) was designed and built using traditional design standards in Korea.

4.2 *Summary of results of the Jindo Bridge*

The results summarized below in Fig. 4 show the expected life-cycle costs, E(*LCC*), of the respective alternative designs of the bridge corresponding to different safety indices.

Observe that the actual bridge (denoted as the standard design in Fig. 4) is slightly more expensive than the minimum E(*LCC*) design with corresponding mean safety indices. Observe also that the E(β) of the actual bridge is slightly higher than that of the minimum E(*LCC*) design.

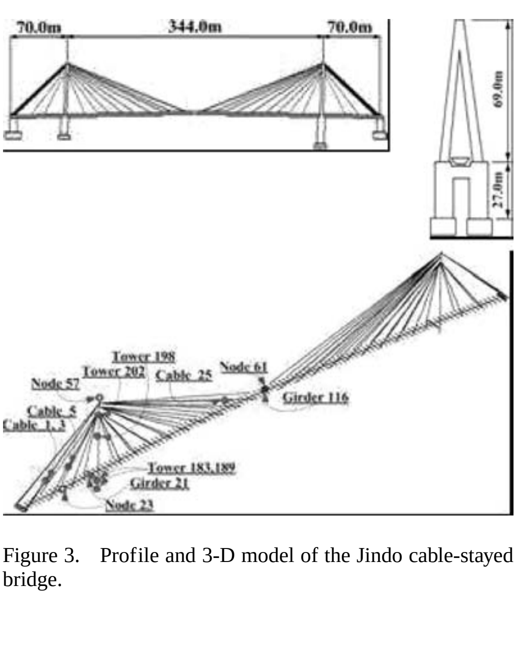

Figure 3. Profile and 3-D model of the Jindo cable-stayed bridge.

For the minimum E(*LCC*) design, the histogram of the safety index, β, is shown in Figure 5, indicating that the mean β is 2.28, whereas the 90% β is 3.23.

Comparing the standard design with the minimum E(*LCC*) design, it can be inferred that the safety index used in the standard design of the bridge would be slightly higher than that of the minimum E(*LCC*) design with a 90% β of 3.23.

The corresponding histogram of the *LCC* for the optimal design is shown in Fig. 6. The 90% *LCC* would give a conservative (or high confidence) estimate of

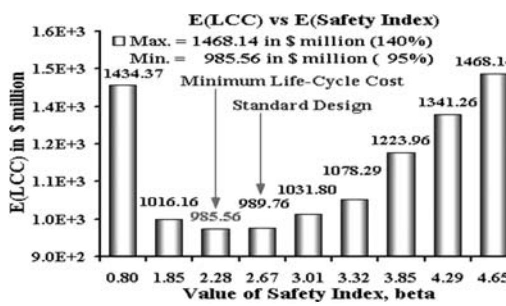

Figure 4. E(*LCC*) vs mean β for different designs of the Jindo bridge.

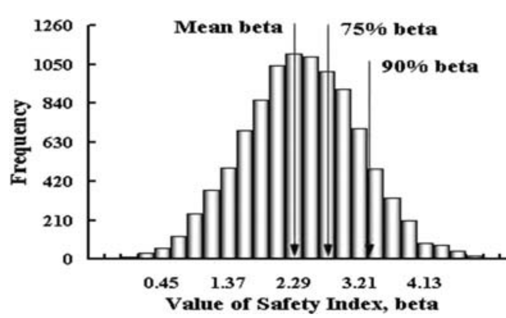

Figure 5. Histogram of β for minimum LCC design of the Jindo bridge.

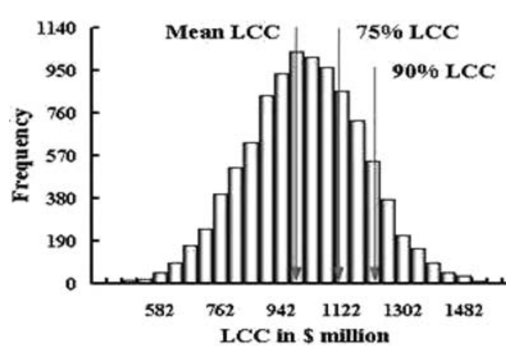

Figure 6. Histogram of LCC of the optimal design.

Figure 7. Typical offshore oil platform in Gulf of Mexico.

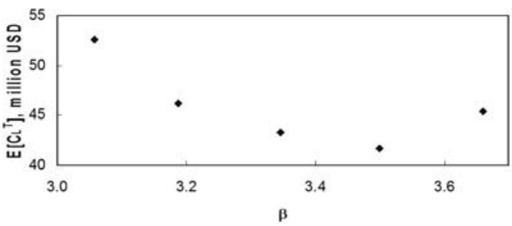

Figure 8. Expected LCC vs 90% β of offshore platforms.

the correct life-cycle cost of the bridge prior to completing its construction. In other words, this implies that by using the 90% *LCC*, the chance that the estimated life-cycle cost will under-estimate the actual cost is 10%.

4.3 *Design of offshore structures* – example 2

Optimal life-cycle cost design of a typical offshore drilling platforms for oil production in the Bay of Campeche, Mexico (see Fig. 7) was determined by DeLeon, & Ang, (2008).

Standards for the design of such systems are widely available; for example the American Petroleum Institute (API, 1993) and the Mexican standard PEMEX (2000).

Figure 8 above summarizes the results for the platform showing the E(LCC) of different designs versus the respective 90% β indicating that the optimal design is obtain with the 90% β around 3.5.

The Fig. 9 below shows further the complete histogram of the range of possible safety indices of the minimum life-cycle cost design of the platform, indicating that the 90% value of β is 3.45.

For a typical offshore production platform, the 90% safety index of 3.45 is consistent with the existing standards of the petroleum industry (API, 1993; PEMEX, 2000) for important platforms which requires $\beta = 3.3 - 3.5$

On the other hand, if the aleatory and epistemic uncertainties were combined, the optimal safety index for marine structures would be 2.96, i.e. the mean value. Clearly, this mean safety index is much lower than the requirement of the current standard for design of offshore platforms.

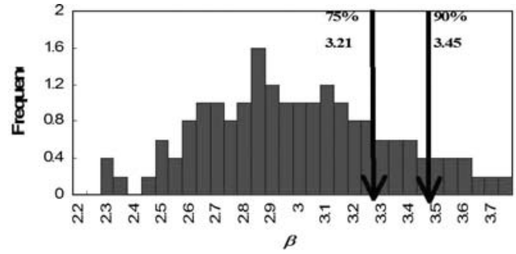

Figure 9. Histogram of β for the minimum *LCC* design of off-shore platforms.

4.4 *Implications of results from real structures*

We might emphasize (based on the two examples illustrated above) that the 90%–95% value of β appears to be consistent with the risk-averseness or degree of conservativeness underlying current standards for structural design.

It bears emphasizing that by prescribing the 90% safety index, the chance that the design safety may be inadequate is around 10%. However, in contrast, if the mean or median value of β is prescribed for design, the chance that the specified β may be inadequate is about 50%.

In other words, we may reduce the epistemic uncertainty by using improved models, or better knowledge – although this is seldom practically feasible. However, we can always minimize the effects of the epistemic uncertainty by specifying high-percentile values of β (e.g., the 90% or 95% value), and derive corresponding conservative safety factors or load-and-resistance factors for design.

Finally, the results of the two examples serve to show that the 90%–95% value of β is consistent with the implied safety level underlying current design standards for major structures.

5 CONCLUSIONS

Design of engineered systems must invariably contend with uncertainties, which can be associated with the variability in available data and information or with insufficient knowledge of reality – known, respectively, as aleatory and epistemic types.

The effect of the aleatory type can be measured in terms of a probability; e.g., probability of failure or safety index; whereas the effect of the epistemic type would yield a range of possible failure probabilities (or safety indices).

By distinctly separating the two types of uncertainty, the range and its distribution of the possible safety indices allows the specification of high percentile values (e.g., the 90% or 95% value) of the safety index for conservative design. This effectively serves to minimize the effects of the epistemic uncertainty.

The two types of uncertainty may also be combined into a total uncertainty; however, in this case the procedure will yield only the mean failure probability or mean safety index which is a single value.

By specifying the 90% safety index for design, the chance that it will be inadequate is 10%; whereas using the mean safety index in design the chance that it will be inadequate is around 50%; clearly using the latter is too risky for safety purposes.

In short,

with the 90% β, risk of inadequate safety = 10%;
with the mean β, risk of inadequate safety = 0%.

Based on the two examples illustrated of existing standards in practice, the 90–95% safety index appears to be the appropriate level of conservativeness for formulating practical design criteria for civil structures.

In essence, the proposed approach of minimizing the effects of uncertainty can be considered as providing a reliability-based procedure for systematically determining the required conservative (or high-confidence) safety factors for design with prescribed risk-averseness. In other words, this is a rational alternative to the traditional approach of determining design safety factors to cover the uncertainties that is based entirely on judgments.

ACKNOWLEDGEMENTS

The results for the cable-stayed bridge in Jindo, Korea were obtained by J.H. Han (2008), and those for the offshore platform in the Gulf of Mexico were obtained by D. De Leon (2008). Their respective contributions to this paper are gratefully acknowledged.

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Pervasive lifetime inadequacy of long-span box girder bridges and lessons for multi-decade creep prediction

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ABSTRACT: The present study was stimulated by the paradigm of KB Bridge in Palau, a record span segmentally erected concrete box girder, which deflected excessively within 18 years, and collapsed 3 months after remedial prestressing. Computations at Northwestern showed that obsolete creep and shrinkage models in standard recommendations are largely to blame. A literature search found that 68 further spans with similar excessive deflections and service lives shortened to 20–40 years were identified. Outlined is a method of 3D FE computation of multi-decade creep effects. It is shown that the computations match the multi-decade bridge observations of the KB and several other bridges provided one uses a recalibrated B3 model, while the obsolete creep and shrinkage models lead to severe underestimation of the multi-decade deflections and prestress losses. A recalibration of the B3 model is achievable through statistical coupling of partial data on bridge deflections with a world-wide laboratory database.

1 INTRODUCTION

Since its collapse in 1996, only very few articles (Bažant, et al., 2010, 2011a, b) have been written until recently about the Koror-Babeldaob Bridge in Palau. 18 years after erection of this record 241 m span, the midspan deflection reached 1.61 m from the designed camber, as pictured in Fig. 1. The prestressing tendons were measured to have lost 49% of the initial applied force. In response, remedial prestressing was performed. Unexpectedly the bridge collapsed 3 months later. This catastrophic event raised legal, theoretical, and design questions.

As a result of ethical arguments, the construction information of the KB Bridge was released to give enough detail to compare the deflections predicted by the ACI, CEB-*fib*, GL, JSCE and B3 models for creep and shrinkage prediction to the observed behavior (B3 is a 1995 RILEM Recommendation based on the solidification-microprestress theory) (ACI Committee 209 2008, FIB 1999, Gardner 2000, 2001). This was accomplished through a detailed finite element analysis that captured the contributions of multi-decade creep effects with aging, diffusion, cracking, tendon viscoplasticity, thermal effects, cyclic creep, and included uncertainty estimation. All the code formulations considered were found to consistently underestimate the effect of creep and shrinkage in the long term by a factor of 2–3. Systematically improving such recommendations in combination with more accurate and efficient numerical calculation procedures can provide the design solution to avoid future cases of excessive bridge creep.

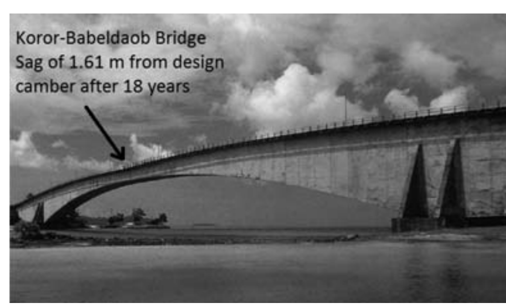

Figure 1. Image of the Palau Bridge shortly before collapse.

The analysis of bridge collapse and the related design formulations is important for the study of the lifetime of structures. Most large structures are designed for a 100 year lifetime, but bridges such as the KB bridge can show excessive deflections at <30 years. Excessive deflections require costly retrofits that should be avoidable. Thus, improved creep and shrinkage models could extend the lifetime of new structures which are susceptible to such effects. Additionally, predictive formulations assist in the assessment of existing structures.

The B3 model was selected for improvement since it is the only one with a theoretical and basis and its time function has the best match to the form of the observed bridge deflection data. This model has the further advantage of having adjustable scaling parameters associated with different creep mechanisms, represented by aging and non-aging viscoelastic terms and drying creep term. Thus, the parameters affecting the long-term slope could be scaled to statistically match the mean slope observed in the bridge deflection data. In order to perform such an analysis and to address safety concerns regarding similar bridges, a number of other bridge deflection records were obtained. A study of the deflection trends of 68 other spans (Bažant et al. 2011c, 2011d) revealed that the KB Bridge is not an isolated case of excessive deflections. A full recalibration of the B3 model can be performed through a joint optimization of the partial data available for long-term bridge deflections combined with an expanded short term laboratory creep and shrinkage test database covering both, traditional normal concrete and modern high performance concretes with admixtures.

2 3D FEM COMPUTATION OF KB BRIDGE

2.1 *Modeling of multi-decade creep effects*

In ideal situations, concrete creep in the service stress range can be described by a linear viscoelastic stress-strain relation in the form of a Volterra integral equation. However, this integral type approach to aging viscoelasticity is not only computationally inefficient but also unable to take into account additional phenomena such as cracking, damage in concrete, cyclic creep or changes in humidity and temperature in slabs of different thickness which cause major deviations from the principle of superposition. An equivalent rate-type formulation with internal variables that account for the previous history of elastic strain can overcome these problems and increase computational efficiency.

A rate-type law can be visualized as a Kelvin chain model. This model consists of a series of Kelvin units $\mu = 1, 2, 3, \ldots, N$, each of which involves a spring of stiffness $E_\mu(t)$ coupled in parallel with a dashpot of viscosity $\eta_\mu(t) = E_\mu(t)\tau_\mu$, where τ_μ are the retardation times (Bažant 2011a, 2011b). In a stepwise analysis $E_\mu(t)$ and $\eta_\mu(t)$ can be assumed to be approximately constant within each step, although they change from step to step. For stable and accurate results, equidistant discrete retardation times in log scale should be chosen. The algorithm is outlined in Fig. 2.

The retardation spectrum can be either obtained in a discrete form as $A(\tau_\mu) = E_\mu^{-1}$ or as continuous retardation spectrum, analytically derived from the compliance function by Laplace transform inversion. The latter approach proves to not only be simpler but also avoid problems of ill-conditioning and nonuniqueness which may afflict the discrete approach.

The solution to the Laplace transform inversion can be obtained by Widder's approximate formula (Tschoegl 1989, Bažant & Xi 1995), which is given by

$$L(\tau_\mu) = -\frac{\lim\limits_{k\to\infty}(-k\tau_\mu)^k C^{(k)}(k\tau_\mu)}{(k-1)!} \qquad (1)$$

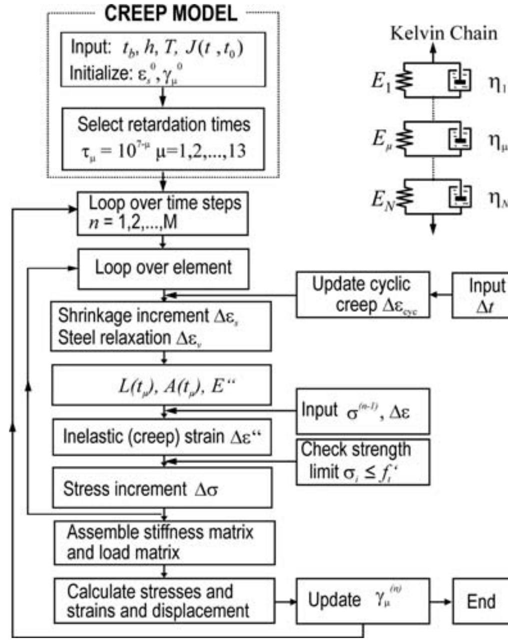

Figure 2. Kelvin chain model and flow chart of the algorithm of finite element creep analysis based on rate type creep model.

where $C^{(k)} = k$-th order derivative of the creep part $C(t, t_{n-1/2})$ of the compliance function $J(t, t_{n-1/2})$; $C(t, t') = J(t, t') - 1/E_0$ and $E_0 =$ instantaneous (or short-time) elastic modulus. In practice it is sufficient to use $k = 3$. The discrete spectrum with a finite number of Kelvin chain units for numerical computation can be derived by approximation of the continuous spectrum $L(\tau_\mu)$.

$$A(\tau_\mu) = \frac{1}{E(\tau_\mu)} = L(\tau_\mu)\ln 10\,\Delta(\log\tau_\mu) = L(\tau_\mu)\ln 10 \qquad (2)$$

The time steps necessary for the typical retardation times unfortunately do not satisfy the requirements of traditional algorithms for stability of numerical integration of first-order ordinary differential equations; they are $\Delta t << \tau_1$. Hence, the unconditionally stable exponential algorithm was used (Bažant 1971, 1975, 1982, RILEM 1988, Jirásek & Bažant 2002). The finally obtained inelastic strain increments (eigenstrains) can be augmented by inelastic strain increments due to smeared cracking as well as shrinkage and thermal strains in the current time step. Note that the shrinkage strains depend on the local environmental humidity and, in particular, on the thickness of each plate in the cross section.

In Figs. 3 and 4 the three-dimensional ABAQUS model (8-node brick elements) of the KB Bridge is presented in comparison with a simplified 1D beam Model in SOFiSTiK. Both models are based on the provided concrete compressive strength, geometrical data, prestressing sequence and construction

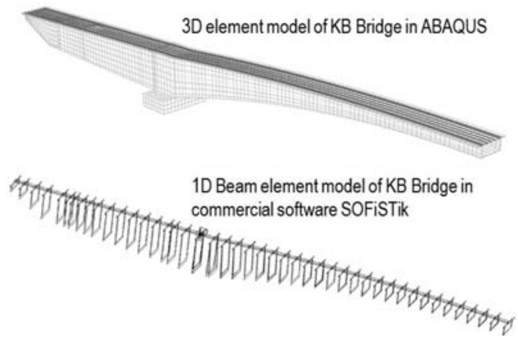

Figure 3. 3D Finite element models of the KB Bridge using ABAQUS and SOFiSTik.

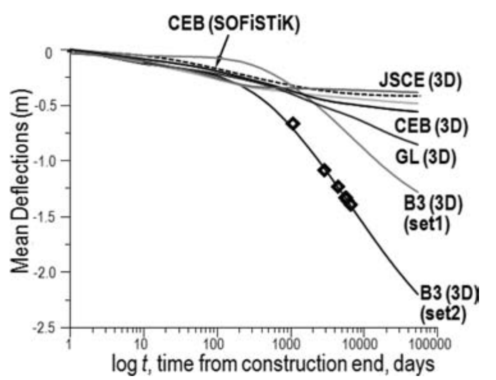

Figure 4. Deflection curves for the KB Bridge computed with finite elements using various creep and shrinkage models. The data shows a linear trend in logarithmic time.

sequence. Compared to the three-dimensional analysis, the traditional beam-type analysis of box girder deflections is found to have errors up to 20%, largely due to the effect of shear lag (which would cause greater errors for bridges with a higher box width-to-span ratio than the KB Bridge).

However, even the long-term deflection prediction obtained by the 3D model and the presented rate-type creep approach which included cracking and cyclic creep cannot explain the observed deflections when the current ACI, JSCE, CEB-fib and GL prediction models for creep and shrinkage are used. In addition to the unrealistic shape of deflection history, these models predict 18-year deflections 50% to 77% lower than the measured values. Similarly the 18-year prestress loss is found to be 22% to 27% lower than the measured loss, which was 50%.

Although capturing the shape of the bridge deflection data, the B3 model also underestimates the 18-year deflection by 42% (Fig. 4) and gives a prestress loss of 40% when the default parameter values (for creep solely based on the compressive strength) are used. However, the B3 model allows the consideration of composition parameters, which, if adjusted according to the long-time tests of Brooks (Brooks, 2005), provide a close fit of the measured deflection history. For early deflections and their extrapolations, it is important that the B3 model can realistically capture the differences in the rates of shrinkage and drying creep caused by the differences in the thickness of the wall cross sections. The differences in temperature and possible cracking of the top slab also need to be taken into account. Other paradigms for which data have recently been released are four bridges in Japan and one in the Czech Republic. Their deflections can also be explained with a similar analysis.

The main reason for the good agreement of shape (final slope in log-scale) between observed bridge deflections and the B3 model is the fact that this model is theoretically based on the physical processes involved and separately captures different mechanisms, some of which are long-term and some almost die out after a few years. The effects of age differences between segments, the variation of self-weight bending moment during cantilever construction, the differences in slab thicknesses and the change of structural system at span closing lose importance the passage of a few years, as do the transient processes, particularly the drying effect on creep and shrinkage, the gradual filling of capillary pores by cement hydration products, and the prestressing steel relaxation rate, which greatly attenuate creep within the first few years. Since drying effects greatly influence the short-time deflection predictions and also distort the deflection curve, it is essential that a sophisticated finite element creep analysis be performed.

3 BRIDGE DEFLECTION STUDY

3.1 *Collection of excessive deflection histories of large bridge spans*

The Infrastructure Technology Institute of Northwestern University has collaborated with the RILEM Committee TC-MDC in an effort to collect data on other bridges. The data was obtained through private communications from construction firms (Yasumitsu Watanabe, Shimizu Corp., Tokyo; Jan Vítek, Metrostav, Prague), consultants (Vladimír Krístek, Lukáš Vráblík, CTU Prague, Miloš Zich, Brno), and through the scanning of various papers and reports (Manjure 2002, Vítek 1997, Burdet & Muttoni 2006, Pfeil 1981, Fernie & Leslie 1975, Japan International Cooperation Agency 1990, Patron-Solares 1996, Navrátil & Zich 2010, Podeyn et al. 2011, Watanabe 2008, Kalny et al. 2010, Papa et al. 1993, Ohno et al. 2012, Dong & Robertson 1999, and private communications with Dr. Goangseup Zi and Dr. Ane de Boer). This effort resulted in a collection of histories of deflections of 69 bridge spans of which 56 are shown in Fig. 5. These, and potentially hundreds of other bridges, would have shortened service lives of 20–40 years instead of the required >100 years. Most of the bridges are large-span segmental prestressed box girders, with midspan hinges in a few

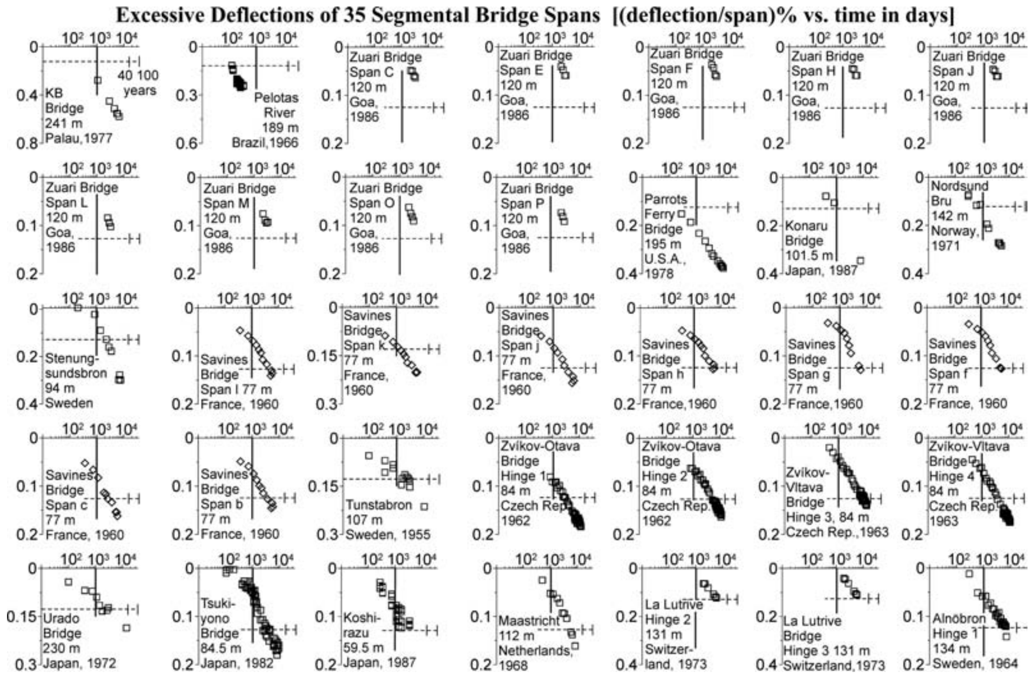

Figure 5. A collection of logarithmic time scale plots of the bridge deflection records. A linear trend is observed beyond the marked 1000 day line. The horizontal dotted line represents the excessive deflection limit.

exceptions. The Gladesville span is an arch, and the Parrots Ferry, Grubbenvorst, Wessem, Empel, Hetern, and Ravenstein spans are continuous, with no midspan hinge. While the elimination of a midspan hinge reduces deflection, it is not necessarily enough to avoid excessive creep and shrinkage effects.

The most prominent feature in Fig. 2 is that all of the deflection histories terminate with a straight or nearly straight line in the logarithmic time scale. This observation has been introduced in 1975 in an analysis of nuclear containment (see Fig. 3 in Bažant 1975) on the basis of L'Hermite's test data (L'Hermite et al. 1968, 1969) and is also informed by other long-time laboratory tests (Brooks 2005, Burg 1994, Russell 1989, Browne & Bamforth 1975, Hanson 1953, Harboe et al. 1958, Troxel et al. 1985, Brooks 1984, Pritz 1968) (see, e.g., Figs. 2.2, 2.7, 2.10, 2.24, 2.28 in RILEM 1988 or Figs. 1–4 in part 2 and 1, 3, 4 in part 2 of Bažant et al. 1991). The B3 model (Bažant & Baweja 1995, 2000) is the only creep and shrinkage model that incorporates this asymptotic behavior.

In contrast to the B3 model, the existing creep prediction models of engineering societies approach a horizontal asymptote in logarithmic scale. This includes the ACI, CEB-fib and GL models (as well as the Japanese JSCE and JRA models) This form would predicts a finite upper bound on the creep deflections, which is not supported by test data. When plotted in an extended linear time scale, even the logarithmic curve gives an illusion of approaching a bound although none exists. Such linear plots have been common in engineering literature and may have introduced the illusion that a finite bound exists for long-term concrete deformations.

To quantify how many bridges show excessive deflections, the condition that a deflection equal to 1/800 of the span is considered acceptable (ASSHTO 2004) was implemented. According to the time the last measurement was made. By linearly extrapolating in the logarithmic time scale, 32 of the this standard, 31 of the 69 bridges are excessive by 69 bridges would become excessive within the typical design lifetime of 100 years.

4 RECALIBRATION OF B3 BASED ON DEFLECTION

4.1 *Approximate multi-decade extrapolation of medium-term deflections*

The collection of bridge deflection histories can be used to recalibrate the long-term behavior of the B3 model. The inverse analysis of multi-decade deflections of large-span segmental box girders is advantageous because many such structures have been built, are old enough, are highly sensitive to creep, and are dominated by self-weight ensuring almost constant loading conditions. The creep properties of concrete may be characterized by the compliance function, $J(t, t_0)$ which represents the strain at time t caused by a sustained unit uniaxial stress applied at age t_0 (Jirásek 2002). If the deflection w_m at a certain medium time such as $t_m = 4$ years is known from the recorded data, it could be simply extrapolated to long times by assuming similarity to the compliance function. To apply this extrapolation, the age differences among the box girder segments must be ignored and the age of concrete must be characterized by one common effective (or average) age t_c at the span closing; and instead of the gradual increase of the bending moment in the cantilever segments during the erection one must consider one common effective age t_a at which the self-weight bending moments are introduced in the erected cantilever. For the current analysis we consider for all the bridges the values $t_c = 120$ days and $t_a = 60$ days.

We can assume that the bridge span behaves as a nearly homogeneous structure when drying effects die out. However the data necessary to calculate the true bridge stiffness from the material properties, geometry and construction sequence is not available. Using a general stiffness constant, C, the growth of the deflection after the termination of drying effects may be calibrated on the basis of the measured increment of the compliance function starting from the closing time t_c. i.e., $w = C[J(t, t_a) - J(t_c, t_a)]$. This relation allows us to obtain the extrapolation from time t_m assuming that the deflection at the end of drying is known. C is calibrated experimentally as $C = w_m / J(t_m, t_a) - J(t_c, t_a)$.

The extrapolation formula of the deflection from t_m thus reads (Bažant 2011d):

$$w(t) = w_m \frac{J(t,t_a) - J(t_c,t_a)}{J(t_m,t_a) - J(t_c,t_a)} \tag{3}$$

This formula can be verified using the finite element solution for the deflection of the KB Bridge. The B3, ACI and CEB-fib material models have been used to compute the compliance function for the extrapolations in Fig. 4. For each curve, we use Eq. (3) to extrapolate from the computed deflection w_m using the compliance function from which the curve was computed. The resulting extrapolations are very close to the finite element solution as seen in Fig. 6. This verifies the procedure for extrapolating the deflections according to this formula.

4.2 *Updating the long-time prediction capability of model B3*

A scaling of the B3 model is achieved by an inverse statistical analysis based on the terminal deflection trends of the 69 bridges. Lacking the concrete composition information for all the bridges, we can make a comparison in the mean sense by incorporating all bridge deflections. When the same average composition and environmental parameters are applied to all the bridges, an approximate extrapolation may be obtained for each case, although it does not uniquely characterize that bridge. The assumptions that must be made to compute the compliance function include: the concrete design strength, the average effective cross-section thickness, the environmental humidity based on the bridge location, and the cement composition. Errors stemming from these simplifying assumptions compensate each other in a statistical sense when the combined behavior of all the bridges is analyzed and the assumptions roughly correspond to the true mean of the entire bridge set.

To correct the systematic underestimation of the long-term extrapolation of creep deflections, the mean error in final slope is minimized. This can be done with the B3 model in which the parameters q_3 and q_4 control the long-term slope without affecting the short term curve. Each bridge span deflection extrapolation provides a ratio of the actual observed terminal slope to the deflection slope extrapolated with the B3 model. The parameter values obtained from minimizing the deflection error may be updated with a factor r.

$$q_3 \leftarrow \bar{r} q_3, \; q_4 \leftarrow \bar{r} q_4 \;\; \text{with} \;\; \bar{r} = 1.6 \tag{4}$$

The coefficient of variation of r is large, 0.45, due to the assumption that the concrete properties for all the bridges are the same.

5 B3.1 OPTIMIZATION

5.1 *Need to circumvent the short-time bias of laboratory database*

In order to update the empirical fitting of the full B3 model extensive creep and shrinkage experimental data is needed. The Infrastructure Technology Institute at Northwestern University currently holds the largest creep and shrinkage database (Bažant & Li 2008). This database encompasses an enlargement of the RILEM database (Muller 1993). The new database contains 1026 creep test curves and 1068 shrinkage test curves. An additional 131 creep test curves and 280 shrinkage curves with admixtures and have been added.

Fitting the B3 formula to this data is not sufficient due to restrictions that arise from limits of laboratory scale tests. Only 8% of the existing database (Bažant & Li 2008) are creep curves for durations >6 years, and only 5% for >12 years. The only available data with a duration exceeding 10 years are the aforementioned 30-year tests of Brooks (Brooks 2005) which represent only 3% of the database creep curves, are thus the only multi-decade source for modern concretes. Furthermore, the existing long-time tests only cover a limited range of concrete types, specimen thicknesses, environmental humidities and ages at loading. Hence, only combined statistical fitting of the laboratory database for short-term creep with material composition information, and of the bridge deflections for the long-term deflections, properly weighted against bias, can provide enough information.

5.2 *Systematic recalibration*

In the update of the long-term slope of the B3 model the q parameters were scaled to meet the long-term slope of the collected bridge data. However, the composition

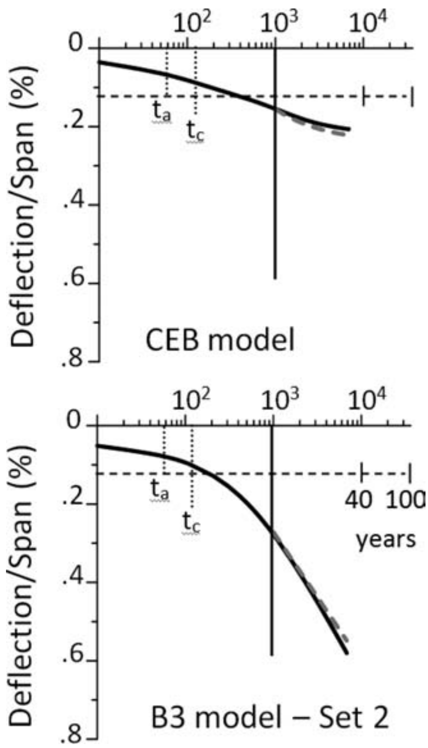

Figure 6. Deflection estimates applied to the predictions of the KB Bridge deflection obtained from 3D FEM studies. The dashed line is the estimate and the solid line is the computational solution.

of these existing structures does not match modern concretes with the admixtures and fillers that are used today. To address this complication, an expanded database of laboratory test data, covering both traditional and new concretes, provides the necessary information for increasing the range of applicability of the B3 model. This leads to a multi-objective optimization strategy based on the comprehensive laboratory database and the asymptotic properties of multi-decade creep, gained by analyzing the bridge deflection data. Such a procedure is used to develop an improved model B3.1, intended for the design of new structures and performance assessment of existing structures.

6 CONCLUDING REMARKS

As seen from the present discussion, most empirical formulations based on laboratory tests for the prediction of creep and shrinkage is not sufficiently informative to model long-term behavior. Thus, it is important to consider the theoretical basis of concrete creep. Some theoretical aspects have already been clarified, but a predictive model for hindered adsorption in nanopores of concrete is missing. The roe of hindered water adsorption was long ago proposed by Powers (Powers 1966), and the corresponding formulation based on continuum thermodynamics was developed in (Bažant 1970a, 1972a, 1972b) and reviewed in (Bažant 1975). But this theory of hindered adsorption does not explain the hysteresis observed in the wetting and drying of concrete. The sorption hysteresis in hardened Portland cement paste, concrete and various solid gels (Scherer 1999, Jennings et al. 2008) has for decades been vaguely attributed to changes in the nanopore structure, such as pore collapse due to drying, particularly the exit of water molecules from the nanopores (Feldman & Sereda 1968, Espinosa & Franke 2006, Baroghel-Bouny 2007, Thomas et al. 2008) (see Figs. 13 and 16 in Rarick 1995, Fig. 1 in Espinosa & Franke 2008, or Fig. 9 in Jennings 2008). However, a mathematical model of such a mechanism predicts enormous macroscopic deformations, far larger than the observed shrinkage caused by drying. To adequately describe the creep mechanism one would need to develop a theoretical model capturing the nano-porosity of hardened cement paste, hindered water adsorption, disjoining pressure, nano-scale rate and water sorption processes, diffusion, chemical processes, microcracking and asymptotic scaling considerations.

The tragic collapse of the Palau Bridge has exposed new sources of data to improve our prediction capabilities of creep and shrinkage in concrete structures. The combination of previously used laboratory test data with studies of the behavior of real structures not only allows us to improve empirical formulations but provides insight for theoretical assumptions. Further insights may be found in full simulations of structures for which sufficient material and geometrical information is available to model time dependent effects similar to what was accomplished for the KB Bridge.

While box girder bridges form the primary example of structures affected by creep and shrinkage effects, the present analysis may also be useful for the new super-tall concrete buildings which might be similarly afflicted by creep and shrinkage once they reach a similar age as the bridges analyzed.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)
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Life-cycle design for the world's longest tunnel project

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ABSTRACT: The paper focuses on major issues regarding the life-cycle design of the world's longest tunnel project, the Brenner Base Tunnel. The design of this tunnel, a project of European significance, is based on European code standards and designed for a life time of 200 years for the tunnel structure. The Brenner tunnel will become the longest underground railway infrastructure world-wide with a length of 64 km. It forms the central part of the high-capacity railway between Munich and Verona, and in general the link along the Trans-European Corridor no. 5 between Helsinki-Finland and Valletta-Italy (Bergmeister 2011). The design and the structural detailing of such an important infrastructure in order for it to comply with defined life time limits is of high importance. In particular, the time-dependency and uncertainties in the properties of structural materials, of rock and its behavior and of loads for the expected tunnel life time need to be taken into consideration. Monitoring and inspection data have been gathered continuously during the construction period in order to calibrate modeling input parameters and to elaborate a realistic design model. For instance, response surface methodologies (Myers, Montgomery 2002), among others, were applied for the required optimization procedures. Through a specific safety evaluation, partial safety factors for the dominant material parameters were worked out for a life time of 200 years, aiming at the rather small probability of failure of the order 10^{-8} per year. During the realization phase, an overload failure caused by small variations within the material (barrier = resistance) or induced by rock deformations is more likely. This "barrier failure dominance" (published by Beck, Melchers 2005) is quite dominant during the construction time, but will not occur during the service life time. For the service life time design, specific structural detailing (concrete cover etc.), increased partial safety factors (material) and periodic inspection and monitoring programs must be considered. A novel "gradient limit state approach" considering gradually serviceability, ultimate loads, durability and robustness has been developed and will be the basis for the life time design of this world longest tunnel project.

1 INTRODUCTION ON LIFETIME DESIGN

Life time design for new infrastructure and life time extension of existing structures have become very important on a worldwide scale. In general, intense research regarding life-cycle design oriented on systems, structural and material optimization or inspection and monitoring has been carried out by Bergmeister & Santa (2000), Frangopol (2000), Bongfiglioli et al. (2005), Biondini (2005), Andrade (2006), Stangenberg et al. (2009), Wendner, Strauss, Bergmeister (2010), Auer (2010) and others. Impressive progress has been made in the field of design optimization, service life design and global monitoring, but nevertheless real applications on important structures are limited in their number. The most common approach is to analyze existing structures through a life-cycle assessment in order to increase their life time.

The necessary handling and treatment of in-situ measured data, the statistical variation of such data, the loading and resistance in general – all these require special design methods and processes. In general, the measured monitoring data are used to suit the design model and to validate the numerical model assumptions. Once this tuning has led to a certain level of completeness and validity, analytical prediction provides a quantitative knowledge and hence is a useful tool to support structural evaluation, decision making, and maintenance strategies as pointed out by Santa et al. (2002), Strauss et al. (2008) and Strauss et al. (2012). Design methodologies that use probabilistic analyses are available and have been proven to work in practice. Suitable tools based on integrated systems (e.g. software package "Structural Analysis and Reliability Assessment" SARA (Bergmeister et al. 2006, Strauss et al. 2006)) of non-linear fracture mechanics (e.g. software package ATENA (Cervenka et al. 2001) and probabilistic models (e.g. software package FREET (Novak & Rusina 2003) are very rare. Latin Hypercube Sampling techniques, due to the small sample number necessary with respect to the required sample number of classical Monte Carlo simulation, are basic tools for efficient, highly accurate probabilistic reliability assessment methods in practical engineering as depicted in Figure 1.

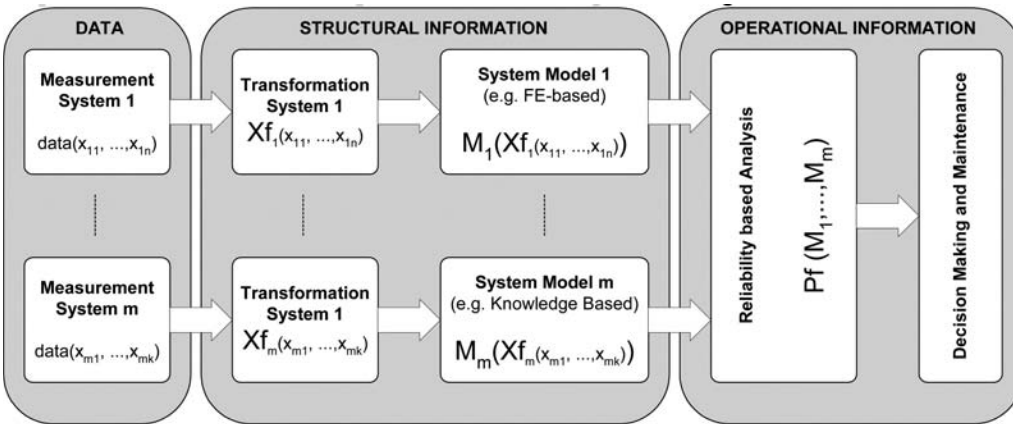

Figure 1. Procedure of tunnel design taking into account reliability based analysis.

Table 1. Example of correlations between random variables for concrete.

	E_c	f_t	f_c	G_f
E_c	1	0.7	0.9	0.5
f_t	0.7	1	0.8	0.9
f_c	0.9	0.8	1	0.6
G_f	0.5	0.9	0.6	1

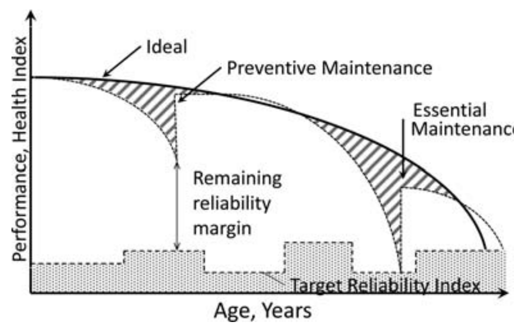

Figure 2. Life-cycle and maintenance considerations (CEB 1998).

The before mentioned SARA software tool, for example, incorporates the LHS technique of the FREET software in order to perform computationally intensive non-linear reliability analyses on real engineering structures. For instance SARA allows taking into account the reality interactions between the random parameters by simulating their real behaviors. In particular, it allows and encourages the formulation of correlations between the most important material parameters by using a specific matrix, as shown in Table 1.

On the basis of a nonlinear structural analysis with integrated probabilistic modeling over the whole life time, preventive action and lifetime planning considering monitoring can be worked out. For tunnels, at least three different levels of monitoring need to be considered (Bergmeister 2006):

- geotechnical monitoring
- structural health monitoring: structural and durability parameters
- service or operational monitoring

From a geotechnical point of view, the necessity for instrumentation starts with the excavation, in particular for tracing and evaluating e.g. rock decompression, movements of the tunnel lining, strains and deformations around the tunnel, water level and settlement control, vibration monitoring during excavation, etc. From the structural point of view, special emphasis is given to the monitoring of concrete stress changes, cracking, profile changes and deformation (see Figure 3 and Figure 4), but also to material degradation due to aggressive environmental agents and durability aspects in general. Structural integrity has to be guaranteed by the structural safety under ultimate and serviceability conditions in order to ensure the safety and robustness of the structure and its users. For the purpose of developing adequate life extension and replacement strategies, issues such as whole-life performance assessment rules, target safety levels and optimum maintenance strategies must be formulated and resolved from a lifetime reliability viewpoint and life-cycle cost perspective. In order to maintain the health of a structure up to a given time into the future, a cost-effective maintenance strategy has to be used, such as proposed in CEB 1998 (Figure 2).

Degradation functions describing the variable life time performance of structures can be captured by Weibull-functions that allow the inclusion of periodical inspection results by the adjustment of the Weibull-parameter. In other words, each inspection, allows a continuous updating of the degradation process, and an assessment of the present safety index with respect to the desired safety level over the planned lifetime. Finally, there is a third category of monitoring

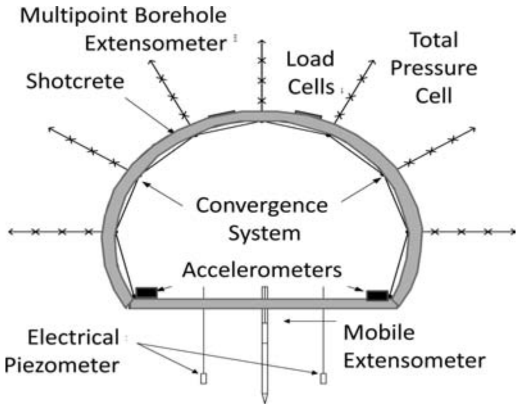

Figure 3. Typical instruments for tunnel monitoring.

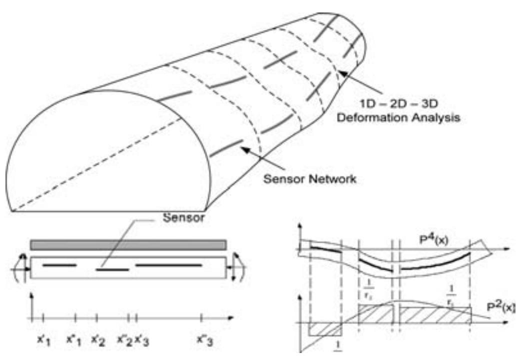

Figure 4. Strain, curvature and deformation analysis (Strauss 2003).

in tunnels which is related to the safety aspect for the users: video-surveillance for critical event detection, fire detection and control of suppression systems, gas monitoring and ventilation control etc. Tunnels usually are very stiff structures; however displacements and stresses in the lining may be induced by certain rock or other underground deformations. Excessive and non-stabilized deformations are often observed, and even though they rarely affect the global structural security, they can lead to durability or serviceability problems. Furthermore, accurate knowledge of the behavior of the tunnel is becoming more important when the aim is to insure the specified service life time. The observation of local deformations and material properties in tunnels made by a series of sensors can be extrapolated to the global behavior of the whole structure, see Figure 3. While strain sensors on a short base length are usually used for material monitoring rather than structural monitoring, long-gauge sensors give information on the behavior and response of the structure. However, material degradations, such as cracking, are only detected when they have an impact on the shape of the structure.

By continuous observation of the strain evolution, eventual changes in the structure become detectable in the domains of both short- and long-term stresses.

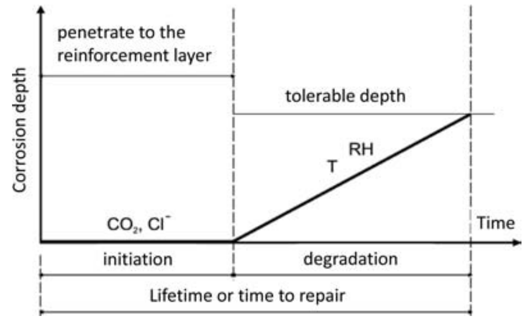

Figure 5. Service life model for degradation through corrosion.

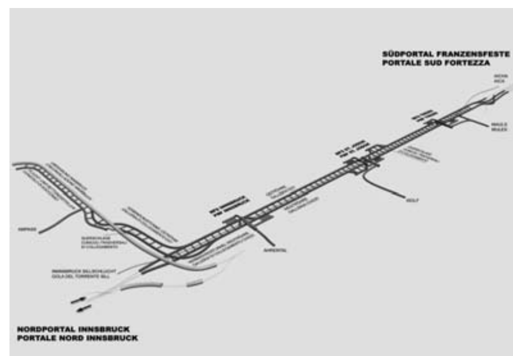

Figure 6. Layout of the Brenner Base Tunnel (BBT).

In an advanced evaluation step, the strain measurements of a whole network of appropriately installed sensors form the basis for calculating the displacement and curvature profiles of a whole tunnel structure, as lined out in Figure 4. Therefore, the acceptable level of deterioration (for example: corrosion etc.) to estimate the service life of tunnels, the probabilistic estimation of service life considering both the serviceability and the safety of structures using Monte Carlo techniques can be carried out, describing design parameters such as concrete cover and specific material and geometrical constraints to a sustain a certain service life time.

2 DESIGN OPTIMIZATION OF THE BRENNER BASE TUNNEL

2.1 *Technical properties*

The 64 km Brenner Base Tunnel (BBT) is a gradient-free railway tunnel between Innsbruck, Austria, and Franzensfeste, Italy, see Figure 6. It forms part of Europe's TEN corridor, which aims at providing an upgraded rail corridor across national boundaries. The Brenner Base Tunnel, an element in the 3500 km TEN No. 5 link from Helsinki (Finland) to Valletta (Sicily in Italy) is being pursued by the BBT SE European project corporation. The BBT consists of two single track rail tunnels, and an exploratory tunnel below them. The distance between the tunnels is 70 meters for almost all of their extension and decreases to approximately 40 meters when approaching the portals. In Innsbruck, the BBT will connect with the existing underground bypass. Together with the Innsbruck bypass, the Brenner Base Tunnel will be the longest underground railway line world-wide, with a total length of 64 km. The BBT crosses the Brenner Pass – the lowest Alpine pass with an elevation of 1371 m – at 794 m. The gradient of the Innsbruck bypass and the Base Tunnel does not exceed 6.7‰. An exploratory tunnel will be located 11 m below the 2 main tunnels. This continuous exploratory tunnel is to be driven prior to construction of the main bores, primarily for the purpose of geological exploration. The most important technical parameters for the BBT are as follows:

Total length:	64 km
Gradient:	5.0‰ to 6.7‰
Altitude of the tunnel crown	795 m above sea level
Net cross-section of main bores:	approximately 43 m^2
Minimum cross-section of exploratory tunnel:	26 m^2
Transverse-gallery spacing:	300 m

2.2 *Construction time*

First ideas about a Base Tunnel were developed as early as 165 years ago by the Italian engineer Qualizza. Subsequently, more than 300 different studies were carried out on the subject. The project that was finally approved has been developed only in the course of the past year:

Feasibility study	1985–1987
Optimization study of the corridor Munich – Verona with the BBT	1995–1996
Preliminary project and prospection	1999–2003
Final project and Environmental Impact Assessment	2003–2010
Construction of Exploratory Section	2007–2011
Construction of Exploratory and Main Tunnels	2011–2026

The construction program was revised and improved several times in course of the project. On the basis of detailed analyses of the applicability of different excavation methods, taking into consideration exploration results also from the exploratory tunnel and expert opinions, the working program was finalized. The first excavation of the exploratory tunnel was started on the Italian site in April 2008 and in Austria in December 2009. Meanwhile, more than 22 km of exploratory and access tunnels, shown in Figure 7, have been excavated.

2.3 *Improvement through the exploratory tunnel*

The exploratory tunnel accompanies the BBT along all of its length. It will be used primarily for preliminary

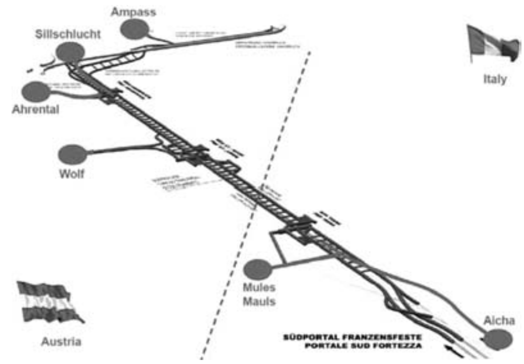

Figure 7. Construction situation in June 2012.

geological and hydrogeological exploration and secondarily for separate drainage. It might be used in the future as a service tunnel for transporting electrical lines etc. In addition, it will facilitate certain procedures during the construction period and also enable maintenance during the operation of the main tunnels.

Geological documentation for the exploratory tunnel will be performed mainly by BBT personnel, with the aim of improving knowledge and accumulating experience for future planning. Every face section is to be documented, and geologically difficult sectors (fault zones) can be studied in-situ (Quick et al. 2010). Geological features, such as the physical rock parameters like hardness and weathering, the configuration and disposition of the definitive cleavage planes, and the underground water conditions will be observed, measured and recorded for a detailed geological mapping of the rock face. Through the exploratory tunnel, it was possible to identify several such fault lines much more precisely. In addition, through the periodic measurement the sulfat content of the rock-water has been analyzed. For a total length of 31 km (from 14 km to 45 km) the sulfat content is higher than 400 mg/l; therefore specific sulfatresistent cement will be used.

2.4 *Design optimization*

In general, a mathematical description which takes into account various design parameters has been published by Bhatti (2000) and Fischer (2010). The objective function $Q(x)$ for attaining a maximum service life time can be transformed into a minimum amount of time for operation T_{OT} as follows:

$$\max_{\boldsymbol{x}\in S\subset\mathbb{R}^n}\left\{Q(\boldsymbol{x})\left|\begin{matrix}\boldsymbol{g}(\boldsymbol{x})\leq 0\\ \boldsymbol{h}(\boldsymbol{x})=0\end{matrix}\right.\right\} \rightarrow \min_{\boldsymbol{x}\in S\subset\mathbb{R}^n}\left\{-T_{OT}(\boldsymbol{x})\left|\begin{matrix}\boldsymbol{g}(\boldsymbol{x})\leq 0\\ \boldsymbol{h}(\boldsymbol{x})=0\end{matrix}\right.\right\} \quad (1)$$

where $g(x)$ describes the maximum permitted geometrical deformations of the rock mass, or the design resistance of the concrete shell, presented as follows

$$\boldsymbol{g}(\boldsymbol{x})\leq 0 \;\Leftrightarrow\; g_i(x)=0 \quad \text{with: } i=1\dots n \quad (2)$$

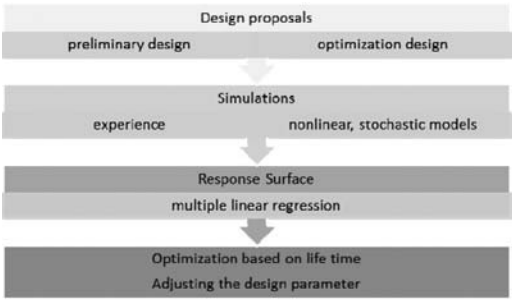

Figure 8. Work flow for the optimization method using the least square approach.

Several constraints or limits of secondary order can be described with a deterministic value by $h(x)$

$$\boldsymbol{h}(\boldsymbol{x}) = 0 \Leftrightarrow h_j(x) = 0 \quad \text{with: } j = 1 \ldots m \tag{3}$$

Several authors have proposed theoretical solutions to solve such an optimization problem (e.g., SQP-method by Han (1977), Biggs (1973), Powell (1979), Powell (1983) or by using e.g. software tools, such as provided by MAPLE. A direct approach that uses results of finite or discrete element methods has been proposed by Myers and Montgomery (2002): "Response Surface Methodology – Process and Product Optimization Using Designed Experiments". The proposed method uses a multiple linear regression analysis to correlate with either the analytical derivations or the in-situ measured results.

$$\begin{aligned} T_{ND}(\boldsymbol{x}) &= \beta_0 + \beta_1 x_1 + \cdots + \beta_k x_k + \varepsilon \\ &= \beta_0 + \sum_{j=1}^{k} \beta_j x_{ij} + \varepsilon_i \end{aligned} \tag{4}$$

with β_k = regression coefficient derived by the method of least squares; k = quantity of regression coefficients; x = vector for adjusting the design parameter taking into account the assumed life time period of the tunnel, $i = 1, 2, \ldots, n$. The response surface is formed through the simulation results of the design proposal and optimization is achieved by direct use of the response surface, shown in Figure 8.

3 LIFE-CYCLE DESIGN FOR A 200 YEAR LIFE SPAN

3.1 *Life span for structures and equipment*

Due to its strategic importance within the Trans-European Network, a life span of 200 years was considered for the Brenner Base Tunnel. By comparison, the 32.9 km long Koralm Tunnel and the 57 km long Gotthard Tunnel were built to last 150 years and 100 years respectively. It is crucial to align the life span of the civil work and the equipment with the monitoring, inspection and maintenance activities. In addition, the required robustness assessments are necessary and have to be treated on must be scenario based with due consideration of probability approaches incorporating and consequences of failures and collapses (robustness-index = direct risk/(direct + indirect risks).

3.2 *Gradient limit state approach*

The tunnel project provides the occasion for the development of a novel gradient approach incorpo-rating various limit state functions. In particular, the gradient limit state approach guaranties the follow-ing balanced safety indexes for the planned 200 year life time of the BBT tunnel:

- Operational security $\beta > 1.2$
- Durability aspects: $\beta > 1.5$ for XC3
- Serviceability limit state $\beta > 2.1$
- Robustness: $\beta > 2.7$
- Ultimate limit state: $\beta > 4.2$

3.3 *Probability of failure for structural limit states*

In the *Eurocodes*, partial safety factors for a specific life span and the relevant failure probability per year (ULS: $P_f = 10^{-6}$; SLS: $P_f = 10^{-3}$) were defined on the basis of European standards (see Table 2). If the life span amounts to 200 years, the safety level needs to be analyzed specifically and the necessary partial safety factors have to be ascertained by means of limit state functions. In the case of deep-lying tunnels, taking into consideration the impacts and the resistance during tunnel construction, the impact values are substantially stable for a long time (except as caused by exceptional, unforeseeable occurrences). Only the resistance side can change due to degradation (e.g. decrease in resistance through cyclic loadings, crack formation and reduction of the cross section through corrosion processes). During tunnel construction, the rock is morphed according to the tunneling method used (mechanized or conventional drilling and blasting techniques) until a state of equilibrium between the forces of the rock and the supportive forces of the lining process has been reached: S (rock action) = R (breakthrough reaction) (ITA 2008).

This behavior can easily be illustrated using a characteristic curve. For instance the graph of Fig. 9. shows the creation of the state of equilibrium between the rock characteristic line (impact) and the breakthrough line (resistance) as a result of radial displacements of the tunnel wall. This characteristic curve is essentially based on the theory of the even distortion condition of an isotropic elastic disc with a circular hole under an initial state of uniform stress. The ground-response-curve procedure is suitable for estimating system performance in deep tunnels. This curve depicts the correlation between radial cavity-boundary distortion and the internal supporting action of the support system. The aim of this observation was to place the risk of structural damage (ultimate limit state) as far as possible below the probability of other operational events

Table 2. Expected useful life of buildings, structures, bridges, tunnels and the respective equipment.

Structure & Equipment	Lifetime [years]	Maintenance Program
Important infrastructure e.g.: Brenner Base TunnelBridges, tunnels	➢ 200	Periodic inspection, maintenance
	➢ 100	Periodic inspection, maintenance
Residential buildings and industrial sites	➢ 50	Periodic inspection
Farm buildings	➢ 30	
Temporary buildings	➢ 10	
Rails	➢ 100	
Ballastless track, reinforcements, gates	➢ 50	
Ventilation systems	➢ 25	Periodic inspection (once a year)
Lighting systems	➢ 15	Periodic inspection (once a year)
Wearing parts	ca. 10	

Table 3. Operative probability of failure P_f and corresponding safety indices β.

P_f	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}
β (1 year)	3.09	3.72	4.27	4.75	5.20	5.8
β (50 year)	1.67	2.55	3.21	3.83	4.41	5.15
β (200 year)	1.2	2.1	2.7	3.3	3.7	4.2

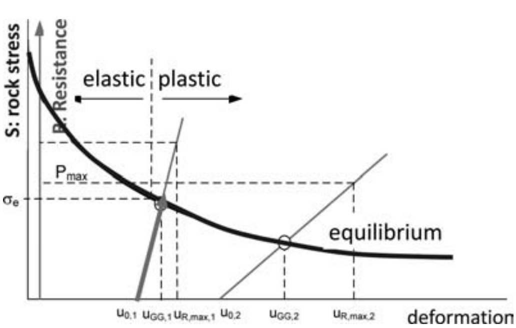

Figure 9. Ground-response-curve.

(track accidents, fire etc.). Various scientific investigations indicated that the risk of events occurring within the rail transport system outside of the train stations was either $< P = 1.8 \cdot 10^{-6}$ (Proske, 2004), or 10^{-7} according to a statistical survey on train accidents in Europe.

For the case of single-shell tunnels, the external load-bearing shell should be measured at "Ultimate Limit State" with $P_f = 10^{-8}$ per year. At the same time, the structural load of this shell in case of fire also needs to be measured. The most commonly used approach is to refer to Eurocode 2 – Part 1–2, General: Structural fire design. The principle there is to reduce the characteristic strength of the material (concrete and steel) in accordance with the maximum temperature achieved and the time of exposure. With these two parameters, it is possible to establish the temperature distribution inside the reinforced concrete element and to design the element accordingly.

For tunnels with two shells, it is usually not permissible to share the load between the outer and the inner shell. When considering both structural elements, difficulties arise regarding the durability of the temporary, so-called primary support (anchors etc.) and the differences in stiffness of the two shells, as the stiffer shell will be loaded higher. Concrete design

Compressive strength: The time related compressive strength of concrete depends on the type of cement used, the temperature and the storage conditions as shown in Strauss (2003). An average temperature of 20°C in combination with storage conditions that are in accordance with EN 12390 should allow the compressive strength of concrete at various ages $f_{cm}(t)$

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \tag{5}$$

with

$$\beta_{cc}(t) = \exp\left\{s\left[1 - \left[\frac{28}{t}\right]^{1/2}\right]\right\} \tag{6}$$

with $f_{cm}(t)$ = mean compressive strength of concrete at an age of t days; f_{cm} = the mean compressive strength after 28 days according to EN 1992-1-1, table 3.1; s = a coefficient which is dependent on the cement in use: (0.2 for cement in strength class CEM 42.5 R, CEM 52.5 N and CEM 52.5 R (class R); 0.25 for cement in strength class CEM 32.5 R, 42.5 N (class N); 0.38 for cement in strength class CEM 32.5 N, (class S).

Alternatively: a sufficient level of resistance to fatigue should be accepted for concrete under pressure if the following condition is met (note values differing from EN 1992-1-1 Table 3.1):

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.5 + 0.40\frac{\sigma_{c,min}}{f_{cd,fat}} \\ \leq 0,85 \text{ für } f_{ck} \leq 50\ \text{N/mm}^2 \\ \leq 0.75 \text{ für } f_{ck} > 50\ \text{N/mm}^2 \tag{7}$$

with $\sigma_{c,max}$ = maximum compressive stress in a fiber in terms of the frequently used impact combination (compressive stress deemed positive); $\sigma_{c,min}$ = minimum compressive stress in the same fiber where $\sigma_{c,max}$ is effective. If the tensile stress is $\sigma_{c,min}$, then generally $\sigma_{c,min} = 0$.

Partial safety factor: A reliability rating defined above the safety index of $\beta = 5.8$, and with a $P_f = 10^{-8}$, can be used to calculate the partial safety coefficients γ_R which are used to reach a defined reliability rating above $\beta = 5.8$. The design value can be derived by applying a lognormal density function [Fischer 2010].

$$f_{cd} = m_c \exp\{-0{,}5 \ln(1 + v^2) - \alpha_x \beta [\ln(1 + v_c^2)]^{1/2}]\} \tag{8}$$

or simplified: $m_c \exp(-\alpha\, \beta\, v_c)$

The partial safety factor for lognormal distributed basis parameters, as for exampleconcrete can be derived as follows:

$$\gamma_c = \frac{m_c \exp(-k_R\, v_c - 0.5 v_c^2)}{m_c \exp(-\alpha_c\, \beta\, v_c - 0.5 v_c^2)} = [v_c\, (\alpha_c\, \beta\, v_c - k_R) \tag{9}$$

Considering a maximum variation coefficient of $v = 18\%$, an unlimited quantity of samples for the 5%-fractile, $k_R = 1.645$ and assuming a sensitivity factor $\alpha_c = 0.8$, the partial safety factor for a life time period of 200 years ($P_f = 10^{-8}$/year) can be calculated as follows:

$$\gamma_c = \exp[0.18\,(0.8 \times 5.8 \times 0.18 + 1.645)] = 1.56 \tag{10}$$

Consequently, a partial safety coefficient of $\gamma_R = 1.6$ is applied for concrete.

Concrete cover: An additional amount of concrete cover should also be applied throughout the process for both pre-cast and in-situ concrete, which is used for nominal concrete covering "c_{nom}". This additional concrete covering is justified as it increases service life during abrasion, cleaning processes, CO_2-penetration and also offers protection against corrosion. The following has been used to establish a suitable quantity of concrete covering for the exposure class XC3:

$$P_{f,d}[c - x(t_{lt}, R, x) > 0] = \beta_d = 1.5 \tag{11}$$

with $c_{nom} = 50$ mm, $c_{min} = 35$ mm, and the inverse *carbonation* resistance $R^{-1}_{ACC,0} > 3.200$ [(mm²/a)/(kg CO_3/m³)] (Gehlen 2012).

3.4 *Reinforcing steel design*

The following partial safety coefficient can be ascertained for reinforcing steel (cold formed steel). Considering a maximum variation coefficient of $v = 8\%$, an unlimited quantity of samples for the 5%-fractile, $k_R = 1.645$ and assuming a sensitivity factor $\alpha_c = 0.8$, the partial safety factor for a life time period of 200 years ($P_f = 10^{-8}$/year) can be calculated as follows:

$$\gamma_c = \exp[0.08\,(0.8 \times 5.8 \times 0.08 + 1.645)] = 1.18 \tag{12}$$

Consequently, a partial safety coefficient of $\gamma_S = 1.2$ is applied for reinforcing steel.

4 CONCLUDING REMARKS

It is important, especially regarding infrastructure projects such as the Brenner Base Tunnel, to reflect on the life-cycle design and to program inspection and maintenance. In order to ensure an useful life of t_d, it is vital to implement comprehensive quality assurance (QA) during the entire design, realization and operation phases. The inspection program can be defined be applying Markov Intervals, where the intervals are marked by their equal length (e.g. one year). The Markov Chain assumes that transition matrix **M** has a uniform timescale between the 'State of the Nature' θ^n in stage n and θ^{n+1} in stage $n+1$. This feature can be expressed in the following mathematical formula:

$$P_f(\theta_{n+1} = y|\theta_n = z) \ldots = P_f(\theta_n = y|\theta_{n-1} = z) \tag{13}$$

$P_f(\ldots)$ is the distribution of probability. Generally speaking, an optimized decision should be taken as a result of gradually developed actions (stage θ^n and θ^{n+1}). The stages result from either one or several inspections $i \in I = \{i_0, i_1, \ldots, i_p\}$ (e.g., i_0 = no inspection, i_1 = visual inspection) and sequentially executed measures $a \in A = \{a_0, a_1, \ldots, a_a\}$. The annual costs of inspection and maintenance, excluding the costs for upgrading the structure and all technical equipment, were estimated on the basis of current experience with other long tunnel projects with approx. €18 Mio/year. The appropriate adjustment of the Markov decision making process using the Bayesian approach enables information taken from periodical monitoring (Strauss, Frangopol 2012) to be taken into consideration. A lifespan of over 200 years can be guaranteed by increased partial safety coefficients, by continuously carrying out inspections on a regular basis and by implementing scheduled maintenance measures.

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Computational methods for time-variant structural reliability analysis

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ABSTRACT: Life-cycle oriented structural design required the consideration of time effects regarding the safety end reliability. In order to account for time effects properly in the design optimization process, computationally efficient methods must be devised. The paper establishes the framework fork time-variant reliability analysis from a conceptual perspective. It then discusses several mainly Monte-Carlo based computational reliability methods in detail. The application of these concepts and methods is demonstrated using an example involving dynamic loading and another sample involving a crack propagation process.

The design of structures and infrastructural systems increasingly requires the consideration of life-cycle aspects. From a safety perspective, this implies that reliability analysis must take into account all possible time-dependent factors affecting the performance of these systems. This leads to so-called time-variant reliability analysis. Moreover, life-cycle considerations play an important role in the optimal design of structures and their maintenance planning (Frangopol, Kallen, & van Noortwijk 2004, Macke & Higuchi 2007). In this context it is essential to take into account the time-dependence of the statistical uncertainty of loads and resistances in the expected life-time of a structure. This is particularly important for overall life-cycle cost minimization including the impact of maintenance planning and repair strategies. A very comprehensive survey on life-cycle oriented structural optimization utilizing a probabilistic basis is given by Frangopol & Maute 2003, Frangopol, Kallen, & van Noortwijk 2004. Further studies involving optimization under random actions and maintenance include Higuchi & Bucher 2004, Bucher & Frangopol 2007, Bucher 2009c. At a fairly theoretical level, the optimal design of structures under consideration of time-variant reliability constraints is in the focus of the work by Kuschel & Rackwitz 2000. This is followed up on in Streicher & Rackwitz 2004.

In the above mentioned approaches, it is necessary to have access to statistical information on the state of the structure, mainly regarding deterioration due to corrosive effects or damage due to possibly large loads. For this purpose, it is useful to introduce the results of structural monitoring on a probabilistic basis. Some discussion of the effect of monitoring on time-variant reliability estimates can be found in Bucher 2010, Catbas, Gokce, & Frangopol 2012.

The safety analysis (or its complementary, the failure analysis) tries to identify the probability that rare combinations of the basic variables lead to structural failure. If properly designed, this failure probability should be rather small (say of the order 10^{-4} per year). The actual value would have to depend on the consequences of failure, thus introducing a cost element into the design. On this basis, the target safety level can be based on minimal expected cost during the structural life time including initial cost, cost of maintenance, and cost of failure if it occurs. Typically, large failure costs lead to higher target safety levels (or smaller failure probabilities). Since the expected cost of failure can be computed as the product of failure probability and cost of failure, again there is a need to be able to compute small probabilities efficiently. A general review of structural reliability analysis methods is given in Rackwitz 2001.

Specific aspects regarding the material and type of structure as well as possible deterioration mechanisms play an important role in the choice of appropriate methods. As an example, the consequences of time-dependent randomness on the service life of concrete bridges are discussed in Enright & Frangopol 1998 whereas time-variant reliability profiles for steel bridges are discussed in Czarnecki & Nowak 2008. Corrosion in a marine or otherwise aggressive environment and its effect on reliability is considered together with repair actions in Soares & Garbatov 1999. An application of time-variant reliability analysis to cooling towers in conjunction with the finite element method is shown in Sudret, Defaux, & Pendola 2005.

The focus of the full paper is on conceptual and computational details of various simulation methods for the estimation of small probabilities from reasonably small sample sets, for both time-invariant and time-variant problem classes. The major advantage of such Monte-Carlo based simulation techniques is that there are virtually no restrictions on the area of applicability. Essentially, these sampling techniques consist of simple repetition of the deterministic analysis with suitably modified parameters. Unfortunately, this enormous flexibility comes with an increase in computational effort. Specialized simulation techniques allow control over the effort required thus rendering advanced Monte-Carlo methods quite competitive.

Clearly, there are cases in which immediate application of Monte Carlo methods are not actually feasible. This will most likely occur whenever the structural model under consideration requires substantial computational resources for one single run. In such cases, response surface methodology may provide a convenient way for obtaining reasonable accurate answers with acceptable computational effort (Bucher & Macke 2005). The application and necessary improvements for the response surface method in the context of time-variant reliability analysis is discussed e.g. in Gupta & C.S. Manohar 2004.

When considering the reliability of structures or infrastructure systems, the performance over the lifetime is significantly affected by deterioration due to corrosion or repeated overloading. Naturally, the magnitude of the deterioration depends on the time passed since the structure was built, and consequently the probability of failure will increase with increasing time. Methods for time-variant reliability attempt to address this problem in a computationally efficient manner. For the ease of presentations, in this section it will be assumed that the reliability problem can be simplified to

$$g(R,S) = R - S \tag{1}$$

in which R denotes the structural resistance, and S is the load effect. Both variables may either be constant or change over time. Cases with $S = \text{const.}$ are fairly simply treatable and are therefore not considered in the following.

The remaining time-variant reliability problems may roughly be categorized as follows:

A) The loading is applied repeatedly in time with randomly changing intensity $S(t)$, the resistance R is constant over time. Here the main problem lies in the determination of the possible correlation between different load applications.
B) The loading is applied repeatedly in time with randomly changing intensity $S(t)$, the resistance changes over time due to load-independent deterioration such as e.g. corrosion.
C) The loading is applied repeatedly in time with randomly changing intensity $S(t)$, the resistance changes over time due to load-dependent deterioration such as e.g. low/high cycle fatigue.

These cases are schematically sketched in Fig. 1. All the cases can be characterized as *first-passage problems*, i.e. failure occurs whenever the current load effect exceeds the current resistance for the first time.

The cases as described here are further complicated if inspection and repair strategies are taken into account. Repair should basically increase the structural resistance whenever inspection reveals possible damage, either due to corrosion or excessive loading.

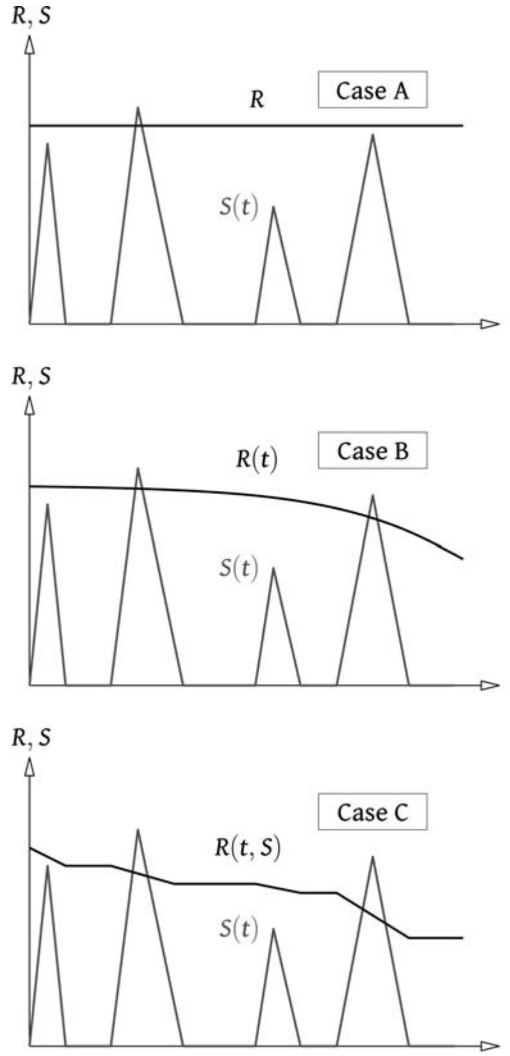

Figure 1. Classification of time-variant reliability problems.

This is not considered in the following. The complexity of Case A is mainly determined by the correlation structure of $S(t)$. If the loading process consists of a sequence of independent pulses (such as a Poisson process) then the first-passage problem can be reduced to a sequence of independent failure events whose individual probabilities may possibly be computed even in analytical form. If the load application does not lead to immediate load effects, but is delayed and accumulated over a long time period (such as due to inertia and damping in the case of dynamic response to random excitation), then the first passage problem needs to be formulated in terms of a high-dimensional reliability problem. For Case B, the situation is fairly similar, the essential difference being the changing probability of individual failure events. Still, there is the possibility of achieving accurate results with rather small efforts. In Case C, however, the resistance and the load are physically coupled, and therefore statistically dependent. It is usually not possible to remove that dependence without crude oversimplification of the problem. Therefore, in this case a high-dimensional reliability problem arises in which all load applications had to be considered simultaneously in one high-dimensional random vector.

Common to all types as mentioned above is that the reliability problem can be expressed as a *first-passage* problem. This means that the time-dependent first passage probability

$$P_E(t) = \mathbf{Prob}[\inf_{\tau \in [0,t]} \{R(\tau) - S(\tau)\} \leq 0] \tag{2}$$

needs to be computed for all values of t between 0 and the expected service life T.

The computation of the first passage probability as defined in Eq. 2 can be substantially simplified if the so-called expected up-crossing rate (first-passage rate) approach is applied. Simply speaking, this implies that passage events are assumed to occur independently such that the up-crossing process can be described as Poisson process. If we define the random process $Z(t) = R(t) - S(t)$ (safety margin), then the first passage probability can be written as

$$P_E(t) = \mathbf{Prob}[\inf_{\tau \in [0,t]} Z(\tau) \leq 0] \tag{3}$$

For the special case that $Z(t)$ is a stationary random process, the expected number of zero crossing of the safety margin per unit time (i.e. the zero-crossing rate) can be computed from the power spectral density function $S_{ZZ}(\omega)$ of the safety margin (Rice 1944):

$$\nu_0 = \frac{1}{2\pi}\sqrt{\frac{\int_{-\infty}^{\infty} \omega^2 S_{ZZ}(\omega)\mathrm{d}\omega}{\int_{-\infty}^{\infty} S_{ZZ}(\omega)\mathrm{d}\omega}} \tag{4}$$

For non-stationary processes as given in cases B and C above, this concept needs to be adapted. If the joint probability density function $f_{Z\dot{Z}}(z,\dot{z})$ of the safety margin Z and its time derivative $\dot{Z}$ is known, then the expected zero crossing rate of Z can alternatively be computed from (Rice 1944, Lin 1976):

$$\nu_0 = \int_0^{\infty} \dot{z} f_{Z\dot{Z}}(0,\dot{z})\mathrm{d}\dot{z} \tag{5}$$

Unfortunately, the required joint density is not available readily for relevant practical situations. A reasonable simplification can be based on the assumption that Z and $\dot{Z}$ are independent, which is true for stationary Gaussian processes. In this case, the expressions of the last two equations yield identical results. For non-Gaussian situations, the probability densities need to be computed by numerical means. This can actually be achieved by re-formulating the problem in terms of a time-independent reliability analysis.

Based on the Poisson assumption and with the expected uncrossing rate, the first-passage probability may be approximated by

$$P_E(t) = \int_0^t 1 - \exp(-\nu_0\tau)\mathrm{d}\tau \tag{6}$$

This excludes the case of instantaneous initial failure which is not related to time effects.

An excellent overview of the expected uncrossing rate approach is given by Beck & Melchers 2004. The same authors also discuss the situation of the so-called Barrier failure dominance (BFD) "which characterizes those problems where an out-crossing or overload failure is more likely to be caused by a very small realization of the barrier (resistance) than by an exceptionally large realization of the load process" (Beck & Melchers 2005). This means that in many cases the resistance variation may be significantly more important than the load variation.

From a computational perspective, it may be advantageous to treat the zero-crossing rates as mentioned above as functions of the particular value of the resistance R so that the up-crossing rates and the probabilities may be interpreted as conditional on R. Unconditional probabilities can subsequently be obtained by integrating over the probability density of R:

$$P_E(t) = \int_R P_E(t|r) f_R(r)\mathrm{d}r \tag{7}$$

In the process of time-variant structural reliability analysis there is substantial need to take into account complex interactions of externally applied loads and structural resistance changes. These changes may be due to deterioration based on external corrosive agents, but also due to load-induced damage processes such as fatigue. Furthermore, human-induced changes of the structural resistance due to maintenance and repair work have substantial influence on the life-cycle performance of the structures.

All these effects can be subsumed into a probabilistic formulation describing a first-passage problem. However, in order to solve this problem, specific computational methods need to be applied. While classical first-order reliability analysis (FORM) or, somewhat improved second-order analysis (SORM) may provide valuable first results, these approaches are in many cases not quite flexible enough to accommodate complex what-if scenarios, particularly when dealing with maintenance and repair decisions.

It turns out that Monte-Carlo based simulation methods certainly have all the flexibility required, but typically induce substantial computational effort. Efficacy of the simulation methods is therefore the primary property needed. Several highly efficient methods have been presented and the application to selected time-variant reliability problems demonstrated their use.

Future developments will need to focus on unified response surface models which include design variables for the life-cycle oriented optimization process simultaneously with the random variables describing the loads and the deterioration processes. Together with reliability updating based on structural health monitoring this will allow for long-term strategic decisions regarding inspection and maintenance as well as for short-term decisions for case-based repair actions.

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Durability related life-cycle assessment of concrete structures: Mechanisms, models, implementation

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ABSTRACT: The durability of concrete structures becomes more and more an aspect of monitoring and prognosis with regard to life-cycle assessment. Key components of life-cycle analysis of concrete structures are on one hand models for the assessment of the actual condition and for the prediction of its further development and on the other hand methods to observe changes in condition during use. There are different challenges. Durability models have to consider the different influencing effects and degradation processes throughout the entire life-cycle of a structure taking into account mutual dependences, interactions, scattering and uncertainties. Moreover they should be able to consider monitoring or inspection data, even if their parameters differ from model parameters. If reinforced concrete structures have been repaired during life time also aging and deterioration of maintenance measures like surface coating or concrete cover replacement must be taken into account. An overview on today's situation is given.

1 INTRODUCTION

Understanding and allocation of durability of concrete structures is in the focus of research and construction practice since several decades. Durability, understood as the ability of a structure to remain during its scheduled service life without requiring an inordinate degree of maintenance, is so far proved by choosing materials and methods of construction which promise by experience to be suited (descriptive concept).

More and more modern codes offer a service life design approach which is comparable to well established load design rules, fib (2006). Such approaches are based on quantifiable models on the load side (e.g. environmental actions) as well as on the resistance side (e.g. concrete resistance against environmental actions). For the latter task realistic models are needed describing the deterioration processes physically and chemically based with reasonable accuracy. For some deterioration processes (e.g. ingress of carbonation or chloride ions) such models are available today. For others (e.g. frost, de-icing salt damage) valuable models are still less developed. During service life material properties are changing and their scattering will increase. Also the environmental actions (e.g. temperature, relative humidity, deicing salt, splash rain events etc.) are spatially and temporally variable. So, for most concrete structures a service life design, which is performed during the design period, should be attended by a system of inspection, monitoring and prognosis as a basis for decision on time, location and method of maintenance or repair in order to arrive at the scheduled service life with minimal effort, e.g. as shown by Santa (2004) in Figure 1. In this contribution an overview on today's situation of durability related life-cycle assessment is given.

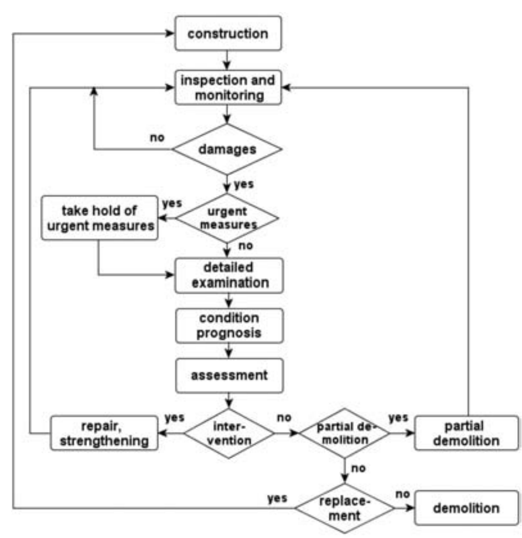


Figure 1. Scheme of structural maintenance during service life, Santa (2004).

2 DURABILITY SITUATION OF RC/PC HIGHWAY BRIDGES

In Germany, the condition of infrastructure buildings like bridges and tunnels forms a great demand on assessment of durability. The German highway

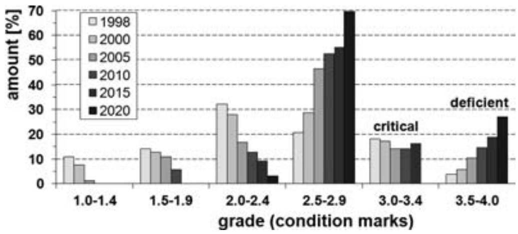


Figure 2. Expected condition development of German highway bridges without enhanced maintenance.

network is about 50.000 km long and includes actually about 38.900 road bridges with a total length of 2.075 km, Friebel (2012). 88% of the highway bridges are erected in RC/PC. Most highway bridges were built after World War II in the 1960s to 1980s.

The bridge condition usually is evaluated in regular inspections. Representative inspection results from numerous bridges in Figure 2 show the change to the worse over the years and the expected condition development if the present expenditure should not be increased in future.

The serious deterioration is caused by normal aging, traffic increase, unconsidered actions, inadequate concrete quality of elder structures, but mainly by time dependent damaging processes. Due to fib (2011) about 70% of damage to bridge structures is caused by chloride or carbonation induced steel corrosion.

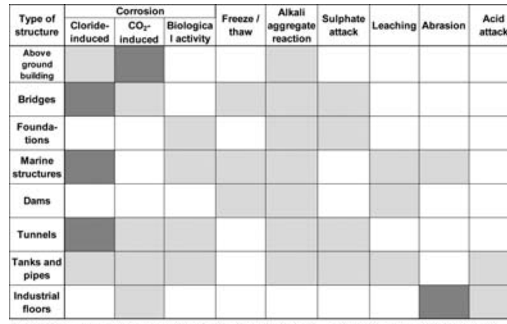
So, maintenance measures are necessary, to prevent corrosion onset or to stop corrosion resp. to reduce the progress of corrosion or to remedy the corrosion induced deficiencies. The aim is to provide the needed reliability and usability during service life, based on economic efficiency. For decision on intervention measures information on the damage state and the expected damage development is required.

3 DURABILITY RELATED CONCRETE DAMAGE MECHANISMS

Generally distinction is made between the function of structures to carry loads and to endure different exposures, both resulting from ambient conditions and from using conditions and both primarily being responsible actions for the structures durability and service life.

Since the exposures of different types of concrete structures differ, also their predominant deterioration mechanisms are different. Figure 3 gives an overview on the predominant deterioration mechanisms for different types of structures, Buenfeld et al. (2008).

Any durability related deterioration mechanism is regulated by characteristics of attack and resistance. The most relevant impact parameters are: types of substance or action, concentration of substances, intensity of action and combination of impact parameters (e.g. temperature and substances). The resistance primarily depends from concrete composition, concrete cover of reinforcement, porosity, permeability and transport parameters. For most reinforced concrete structures of



Shading: commonly affected (dark grey), sometimes affected (light grey), uncommon (no shading)

Figure 3. Predominat deterioration mechanisms for different concrete structures, Buenfeld et al. (2008).

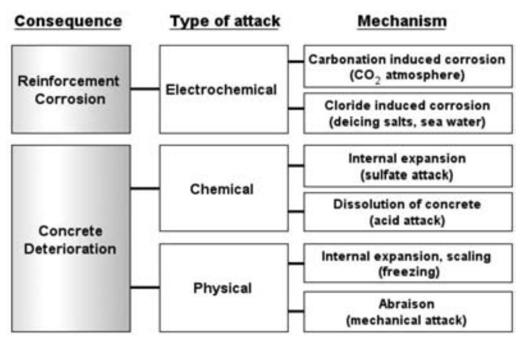


Figure 4. System of deterioration mechanisms of reinforced concrete, fib (2006).

infrastructure or industry the relevant damage mechanisms may be summarized and systematized into those affecting the durability of the reinforcement and those affecting the durability of the concrete. Electrochemical attacks are targeted at the reinforcement, chemical and physical attacks at the concrete, cp. Figure 4.

4 DURABILITY PREDICTION MODELS FOR LIFE-CYCLE ASSESSMENT

4.1 *Elements of life-cycle assessment*

Durability related life-cycle assessment is directed to the provision of information on the nature, cause, extent, location and time development of any deterioration process. The inventory of life-cycle assessment tools consists of as-built-information (birth certificate), current condition information (inspection, monitoring, condition assessment), prognosis (deterioration model) and intervention (limit states, measures) as illustrated in Figure 5.

4.2 *Types of deterioration models for life-cycle assessment*

Any deterioration process reduces the durability as well as the integrity and reliability of a structure.

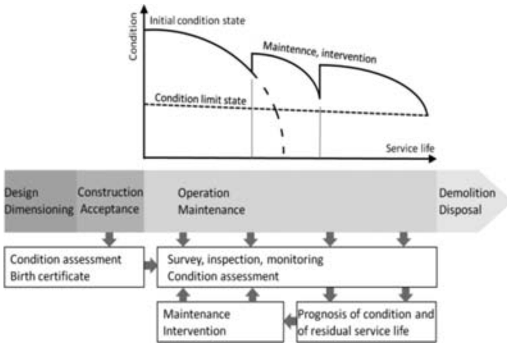


Figure 5. Inventory of life-cycle assessment.

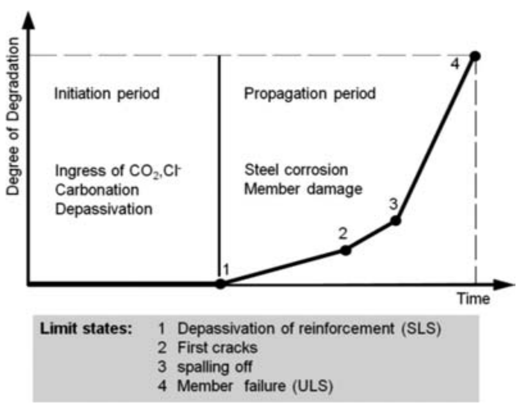


Figure 6. Periods of deterioration of RC structures due to reinforcement corrosion with possible limit state definitions.

Deterioration models are key components of life-cycle assessment. They should predict the durability affecting deterioration processes sufficiently accurate, physically and chemically correct, as simple as possible and based on information that is available from the structure.

Reinforcement corrosion, the most frequently observed deterioration cause of RC structures, can be regarded as a two periods process, consisting of an initiation period and a propagation period. The first stage means the time of ingress of harmful substances like carbon dioxide or chloride ions, the second phase is concerned with the steel corrosion itself, the loss of cross section, the cracking and spalling off of the concrete cover, cp. Figure 6.

Compared to reinforcement corrosion models the description of deterioration processes of the concrete itself is less developed. This is primarily attributed to the complexity of chemical and physical processes like acid attack, sulphate attack, freeze-thaw attack and alkali-aggregate reaction, in which multiple processes are interacting which could not yet be described by simple engineering models with respect to well defined limit states, suited for monitoring. Table 1 gives an overview on the deterioration processes and on possible limit states.

Table 1. Overview on deterioration processes and limit states, Buenfeld et al. (2008).

Deterioration process	Serviceability limit state	Ultimate limit state
Reinforcement corrosion	Corrosion initiation	Flexual failure
	Cover cracking	Shear failure
	Spalling	Anchorage failure
	Excessive crack widths	
	Excessive deflections	
alkali aggregate reaction	Cover cracking Spalling	Degradation of mechanical properties
Sulphate attack	Cover cracking Spalling	Degradation of mechanical properties
Freeze-thaw attack	Cover cracking Spalling	Degradation of mechanical properties
Leaching	Loss of alkalinity at depth of rebar	Degradation of mechanical properties
Acid attack	Loss of alkalinity at depth of rebar	Degradation of mechanical properties
Abrasion	Loss of cover	

Shading: Models widely available (dark grey), limited modeling (light grey), no models available (no shading)

Generally, available deterioration models have been developed on different approach levels, which have been classified by Buenfeld et al. (2008) according to their suitability for monitoring:

- empirical models,
- analytical models,
- numerical models,
- 'direct concrete behavior' models.

The simplest approach is the 'direct concrete behavior' model, which does not need any mathematical model but just a suited monitoring system directly at the 'center' of a durability threatened structural area. It is comparable to the approaches which can be found, called 'descriptive rules', in the standards nowadays. They are based on long-time practical experience, but they are not really calibrated against more sophisticated probabilistic approaches. With certain restrictions monitoring based on such rules is possible, controlling if or when a condition is arising which should not have been occurred if the above mentioned rules would have been successful. An example for this is the propagation of the corrosion front towards the reinforcement, which can be monitored by a new sensor type as described in chapter 5.

Empirical models are understood as simple equations, usually derived from experimental results or from on-site observation (e.g. by monitoring) by regression. By help of them results from the past at a certain spot can be extrapolated to the future in order to estimate the future development from those of the past, without regarding changing attacks or resistance of the structure. Combined with a threshold monitoring they may be a workable solution for certain tasks.

Analytical models describe more or less realistically and based on the relevant physical or chemical mechanisms a degradation process by solving the appropriate differential equations, Li & Melchers (2006). Of course, material properties and environmental attacks can only be considered in a simplified manner. Usually the input parameters of these models are determined from laboratory or field tests. For the description of transport controlled processes like carbonation of concrete or chloride ions ingress (initiation period) such models have been developed to an applicable performance, see fib (2006) and fib (2011). The advantage of such analytical models is that the stochastic nature of all parameters of the model can directly be considered in a full probabilistic approach, Marchand & Samson (2009).

For more complex processes which are transport controlled but moreover characterized by chemical reactions, like acid attack or sulphate attack, such analytical models are not available today on a practically applicable performance.

This is also true for the propagation period of RC structure's deterioration, in particular the reinforcement corrosion. According to fib (2011) the loss of steel cross section can be modeled as a function of the corrosion current density, cp. chapter 5.2. But, this parameter depends on many other parameters, e.g. the electrical resistance of concrete, and a generally accepted model does not yet exist. Furthermore the propagation period is not only characterized by cross section loss, but by the development of corrosion expansion pressure, leading to cracks, loss of bond and spalling, Andrade et al. (2011).

Numerical models describe complex processes and they are complex themselves. They are commonly used for the description of combined chemical and physical processes and include transport laws, e.g. Fick's second law of diffusion and further laws to describe reactions and property changes, Cusson et al. (2011). Using finite element methods complex systems of nonlinear equations can be solved numerically. Numerical models usually consist of a large number of input parameters, not all of them can be obtained from the structure. An advantage is that the spatial and time dependency of parameters can be considered in the analysis. Generally numerical models offer the opportunity to use data, gathered from the structure by monitoring, for an up-dating of input parameters at each time step in the calculation (adaptive modeling). A numerical model describing transport and reaction processes of concrete was developed at iBMB and has been applied to life-cycle assessment several times, e.g. Rigo et al. (2006) and Rigo et al. (2011).

4.3 *Limit state approaches*

Generally, for any life time assessment the end of service life has to be specified in terms of limit states. Unlike ultimate the limit state (ULS) for the service limit state (SLS) there are now rigid criteria. Individual ideas of the appearance of a structure or of certain usability criteria may influence the specification.

According to fib (2011) three different definitions can be characterized:

- condition related limit states,
- optimum time for intervention limit states,
- Defined performance limit states.

Commonly used condition related limit states are shown in Figure 6 and Table 1. From the view of practical hand-ling a plausible condition state is the time of depassivation of reinforcement, expressed by the carbonation of concrete or critical chloride content at the depth of reinforcement, Zhang et al. (2009). This definition of SLS of RC structures has already been established for a long time, Hansson et al. (2004). But any concentration criterion contains considerable uncertainty about its relevance. So, it is well known that the chloride content which triggers corrosion may vary between less than 0,1 and 3 in % per weight of binder, with the Cl^-/OH^- ratio, influenced e.g. by carbonation, being one important influence parameter, Angst & Vennesland (2009). Hence, using only the chloride content at the depth of reinforcement for the definition of SLS may lead to huge miscalculation of expected service life time. Moreover, structures may provide a long continued service phase even on corrosion preconditions achievement. It may vary between months and decades.

For chemically and physically induced damage mechanisms on concrete, like e.g. acid attack, sulphate attack, freeze-thaw attack and alkali-aggregate reaction, condition related limit states can be referred to e.g. concrete erosion, expansive strains or the degradation of concrete properties. But such indicators are not obviously related to the mechanisms regarded. Due to this fact it is difficult to define reasonable limit values. Furthermore, inspection methods or monitoring sensors are only partially available. There is still great need for research.

5 UPDATE OF DURABILITY PREDICTION MODELS BY INTEGRATION OF STRUCTURAL DATA

5.1 *General procedure*

For the validation and improvement of the prediction accuracy of deterioration models real statistical data of the structure are obligatorily needed. Degradation models contain mostly many parameters for material, structural and environmental properties. A basic problem is the fact that only a few model parameters can be directly measured on site or at separate concrete specimens in the lab, Marchand & Samson (2009).

Typically the assessment process for an existing structure consists of the following steps, fib (2003):

- preliminary on-site inspection to ascertain location, condition, loadings, environmental influences and the necessity for further testing,
- recovery and review of all relevant documentation, including loading history, maintenance, repair and alterations,
- specific on-site testing and measurements, e.g. proof loading,
- analysis of collected data to refine the probabilistic models for structural resistance,

– accurate (re-)analysis of the structure with updated loading and resistance parameters,
– decision analysis.

The conventional procedure of structural condition assessment is confined to periodic visual inspections. Although visual inspections do not yield quantitative results for the update of the deterioration prognosis, they play an important part of the acceptance inspection as they help to identify potential 'hot spots' due to cracking, risk of ponding or other irregularities, fib (2011).

A condition assessment concept includes besides visual inspections and the object specific defect and damage analysis also the long-term monitoring by means of embedded sensors. In some cases also permeability or loading tests are useful. In Table 2 important inspection and testing methods for concrete structures are specified.

Results of quality control during construction are important indicators for the assessment of the initial structural durability. Within the framework of a 'birth certificate' the initial material resistances of the concrete are measured, Mayer & Schießl (2009). As quality control after the completion of the concrete structure the actual dimension of concrete cover and its resistance towards chloride ingress (e.g. as chloride migration coefficient of separate concrete specimens) have to be determined and incorporated into the calculation by an initial Bayesian update of model parameters, Schiessl et al. (2011).

Especially for properties and parameters that are not changing with time (e.g. concrete cover) extensive testing is recommended. As concrete cover is one of the key parameters for the time until depassivation both due to chloride ingress and carbonation, larger parts of the concrete surfaces should be inspected, Dauberschmidt et al. (2009).

Depending on the exposure of the structural elements, non-destructive techniques (NDT) are used in addition to concrete cover mapping, for instance electrolytic resistivity measurements or crack and potential mapping, as well as the sporadic determination of chloride content and carbonation depth.

Table 2. Typical inspection and non or minor destructive testing methods for concrete structures, fib (2003).

Property under investigation	Test method
Material properties:	Cores, pull-out, pull-off, break-off, internal fracture, penetration resistance, rebound hammer, maturity
Concrete quality, Durability, deterioration, reinforcement:	Surface hardness, ultrasonic pulse velocity, radiography, radiometry, neutron absorption, relative humidity, permeability, absorption, petrography, sulphate content, chloride content, expansion, air content, cement type and cement content, abrasion resistance
Corrosion:	Half-cell potential, concrete resistivity, linear polarization resistance, A.C. impedance, cover depth, carbonation depth, chloride concentration
Integrity and performance:	Tapping, pulse-echo, dynamic response, acoustic emission, thermoluminescence, thermography, radar, strain or crack measurement, load test

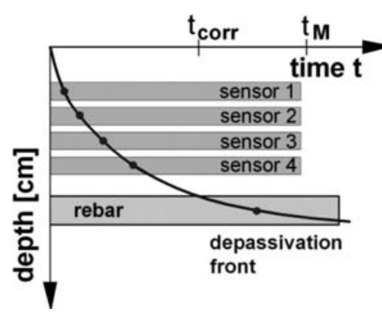


Figure 7. Operating principle of integrated corrosion sensors in different depths within the concrete cover of a rebar with t_{corr} = corrosion initiation (depassivation) time and t_M = maintenance (rehabilitation) time, if $t_M < t_{service\ life}$.

5.2 *Corrosion monitoring*

The rebar corrosion is the dominating effect of degradation of RC/PC structures. The output of many service life models for rebar corrosion is frequently limited to the time until the onset of corrosion, Hansson et al. (2004). Contrarily, in-place probes and sensors permit the monitoring of environmental conditions, the quality of concrete cover and actual corrosion rates, thus providing better information for the input of these models, updating and improving their reliability, Vennesland et al. (2007).

A number of probes and sensors have been designed for the installation into concrete, particularly to determine critical depth of the depassivation front, the time-to-corrosion onset and the corrosion rate (with seasonal and daily changes in temperature and humidity) as integral key parameters, Martinez & Andrade (2009). Monitoring allows also the assessment of the efficiency of repair and rehabilitation measures, Schiessl et al. (2011).

In Figure 7 the general working principle of deep dependent arranged corrosion sensors for the assessment of the corrosion initiation time and progress is shown. Based upon the measurement of the corrosion activity by extrapolation of the depassivation time at each sensing element the intervention time t_M can be estimated. Once the corrosion at the rebar has been initiated, the corrosion induced steel loss at the rebar can be estimated by means of the measurement of the corrosion rate i_{corr} at a sensor element in the same depth close to the rebar.

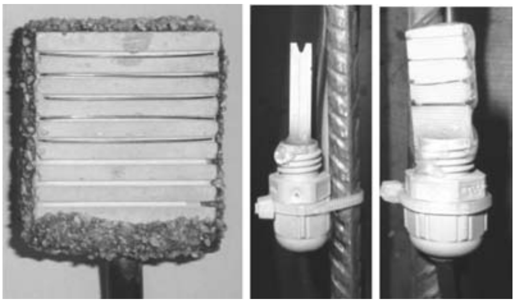


Figure 8. Innovative filament sensors for integrated corrosion monitoring of concrete structures – corrosion initiation sensor: *a*): deep dependent 5-wire sensor in the concrete cover, *b*): single steel wire sensor mounted at a rebar and *c*): multi-wire corrosion progress sensor at rebar with different wire diameters, cp. Holst & Budelmann (2012).

The corrosion rates at the steel reinforcement in concrete according to Langford & Broomfield (1987) can be subdivided into four levels:

- $i_{corr} < 0.1\,\mu A/cm^2$ small corrosion rate,
- $0.1 < i_{corr} < 0.5\,\mu A/cm^2$ small to moderate rate,
- $0.5 < i_{corr} < 1\,\mu A/cm^2$ moderate to high rate,
- $i_{corr} > 1\,\mu A/cm^2$ high rate.

For identifying the deep dependent penetration of the corrosion front into concrete as well as for the assessment of the corrosion progress at the steel bar a novel type of embeddable sensor for long term corrosion monitoring has been developed, Holst & Budelmann (2012). Different sensor types are available. In Figure 8 this dummy sensor is shown, which is based upon the recognition of the corrosion induced break of very thin steel filaments by resistance measurement.

The sensor gives a signal if corrosion occurs at a single sensor element which is located in a certain depth. With this information the possible location of the depassivation front penetrating down from the member surface can be determined. The statistical evaluation of monitoring data for each sensor element leads to a cumulative frequency of active and passive depths.

6 INTEGRATION OF REHABILITATION MEASURES INTO DURABILITY PREDICTION FOR LCA

The most commonly used rehabilitation measures of concrete structures are the application of repair mortar and/or of surface coating (surface protection systems).

To protect concrete members from the ingress of harmful substances, such as carbon dioxide or chloride ions, surface coating is an established measure. The material composition and the thickness of the coating layer are chosen according to the exposure and to the protection objective. Important requirements are e.g. resistance against mechanical abrasion or chemical attack. The interaction between concrete and surface

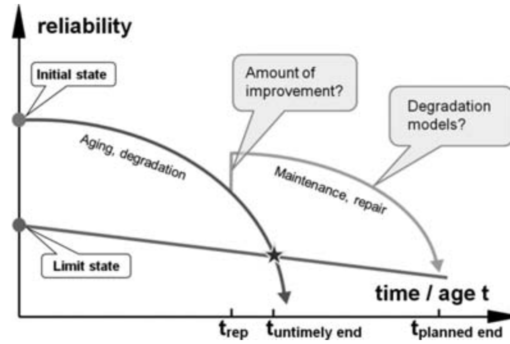
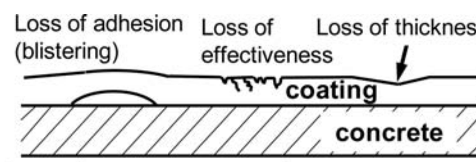


Figure 9. Principal influence of concrete repair measures on service life.



Model	Damage mechanisms	Criteria
SC-I	Loss of adhesion between coating and concrete	Adhesion tensile strength
SC-II	Loss of effectiveness of the coating	Diffusion equivalent layer thickness
SC-II	Loss of layer thickness of the coating due to mechanical abrasion	Layer thickness

Figure 10. Failure mechanisms of surface coating for concrete.

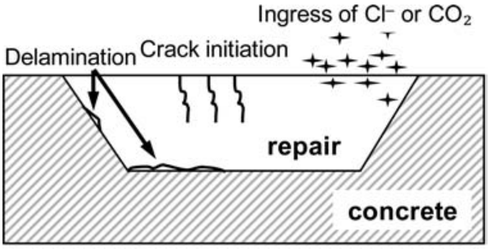
coating is ensured by the bond due to mechanical adhesion.

For reprofiling of damaged concrete surface near areas repair mortar is used. Repair mortar normally is a polymer modified mortar in which the hydraulic binder component is partly replaced by polymers. As in the case of coating, the bond between the repair system and the concrete underneath is encompassed by adhesion. The bond interaction between the old and the rehabilitated areas must resist against shear stresses at the interface between old concrete and rehabilitation mortar for the whole lifetime period of the measure.

As indicated in Figure 9, repair layers offer for a certain lifetime an improved reliability, which needs to be quantified in terms of reliability and service life. But there are no models for the behavior of those maintenance measures available.

As a first approach repair measures may be regarded as additional barriers, which eliminate harmful ingress into the concrete underneath. With increasing aging of the rehabilitation materials or in case of other damages the protection decreases and the original deterioration processes accelerate again.

In order to describe the time and condition dependent protection degradation of the repair materials material models are needed which take into



Model	Damage mechanisms	Criteria
RM-A	Crack initiation in the repair layer	Tensile strength
RM-A	Delamination bond interface concrete/ repair mortar	Adhesion tensile strength
RM-B	Depassivation of reinforcement	Depth of carbonation
RM-B	Reinforcement Corrosion	Chloride content

Figure 11. Failure mechanisms of repair mortar for concrete.

consideration the most important degradation phenomena. Figures 10 and 11 show the most relevant mechanisms of deterioration of both measures.

In Budelmann et al. (2010) a first approach is described assuming that the individual damage processes occur independently of each other. More details and calibration tests of the models are published in this conference proceedings, cp. Wachsmann & Budelmann (2012).

7 NEEDS AND CHALLENGES

Even if impressive progress has been made in the field of life-cycle assessment of concrete structures based on durability modeling, there are still lots of restrictions and challenges left. Only some of them should be mentioned afterwards without discussion in detail:

– There is a considerable interaction between degradation mechanisms, which is not yet regarded by analytical models and only partially considered in numerical models.
– Transport oriented degradation mechanisms have only been considered for non-cracked concrete with limited validity only for small crack widths.
– Spatial and temporal scattering of all variables of attack and resistance is considerable, leading to imprecise results with limited value for decisions as shown in Figure 12, Lohaus & Gerlach (2011).
– Efficiency, robustness and resilience are necessary features of methods and tools, not yet dealt with systematically.
– Up-scaling/calibration of available models on the micro-scale to the macro-scale (member scale) behavior is still pending, e.g. interaction between physicochemical processes and mechanical behavior of members.

Inherent Variability	Estimation Errors	Model Imperfections	Human Errors
-material properties -environmental conditions	-incompleteness of statistical data -inability of parameter estimation -measurement errors	-lack of understanding -simplified models	-errors made by engineers and operators -in design, construction and operating stage

Figure 12. Sources of uncertainty in reliability assessment, according to Lohaus & Gerlach (2011).

– Integration of real environmental actions into existing model approaches and calibration of models with the behavior in real conditions.
– Fatigue problems of concrete structures and their influence on life-cycle have not yet been studied carefully, neither concerning mechanisms nor modeling or monitoring.
– Novel materials and multilayer systems as well as the implementation of retrofitting and rehabilitation measures have to be considered in the durability models, cp. also chapter 6.
– There is a lack of measurement and monitoring methods for long term performance testing of concrete structures. Also conventional laboratory methods show some deficits, Grantham (2010).

8 CONCLUSIONS

There is worldwide an increasing demand for life time extension of existing concrete structures. This can be done only with improved prediction and estimation of future expected structural behavior and creating awareness of uncertainty in such estimations. This requires improved understanding and modeling of load processes, of structural system behavior, of largely unknowable boundary conditions and of processes such as fatigue, corrosion and the effectiveness of protection systems. An overview on today available methods and tools for durability related life-cycle assessment of concrete structures is given in this contribution.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)
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Life-cycle performance goals for civil infrastructure intergenerational risk-informed decisions

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ABSTRACT: Civil infrastructure facilities, including buildings, bridges, locks and dams on inland waterways, port and harbor facilities and other infrastructure, play a central role in the economic, social and political health of modern society. Such facilities are susceptible to aging, which is stochastic in nature and makes their reliabilities time-dependent. In recent years, there has been a growing recognition that for certain civil infrastructure projects, the required service periods might be substantially longer than what is typically is expected of buildings, bridges and similar infrastructure, extending the potential consequences of life-cycle engineering decisions far beyond the limits for which there is practical experience. Life-cycle engineering analysis and risk-informed decision tools have advanced in recent years for managing public investments in performance assurance and risk mitigation of civil infrastructure. However, current assessment procedures will require modification to evaluate performance of civil infrastructure facilities over extended time frames and to support sustainable and equitable decisions regarding long-term public safety. This paper considers a number of key issues that must be addressed in the course of life-cycle reliability assessment of civil infrastructure that must remain functional for service periods on the order of several generations, introducing perspectives on risk that are germane to ensuring sustainability and inter-generational equity in risk-informed decision-making.

1 INTRODUCTION

Civil infrastructure facilities play a central role in the economic, social and political health of modern society and must maintain their safety, integrity and functionality at manageable cost over their service lives. Such facilities, including buildings, bridges, locks and dams on inland waterways, port and harbor facilities and other infrastructure, may be difficult or impossible to inspect or service on a periodic basis, or to replace, and their failures may impact other essential systems (common-cause failures) and broad segments of the public. In addition, such facilities are susceptible to aging, where deterioration in performance may occur as a result of natural internal processes, service conditions, aggressive environments, or improper construction or usage. These aging and degradation mechanisms and the structural demands that arise from operating environments and natural and man-made hazards invariably are stochastic in nature, making the reliability of civil infrastructure facilities time-dependent. The challenges to civil infrastructure posed by aging have prompted numerous research programs that address risk management issues for buildings and bridges (Frangopol et al 1997; Vu & Stewart 2000; Petcherdoo et al 2008), offshore and inland riverine navigation facilities (McAllister & Ellingwood 2001; Melchers 2003), port and harbor facilities, and nuclear plants and other critical facilities (Mori & Ellingwood 1993; Naus et al 1999; Ciampoli & Ellingwood 2002). Despite the advances in facility risk management made possible by this research, significant challenges to practical implementation of reliability-based condition assessment remain [Ellingwood 2005]. While civil infrastructure seldom has been designed and constructed with specific service lives in mind, it is commonly understood that buildings, bridges and similar facilities should perform their intended functions for service periods of 50 to 75 years. In recent years, there has been a growing recognition that the service periods required might be substantially longer than this for certain facilities, and indeed may extend to centuries rather than decades. Such considerations extend the potential consequences of life-cycle engineering decisions to future generations, far beyond the customary utilization horizon, budget cycles for public investment, terms of office of elected public officials, or lifetimes of responsible decision-makers. Current models for time-dependent reliability analysis are inadequate for making forecasts of facility performance over service periods of this magnitude.

Risk-informed decision-making for civil infrastructure performance and integrity has three essential ingredients: physics-based models of material aging and structural deterioration; time-dependent reliability models to capture the uncertainties in facility behavior over its projected service life; and a decision framework that integrates the uncertain time-dependent behavior for purposes of design and condition evaluation and risk management [Ellingwood 2005].

Quantitative time-dependent reliability assessment tools have matured in recent years to the point where they can assist in establishing strategies for analyzing risk of aging infrastructure exposed to natural and man-made hazards. At the same time, life-cycle cost analysis has become an accepted tool for managing public investments in performance assurance and risk mitigation. However, these current quantitative risk-informed assessment procedures account only for decision preferences of the current generation. They will require modification to evaluate performance of critical infrastructure facilities over extended time frames, to recommend alternative design and maintenance procedures, and to support sustainable decisions regarding long-term public safety [e.g., Rackwitz et al 2005; Nishijima et al 2007]. The enormous investments of public funds that are necessary to construct and maintain civil infrastructure must be based on rational analysis and should not be made purely on the basis of subjective or political judgment.

A number of key issues must be addressed as part of life-cycle engineering, reliability assessment and risk-informed decision-making: How does one measure acceptable performance in reliability terms? How can one deal with the non-stationarity in demands from natural hazards that arise as a consequence of climate change and in changes in structural behavior that arises from aging? What additional requirements, if any, should be imposed as conditions for extending the service life of a facility over extended periods? How can uncertainties in future demands and structural aging mechanisms, in in-service inspection, flaw detection and measurement be integrated in time-dependent structural reliability analysis to demonstrate compliance with reliability-based performance objectives? How does one deal with life-cycle cost issues – discounting, decision preferences, social goals – that arise when service periods in which life-cycle analysis has been successfully applied to periods extending over multiple generations? How does one measure costs consistently and distribute benefits from generation to generation in an equitable fashion? These issues must be addressed to achieve sustainable solutions to many pressing infrastructure problems. The following review will focus on answers to these key questions, touching upon common structural degradation mechanisms and demand models, practical time-dependent reliability analysis tools, the role of nondestructive evaluation in ensuring life-cycle performance, and perspectives on risk that are germane to inter-generational equity in civil infrastructure decision-making for the long term.

2 COMMON STRUCTURAL DETERIORATION MECHANISMS AND MODELS

The service life of structural components and systems depends on how the material properties change over time. Aging mechanisms that cause deterioration of concrete structures may be produced by chemical or physical attack of either the cement-paste matrix or aggregates. Chemical attack may occur by efflorescence or leaching, sulfate attack, attack by acids or bases, salt crystallization, and alkali-aggregate reactions. Physical attack mechanisms include freeze/thaw cycling, thermal expansion/thermal cycling, abrasion/erosion, shrinkage cracking and fatigue. Degradation of mild steel shapes or steel reinforcement in concrete can occur as a result of corrosion, elevated temperature or fatigue. Pre-stressing or post-tensioning systems are susceptible, in addition to the above, to loss of pre-stressing force due to relaxation. Some deterioration mechanisms are synergistic (e.g., corrosion/fatigue); others may interact with or impair the effectiveness of protective systems such as coatings or cathodic protection (corrosion, cracking). These interactions and synergisms are poorly understood.

Much of the literature on structural deterioration mechanisms has been developed from small specimens tested under laboratory conditions. These tests have been conducted in different environments and at different scales than would be typical for civil infrastructure [Pommersheim & Clifton 1985; Liu & Weyers 1998; Naus et al 1999], and the relevance of these data to structural engineers seeking to manage risks in aging civil infrastructure is questionable. Data for measuring deterioration of structures *in situ* are limited. Accordingly, models of structural deterioration that have been used in time-dependent reliability analysis of aging infrastructure are, for the most part, rudimentary (Mori & Ellingwood 1993; Frangopol et al 1997; Petcherdoo et al 2008]. Deterioration usually has been modeled for time-dependent reliability assessment using simple rate equations of the form:

$$X(t) = \alpha (t - T_i)^{\beta} + \varepsilon_1(t); \; t \geq T_i \tag{1a}$$

or

$$X(t) = \alpha (t - T_i)^{\beta} \, \varepsilon_2(t); \; t \geq T_i \tag{1b}$$

in which $X(t)$ = deterioration parameter (loss of section, depth of penetration, etc), T_i = initiation or induction period (often modeled as a random variable), α and β are parameters determined from a regression analysis of experimental data, and $\varepsilon_1(t)$ and $\varepsilon_2(t)$ = random processes describing the uncertainty in the deterioration with time. In Eq (1a), ε_1 customarily is assumed to be modeled at present by a normal random variable, with $E[\varepsilon_1] = 0$ and $SD[\varepsilon_1] = \sigma_1$ while in Eq (1b), ε_2 is modeled by a lognormal random variable, in which $E[\varepsilon_2] = 1$ (assuming the model is unbiased) and $SD[\varepsilon_2] = V_2$, the coefficient of variation describing the variability in deterioration. Such relations are empirical in nature, depend on experimental data, and are sensitive to the environment (temperature, atmosphere, aeration, humidity). They are difficult to generalize to environments not reflected in the database. While, certain deterioration mechanisms, such as general corrosion of reinforcement in concrete, are becoming more amenable to physical modeling [Bentz et al 1996], *in situ* service conditions are difficult to reproduce in laboratory tests and scaling to prototype conditions raises additional uncertainty which is difficult to model. Thus, it is not clear whether such modeling offers any practical improvement in the reliability assessment over the simple models reflected by Eqs (1). Furthermore, a more complete description of the deterioration in time (and better estimate of time-dependent reliability) requires, in addition to Eqs (1), the covariance, $C[\varepsilon(t_1), \varepsilon(t_2)]$, which describes the correlation in deterioration at two points in time t_1 and t_2. Information to describe the covariance for practical time-dependent reliability analysis does not exist, although one might expect that a simple exponential decay model:

$$C[\varepsilon(t_1), \varepsilon(t_2)] = \sigma^2 \exp\left[-\left(|t_2 - t_1|/T_c\right)^2\right] \tag{2}$$

in which $\sigma^2 = \mathrm{Var}[\varepsilon]$ and T_c = correlation period, would capture the observed behavior adequately.

3 TIME-DEPENDENT RELIABILITY ASSESSMENT

Structural loads, engineering material properties and strength degradation mechanisms are random functions of time. The time-dependent margin of safety, M(t), is

$$M(t) = R(t) - S(t) \tag{3}$$

in which R(t) = strength and S(t) = dimensionally consistent structural action (moment, shear, etc) from the applied loads. Methods for evaluating the reliability of a structure at an arbitrary point in time are well-established (e.g., Ellingwood 2005). For service life prediction and reliability assessment, the probability of satisfactory performance over some service period is a more relevant metric of performance. If, for example, n discrete loads $S_1, S_2, \ldots, S_n$ occur at times $t_1, t_2, \ldots, t_n$ during (0,t), the probability that a structure survives during this interval is defined by the reliability function, L(t):

$$L(t) = P[R(t_1) > S_1, \ldots R(t_n) > S_n] \tag{4}$$

which can be related to the conditional failure rate or hazard function, h(t) [Mori & Ellingwood, 1993]:

$$L(t) = \exp\left[-\int h(\xi)\, d\xi\right] \tag{5}$$

in which h(t) defines the probability of failure in interval (t, t + dt), given survival through (0,t). If the load process is continuous (or is modeled as continuous) rather than discrete, L(t) can be approximated by replacing the conditional failure rate, $h(\xi)$ with the mean down-crossing rate of level zero. The conditional failure rate can be used as a tool for maintenance planning and condition assessment of existing infrastructure that already has performed successfully for an extended period of time.

3.1 *Structural load modeling*

Practically all reliability analyses to date (time-dependent and otherwise) have considered the load (demand) on the system to be a stationary random process, where the distribution and its parameters are invariant in time. The idea that (static) loads (e.g., annual extreme winds) can be modeled as a sequence of identically distributed and statistically independent random variables is embedded in first-generation probability-based limit states design. Under the assumption that the service load can be modeled as a statistically stationary random process and aging does not occur, then $h(t) = \nu$ (a constant), and failures are said to occur randomly during the service life; otherwise, the failure rate for an aging structure generally is increasing, and time to failure can be described by a Weibull distribution.

The assumption that the loads on a structure comprise a stationary random process becomes untenable when the effects of global climate change [IPCC 2007] are considered. Such effects are postulated to include more frequent and increasingly severe wind storms and extremes of temperature and precipitation. The structural engineering research community is just beginning to consider the effects of such increases on structural loads [Stewart et al 2011; Bjarnadottir et al 2011], and only very simple models of non-stationary (e.g., time-dependent characteristic extremes) have been postulated. Such models might be justified for periods of 50 years or less, provided that the rate of climate change is small, but are unlikely to be adequate for modeling structural demands over a period of centuries. Furthermore, models employed by scientists modeling climatology are extremely large in scale (greater than 10^6 km^2); local climatology of the scale of interest to structural engineers is orders of magnitude smaller (10^4 km^2). Bridging the gap between models of these scales is problematic but is essential for risk-informed decision on a multi-generational scale.

The numerical analyses of Eqs (4) and (5) for realistic civil infrastructure systems can be quite complex computationally (Mori & Ellingwood 1993). However, efficient methods for determining the limit state probabilities of structural components or systems as functions (or intervals) of time from stochastic process models of deterioration, residual strength and service and environmental loads have advanced considerably in recent years (e.g., Petcherdchoo et al 2008). Computation is no longer the formidable barrier that it once was.

3.2 *In-service inspection and maintenance*

Forecasts of time-dependent limit state probability enable the manager to judge when the facility should be inspected, maintained or repaired. If the facility has withstood previous operational or environmental demands, especially those that may have approached or exceeded the design basis, that information should be reflected in condition assessment (Faber et al 2000; Melchers 2001). Nondestructive evaluation (NDE) or prior successful performance enables the prior probability density function of capacity to be updated using Bayesian methods. For this process to be informative for time-dependent reliability assessment, performance-degrading defects must be detected, located and measured accurately. Detection and measurement of defects and estimation of their impact on updated capacity pose significant research issues for civil infrastructure. Most quantitative NDE procedures have been developed for use in manufacturing QA/QC, particularly in the aircraft industry, where conditions are tightly controlled and inspectors are highly trained. Inspection of civil infrastructure normally is performed in the field, which is far more difficult. In addition, the current strength must be estimated, either directly or indirectly. It is common to postulate a degree of structural restoration, but the restoration is, in fact, a random variable, and there is very little information available to define its characteristics for reliability analysis purposes. Finally, a host of additional problems arise when service periods extending to several generations are considered. In addition to the need to extend existing capacity and demand models well beyond the range of observation and possible validation, the civil engineering profession has yet to identify the ingredients of a plan for performance and maintenance that would extend to such periods. It seems clear, however, that a proper plan for investing in-service inspection and maintenance will be essential to ensure that the plan is implemented as intended in the initially planning stages and will achieve the performance expected of civil infrastructure over multiple generations.

4 PERFORMANCE LIMITS FOR CIVIL INFRASTRUCTURE

Performance objectives for civil infrastructure must be related to results produced by structural reliability analysis. Public safety is, and has been, paramount in structural code development, and so assurance of structural strength, stability and overall integrity is the first concern in public regulation of the built environment. However, there is a growing awareness, prompted by recent experience following natural disasters and the new paradigm of performance-based engineering (PBE), that public safety is necessary but not sufficient for satisfactory infrastructure performance. The likelihood of business interruptions, social disruptions, and unacceptable economic losses also must be minimized. Durability and maintainability issues also become important. The expectations of individuals and the public toward performance of buildings and other civil infrastructure have evolved during the past three decades; if the framework for decision-making involves a time horizon of a century or more, it is virtually certain that those expectations will evolve further. The only reasonable way to address this issue economically is for facilities to be designed for maximum future adaptability to changing circumstances and expectations. Such solutions are likely to be sustainable (under the current definition of the word) but require a mode of thinking at the conceptual stages of design that is foreign to what is typical at present.

5 RISK BENCHMARKS AND RISK-INFORMED ENGINEERING DECISION

In current risk-informed decision-making, performance goals generally are expressed in terms of probabilities, expected losses, or a combination of these metrics. Most studies of time-dependent reliability analysis of aging infrastructure indicate that the cumulative limit state probability or conditional failure rate increase over time by orders of magnitude, absent any rehabilitation. The analysis of time-dependent reliability has received far greater attention in the technical literature than have risk acceptance and decision criteria. When looking toward the future, there are far fewer answers than questions: What is acceptable risk in specific circumstances? What is an acceptable increase in failure rate? How much of a decrease in conditional failure rate should one expect (or insist on) when stipulating repair or renewal as a condition of continued service? Does the original design basis even have any relevance to this assessment when future performance requirements and expectations are factored into the decision process? When expected cost is used as a basis for decision and the usual discounting is applied, it may be found that events in the far-distant future have little impact on present value, leading to the erroneous conclusion that such events need not be considered. How should expected cost (or utility) analysis be modified to allocate costs and benefits equitably between the current and future generations? How does one persuade a decision maker to invest in achieving life-cycle objectives vs immediate financial gratification; in achieving phased reductions in risk vs immediate reductions? The investigation of such issues is still in its infancy [Nishijima et al 2007].

The assessment of loss probabilities or expected losses, once determined, and decisions regarding risk mitigation depend on the decision-maker's view on the acceptability of risk and on whether/how investments in risk reduction should be balanced against available resources. Most individuals are risk-averse, while governments and large corporations tend to be more risk-neutral (Slovic 2000). Recent studies (e.g., Corotis 2009) have indicated that *acceptance* of risk is based more on its *perception* than on the actual probability of occurrence and that biases in perception, whether or not they are well-founded, shape decisions. Consideration of acceptable risk in quantitative terms for civil infrastructure facilities, the construction of which often has been regulated by public codes, is a relatively new development (Ellingwood 2007). Furthermore, the conventional minimum expected cost approaches presume that the risk can be entirely monetized. There is evidence to indicate that individual decision-makers and many public agencies are risk-averse, meaning that they demand increasing payments for accepting marginally increasing risk (Cha & Ellingwood, 2012). Risk aversion is likely to play an increasing role in decisions extending to multiple generations, due to the substantial uncertainties in extrapolating stochastic models of demand and capacity far beyond their previous applications or supporting databases, as well as the uncertainties in decision consequences that might not occur until after decades into the future.

Risk communication requires a continuing dialogue among the members of the project team and other project stakeholders, aimed at facilitating understanding basic issues and enhancing the credibility and acceptance of the results of the risk assessment. Performance objectives and loss metrics must be clearly identified and agreed upon, and uncertainty analysis should be a central part of the decision model. Tradeoffs that occur between investment and risk reduction must be treated candidly, and the entire decision process must be made as transparent as possible. All sources of uncertainty, from the hazard occurrence to the response of the structural system, must be considered, propagated through the risk analysis, and displayed clearly to obtain an accurate picture of the risk.

6 CLOSURE

Civil infrastructure in the United States and in other industrialized countries is subjected to continually increasing operational and social demands at a time when diminished budgets cause essential maintenance and repair to infrastructure to be deferred or eliminated. Continued serviceable infrastructure performance is essential for the social, political and economic health of a society. Decisions aimed at ensuring performance or maintaining integrity of civil infrastructure systems affected by natural or man-made disasters, such as floods, hurricanes, earthquakes, fire, collisions or terrorist acts, can entail billions of dollars and expose thousands of people to the possibility of injury, death or financial ruin. Significant knowledge gaps exist between capabilities for analysis and prediction of infrastructure performance during or following low-probability, high-consequence events, and public policy and decision-making in the public interest. Quantitative risk-informed methods are required to assess alternative maintenance policies and engineering strategies for civil infrastructure at risk and to establish the necessary investment priorities at a time of constrained resources. Sustainable decision-making for periods extending over multiple generations require new ways of thinking about life-cycle engineering. Such decisions involve ethical decisions that heretofore have not been considered, and should be aimed at maximizing the likelihood of successful future performance at a reasonable, if not a minimum, cost.

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Structural damage accumulation and control for life-cycle optimum seismic performance of buildings

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ABSTRACT: Seismic reliability and expected performance functions of structural systems are sensitive to the process of damage accumulation associated with the random sequences of ground motion excitations that those systems may experience. Optimum life-cycle-based engineering decisions must examine the influence of concepts related to both the target safety level of the initial system and the eventual repair and maintenance actions that may be undertaken during the life of the system. This presentation includes an overview of the general framework supporting these decisions, as well as some available results about a) the influence of damage accumulation of the seismic vulnerability functions of building structures and b) approximate estimates of accumulated damage, c) optimum damage threshold values for repair of structural frames or replacement of energy dissipating devices. Some comments are presented concerning desirable studies about life-cycle optimization of systems exposed to different types of excitations.

1 INTRODUCTION

Optimum life-cycle engineering decisions for structural systems to be built at sites affected by significant seismic hazard conditions are made by selecting the combination of seismic design criteria, quality control and repair and maintenance strategies leading to the minimum present value of the sum of initial-construction costs and those that may occur during the life-cycle of the system. The latter include those due to possible damage and failure, as well as to repair and maintenance actions in both structural and non-structural elements (Rosenblueth, 1976). All these costs depend on the evolution of the seismic vulnerability function of the system with time, as a consequence of the damage accumulation process (Esteva & Díaz, 1993; Esteva *et al*, 2011). For structural elements, repair and maintenance actions include, for instance, the restoration of the strength and stiffness of structural members and joints and the preventive replacement of energy-dissipating or seismic-isolation devices. Within this framework, optimum seismic design criteria also include the selection of the potential damage distribution patterns and of the locations in the structural system where damage can be made to concentrate, in case it occurs, aiming at repair actions that are more reliable, efficient and easy to perform.

2 SEISMIC VULNERABILITY FUNCTIONS

In practical engineering applications related to performance-based earthquake-resistant design, the estimation of the seismic reliability of nonlinear multi-story buildings for given values of the ground-motion intensity is ordinarily based on a measure of the probability that the lateral deformation capacity of the system, determined by conventional pushover analysis, is greater than the peak value of the corresponding nonlinear response demand for an ensemble of earthquake excitations with the specified intensity. However, probabilistic estimates of the deformation capacities of multi-story buildings obtained by means of pushover analysis are tied to severe limitations, because according to this approach it is not possible to account for a) the influence of cumulative damage associated with the cyclic response, and b) the dependence of the lateral deformation capacity on the response configuration of the system when it approaches failure.

Esteva *et al* (2003, 2006, 2010, 2011) presented a secant-stiffness-reduction index to be applied in the seismic reliability assessment of multi-storey buildings. According to it, the reliability of the system under the action of a ground motion of known intensity but uncertain detailed ground motion time history is expressed in terms of the probability density function of a secant-stiffness-reduction index (I_{SSR}):

$$I_{SSR} = \frac{K_0 - K_S}{K_0} \tag{1}$$

Here, K_0 is the initial tangent stiffness associated with the base-shear *vs* roof displacement curve resulting from pushover analysis and K_S is the secant stiffness (base shear divided by lateral roof displacement) when the lateral roof displacement reaches its maximum absolute value during the seismic response of the system. The failure condition is expressed as $I_{SSR} = 1.0$.

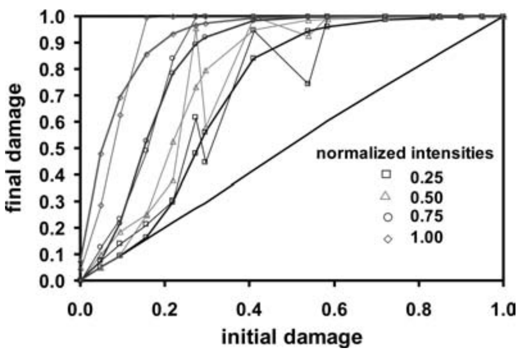

Figure 1. Influence of initial conditions on the damage potential of new earthquakes.

For a given value of the intensity (y), the probability density function of $Q = \ln I_{SSR}$ is equal to $f_Q(q)$, which is continuous for $q < 0$ and includes a discrete concentration at $q = 0$, which is equal to $p_F(y) = P[Q = 0 \mid y]$, the failure probability for an intensity equal to y. This approach has been adopted by Esteva *et al* (2011) in an exploratory study about the influence of initial damage conditions on the damaging potential of new earthquakes. Their results are presented in Figure 1, taken from the mentioned reference, where the normalized intensity is equal to the ratio of the ordinate of the linear displacement response spectrum to the deformation capacity of the system, in this case a large-span single-story frame with natural period equal to 0.43 s, subjected to simulated ground motion acceleration time histories with frequency contents and durations similar to those typical of a Mexico City soft soil site. The damage at the end of each seismic excitation, for given initial damage conditions, is measured by the value of I_{SSR}, as given by Equation 1.

3 INFLUENCE OF DAMAGE ACCUMULATION ON SEISMIC VULNERABILITY FUNCTIONS

3.1 *Damage indicators*

For purposes related to the assessment of seismic vulnerability functions of structural systems of multistory buildings, it is convenient to measure physical damage by means of indicators that are strongly correlated with the potential reductions in strength, stiffness, deformation capacity and energy-dissipation capacity of those systems when subjected to seismic excitations. Those indicators can be referred to the global or to the local properties of the system, either to critical sections of individual members or to assemblages of them, such as building stories. In those cases when the spatial distribution of damage is approximately uniform, global indicators of damage are sufficient to obtain reasonable estimates of the influence of the latter variable on the seismic vulnerability function of the system. In other cases, it will be necessary to identify the potential failure modes for the whole system or for significant portions of it that may be triggered by local failures at the members or system segments where damage concentrations take place; failure probabilities will then have to be estimated taking into account the possibilities of occurrence of all significant failure modes.

Both global and local damage can be measured by different types of indicators; some of them are based on peak amplitudes of response demands, while others take into account concepts related to dissipated energy or to low-cycle fatigue indexes.

3.2 *Damage identification and estimation*

Severe local damage concentrations at critical sections of structural members can often be visually identified. If the visual information suffices to make reasonable quantitative estimates of the damage levels at those sections, they can be incorporated into an updated mechanical model of the whole system and used to assess the expected influence of the residual damage on the potential increase of its vulnerability function under the action of a future external excitation, in this case an earthquake ground motion of unknown intensity. Decisions about repair and maintenance actions can then be made in terms of risk-acceptance and cost-benefit criteria, formulated within a life-cycle framework.

In many cases, visual inspection may not provide enough quantitative information about the local-damage levels and, as a consequence, about their potential impact on the increase of the seismic vulnerability function of the system. This may occur, for instance, if main structural members are hidden by some architectural finishing elements the removal of which might be considered too expensive; it may also happen in hysteretic energy-dissipating devices, which may not show any evidence of damage before the failure condition by low-cycle fatigue is reached. In many of these cases it would be justified to make preliminary estimates of local and global damage after the occurrence of moderate and high intensity events. These estimates can be derived from instrumental response records obtained during those events or from simulated time histories of local strains or distortions at critical sections, generated by means of mathematical models of the system; they can also be based on structural health monitoring studies performed after the occurrence of excitations deemed to be significant. If these damage estimates lead to significant increments in the values of the seismic vulnerability function, further actions should be taken to improve the knowledge of the decision maker about the local damage conditions and their possible impact on the updating of the latter function. The first step might be to perform a detailed visual inspection, covering all the critical sections that form part of each relevant global failure mechanism. Final estimates of the vulnerability function should include the information coming from the visual inspection as well as that derived from the previous steps. For this purpose, probabilistic models of the epistemic uncertainties associated with the local damage levels and the corresponding mechanical properties should be developed in accordance with Bayesian probability concepts.

3.3 *Spatial distribution of damage: Its implication on seismic vulnerability functions and long-term expected performance*

Spatial distribution of damage throughout a structural system is often characterized by significant irregularities. Damage concentrations tend to occur at those elements or sub-assemblages with the highest ratios of the magnitudes of the internal forces associated with the accidental excitations (earthquake ground motions, in this example) to those corresponding to the permanent loads. Because the mentioned irregularities determine the dominant potential failure mechanisms of the system under the action of future earthquakes, the functions relating the expected increments in the seismic vulnerability functions to the global indicators of initial damage are affected by very large uncertainties.

The influence of damage accumulation on the expected seismic performance of the systems can be expressed in terms of the increasing values of the expected failure rates ν_F, determined in accordance with the following equation:

$$\nu_F = \int_0^{\infty} -\frac{d\nu_Y(y)}{dy} P_F(y)dy \tag{2}$$

Here, $\nu_Y(y)$ is the seismic hazard function at the site, expressed in terms of the mean annual rate of occurrence of intensities greater than y, and $P_F(y)$ is the probability of failure given that the intensity is equal to y. This probability grows with time, as the accumulated damage grows.

4 LIFE-CYCLE ANALYSIS INCLUDING DAMAGE ACCUMULATION

Within a life-cycle framework, a utility function as given by Equation 3 is defined as the sum of the present values of the expected benefits and costs to be generated since the construction of the system to its eventual demolition or abandonment:

$$U = B - C_0 - E\left[\sum_{i=1}^{\infty} D_i^{-\gamma T_i}\right] \tag{3}$$

In this equation, C_0 is the initial construction cost, B is the present value of the expected benefits to be received while the system remains in operation, $E[\cdot]$ is the expected value, T_i is the unknown time of occurrence of seismic event i, D_i is the expected value of the costs associated with the potential damage generated by this event and γ is the discount coefficient that transforms instantaneous values at any time to present values at the decision-making time; as explained by Esteva *et al* (2011), D_i includes both the expected costs of damage in case of survival of the system and the expected cost of collapse multiplied by the failure probability for the intensity of the seismic event considered. In applications of Equation 1, the influence of damage accumulation is taken into account by adjusting the model used to estimate $E[D_i^{-\gamma T_i}]$ as a function of the damage indicator D_{i-1}, at the end of event $i-1$, and the potential reductions in the value of this indicator arising from the repair and maintenance actions that may be performed after that event.

Esteva and Díaz (1993) and Esteva *et al* (2011) present detailed probabilistic models to be used in the formulation of optimum decisions about minimum threshold values of residual damage levels demanding repair and maintenance actions, including possible preventive replacement of energy-dissipating devices. Those models include expressions for the transition probabilities of the value of D_{i-1} to that of D_i, as functions of the intensity of the i-th event. Because of the epistemic uncertainties affecting the values of D_{i-1}, as estimated from the information available after the occurrence of the i-th event, the probability of performing the required repair and maintenance actions when the specified threshold value is reached or exceeded will often be smaller than unity. This concept has to be taken into account in the formulation of practical decision criteria.

5 APPLICATIONS

5.1 *Reliability and expected-damage functions: Illustrative examples*

In the following paragraphs, a summary is presented of the main results of a study about a) the expected-damage produced by earthquakes of different intensities on several reinforced concrete buildings and b) the influence of damage accumulation on the seismic reliability and vulnerability functions of those systems, expressed in terms of the ground motion intensity (Vásquez, 2010). Two six-story and two ten-story buildings were considered, assumed to be built at a soft-soil site in Mexico City. Two reduced design spectra were considered for each pair of buildings, corresponding to two different values of the nominal ductility factor ($Q = 2, 4$) intended to account for non-linear dynamic response behavior. In the following, these buildings are designated as B6-2, B6-4, B10-2 and B10-4, where the first number in each case identifies the number of stories and the second number corresponds to the value of Q. The fundamental periods of the systems, including the influence of soil-structure interaction, are equal to 0.892 s and 0.984 s for buildings B6-2 and B6-4, and 1.04 s and 1.302 s for B10-2 and B10-4, respectively.

In order to account for uncertainties about the gravitational loads and the mechanical properties of structural members, a sample of random values of those variables was generated by Monte Carlo simulation. Each system generated in this manner was subjected to an artificial ground motion time history simulated in accordance with the criterion proposed by Ismael and Esteva (2006). In the dynamic response studies, the nonlinear behavior of the systems considered was assumed to be concentrated at plastic hinges at the ends of beams and columns. A stiffness degrading model proposed by Campos and Esteva (1997) was used to represent the constitutive functions relating moment and curvature at those hinges. For each structural member, the damage index was taken equal to the average of the local damage at its two ends. The global damage for the whole system, designated as I_{DF} in the following, was taken equal to the average of the damage at all the members. Considering that the global damage determined in this manner does not provide any information about the spatial distribution of damage and, therefore, about its implications on the reliability of the system as a whole, it was decided to define a normalized damage index, $I_{DFN} = I_{DF}/I_{FE}$, where I_{FE} is the value of I_{DF} required to produce global failure of the system. For practical reasons, it would have been convenient to determine the value of I_{FE} for any given system taking the values of the gravitational loads and of the mechanical properties of the structural members deterministically equal to their expected values; however, in the examples that follow, the value of I_{FE} for each particular system was determined using the random values of the system properties that were generated by Monte Carlo simulation, as proposed in the preceding paragraphs.

A comparison between the values of the damage and normalized damage indexes for systems B6-2 and B10-4 as functions of the intensity y has been presented by Vásquez (2010). The latter variable is measured by the ordinate of the linear pseudo-acceleration response spectrum of the ground motion excitation, for the fundamental natural period of the system considered and for a damping ratio equal to 5 percent; it is expressed as a factor to be applied to the acceleration of gravity. Large values of the ratio I_{DFN}/I_{DF} resulted from non-uniform distribution of local damage along the building height. These values are higher for $Q = 4$ than for $Q = 2$, as it should be expected, as a consequence of the higher levels of nonlinear behavior expected to be experienced by the former systems. For high intensities the values of I_{DF} are concentrated at two opposite ends: one corresponding to collapse and another covering a narrow band of low values. The wide variability in the values of I_{DF} can be ascribed to the statistical deviations in the mechanical properties of the members of the random population of structures used to account for the uncertainties about those properties for each system studied; however, the reason for having the values of I_{DF} concentrated near its upper and lower limits is not clear.

For a number of members in the sample of simulated systems associated with each of the basic systems (B6-2, B6-4, B10-2 and B10-4), the influence of initial damage I_{DF} on the final normalized damage as a function of the intensity was estimated by subjecting each of those simulated systems to ten ground motion time histories corresponding to each given intensity. The information obtained in this manner can be used to obtain cumulative damage functions for the basic systems considered, in terms of intensity and initial damage; these functions can be applied in engineering decisions related to optimum threshold damage levels for repair and maintenance actions.

The influence of initial damage on the failure probability functions of systems B6-2 and B10-4 is shown in Figure 2. These functions were derived from previously determined seismic reliability functions $\beta(y)$, following the definition by Cornell (1969). The latter functions were estimated using the maximum likelihood approach proposed by Esteva and Ismael (2003).

The seismic hazard function for the four basic systems considered are similar to that shown in Figure 3,

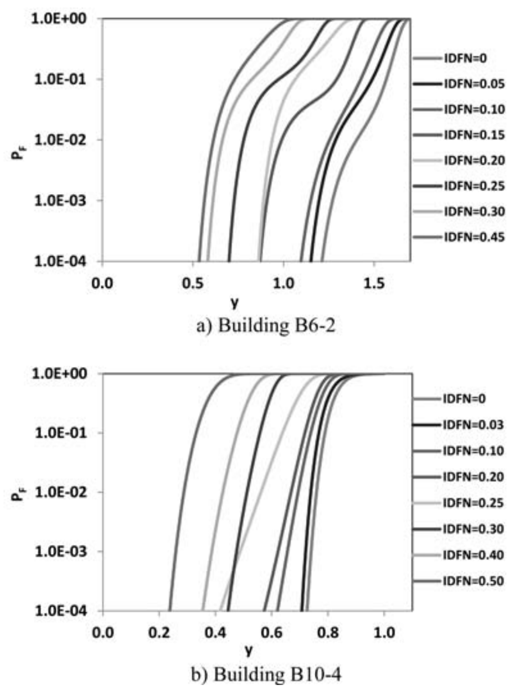

Figure 2. Influence of initial damage on the failure probability functions of systems B6-2 and B10-4.

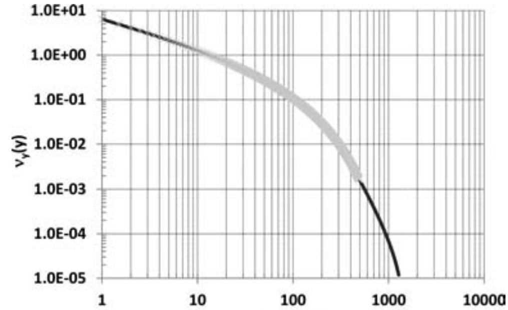

Figure 3. Seismic hazard function for system B6-2.

which corresponds to system B6-2; the intensities are measured by the ordinates of the linear pseudo-acceleration response spectra for 0.05 damping, for the fundamental natural periods of those systems, calculated taking the gravitational loads and the mechanical properties equal to the corresponding mean values. The resulting expected failure rates were determined according to Equation 2, in terms of the initial damage; they are presented in Figure 4 for systems B6-2 and B6-4. It was observed that a normalized damage index of 0.15 produces an increase by a factor of 10 in the expected failure rates in buildings B6-2 and B10-2 and by a factor of 4 in buildings B6-4 and B10-4.

5.2 *Optimum repair and maintenance strategies*

Several illustrative examples have been developed by Vásquez (2010) and León (2010), aiming at showing the impact of the damage-threshold level adopted for repair and maintenance actions within a decision-making criterion based on the maximization of the utility function given in Equation 3. The evaluation of D_i is made in accordance with the approach proposed by Esteva and Díaz (1993).

For simplicity, the following assumptions are made:

a) The expected damage terms D_i do not include the costs associated with non-structural damage for the condition of system survival, but only those when it collapses.
b) The initial cost C_0 corresponds to a system with mechanical properties resulting from previously established design requirements.
c) The damage threshold level is expressed in terms of the global indicator. It is assumed that when the repair and maintenance actions are performed the system as a whole returns to a condition of zero damage.
d) The probability of repair is equal to 1.0 once the specified threshold level is reached. This implies that the decision maker counts with perfect information about the global damage level at the end of each seismic event.

Life-cycle expected-damage-cost functions U/C_0 in terms of the damage-threshold repair index D_r were determined for systems B6-2, B6-4, B10-2 and B10-4, in accordance with Equation 3. Extensive results are presented by Vásquez (2010) and León (2010), both for RC frame systems and for cases including energy-dissipating devices. One particular case is shown in Fig. 5, where it is easy to appreciate that the utility function is not very sensitive to D_r for cases designed assuming a nominal value of the ductility factor $Q = 2$. This is a consequence of the lower damage levels expected for a system designed for higher lateral forces, because in these cases it was assumed that the moment-curvature constitutive functions for the structural members in the systems designed for $Q = 2$ and $Q = 4$ are the same. These results are consistent with those presented by Esteva *et al* (2011).

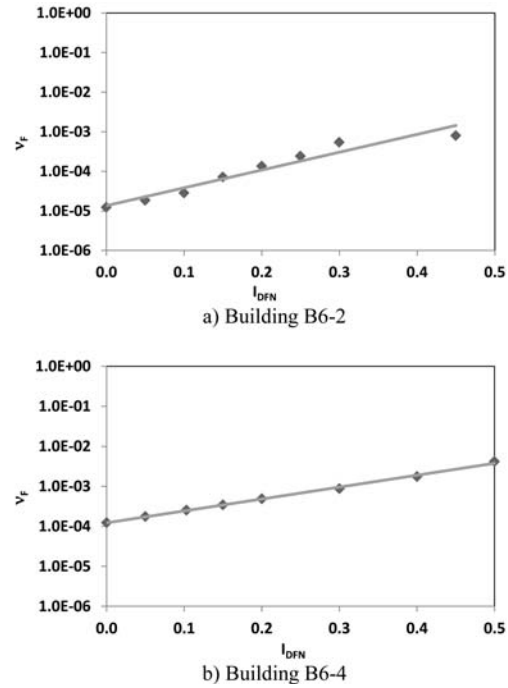

Figure 4. Influence of initial damage on expected failure rates.

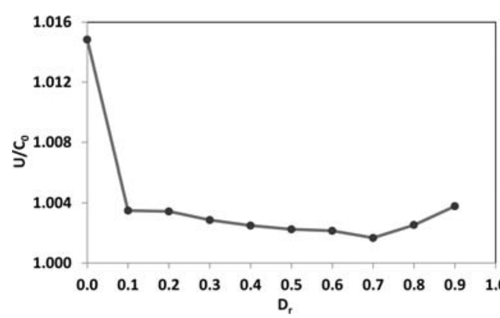

Figure 5. Life-cycle utility functions in terms of damage-threshold repair index, system B6-2.

6 DISCUSSION AND CONCLUDING REMARKS

6.1 *General conclusions*

The studies reported here are intended to provide some gross information about the potential influence of structural damage accumulation on the expected seismic performance of multistory buildings. Its scope is limited to structural damage due to seismic excitations; however, it is recognized that damage due to other excitations, such as differential settlements, can also be of high relevance in this respect.

For simplicity, in the life-cycle studies presented here structural damage is expressed in terms of a global indicator; the influence of spatial distribution is disregarded. Therefore, decisions about repair of structural members and replacement of EDD's are not performed on a member-by-member basis, but in accordance with the global-damage indicator. This simplification may contribute to significant uncertainties in final damage estimates.

For the cases studied here, it was found that increments by a factor as high as 10 in the expected failure rates can be generated by residual damage levels of about 0.15.

Because of the many variables that affect spatial distribution of damage on different structural systems, it is not considered feasible to perform parametric studies about the concepts studied in this article; it is only intended to present a glimpse about the orders of magnitude of the influence of damage accumulation on some relevant indicators of life-cycle seismic expected performance for some typical building systems.

6.2 *Recommendations for future studies*

Near-future research activities are should be aimed at improving engineering tools for the assessment of the influence of initial structural damage on the seismic reliability and expected performance of multistory buildings and for the selection of efficient repair and maintenance strategies. Some specific concepts requiring attention are:

a) Efficient criteria and methods to estimate and identify local damage and its relation with global damage, including both instrumental records and analytical models.
b) Selection of "efficient" damage distribution patterns; *i.e.*locations where damage should be made to concentrate, so that they lead to easy, reliable and efficient retrofit actions.
c) Application of the foregoing concepts to three-dimensional systems subjected to two simultaneous horizontal components.

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Sustainable design of structures: The outcomes of the COST Action C25-WG3

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ABSTRACT: This lecture reports the outcomes of four years of activity carried out within the framework of the COST Action C25 by the members of the Working Group 3 (WG3). After an introduction of the project as a whole, the main results achieved by WG3 are presented and discussed. The main topics covered within WG3 dealt with life-time damage processes, life-time assessment and design, life-time monitoring and condition assessment, maintenance repair and rehabilitation as well as demolition and deconstruction of buildings and bridges. More than 50 scientists and experts coming from all over the world contributed to the work of WG3 with the aim to collect methods and practices in the field of sustainable structural design. The Working Group Members have collaboratively examined theories, methods and tools sharing past acquired knowledge and cooperating for the development of joint case studies within the project.

Nowadays, the enhancement of sustainability of the built environment has become a pressing issue touching all the construction industry and related activities.

In this framework, the Action C25 "Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering" (Braganca et al., 2011a), was launched in 2006 with the aim to promote a scientific understanding of life-time engineering and to boost science-based advancement of sustainable construction in Europe.

The project involved more than one hundred researchers, engineers and architects from 28 countries and it was focused on the problem of "Sustainability of Constructions" which refers to the combination of methods of structural engineering with those of environmental impact assessment and life-cycle economy.

The Action C25 was organized in three Working Groups in accordance with the main research areas identified as necessary for the objectives:

- WG1 – Criteria for Sustainable Constructions;
- WG2 – Eco-efficiency;
- WG3 – Life-time structural engineering.

WG1 was devoted to the collection and review of global methodology for the assessment of Sustainable Design and Construction with a specific focus on of standards and literature on Life-cycle cost and environmental impact analysis (Braganca et al., 2011a).

WG2 dealt with the application of new materials and technology for the improvement of the environmental performance, the comfort and the energy performance of buildings (Braganca et al., 2011a).

WG3 dealt mainly with topics related to design for durability, life-cycle performances, condition assessment, maintenance and repair techniques as well as problems related to end of life of constructions (Landolfo, 2011).

Figure 1. The COST Action C25 logo.

The main objective of WG3 was the collection of methods and practices in the field of life-cycle structural engineering with a strong emphasis on the question related to sustainability.

More than 50 scientist and experts coming from all over the world contributed to the work of WG3 with the aim to collect methods and practices in the field of sustainable structural design.

The Working Group Members have collaboratively examined theories, methods and tools, sharing past acquired knowledge and producing novel scientific results within the project.

In order to cover to analyze the different stage of the life-cycle according to a performance based approach, WG3 has been organized in two Working Packages, namely WP8 and WP9.

The main aim of WP8 (Landolfo & Vesikari, 2011) was to analyze the different procedures and methods for service life design of structures.

In particular, the main tasks of WP8 were the survey of the state-of-the-art concerning verification methods for service life design and the analysis of degradation models for prediction of durability performances over time.

WP9 (Hechler, 2011) was mainly concerned with the problem of monitoring the life-cycle performances, maintenance and end-of-life scenarios.

The main tasks of WP9 were: the analysis of maintenance, repair and rehabilitation techniques and planning; the survey and condition assessment of structures in terms of safety and functionality; the classification and planning of demolition and deconstruction systems.

In this context, several contributions were prepared by WG3 members and invited experts during the four years of the Action, which have been collected in a final summary report (Braganca et al., 2011b).

The report has been organized in five main sections, each devoted to a specific stage of the construction life-cycle, from the design stage until the end of life.

In line with that, the first part of the summary report is devoted to a critical review of the different methodologies developed in the framework of international scientific literature for the verification of the structural performance over the life-cycle An overview of service life design methods outlined in international standards and codes has been presented as well.

The second part of the report deals with the problem of modeling the deterioration of structural materials which represent one of the most critical issue for the life-cycle performance evaluation of structures.

In this context, WG3 attempted to provide an overview of the degradations models developed in the framework of scientific literatures for concrete, steel, masonry and timber structures.

The third part collects information concerning the state of art, the specification, the implementation and the operation of monitoring systems. Besides different methodologies developed for the condition assessment of buildings and bridges are reported as well.

The fourth section is devoted to the problem of maintenance, repair and rehabilitation of building and bridges. Following a general presentation of computer based systems for the management of constructions, different theories and techniques of structural interventions are reported.

Finally the last part of the report is devoted to the end of life of the constructions covering topics related to demolition and deconstructions methods and practices. In particular, the key principles of the design strategies aimed at improving the re-use and recycling of buildings and components are presented, including design for adaptability, for dismantling and design for deconstruction.

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Sustainable asset preservation at the Austrian Federal Railways

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ABSTRACT: Increasing degree of track volume, higher transport tonnage, shorter intervals for suburban trains and increasing speeds are the most important features of the Austrian rail-network from the customers' point of view. The basic conditions mentioned before and the rising cost pressure call for higher requirements for the maintenance of the track system. One of the most important aspects for the ÖBB-Infrastruktur AG is the sustainability of the track maintenance. To achieve this sustainability it was necessary to implement a batch of several tasks that are based on LCC analyses. LCC analyses to gather the cost drivers, LCC strategies for the main fields of application, Life-Cycle Management of the Permanent Way, New track machinery, further development of the Condition Monitoring, Design Engineering with the focus on LCC reduction, Implementation of a Life-Cycle Management group. ÖBB infrastructure is doing the fully decision making based on LCC. The positive results of the work done in the last years can be shown in many figures, e.g. the reduction of the number of slow orders.

1 LCC COST ANALYSES

Since 1998 ÖBB infrastructure cooperates with the Technical University of Graz in the field of LCC analyses [1]. These LCC analyses were done for the relevant fields of application, starting with the permanent way (PWay). The calculation brought up a deeper understanding of the cost drivers.

The PWay analysis delivered the normalised annual costs divided into depreciation costs (investment costs divided by the life span), operational hindrance costs (costs for the effect of maintenance based track possession) and the core maintenance costs. The LCC analysis of bridges, tunnels and the catenary show the same results. The depreciation costs are always the main cost drivers. But also the high depreciation costs have to be taken into account when choosing the right track maintenance and renewal method.

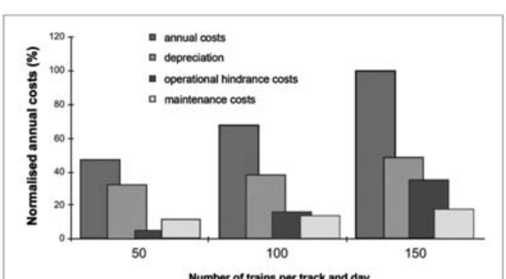

Figure 1. LCC analysis of the permanent way.

2 STRATEGIC OBJECTIVES

The strategic objective of the ÖBB-Infrastruktur AG is to increase the reachable service life of the infrastructure components and the reduction of the need of maintenance at once. This is not an impossible task, but it requires an appropriate design and construction and a consequent realisation of the maintenance strategy.

Track closures have to be reduced as much as possible. To achieve this aim, different maintenance works were concentrated to one track closure during the last years as far as possible. The long-term planned increase of the maximum axle load from 22.5 to 25 tons also has to be considered when maintenance work is performed.

3 LCC STRATEGIES FOR THE MAIN FIELDS OF APPLICATION

Based on the LCC calculations a LCC strategy was developed for the main fields of application. Due to the influence of the depreciation costs the goal must be a prolonging of the service life of the main technical equipment. Of course you should pay attention to the investment costs, but installing products with a reduced life span is never economical. To reduce the operational hindrance costs it is necessary to install

Figure 2. Reinforced concrete bridge at ÖBB.

components with low maintenance needed. A prolonged service life in combination with a reduced maintenance is the goal to be achieved, even the investment costs are a bit higher. Fig. 2 shows a modern bridge installed at ÖBB. The calculated life span is 100 years. Instead of the former steel bridges just little maintenance is needed to provide the functionality over this long time [2].

4 LIFE-CYCLE MANAGEMENT OF THE PERMANENT WAY

Since 2005 technical investment strategies exist for the main fields of application. Due to the situation that a big amount of the maintenance budget is used for the permanent way special attention was paid for that subject. In the last 10 years a modern Condition Monitoring was developed to give the precise track condition and the need of maintenance. A focus was also laid on the Design Engineering of the tracks, esp. in sharp curves. Additionally ÖBB has installed a life-cycle management group that is regularly doing LCC analysis when tracks have to be renewed.

5 NEW TRACK MACHINERY

Due to the influence of the operational hindrance costs ÖBB is forced to use the state-of-the-art maintenance and renewal machines with highest output and highest quality.

An important maintenance work of the permanent way is to ensure the correct track geometry quality. In the ÖBB-network this tamping work is done with the so called MDZ machinery. MDZ 3000 exists of a 4-sleeper-tamping-machine 09-4X, the track stabiliser DGS and the ballast distribution system BDS.

The tamping machine 09-4X delivers highest output with highest quality. Within one operation cycle 4 sleepers are tamped at the same time. After this work the track has to be stabilised by the track stabiliser that is doing the initial compacting and lateral stabilising of the track. The BDS reinstalls the correct ballast profile and due to the possibility of temporarily ballast storing the ballast can be moved to sections where it is needed in an economic way.

The track renewal is also done by rail-mounted machinery. Highest initial quality is the goal to be

Figure 3. MDZ machinery at work (tamping machine, track stabiliser, ballast distribution system).

Figure 4. Subsoil rehabilitation with PM 1000 URM – 5 layers can be installed in one working cycle.

achieved. With the modern machinery a track renewal of a 5 km section lasts only 10 days, including the additional side work.

The economic renewal length in most cases is the renewal of a whole section between two stations (in Austria approx. 4 to 5 km). Additional drainage and subsoil works should be finished within the renewal time.

The high initial quality can be measured by the PWay measurement coach. The longitudinal level of the track (vertical irregularities) is showing values between 0.3 and 0.4 mm standard deviation.

In the last 18 years about 700 km of the tracks in Austria were subsoil-rehabilitated with high economic and high output machinery. Instead of the classic way – to use diggers and lorries – nowadays the formation layer of 0.4 m is installed with rail-mounted machinery. The machinery uses the former existing and then crushed ballast material to reinstall it as formation layer material. The optimum moisture content helps to get an optimum of the compacting of the formation layer.

With the newest system – the so called PM 1000 URM – the machinery was further developed. 5 layers can be installed in one working cycle and the removed material can be 100% recycled.

6 FURTHER DEVELOPMENT OF THE CONDITION MONITORING

Two to four times a year the Permanent Way measuring coach EM 250 is running on the tracks of ÖBB. EM 250 is doing it up to a speed of 250 km/h and is measuring more than 100 signals every 25 cm, e.g. longitudinal level of the track, the wear of the rail and the curvature.

Figure 5. The Permanent Way measuring coach EM 250 is running with a speed of up to 250 km/h.

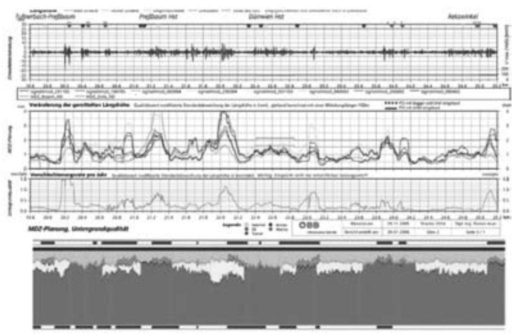

Figure 6. Example of the NATAS (new Austrian track analysing system) chart – overlay of track measuring signals and georadar information.

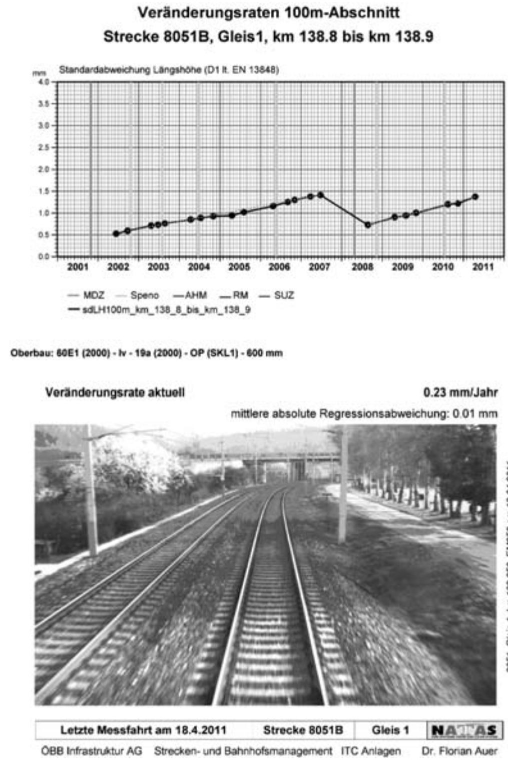

Figure 7. History of a certain 100 m section of the track.

Since 2001 the data of the PWay coach is stored in digital form. The measurement signals can be easily combined with other information. Fig. 6 shows one of the so called NATAS (new Austrian track analysing system) charts. The history of the track degradation (sections with bad subsoil show higher settlement rates) is overlayed with a diagram coming from georadar measurements. This analysis tool helps to do maintenance planning with a new quality. Not the effect has to be maintained and repaired but the root cause. In many cases the condition of the drainage is influencing the track quality in a bad way.

For every 100 m section of the ÖBB network it exists a "digital history" of 10 years (Fig. 7). The degradation rate can be recalculated from this diagram. Long term effects, e.g. coming from the formation rehabilitation, can be shown in a modern way.

Fig. 8 shows another NATAS chart. Due to the poor condition of the fastenings the rail foot distance (inner distance between the rails) becomes bigger. The track masters can find these sections via analysing the NATAS charts. A big step in the Condition Monitoring.

The measuring signal rail inclination allows detecting the condition of the rail pads – the plastic or rubber material that is used for the load distribution between rail and sleepers.

The lateral forces in curves lead to a one-sided load of the rail pads. The pads therefore show this one-sided wear behaviour. With the measuring signal rail inclination the condition of the rail pads can be recalculated. The signal allows doing a strategic planning of the pads. The pads are not very costly itself, but the repading work. And – if the repading was done too late – the life span of the whole track can be reduced significantly.

An additional effort is that with consequent analysing of the signal rail inclination some rail pad material with a better wear behaviour was found.

This is the output of a test track in Upper Styria where ÖBB had installed a test run with 22 different rail pads. The new pads cost a bit more than the former one, but show a significantly better wear behaviour.

The rails all over the world suffer more and more from rolling contact fatigue, this is a result of using more powerful locomotives. The failure type is not serious at the early stage but can lead to a breakage of the rails over the time if the countermeasure rail grinding was done too late. Therefore it is necessary to detect the early rail cracking in the very beginning. A special test method was developed in the last years. This eddy current testing is more and more used to detect sections with beginning rolling contact fatigue on the rail head. The results are consequently used for the maintenance planning.

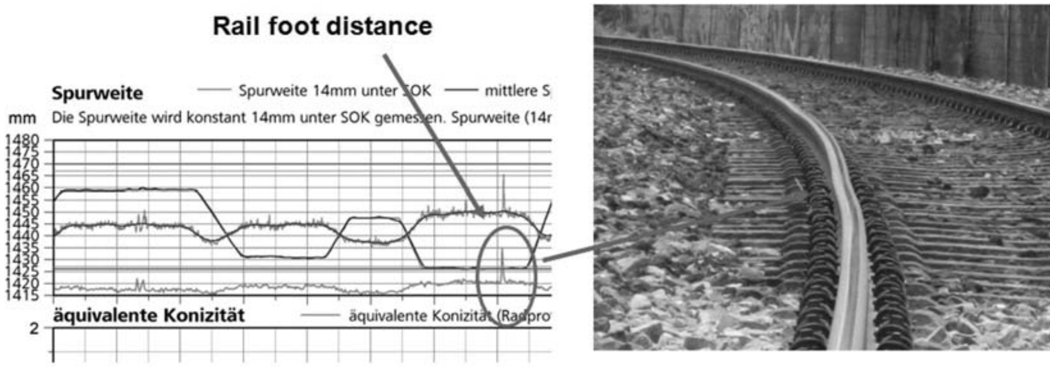

Figure 8. With the measuring signal rail foot distance the track masters can easily find sections with a poor condition of the fastening system.

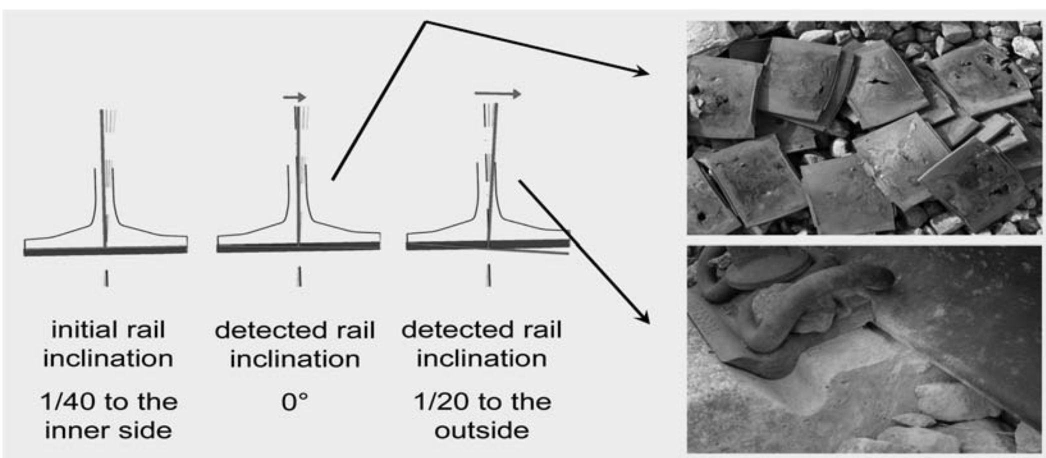

Figure 9. The measuring signal rail inclination allows detecting the condition of the rail pads.

7 DESIGN ENGINEERING WITH THE FOCUS ON LCC REDUCTION

One aspect of railway systems – including the track system – are long service life. The long life-cycles can only be achieved with a suitable design of the track and other technical lineside equipment combined a high-quality maintenance.

The designated life-cycle for track systems depends on the curve radius and traffic and is between 30 and 37 years.

Unfortunately, many tracks had to be renewed earlier due to different reasons. Two of those reasons are decisions which were made in the past and have not been sustainable.

- For the change from the 49-kg rail to the new 54-kg rail, the ÖBB-Infrastruktur AG used a geometric unbalanced rail 54 E2.

This rail has a too low bottom width of only 125 mm compared to its weight per metre. This lead to higher pressures in the rail fastening caused by higher rail head deflections. This is a disadvantage in the curvy Austrian rail-network

- Furthermore, the rail fastening on concrete sleeper tracks was changed from ripped plates to the W-fastening which has not yet been strong enough.

40% of the Austrian track is located in curves. Therefore it is important to install rails with reduced wear rates. Modern rails are head hardened, a special production method as shown in Fig. 11. The Brinell hardness of these rails increases to a value of 350 and higher. Additional investment costs of 10% on the one hand, an increase of the service life of the rails with the factor of 3 (sharp curves) on the other hand.

A new track construction for narrow curves was developed in the last years. The new track shows little maintenance needs and an expected prolonged service life. The rails do not show the resonance phenomena of rail corrugation – they are typical for narrow curves –

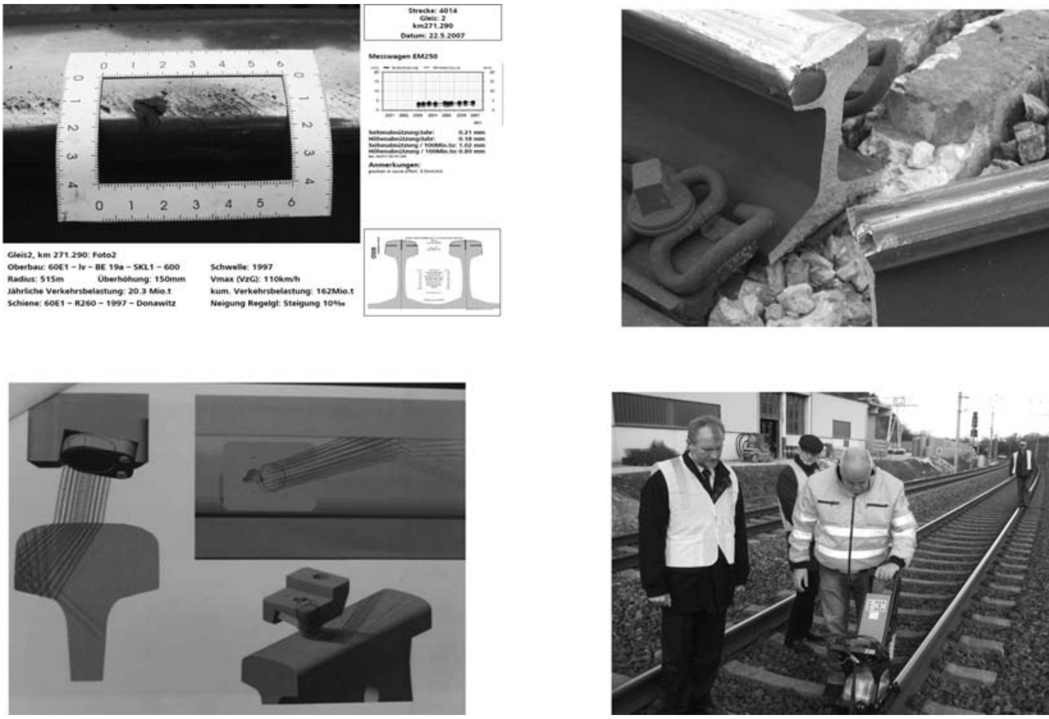

Figure 10. Eddy current testing is more and more used to detect sections with rolling contact fatigue on the rail head.

Figure 11. Head hardening of rails.

anymore. Corrugated tracks are up to 15 dB louder than non-corrugated ones [3].

An additional component is used regularly in the ÖBBs' tracks. Under sleeper pads help to reduce the forces within the ballast bed. The output is a prolonging of the service life of the ballast bed (Fig. 13).

8 LIFE-CYCLE COST ANALYSIS FOR EVERY TRACK RENEWAL

ÖBB has established a life-cycle management department that is analysing all bigger investment projects. The decision to do a complete track renewal is therefore done by the new analysing tools (the technical documentation) in combination with an LCC analysis (the economical documentation). This helps to do the

Figure 12. Effect of the new track design – the reduction of the rail corrugation: track construction 2000 (top) and 2010 (bottom).

Figure 13. Reduction of ballast wear due to the installation of under sleeper pad.

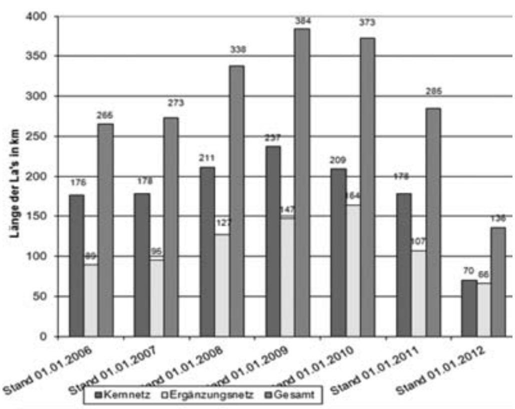

Figure 14. Reduction of the number of slow orders since 2009 – one output of the consequent life-cycle decision making of ÖBB-Infrastruktur AG.

decision making with a new quality – the goal "to do the right measure at the right time" is achieved.

9 QUALITY FIGURES

Fig. 14 shows the output of the consequent LCC-based decision making. Since the implementation of the ÖBB-Infrastruktur AG on Jan 1st 2009 the number of condition based slow orders could dramatically be reduced.

10 SUMMARY

Much efforts have been put on the life-cycle cost aspect in the last years. The ÖBB-Infrastruktur AG is trying for several years to provide a sustainable maintenance of their infrastructure. The aim is to plan the maintenance and renewal works so, that the best service life with proper dimensioned track systems can be achieved.

The results and quality figures show that the Austrian Federal Railways are on the right way regarding the sustainability of the technical equipment. ÖBB infrastructure is ready to tackle the tasks of the future.

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MINI-SYMPOSIA

Vibration-based health monitoring, damage identification, and parameter estimation for civil engineering structures

Organizers: C. Papadimitriou, G. Lombaert, G. De Roeck & E. Reynders

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Dynamic methods for health monitoring and structural identification of bridges

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ABSTRACT

Assessment of structural conditions of bridges by means of dynamic testing is a crucial task in every effective maintenance program. However, mainly due to economic reasons, bridge structures and infrastructures are usually monitored through visual inspections without performing any experimental test. Dynamic forced vibrations are carried out only in special cases, when the relevance of the structure is high or the damage conditions are critical. On the contrary, dynamic testing in operational conditions is directly related to low costs and simplification of in-situ surveys suitable for a large number of bridges.

Dynamic data are important per se, but they can provide more meaningful results if they are used to validate a mathematical model of the structure under investigation, enabling to estimate important mechanical properties, such as the stiffness coefficient of structural elements and boundary conditions. Identification of structural modifications is another important goal of dynamic testing on bridges. Repeated tests over time can indicate the emergence of possible damage occurring during the structure's lifetime or the effect of restoration works carried out on the bridge, providing quantitative estimates of the level of residual safety.

Recent technological progress has generated accurate and reliable experimental methods, enabling good estimates of the dynamic response of a structural system. Although experimental techniques are now well-established, the interpretation of measurements still lags somewhat behind. This particularly concerns structural identification and damage detection due to their nature of inverse problems. Indeed, in these applications it is desirable to determine some mechanical properties of a system or to improve the description of some structural component on the basis of measurements of its response. Hence, typical aspects of inverse problems arise, such as high nonlinearity, nonuniqueness, or noncontinuous dependence of the solution on the data. When identification techniques are applied to the study of real-world bridges, additional obstacles are found given the complexity of structural modeling, the inaccuracy of the analytical models used to interpret experiments, measurement errors, and incomplete field data.

Development of dynamic methods for health monitoring and structural identification of bridges based on output-only vibrations is the objective of a joint research program conducted by the Universities of L'Aquila, Rome and Udine in cooperation with Local Public Territorial Authorities. In this paper, after describing in section 2 the role of dynamical tests in SHM programs, and also their actual limits, two main case studies have been considered. The results of a series of output-only vibration tests carried out on a new curved, fifteen-span, post-tensioned reinforced concrete viaduct are presented and discussed in section 3. The viaduct belongs to the highway line connecting the cities of Pordenone and Conegliano, in the North East of Italy. Main purpose of the experimental survey was the evaluation of the dynamic performance of the viaduct and, in particular, of the deck super-structure under regular traffic load. Section 4 concerns with the output-only based dynamic characterization of a class of existing twin arches bridges located near the city of Teramo, in Central Italy. In particular, only the case of the Valle Castellana Bridge is described in detail. The main purpose here was to quantify the effects of some restoration works that were needed on the bridge after an exceptional flood on 2008. Both the experiences have shown that dynamic test data contain essential information in order to improve the knowledge of the actual dynamic behavior of a bridge and to evaluate its structural performance.

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Structural health monitoring of a centenary iron arch bridge

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ABSTRACT

The San Michele bridge, also known as Paderno bridge, is an iron arch bridge (Fig. 1) built in 1889 about 50 km far from Milan. It is the most important monument of XIX century iron architecture in Italy and still used as a combined road and railway bridge (Nascè et al. 1984).

In order to assess the structural health condition of the historic bridge, Ambient Vibration Tests (AVTs) were performed in 2009 on the roadway deck of the bridge. These tests (Gentile & Saisi 2011) represented the first experimental survey carried out on the global characteristics of the bridge since the original static proof tests (1889–1892) and highlighted that: (a) the vertical bending modes were non-symmetric with respect to the vertical plane containing the longitudinal axis of the bridge, indicating the different state of preservation of the iron members on the downstream and upstream sides; (b) under the service loads (road traffic), the natural frequencies of vertical bending modes exhibited slight variations, possibly depending on the excitation/response level.

In addition, ambient vibration modal testing and analysis turned out to be effective tools for assessing the structural condition of the historic bridge. Consequently, the main institutional owner of the bridge – the Italian Railway Authority (RFI) – decided to install a continuous monitoring system on the bridge.

In the first part of the paper, after a description of the bridge, some results of the AVTs carried out in March 2010 (with the aim of defining the position of the sensors to be permanently installed in the bridge) and in June 2011 (in order to further verifying the design of the monitoring system) are presented.

In the second part of the paper, some details are given on the monitoring system (Fig. 1) and the software developed to continuously process the raw data received and to automatically extract the modal parameters. In order to perform a reliable and robust automated modal identification, the Frequency Domain

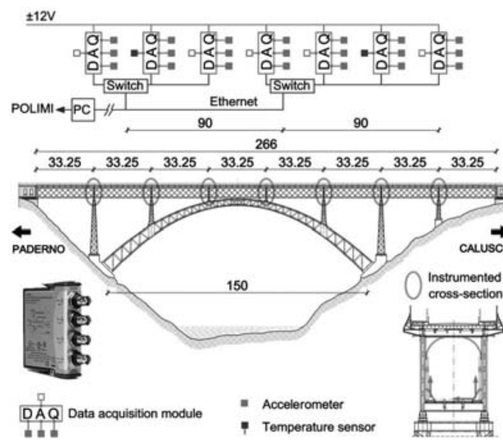

Figure 1. Elevation and cross-section of the San Michele bridge and schematic of the monitoring system (dimensions in m).

Decomposition (FDD, Brincker et al. 2001) technique has been considered and a procedure for its automation has been developed. Finally, the tracking of natural frequencies – obtained by applying the automated FDD technique to the data collected in the first weeks of continuous monitoring – is presented and discussed.

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Structural identification of a super-tall tower by GPS and accelerometer data fusion using a multi-rate Kalman filter

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ABSTRACT

This work outlines a procedure for the effective fusion of different types of sensor data, namely acceleration and displacement records, aiming at the successful simulation and monitoring of the behavior of large structures under dynamic loading. Recent advances in technology such as the improvements in Global Position System (GPS) receivers, have offered easier access to displacement information, while acceleration is commonly available for instrumented civil structures. In this study, a method of combining displacement and acceleration data for the purpose of structural health monitoring is demonstrated and the example of a new innovative high-rise structure in China, featuring a fully functional on-line monitoring system is conceived as a test-case. The overall goal is to successfully fuse the heterogeneous measurements, obtained on-line while sampled at different frequencies. The tools employed involve a multi-rate Kalman Filter scheme, as presented in (Smyth & Wu 2007) and an artificial White Noise (WN) observation technique necessary for eradicating the displacement drift issue which stems from the integration process. As shown in the application section, this methodology is able to increase the accuracy of the estimated quantities against the use of conventional numerical processing methods. The application presented herein is inspired from an actual experimental campaign employed for the case of a super-tall tower structure in China, as reported in (Casciati & Fuggini 2009), (Casciati, Saleh, & Fuggini), (Faravelli, Ubertini, & Fuggini). Although numerically generated, data is assumed available from GPS receivers and uniaxial accelerometers in the manner that these have indeed been installed on the top level of the referenced tower.

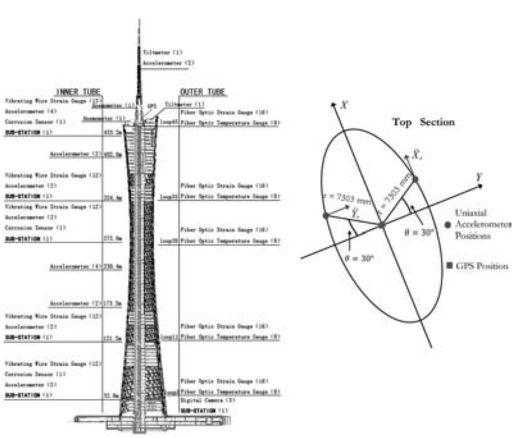

Figure 1. a) Actual tower instrumentation pattern taken from (http://www.cse.polyu.edu.hk/benchmark/index.htm), b) Location of the assumed measurement points for both GPS and accelerometer sensors.

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Maintenance and rehabilitation of 19th century masonry buildings – life-cycle aspects

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ABSTRACT

In Central Europe cities, a considerable high percentage of existing building stock was constructed during the second half of the 19th century in the so called period of promoterism. Considering the (further) life-cycle costs of refurbished buildings of this age, the knowledge about the remaining lifetime of the load-bearing construction is one of the essential input parameters.

In the course of investigations within the scope of the Institute for Building Construction and Technology over more than 20 years it turned out that evaluation and restoration of masonry and timber constructions has a considerable impact on the life-cycle costs for these buildings.

Considering the future life-cycle costs of these buildings the following main aspects have to be considered:

– Evaluation of the actual loadbearing capacity of timber floor constructions and masonry
– Earthquake resistance of masonry constructions due to mechanical properties of the mortar
– Evaluation of methods for repair and consolidation under the aspects of sustainability

Several methods are presented which focus on the detection of sever damage and on the rehabilitation of existing structures in order to extend the lifetime of existing structures for another 100 years.

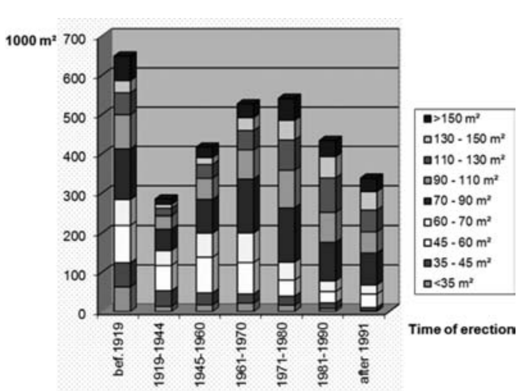

Figure 1. Age of residential buildings in Austria (Statistics Austria).

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Dynamic damage identification using linear and nonlinear testing methods on a two-span prestressed concrete bridge

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ABSTRACT

Today, the reliability of civil engineering structures, especially bridges, is a crucial issue regarding their growing amount due to expanding mobility facilities. Nowadays, as the cost for planning and construction is continuously increasing, service guarantees and regular inspections get more and more important to assure the serviceability of the engineering structures. Furthermore, these inspections encounter sometimes difficulties due to the complex handling, associated with huge costs. Therefore, research is conducted on several inspection techniques to simplify and improve existing procedures and to introduce new methods. The University of Luxembourg also studies in this field of condition control of civil engineering structures by using dynamic and static testing methods. These have the advantage that they can easily be set up or implemented for condition monitoring.

In this paper the dynamic tests are carried out on an artificially damaged two span prestressed concrete bridge of 102 m length, which is afterwards demolished. The artificial damage is introduced by cutting part of the prestressed tendons at a defined location and by additional charging of the bridge with an experimental load. The bridge is excited by swept sine signals generated by special designed mobile machines, which are able to control and measure the excitation force amplitude and frequency in order to generate well determined and clear signals above the noise level. For each damage and loading scenario, changes of the investigated parameters are explored and compared to the reference state to identify damage and thereby validate their individual potential as damage indicator. The focus is held on the analysis of modal parameters, i.e. the eigenfrequencies, modeshapes and their normalisation factors compared to the initial state in order to highlight changes. The normalised eigenvectors permit to calculate the flexibility matrix, i.e. the inverse of the stiffness matrix, of a structure and, thus, to detect and even localise modifications.

Moreover, studies on nonlinear phenomena, such as an excitation-force-amplitude dependency of the eigenfrequencies, which is known to emerge at concrete structures and the appearance of higher harmonics due to increasing cracks in the concrete, are examined. However, regarding these nonlinear parameters, no exact statement on the condition can be made, since the amplitude dependency shown for different eigenmodes does not show any correlation to the damage.

In contrast, for the linear investigated values, the most influenced parameters turn out to be the eigenfrequencies and modeshapes for some specific eigenmodes. Regarding firstly the eigenfrequencies, clear decreases are recognisable and are conform to the damage introduced into the bridge. These variations on the eigenfrequencies identify the changes on the structure, i.e. cracks, but do not afford localisation considering only this parameter. The analysis of the modeshapes is evaluated for this issue. Changes are evident, but the localisation regarding the identified modes rests difficult. Therefore, the flexibility matrices, calculated from the identified modal parameters, are analysed to discover the loss in stiffness. They yield accurate results indicating and even, in contrast to the eigenfrequencies and modeshapes, localising damage in the bridge. Moreover, the differences of the diagonal elements of the flexibility matrices show clearly the location of damage, making this investigated parameter adequate for further research on damage assessment on civil engineering structures in situ.

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Structural health monitoring from on-line monitored vibration measurements

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ABSTRACT

Condition assessment of existing civil infrastructure systems has become a critical issue due to the deterioration of structural performance and the requirements of structural safety. More reliable methods are constantly required to detect and quantify local damage in structures. This study attempts to develop a method for on-line structural health monitoring by detecting local damage in structures. Structural damage is assumed to be associated with the reduction of structural stiffness, which is represented by the change in the coefficients associated with element stiffness matrixes. These coefficients are then used as damage parameters in structural damage identification. Dynamics characteristics of the structure are calculated by using Newmark's numerical integration method based on measured acceleration data. The vibration data of undamaged and damaged structures are directly adopted into the basic equation of motion for the structure. Both the location and extent of the damage are determined based on the inverse predictions of damage parameters of the individual elements. Moreover, real time changes of structural damage in the elements can be determined from on-line monitored vibration measurements.

In this study, a numerical example for a plane frame structure is utilized to demonstrate the effectiveness of the proposed approach. The structure is divided into a number of elements in order to obtain better predictions of structural damage at more detailed level. Figure 1 shows the results for the simulated damage scenario involving the loss of stiffness in three elements, namely element numbers 10, 17 and 32 with amount of 5%, 25% and 10%, respectively. The location and extent of the simulated damage are correctly identified from the continuously monitored vibration data with considerations of damping effect in the structure.

The results for the inverse predictions of various simulated damage scenarios indicate that structural damage can be accurately identified at more detailed level in terms of location and extent in the structure, even in the case when the number of damage parameters is relatively large. The new proposed technique performs well and produces stable and reliable results from the vibration measurements. The proposed method therefore could be used to assess on-line structural condition when local damage exists in structures.

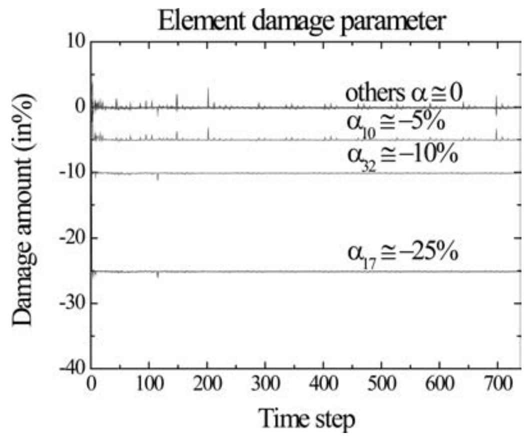

Figure 1. Inversely identified damage parameters in real time for the simulated damage scenario where element 10, 17 and 32 damaged by 5%, 25% and 10%, respectively.

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Damage detection on the Champangshiehl bridge using blind source separation

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ABSTRACT

This paper addresses the problem of damage detection in civil engineering structures using characteristic subspaces obtained from Principal Component Analysis (PCA) of output-only measurements. Damage detection is performed by comparing subspace features between a reference (healthy) state and a current (possibly damaged) state. The damage indicator used in this study is the angular coherence between subspaces.

The considered damage detection procedure is illustrated on the Champangshiehl Bridge which is a two span concrete box girder bridge located in Luxembourg. Before its destruction, multiple damage levels were intentionally created by cutting a growing number of prestressed tendons. Vibration data were acquired by the University of Luxembourg for each damaged state at many locations on the bridge. As previous studies demonstrated the large importance of environmental factors on modal identification, special care was taken to evaluate this influence during the test campaign.

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Fast Bayesian structural damage localization and quantification using high fidelity FE models and CMS techniques

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ABSTRACT

Bayesian estimators (Ntotsios et al. 2009) are proposed for damage identification (localization and quantification) of civil infrastructure using vibration measurements. Damage occurring at one or more structural components can be monitored by updating an appropriately parameterized Finite Element (FE) model with parameters associated with the properties of the monitored structural components. The actual damage occurring in the structure is predicted by Bayesian model selection and updating of a family of parameterized model classes with the members in the model class family introduced to monitor the large number of potential damage scenarios covering most critical parts of the structure. Bayesian inference ranks the plausible damage scenarios according to the posterior probability of the corresponding parameterized FE model classes. The most probable FE model class is indicative of the location of damage, while the severity of damage is inferred from the posterior probability of the model parameters of the most probable model class.

Asymptotic approximations as well as efficient stochastic simulation techniques (Ching & Chen 2007) are presented for estimating the resulting probability integrals. To reliably estimate damage, high fidelity model class, often involving a large number of DOFs, are introduced to simulate structural behavior. The proposed Bayesian estimator requires a large number of FE model simulations to be carried out which imposes severe computational limitations on the application of the damage identification technique.

Component Mode Synthesis (CMS) techniques (Craig & Bampton 1965) are effectively used in this study to drastically reduce the computational effort required to monitor the structure and identify damage locations and severity. Following the CMS formulation, dividing the structure into components and reducing the number of physical coordinates to a much smaller number of generalized coordinates certainly alleviates part of the computational effort. However, at each iteration one needs to re-compute the eigenproblem and the interface constrained modes for each component. This procedure is usually a very time consuming operation and computationally more expensive that solving directly the original matrices for the eigenvalues and the eigenvectors. It is shown in this study that for certain parameterization schemes for which the mass and stiffness matrices of a component depend linearly on only one of the free model parameters to be updated, often encountered in FE model updating and damage identification formulations, the repeated solutions of the component eigen-problems are avoided, reducing substantially the computational demands without compromising the solution accuracy.

The effectiveness of the damage identification methodology was illustrated using simulated damage scenarios from a real bridge. The integrated CMS technique is shown to be very effective in drastically reducing the computational effort required to identify damage locations and severity. The proposed methodology correctly identifies the location and the magnitude of damage. Surrogate models can also be incorporated in the formulation to further alleviate the computational burden. Finally, parallel computing algorithms can be combined with the proposed method to efficiently distribute the computations in multi-core CPUs.

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Output-only structural health monitoring by vibration measurements under changing weather conditions

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ABSTRACT

Vibration-based structural health monitoring relies on the fact that modal parameters of a structure depend on local stiffness changes. A major problem is that also normal changes in temperature, relative humidity, wind speed, operational loading, etc. influence the modal parameters, in particular the natural frequencies. This influence is generally nonlinear (Fig. 1). Here, an output-only technique is proposed for eliminating environmental influences on natural frequencies. It consists of estimating a nonlinear system model, using the versatile kernel principal component analysis (PCA) technique, for the evolution of the modal parameters during a training phase, where the

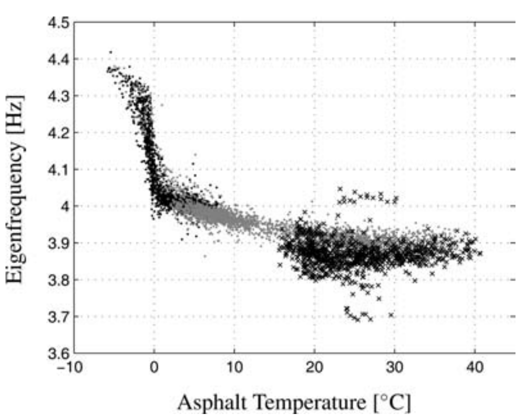

Figure 1. Z24 bridge, fundamental natural frequency as a static function of the temperature of the asphalt layer. Black dots: Training data. Grey dots: Monitoring data in undamaged condition. Black crosses: Monitoring data in damaged condition.

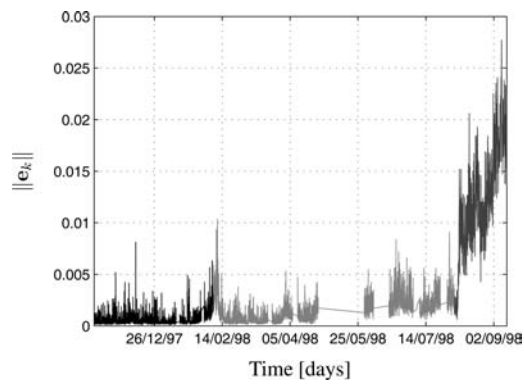

Figure 2. Z24 bridge, misfit of the nonlinear output-only model. Black: Training data. Light gray: Monitoring data in undamaged condition. Dark gray: Monitoring data in damaged condition.

structure is known to be undamaged. Afterwards, the structure can be monitored by comparing the model predictions with the observed modal parameters.

The technique is validated with monitoring data from a three-span prestressed concrete bridge, that was progressively damaged at the end of the monitoring period. Modal parameter identification on hourly sampled batches of vibration data yields four natural frequency time histories. A total of 5652 values for each natural frequency are obtained in this way. The first 2000 samples are used for nonlinear system identification with kernel PCA. As Fig. 2 shows, monitoring the misfit between the predictions of the identified model and the observed natural frequency data allows a very clear discrimination between validation data in undamaged and damaged condition.

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Influence of the prediction error correlation model on Bayesian FE model updating results

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ABSTRACT

Bayesian Finite Element (FE) model updating is a probabilistic method that can be applied for uncertainty quantification in FE model updating (Beck & Katafygiotis 1998). The technique consists of using the well-known Bayes' theorem to update Probability Density Functions (PDFs) of model parameters, accounting both for the information contained in the data and for uncertainties present in the measurements and model predictions. In short, a prior PDF reflecting the prior knowledge about the parameters is transformed into a posterior PDF, accounting both for uncertainty in the prior information as well as for uncertainty in the experimental data and FE model predictions. This transformation is done through the so-called likelihood function, which reflects how well the FE model can explain the observed data. The likelihood function is computed using the probabilistic model of the prediction error (i.e. the discrepancy between model predictions and observations).

Effective application of the Bayesian FE model updating technique in practice therefore requires (1) the selection of a suitable joint prior PDF and (2) the selection of a suitable likelihood function or prediction error model. The first of these challenges has been documented extensively in literature (Jaynes 1985); however, much less attention has been given to the latter, as it is usually assumed that the probabilistic model of the prediction error is known. In most cases, an uncorrelated zero-mean Gaussian error is adopted, even though this often does not correspond to reality. For example, in structural mechanics applications, where use is made of modal data obtained from sensors located closely together along a structure, it is very likely that errors between observations at different sensor locations are spatially correlated.

In this paper, it is shown that accounting for this prediction error correlation (1) has a large influence on the results of the Bayesian updating scheme and (2) is a non-trivial task, as it requires selecting a suitable correlation model that correctly represents the actual errors at the different sensor locations. For example, assuming an exponential correlation model (i.e. with positively correlated errors over the whole sensor grid) would most likely not correspond to reality in practical cases.

It is shown that adopting a prediction error model that is incompatible with the data and FE model at hand yields inconsistent, misleading and erroneous results of the Bayesian scheme. However, the challenge remains in finding methods for determining suitable correlation models, as in most cases very little information is available on the true correlation between the errors.

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Hybrid genetic algorithm to system identification and damage assessment of a high-rise building

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ABSTRACT

Field of system identification has become important discipline due to the increasing need to estimate the behavior of a system with partially known dynamics. Identification is basically a process of developing or improving a mathematical model of a dynamic system through the use of measured experimental data. In addition to updating the structural parameters for better response prediction, system identification techniques made possible to monitor the current state or damage state of the structures.

Most of the identification methods are calculus-based search method. A good initial guess of the parameter and gradient or higher-order derivatives of the objective function are generally required. There is always a possibility to fall into a local minimum. On the other hand, Genetic Algorithms (GAs) are optimization procedures inspired by natural evolution. They model natural processes, such as selection, recombination, and mutation, and work on populations of individuals instead of a single solution. In this regard, the algorithms are parallel and global search techniques that search multiple points, and they are more likely to obtain a global solution. Many GA applications have been performed on a variety of optimization problems in engineering area. However, relatively few applications have been on structural identification. Koh et al. [1] proposed a hybrid strategy of exploiting the merits of GA and local search operator. Two local search methods were studied: an existing SW method and a proposed method called the MV method. The numerical study showed that the hybrid strategy performs better than the GA alone. The author applied the real-coded GA to structural identification problems. The validity and the efficiency of the proposed GA strategy were explored for the cases of systems with simulated input/output measurements. Moreover, the strategy was also applied to the real structure. Genetic Algorithms (GAs) are global search techniques for optimization. However, GAs are inherently slow, and are not good at hill-climbing. In order to accelerate the convergence to the optimal solutions, a hybrid GA identification strategy that employs Gauss-Newton method as the local search technique was also proposed and verified by the author [2].

Since 1993, the Central Weather Bureau has installed 61 Strong-Motion Systems on various kinds of structures, including 50 buildings and 17 bridges in Taiwan area. However, the data collected from the accelerographs installed on buildings before the Chi-Chi earthquake are still remained in elastic range since the intensities of the earthquakes before the Chi-Chi Taiwan earthquake, are not strong enough to trigger the inelastic response. The damage of the buildings induced by the Chi-Chi earthquake, provides a solid evidence that some of the structures has experienced inelastic response. It is intended to perform system identification of linear system to Taiwan Electricity Main Building using measured response data collected during real earthquakes. This building seems to experience no visible damages under the attacks of the earthquakes. However, the dynamic parameters may be altered even though the damage of the structure is slight or invisible. In this study, the Hybrid Genetic algorithms are used to identify the modal parameters in the time domain. To get knowledge of this damage state of the structure, single-input-single-output model is utilized to perform the identification of modal parameters of the structure. As a result, by monitoring the variation of the identified parameters, the damage assessment of the structure is performed and the damage state of the structure is evaluated.

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Non-stationary random vibration for a high-pier bridge under vehicular loads

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In studying the dynamic responses of a bridge-vehicle system, the bridge-vehicle interaction problems were usually solved either in space domain or in time domain. When a vehicle travels with a constant speed, the responses of wheels induced by the road roughness can be simulated as a stationary random process both in space domain and in time domain. However, when a vehicle travels at variable speeds, the responses of the wheel induced by the road roughness are essentially a non-stationary random process in time domain. As a result, the vibration response of the vehicle-bridge system caused by road roughness should be considered as non-stationary random vibration responses. Therefore, introducing and verifying a new method using a field bridge to study this non-stationary vibration is significant. When investigating the dynamic response for vehicles with variable speeds, most of the previous studies either did not realize the non-stationary nature of the random response or simply simplified the non-stationary random response of the bridge-vehicle system as a stationary response in time domain. This simplification avoids the complexity of the non-stationary random processes.

This paper describes an experimental study on comparing the non-stationary and stationary vibrations of bridge-vehicle system under vehicles with variable speeds, and thus develops a new methodology of analyzing the non-stationary random response of bridges. Taking a high-pier Luping Bridge as an example, the dynamic responses of the tested span under the vehicular loads were measured. The non-stationary random responses of the moving wheels induced by the road roughness in time domain were obtained, and then those non-stationary responses were treated as the non-stationary inputs to the bridge-vehicle coupled system. A full-scale vehicle model with 12 Degree-of-Freedoms (DOFs) was used while the vehicle wheels were modeled as patch contacts instead of point contacts with the bridge road surface. The vehicle-bridge coupling equations were established by combining the equations of motion of both the bridge and vehicle using the displacement relationship and the interaction force relationship at the contact patches. The stationary and non-stationary random responses of wheels were treated separately as the two inputs to the bridge-vehicle coupled system. The effect of the two inputs on the mid-span deflection was compared and verified with the measured responses under different parameters including vehicle acceleration and deceleration. Results showed that the proposed method can accurately simulate the vibration of bridge under vehicle moving with variable speeds. The proposed method was then used to study the ride comfort when the vehicle moves with variable speed on the high-pier bridge.

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Monitoring of a riveted steel railway bridge

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ABSTRACT

The European railway networks are an infrastructural system that has been developed for more than 100 years. On the one hand currently new lines are constructed mainly for highspeed traffic while on the other hand lines that have been in service for many decades still form the backbone of many national railway systems. Consequently it is of high economical importance that the safety of the traffic on these lines has to be guaranteed nowadays and in future. These considerations include also the structural safety of all railway bridges. One of the most frequent failure modes of steel railway bridges is fatigue. Therefore the estimation of the remaining life time of such structures is related to fatigue. Several guidelines are available to the engineers to assess the structural integrity of existing bridges. The assessment procedures are mainly based on numerical investigations. Some guidelines allow also additional experimental investigations if the structural safety cannot be proofed by calculations. However, a standardized approach for a measurement-based fatigue assessment of existing structures has not been included in the guidelines. This gave reason for systematic investigations of currently applied approaches and the development of experimental procedures for fatigue assessment of existing steel bridges within a European research project. The tasks include the monitoring of sample structures. One of the considered bridges consists of five skewed riveted plate girder superstructures with spans of 36 to 40 m and a truss girder spanning over 75 m. The focus of the described investigations was put on one of the plate girder superstructures. During regular bridge inspections a crack was discovered in a gusset plate at a location where the stiffness changes sharply due to changes of the cross sections. Based on the assumption that this crack was caused by fatigue the relations between the global structural behaviour of the bridge, deformations and vibrations of structural elements connected to the cracked gusset plate and the local stresses became objective of the investigations.

After numerical investigations a monitoring system was installed. To minimize the possibility of noise contamination in the signals a monitoring system was chosen that digitizes the signals close to the sensors. The digital signals are then collected by a central controller. The acquired data is transferred daily to a central server. Automatically first analyses are performed to obtain a brief information about the general quality of the data and to identify defects such as sensor failures or other technical breakdowns.

Even though not all assessments of the of data acquired by the monitoring system during a period of one year have been finished yet, some observations can already be presented.

Based on strain measurements at the rail not only the trains' speeds but also the respective train type can be identified. Furthermore axle loads are derived from these strain measurements. For speeds up to 65 km/h the identified average axle loads coincide well with the nominal axle loads with a deviation of approximately $\pm 5\,\%$. For higher speeds the average mean axle loads tend to decrease, while the scatter of the mean axle loads increases. It is assumed that dynamic effects such as bridge–vehicle interactions have an increasing influence on the axle load estimates with increasing train speed.

Beside further strain measurements on the cracked gusset plate vibrations due to ambient excitation are acquired. The ambient vibration measurements are analyzed by means of the covariance driven stochastic subspace identification method. For the modal identification a correct extraction of stable poles is required. In this context a hierarchical clustering of the results is applied.

The research presented in this article was carried out within the European project FADLESS (FAtigue Damage controL and assESSment for railway bridges) that was supported by the European Research Fund for Coal and Steel (RFCS). Furthermore the support by DB Netz AG and by the European Regional Development Fund (ERDF) to the technical equipment used within the project is appreciated.

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Fast Bayesian ambient modal identification with separated modes incorporating multiple setups

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ABSTRACT

Modal identification is often the first step to establish the baseline properties of the subject structure, including the natural frequency, damping ratio and mode shape, which are important for calibrating subsequent measurement and analysis results as well as instrumentation planning. In full-scale ambient vibration tests, many situations exist where it is required to obtain a detailed mode shape of a structure with a limited number of sensors. A common feasible strategy is to perform multiple setups with each one covering different parts of the structure while sharing some reference degrees of freedom (dofs) in common. Methods exist that assemble the mode shapes identified in individual setups to form a global one covering all measured dofs. This paper presents a Fast Bayesian method for modal identification that can incorporate the FFT information in different setups consistent with probability logic. The method views modal identification as an infe-rence problem where probability is used as a measure for the relative plausibility of outcomes given a model of the system and measured data. It allows the global mode shape to be determined taking into account the quality of data in different setups. A fast iterative algorithm is developed that allows practical implementation even for a large number of dofs and setups. Using the proposed algorithm, Bayesian modal identification incorporating multiple setups can now be performed practically even on site. The method is illustrated with synthetic data and full-scale field data of a pedestrian bridge where a global mode shape with a large number of measured locations is identified. Figures 1–3 show the assembled mode shapes using the proposed method.

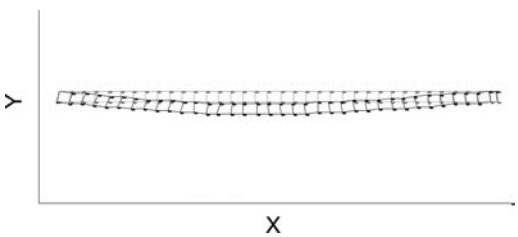

Figure 1. Identified mode shape of Mode 1 (1.67 Hz, 1.4%).

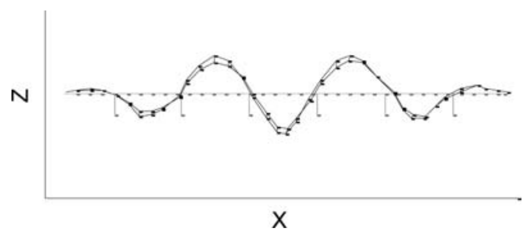

Figure 2. Identified mode shape of Mode 2 (3.76 Hz, 0.8%).

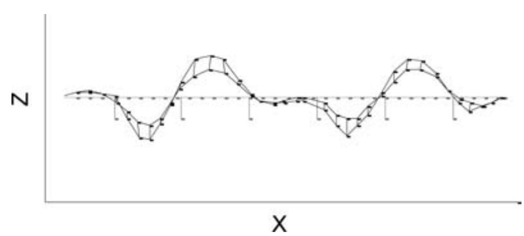

Figure 3. Identified mode shape of Mode 3 (4.16 Hz, 1.3%).

REFERENCE

Au, S.K. & Zhang, F.L. 2012. Fast Bayesian ambient modal identification incorporating multiple setups. *Journal of Engineering Mechanics*, ASCE. In print. DOI: 10.1061/(ASCE) EM.1943-7889.0000385.

Monitoring and assessment of bridges using novel techniques

Organizers: A. Strauss & D.M. Frangopol

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Structural health monitoring system using recurrence quantification analysis of ambient vibration

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ABSTRACT

Recent studies of attractor-based monitoring have demonstrated that the damage-induced change of the feature obtained from the attractor is larger than that of the most sensitive frequency and mode shape of the structure. In order to investigate the damage state, it is, in general, necessary to measure the intact state of structure in advance. This fact means that it is difficult to apply these methods to the structure without reference data. In fact, there are many existing structures which have no such data. To overcome these problems, this study investigates a feasibility of a Reference-less structural health monitoring system that can localize the damage without using any baseline data. An attempt is made to clarify the effectiveness of recurrence quantification analysis (Eckmann et al. 1987) of the response attractor caused by ambient excitation. The recurrence plots (Webber & Zbilut (1994)) were designed to detect non-stationarity in time series data and can be therefore a candidate for detecting damageinduced non-stationarities in structural response data. This study investigates the damage situation in which the 10% and 20% deteriorations of bending rigidity occur at the element 5 and element 19 of the structure shown in Figure 1.

Figure 2 shows the maximum dimensionless % recurrence obtained at all the nodes. It is found that dimensionless % recurrences obtained at node 5 and node 19 is the smallest among all. The reason for this is due to the damage. This study considered deterioration of the stiffness as the damage. According to the reducing of bending rigidity, response displacement of damaged element of the structure gets larger than that obtained from intact element. Therefore, the range or geometric shape of attractors reconstructed from response of damaged element get larger than that reconstructed from response of intact element. As a result, it can be considered that dimensionless % recurrence obtained at damage element decreased with the increase of amplitude of response displacement.

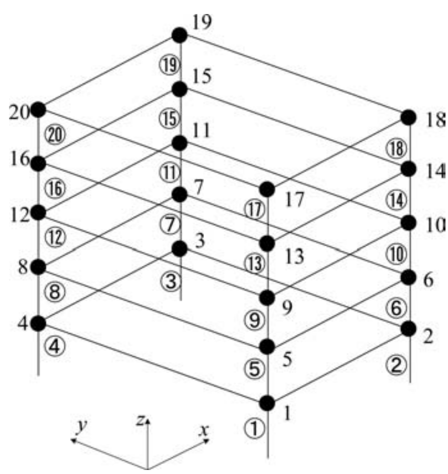

Figure 1. Structural model.

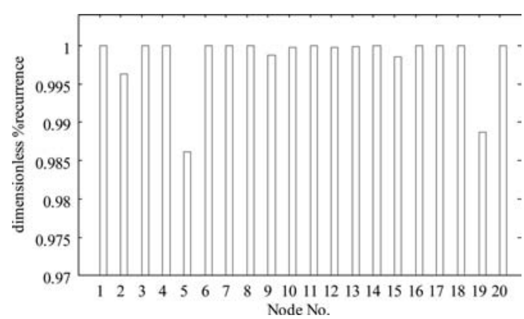

Figure 2. Maximum dimensionless % recurrence.

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Some difficult monitoring problems and some interesting outcomes

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ABSTRACT

Field monitoring of bridges is plagued with poor interaction between engineers and monitoring experts. For monitoring to be effective, it must be capable of revealing any aberrant behaviour.

At Cleddau bridge there was doubt about the behaviour of the tuned mass damper and certainty about damage to the roller bearings. Monitoring the bearings showed that the bridge bends horizontally morning and evening on sunny days and this produces plan rotations at the joint. Monitoring of the movement of the rollers showed that the corrosion and related damage was causing the rollers to flick rotate in plan each time the main movement changed direction. This happened often because vehicle loads caused the bridge to bend and that moved the rollers.

Linear potentiometers were used to measure displacement of the whole joint of up to 600 mm and of the rollers of 300 mm. Records were taken every second and although this produced vast data sets (86400 readings per day) it enabled the result of vehicular traffic to be properly identified.

Wire pull potentiometers were positioned vertically between four corners of the tuned mass damper and these were read ten times each second in order to record both long term movement and oscillation. The long term movement showed that the damper rocks east to west through the day.

A laser light lever system was used to confirm the diurnal movement of the span and this was related to tilt of the damper.

The laser levers were also used on a station refurbishment and proved to be capable of measuring deflection under railway loads of a masonry arch system. The measuring base line was 80 m and the deflections measured to about 0.1 mm.

Moire tell tales were also utilised to provide cheap displacement measurement as confirmation of behaviour.

These monitoring projects provided useful information, including detecting some unexpected behaviour.

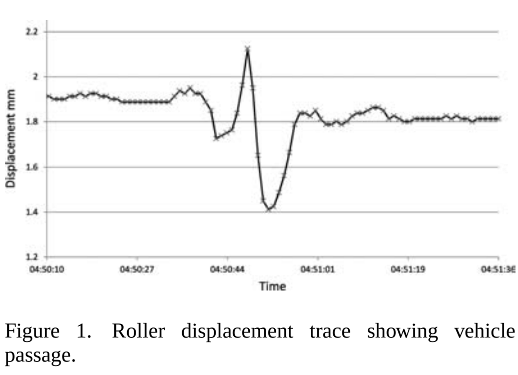

Figure 1. Roller displacement trace showing vehicle passage.

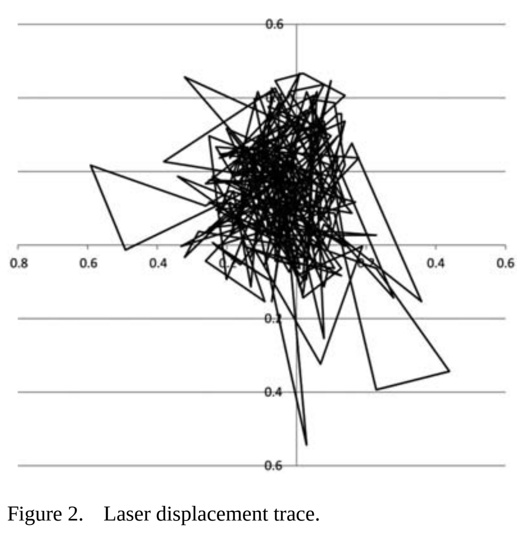

Figure 2. Laser displacement trace.

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Dynamic testing and structural identification of "New People's Bridge" in Verona

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ABSTRACT

To contain the inconveniences due to a live-cycle damage it is necessary to study in depth the causes of damage of infrastructures and in particular of bridges. In the context of the project sponsored by the Municipality of Verona in Italy, a reinforced concrete bridge was subject to experimental and analytical investigation into its dynamic characteristics.

The bridge is located in the center of Verona city and is characterized by three spans with a total length of over 90 m and 7 girders bearing a thin slab. Due to non-workmanlike details and not scheduled maintenance since constructed in 1945, recently the bridge has revealed severe damage on the edge girders at the middle of the spans. Before taking any provision or deciding to retrofit it the Municipality required to analyze the state of the bridge.

Relying on the results of base-line tests a short time monitoring system was to be set up. At first the bridge was subjected to a system of investigation (sensors and data gathering) as to perform an ambient modal test. Multiple non-simultaneously recorded measurement setups were prepared. Applying this identification method, the principal natural frequencies could be identified. Based on identified results finite element models were updated and adjusted in order to extrapolate natural frequencies and mode shapes that mostly accord with the ones that were observed. In order to use best fitted material characteristics in every element of the structure were executed lots of destructive and non-destructive tests as pull-out, ultrasonic pulse wave, rebound hammer and core sampling technique. The model was used than for static and seismic parametric studies for the evaluation of the structural elements. The vulnerability analysis revealed the deficiency of some structural elements and suggested the installation of a permanent monitoring system.

Figure 1. New People's Bridge, Verona, Italy. Photo (2011).

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A year-long monitoring using in-service vibration data from a multi-span plate-Gerber bridge

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ABSTRACT

This study investigates effects of temperature on structural health monitoring of a multi-span plate-Gerber bridge using a year-long monitoring vibration data. Only the effect of temperature fluctuation is considered as an in-service environmental factor, since the previous study (Kim et al. 2011) demonstrates that classifying the observed data according to a specific time can reduce an influence of traffics on the monitoring.

Coefficients of the AR model of signals taken from the bridge are used as a parameter for structural diagnosis. The Bayesian regression method (Bishop 2006) is used to update and detect anomaly from the monitoring data affected by temperature fluctuation. A statistical feature of the parameter considering temperature is firstly identified utilizing the Bayesian regression, and then residual errors between observed and estimated parameters are calculated. The 95 per cent confidence interval of a scaled residual with respect to temperature is adopted as a threshold.

Almost all of the scaled residuals are placed within the threshold and decided as normal (solid circles in Fig. 1). It is also observed that three consecutive anomaly events (asterisks in Fig. 1) are detected even though the bridge is in intact. However, in considering normal events followed by those consecutive anomaly events, those anomalies might not be caused by a structural damage. A noteworthy point is that the anomaly event is frequently observed from March to June even though relatively less number of anomalies is detected from August to February. The reason for the monitoring results is not clear yet. Observations also show that the scaled residual taken from the signal of traffic accident apparently deviate from the threshold (see Fig. 2).

How to make a decision for structural diagnosis from the consecutive anomaly events is a task remaining to be solved, which is the next step for this study.

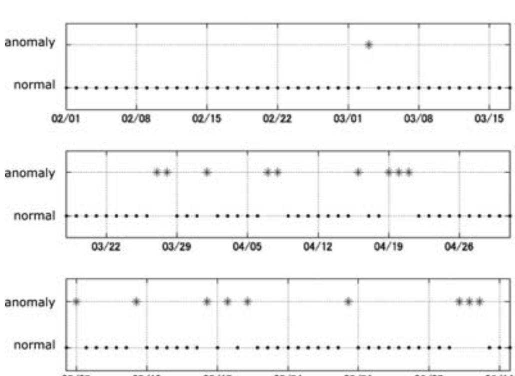

Figure 1. Structural diagnosis from a year-long monitoring data observed at 7:00 AM of the observation point UA1.

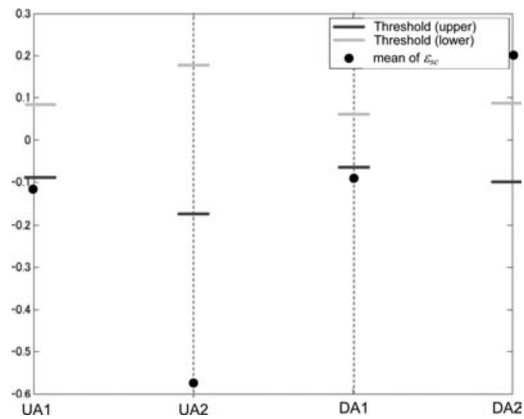

Figure 2. Scaled residuals between observed DI and predicted DI from the data observed at four observation points when a truck collides with a concrete barrier of the bridge.

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Optimised monitoring concepts for historical masonry arch bridges

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ABSTRACT

Practical experience has shown that masonry arch bridges in adequate condition hold remarkable bearing reserves and therewith mostly meet today's safety requirements. Estimating and using the structural reserve of existing bridges to their maximum potential in order to satisfy new requirements implies a precise assessment of the structure and an analysis of the rehabilitation project in order to guarantee structural safety and increase structural service life. Nowadays due to financial restrictions and monument conservation aspects more attention is paid to maintenance and rehabilitation of existing arch bridges than on reconstruction. Since those bridges usually have been planned for different loads according to the respective codes (EC1-2 (2010)) assessment concerning the current bearing capacity and the future usage is required. A recalculation with conventional calculation methods is often insufficient as results can considerably deviate from the actual load capacity due to various influence factors as discussed in Proske & van Gelder (2009). The challenge therefore is to verify accurate methods for determining the actual state of such arch bridges. An important point of interest is the combination of finite element modelling of masonry on the basis of measurement data with inspection and monitoring strategies of existing structures. Within the contribution the basic concept of data acquisition and their usage regarding the model is discussed on a case study object.

As a result of preservation orders and financial restrictions, arch bridges have to be maintained, toughed up during their lifetime and in addition they have to be assessed considering new load scenarios according to recent codes. The recalculation of these structures is quite difficulty, due to the lack of initial plans and adequate data of material parameters. The interaction between the single components of arch bridges (soil, masonry, backfill, structure-soil interaction as discussed in UIC Code 778-3 (2011)) is afflicted to many uncertainties. Thus from the point of view both of the responsible official corporations and of pure research, it is necessary to design well operating concepts for estimating the load bearing behaviour of existing arch bridges.

In this contribution it is shown how an existing arch structure can be analysed on the base of various measurements e.g. ground penetration radar, Laservibrometer (see Figure 1) and Displacement Transducers (LVDT).

In addition, the relation of the measurement data to a adequate finite element model is discussed. Another important issue is the conduction of laboratory tests for proving and verifying the assumptions for numerical modelling.

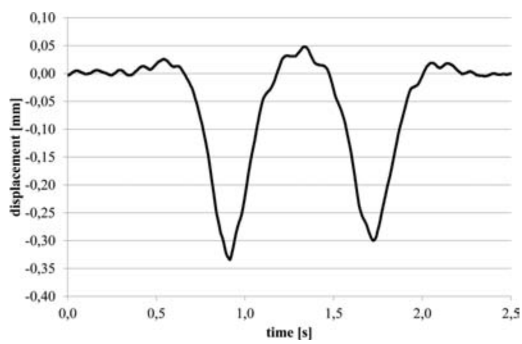

Figure 1. Vertical displacements caused by a crossing single railcar type 5047, determined by laservibrometer.

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Bayesian forecasting of structural bending capacity of aging bridges based on dynamic linear model

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ABSTRACT

The performance degradation of bridge structures is one of worldwide concerned problems in life-cycle civil engineering. Since the deterioration of structural performance is a time-variant process with large amount of aleatory randomness and epistemic uncertainties, it is very important to successively predict structural performance to ensure safety and serviceability. To integrate the past prior information and inspection and/or monitoring data, Bayesian updating techniques are usually used to predict structural performance and condition. However, the traditional prediction functions for processing inspection or monitoring data are normally defined as static polynomial regression functions, which are difficult to realize online, dynamic, and real-time performance prediction. In this paper, the dynamic measure of structural performance with time is treated as a time series, a Bayesian Dynamic Linear Model (DLM) as shown in Figure 1 is introduced. Considering the time-dependent characteristics of structural performance of the considered bridge, a linear growth model is built to predict the short-term variation trends of structural performance. The well-known Kalman filter algorithm is used to estimate and forecast the dynamic performance index for the DLM. The one-step-ahead predictive distribution and the filtering distribution are determined for Bayesian dynamic updating. To allow for the epistemic uncertainty in variance estimation based on monitoring information, use of a discount factor approach is made for specification of unknown variance matrix. Two illustration examples for RC bridge girders are used to demonstrate the applicability of the proposed method.

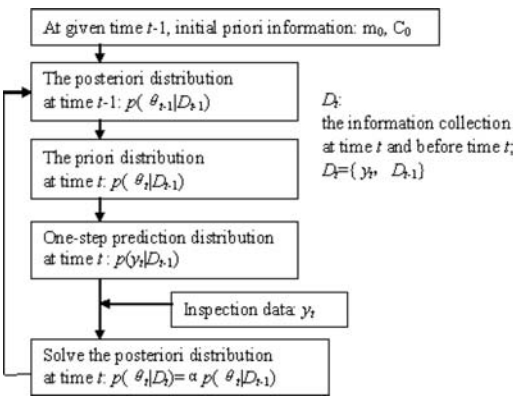

Figure 1. Bayesian dynamic linear model.

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Service life management of infrastructure systems – application of corrosion and moisture monitoring

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ABSTRACT

Repair works on infrastructure systems due to reinforcement corrosion cause annual costs in the range of several billion Euros worldwide. One of the main reasons for these high costs is the fact that in most cases reinforcement corrosion damages will only be detected once corrosion has already proceeded so far that cracking and spalling of the concrete cover occur. Corrosion and moisture monitoring appear to be both very reliable and cost-effective tools for the early detection of potential corrosion, allowing for the use of preventive repair measures instead of extensive concrete repair at later stages of corrosion. In combination with probabilistic deterioration modeling monitoring can be employed to reliably predict the future condition state development of the structure. This way, necessary repair measures can be scheduled well in advance and the available financial means can be used optimally.

The principle of moisture monitoring is based on the measurement of the electrolytic resistivity of the concrete which is directly correlated to the moisture content of the concrete. The most common moisture sensor in concrete, the so-called Multiring Electrode, enables the user to determine moisture profiles over the concrete cover. Typical fields of application for the Multiring Electrode are the monitoring of the functionality of hydrophobic treatments on concrete surfaces or of coating systems on bridge decks. In case monitoring indicates a loss of effectiveness of a hydrophobic treatment, the system can easily be renewed with comparably small effort.

For corrosion monitoring, 'substitute' steel electrodes are placed at well-defined depths between the concrete surface and the reinforcement. The corrosion state of these steel electrodes is monitored via corrosion potential and corrosion current measurements. A drop in corrosion potential along with an increase in corrosion current indicates the depassivation of the respective steel electrode. If the distance of the steel electrode from the concrete surface and the actual concrete cover of the reinforcement are known, the point of time at which the reinforcement itself will start to corrode can be predicted.

Corrosion sensors are normally placed in reinforced concrete elements exposed to chloride impact. Typical fields of application are e.g. bridge pylons, tunnel portals, parking decks, marine structures or the tunnel outside in case the structure is built in a chloride-containing environment. The Anode Ladder as the most common corrosion sensor worldwide has been applied successfully for more than two decades.

For a major inner city tunnel project in Munich, the Tunnel Mittlerer Ring Südwest, a durability approach was selected that combines both deterioration modeling and monitoring. During the design stage, durability calculations were carried out and the required concrete cover and threshold values for the chloride transport properties of the concrete in order to reach a target service life of $t = 100$ years were determined. These threshold values had to be achieved by the contractor during the compliance testing stage. For the execution phase, an additional quality control regime was installed. As a durability indicator the electrolytic resistivity was measured as well on concrete cubes in the lab as on the actual structure itself. In addition, extensive concrete cover measurements are carried out and the results are used to update the original durability calculations.

The corrosion and moisture monitoring system of the tunnel consists of 65 Anode Ladders and 20 Multiring Electrodes which are mainly installed in the tunnel walls and the tunnel ceiling close to the tunnel portals. They are intended to monitor the effectiveness of the hydrophobic treatments and the coatings on the concrete surfaces and to render further information for the update of the durability calculations. The costs for the monitoring system amounted to appr. 0.4‰ of the total costs of the project – which is negligible when compared to the saving potential due to the improved knowledge of the actual condition state.

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Structural monitoring of a steel bridge with the longest arch span in Poland – selected issues

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ABSTRACT

In Poland, structural monitoring systems have been installed in several road bridges. This paper presents the monitoring system installed on a steel arch bridge over the Vistula River along the ring road of Pulawy, Poland. The total length of the crossing is 1038.2 m (a four-span continuous structure) and the main arch river span is 212.0 m being the longest among the arch bridges in Poland.

The system thoroughly measures various physical quantities. The system is composed of three subsystems: monitoring of the structure, meteorological monitoring and video monitoring.

The monitoring of the structure, one of the three subsystems of the monitoring system, is designed to control the behaviour of the bridge by means of continuous electronic measurement of the following parameters: changes in strains, deflections, accelerations, temperature of the structure, wind speed and direction. The scheme showing the localization of test points is presented in Figure 1.

Force changes in hangers are calculated on the basis of strain measurements with the use of sensors installed on ten hangers. These test points are marked P81–P85.

Force changes in three selected hangers (P81NR, P83NR and P85NR) have been analyzed. Presented data concern the time period from 1 May 2009 to 30 September 2010. The greatest changes of forces in hangers have been measured at the point P81NR. On average, the force changes are approximately 90 kN, with the characteristic load capacity of the hanger being 700 kN. It can be observed that the minimal force values have been measured during the night or in the early morning hours, while the maximal values during the day. This phenomenon is assumed to be connected with the change of temperature of the structure.

Stress changes in arches are calculated on the basis of strain measurements with the use of sensors installed in 14 points (points P2–P5). Stress changes in the northern arch in selected points (P2NR, P3NR, P4NR and P5N) have been analyzed. As can be observed, maximum temperature of the structure is achieved round noon and in the afternoon, which confirms the intuitive approach as well as data in literature (Zobel 2003). Certainly, seasonal temperature amplitudes influence the scopes of stress changes as well. The scopes of stress changes in winter are smaller than in summer and are at the level of 5–15 MPa, while in summer at 30–40 MPa.

Gathered data allow concluding that the main factor to influence the effort of the structure being under exploitation is thermal action. Data obtained from the systems can give an overview on the real level of the live and/or environmental loads on bridges. These information is believed to be a foundation for the National Annex of Eurocodes.

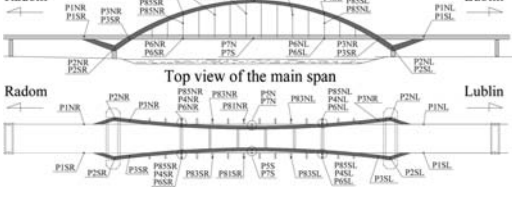

Figure 1. Localization of test points on the bridge.

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New approaches in evaluating vibration and physical monitoring techniques

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ABSTRACT

The interest in developing approaches and possibilities of determining the condition and remaining lifetime of engineering structures, in particular railway and road bridges, has considerably increased over the past decade. Numerous monitoring approaches have been developed based on vibration measurements especially on acceleration measurements. The development of approaches based on vibration measurements are of particular interest because they are easy to perform and therefore an economical solution. The present work introduces methods for automatically evaluating the natural frequencies and the damping of a structure from monitoring data. Additionally the present article shows methods to optimize the location and the amount of sensors used to solve a given monitoring problem. Finally the introduced methods are tested on a monitoring work for a railway bridge in lower Austria (Geier, 2008, Österreicher, 2008, Österreicher 2009).

The natural frequency of a structure is beneath damping and one of the main identification parameters of a structure which can be evaluated from vibrations measurements f.e. accelerations measurements. The results for the analysed structure show that it is a suitable method to evaluate natural frequencies from large data sets automatically.

The second method introduced is an algorithm for estimating the damping from vibration data automatically. The evaluation of the damping value, the percentage of critical damping, is generally more complex than to get the maximum deflection or to evaluate the natural frequency of a structure.

The simulation with both methods indicates that one important thing to get reliable results is to evaluate some measurement data before manually. To find suitable trigger conditions to evaluate the natural frequencies based on the TRWFT a good knowledge about the monitoring data is necessary. For a good estimation of the natural frequencies is important to get meaningful results for the damping from the introduced method.

As a conclusion, both methods could help to get estimates for the damping and the natural frequencies respectively for the analysed structure, but both of them need the user evaluate parts of the data manually to get the calculation parameters for the introduced methods.

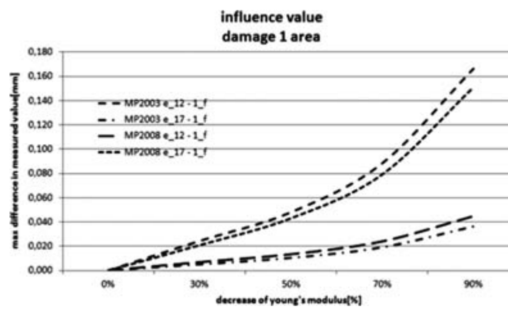

Figure 1. Simulation result

The structures become more complex so that knowledge about the optimum location for the sensors is necessary to find certain damage. This requires special techniques to find the best position for sensors and also the best measured variable for a given structure. One method to find the best position is influence fields. The amount of load positions has a considerable influence on the accuracy of the influence areas. With all the load positions and the influences on the predefined measuring point a contour plot can be created.

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Maintenance life-cycle costs for bridges of Egnatia Motorway, Northern Greece, considering their seismic risk assessment

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ABSTRACT

Egnatia Motorway in Northern Greece has a 670 km main axis length and comprises 210 twin concrete bridges among 1800 structures. As the motorway crosses high seismicity prone areas, the severe damage of their bridges due to intensive seismic events should be considered for predicting extra bridge life-cycle costs and managing their maintenance.

This paper focuses on the seismic risk assessment of the motorway bridges and presents its results and their impact on the typical life-cycle bridge maintenance costs. A seismic risk assessment software package is used, developed for this purpose, for Egnatia Odos S.A., by EQE Ltd (2001). It combines a seismic hazard model, considering historical data across the motorway, prepared by the Greek Institute of Technical Seismology and Earthquake Engineering (ITSAK), and the vulnerability functions determined for all the bridge structural types (ASPROGE, 2007), see e.g. Fig. 1 for the Krystallopigi bridge. Following e.g. Strauss et al (2008), life-cycle analyses for Egnatia bridges have also been realized. Based on these results, the typical expectations for the maintenance life-cycle analysis of the motorway bridges are updated considering the seismic risk.

As shown in the representative Table 1, the results demonstrate the high vulnerability of many bridges, and especially of those built in West Epirus and in Central Macedonia Sections of the Motorway. There, for small return periods (50 and 100 years), their moderate damage is predicted. It is therefore necessary to consider the damage and repair of critical bridge components earlier than predicted by the typical deterioration curves adopted in Egnatia life-cycle maintenance costs. This is the case of major bridges, which shows an 20 years offset of the time of expecting the major necessary repair of the substructure. This results in less effective and more expensive maintenance life-cycle for this bridge.

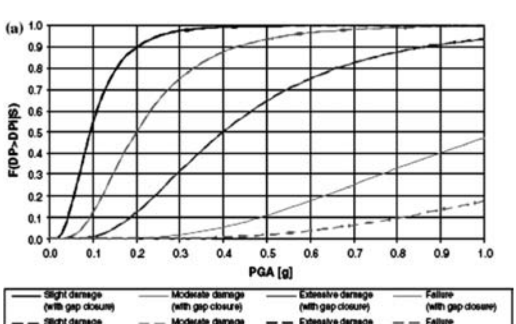

Figure 1. Vulnerability functions of Krystallopigi bridge.

Table 1. Expected seismic damage % for various return periods, for some major Egnatia bridges.

Bridge	Code	Return periods 50	100	475	1140
Metsovo bridge	1	7.87	15.33	25.77	34.20
Aracthos bridge	2	8.06	15.73	26.02	33.49
Polymylos bridge	3	3.55	7.10	18.32	25.95
Krystallopigi bridge	4	33.70	38.67	49.75	54.93
Mesovouni bridge	5	22.34	26.88	37.89	45.36
5th Kavala bypass	6	37.92	44.52	56.17	59.88
Greveniotikos bridge	7	3.65	6.30	13.20	20.27
Votonosiou bridge	8	7.65	10.27	20.25	25.97
Nestos bridge	9	20.60	23.97	32.25	45.40

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Prediction of fatigue damage accumulation in metallic structures by the estimation of strains from operational vibrations

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ABSTRACT

Damage accumulation due to fatigue is an important safety-related issue in metallic structures. Fatigue damage accumulation at critical points of a structure can be estimated using available damage accumulation models that analyze the actual stress time histories developed during operation. Inferring the stress time histories at a structure under actual operational conditions using strain rosettes has limitations since the number of sensors that can be placed on the structure cannot cover the entire structure or critical structural locations. The characteristics of the stress response time histories at a point in a structure can alternatively be predicted by using a finite element model of the structure and the actual excitation time histories. However, for most structures, the excitation time histories are neither available nor can be conveniently measured by a system of sensors.

This work deals with the problem of estimating damage accumulation due to fatigue in the entire body of a metallic structure using operational vibration (output-only) measurements from a limited number of sensors installed in a structure. A first attempt to address this problem can be found in Papadimitriou et al. (2011). An effective solution was provided assuming that the excitation and response can be considered to be stationary. This paper extends this approach to the general excitation case, making no assumptions on the load characteristics and the location of loads. A recently proposed joint input-state estimation filter (Lourens et al. 2012) is extended to estimate the strain response time histories in the entire body of the structure using the output-only vibration measurements collected from the sensor network. Such predictions are integrated with available linear damage accumulation laws (Palmgren-Miner rule), S-N fatigue curves and stress cycle counting methods to estimate fatigue maps covering the entire body of the structure.

The proposed method is validated using simulated vibration measurements from a laboratory steel beam suspended at both ends from a steel frame using flexible springs to simulate free-free boundary conditions.

Table 1. Validation of proposed method using simulated data.

Fatigue	Impulse excitation	Stochastic excitation
True	1.19×10^{-9}	2.00×10^{-9}
Identified	1.20×10^{-9}	2.10×10^{-9}

Simulated measurements are generated for two excitation cases: an impulse and a broadband stochastic excitation. Acceleration measurements at ten locations of the beam are generated using the excitation and the finite element model of the beam and adding noise, with noise level set to 20%, to simulate the effect of measurement and model error. The fatigue damage accumulation results are compared in Table 1 using the strains identified by the proposed input-state filter estimation method and the true strains calculated using the excitation and the finite element model of the structure. An extremely good prediction is noted for both impulse and stochastic excitations, validating the effectiveness of the proposed methodology.

The fatigue damage accumulation maps provided by the proposed methodology are useful for designing optimal maintenance strategies for most critical components of metallic structures using the actual structural vibration information collected from a sensor network.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Reliability-based inspection planning with application to orthotropic bridge deck structures subjected to fatigue

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ABSTRACT

Throughout their service lives, structures are subjected to various deterioration processes such as corrosion and fatigue. Deterioration processes may lead to structural degradation beyond acceptable limits, which are typically related to the safety of humans and risks to environment. In order to ensure that degrading structures comply with given acceptance criteria throughout their service lives, it is generally necessary to control the progress of the deterioration processes e.g. by inspections.

Over the past 25 years reliability-based and risk-based approaches to inspection planning have been developed, see e.g. Madsen et al. (1987), Straub (2004), Straub et al. (2006). These approaches have thus far primarily been utilized for inspection planning for fixed and floating offshore structures.

In order to demonstrate the principles underlying reliability-based inspection planning, the current paper summarizes a reliability-based method for planning inspections for welded steel structures subjected to high cycle fatigue. The method is applied to determine the minimum required inspection effort in terms of number of inspections and their corresponding inspection times for a fatigue prone welded connection of an orthotropic bridge deck.

To ensure that the structure subjected to fatigue complies with the given acceptance criteria throughout its service life, inspections are planned before the predicted annual failure rates exceed a maximum acceptable limit. Additional information about the deteriorating structure gathered through inspections is used to update the predicted failure probabilities through Bayesian updating. Each planned inspection is assumed to result in a "no detection" event. The presented method is applied to determine the minimum required number of inspections and the corresponding inspection times for a selected fatigue prone welded connection of an orthotropic bridge deck structure (Figure 1).

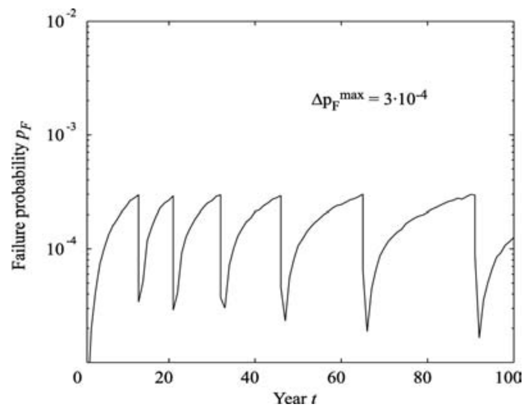

Figure 1. Inspection times for $\Delta p_F^{max} = 3 \cdot 10^{-4}$.

The presented concept can also be extended within the framework of Bayesian decision theory with the aim to minimizing the service life economical risks in terms of expected life-cycle costs, see e.g. Straub et al. (2006) and Thöns (2011). The basic idea underlying risk-based inspection planning is to plan inspections and maintenance activities such that the given acceptance criteria for the considered structure are met and at the same time the overall costs of inspections, repairs and failure are minimized.

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Probabilistic fatigue crack growth modeling for reliability-based inspection planning

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ABSTRACT

Throughout their service lives structures are subjected to various deterioration processes such as corrosion and fatigue. Deterioration processes may lead to structural degradation beyond acceptable limits, which are typically related to the safety of humans and risks to environment.

Welded steel structures subjected to fluctuating loads are prone to develop fatigue damage (Figure 1). Fatigue deterioration is governed by a number of uncertain conditions including material, environmental and loading conditions. Additional uncertainties arise from the modeling of the actual physical phenomenon. In order to properly account for these uncertainties in the fatigue assessment of welded structures it is necessary to apply probabilistic methods, see e.g. Wirsching (1984) and Madsen (1997).

In this paper, a quantitative probabilistic fatigue deterioration model is presented, which is based on Linear Elastic Fracture Mechanics (LEFM) theory and models the fatigue crack growth. This approach is developed in the perspective of updating the fatigue probability of failure by inspections which require a measureable entity, such as the crack size. The probabilistic fatigue deterioration model can then serve as a basis for a reliability based inspection planning (e.g. Madsen et al. (1987) and Straub (2004)).

The quantitative modeling of the phases fatigue crack initiation, fatigue propagation and fatigue failure is discussed and models for each of the phases are introduced on the basis of a literature review.Furthermore, the paper provides an overview of the uncertainties in the loading andin the phases of the fatigue crack growth. Probabilistic models for the fatigue crack growth and the fatigue failure are developed and it is discussed how the time variant fatigue problem can be modeled with aprobability distribution function of the fatigue stress ranges. The paper concludes with the perspective of reliability-based inspection planning which is developed based on this paper in Schneider et al. (2012).

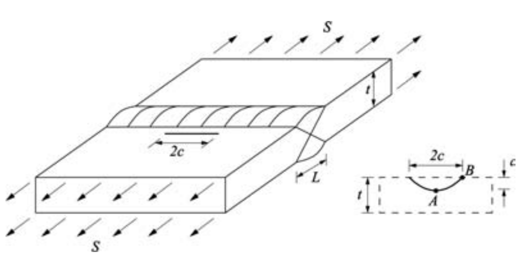

Figure 1. Butt welded plate containing a semi-elliptical surface crack in the weld toe region.

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Detection of traffic loads by structural and geodetic measurements

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ABSTRACT

The continuous increase of traffic loads lead to rising fatigue stresses of existing road bridges. Among others the service life of bridge structures depends on the fatigue stresses influenced by traffic loads in combination with thermal stresses. In this context especially the heavy goods vehicles cause remarkable fatigue stresses. Many existing, aged bridge structures are not designed for the expected fatigue stresses; thereby, the remaining service life of these bridges is unknown. Based on current traffic loads realistic forecasts of the remaining service life can be performed. The current traffic loads can be extrapolated to determine the past, present and perspective fatigue stresses and thereby, the remaining service life can be prognosticated. This is an economic as well as safety relevant aspect of an effective life-cycle management.

This paper describes traffic census and load measurements on a prestressed box girder bridge (Fig. 1) by structural and geodetic measurements. In this context the bridge is used as scale for the passing heavy goods vehicles. The identified traffic loads can finally be used to determine the remaining service life of the examined bridge structure.

The structural measurements are primarily performed by strain gauges and displacement transducers. The detection of passing heavy goods vehicles is applied by peak value analyses of the recorded measurement signals; further vehicle attributes like the velocity, driving direction and the vehicle type are determined by combining different signal information. The global response of the superstructure is used to determine the total vehicle weights of the passing heavy goods vehicles whereas the identification of the axle loads is applied by analyzing the local response of the bridge structure. Therefore, the method of least squares is applied in order to define the axle loads as well as the transversal position of the passing vehicles on the carriageway.

The geodetic measurements are carried out with a terrestrial laser scanner. Measurements of bottom slab deflections are performed and the strains of the bottom slab are obtained by these data. The obtained strains by the geodetic measurements will be compared with the recorded strains by the structural measurement program.

Figure 1. Examined box girder bridge.

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Maintenance and rehabilitation of aged bridges

Organizers: M.A. Ahrens & P. Mark

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Precision-assessment of lifetime prognoses based on SN-approaches of RC-structures exposed to fatigue loads

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ABSTRACT

Today, a great majority of existing infrastructure buildings, especially bridges, has reached its intended lifetime. During the last decades, several damage mechanisms have weakened the structural resistance to some extent (Haveresch 2011). Among others, current focus lies on a reliable assessment of the influence of fatigue on structural bearing capacity. The accompanying structural damage state has to be quantified in every single case by experts involved in the field of structural assessment or rehabilitation. To support their decisions, usually mathematical methods are applied to judge the remaining structural strength after several years of service, which are generally based on reliability theory, to deliver objective safety factors. Finally, according to these evaluated safety factors economical decision are made.

All mathematical models used to predict structural damage states or residual lifetimes are formulated with respect to a broad variety of uncertain input parameters, comprising different domains, e.g. damage mechanisms, material and geometry (Ahrens 2010, Ahrens et al 2010, Ahrens & Mark 2011). Seldom all required data can be measured in situ with acceptable precision. Hence, assumptions have to be made about spatial and temporal distributions to reflect those scattering properties in generic input data adequately.

The current contribution focuses on identification and assessment of the most relevant parameters dominating widely accepted SN-approaches for judging fatigue resistances of reinforced concrete structures. Both domains, impact – usually described by stress amplitudes and e.g. counted by the rain-flow method –, as well as material resistance, are treated. The connection of both, commonly addressed in standards by prescribed SN-curves according to the type of construction material used, is central to the investigation. By mathematical examination employing sensitivity analyses of essential parameters their distinct contribution to the precision of lifetime predictions can be assessed exemplified by means of reinforcement SN-curves (Figure 1).

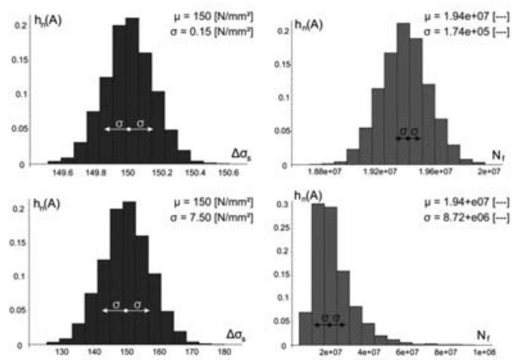

Figure 1. Uncertainty in stress amplitudes of fatigue loads leads to a great scatter of the lifetime prediction. The transformation of thedistributions shape is caused by the power function law acc. to the SN-approach applied.

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Structural flexibility in relation to integrated service life design of buildings

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ABSTRACT

In *Integrated Service Life Design* and other assessments of buildings it is necessary to use an *Estimated Service Life (ESL)* which needs to be as accurate as possible. A building is a rather complicated system of products and elements and the different building elements are likely to have different product life-cycles. The ability to change or replace these parts, influences the Service Life of the building as a whole. The Iso 15686 series define a method to calculate an *Estimated Service Life ESL*. This Factor Method uses a Reference Service Life (RSL). It hardly allows for "Flexibility" influences. In (Nunen 2010) an Improved Factor Method is proposed. Here two extra factors are added to account for economical an functional influence. The accuracy of these methods is yet uncertain. Another approach that can be followed is the calculation of different Service Lives in which the lowest value of a Service Life finally has the largest influence on the expected or Estimated Service Life. This way a separate: *Functional Service Life* is introduced, which is expected to be related to the Flexibility of a building. This Flexibility can be defined as: *The building's capacity (passive or active) to accommodate, in a relatively easy way, (future) changes in use or adaptations to parts of the building.* Relatively easy is here defined:

> *A change to a certain building layer is "relatively easy" if it can be achieved without the necessity to affect or change other building layers as well.*

With a building model *Flexibility* is defined more clearly. This building model is based on earlier work by (BRAND, 1994) and (LEUPEN, 2002). In case of sufficient *Structural Flexibility* the building's *structure* need not to be changed if other building layers do need to be changed or replaced. To quantify Structural Flexibility more precisely it becomes necessary to find and evaluate *indicators* which can describe the flexibility relations of the Structure with the other building layers. The indicators that have been found are: *Integration,Connections, Accessibility, Capacity, Dimensions and Obstruction.* By using the Structural Flexibility indicators on one axis and the extended building layers on another axis, a *Structural Flexibility Matrix* is created in which each of the different relations can be assessed and scored. This method was tested in the assessment of three different buildings that were expected to show different levels of Flexibility. By using two different Experts Surveys both the method, the used indicators as well as the used relations with the other Building Layers were assessed and evaluated. In the graph below the single score results of the four different investigated methods are given. Now that Structural Flexibility can be assessed more precisely it becomes possible to research the Hypothesis: A higher Structural Flexibility of a building ensures a longer Functional Service Life.

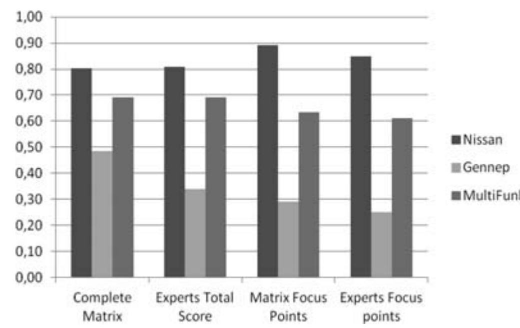

Figure 1. Single Score Structural Flexibility.

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Modern bridge stock management in a regional metropolis – structural assessment, data management and cost control

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ABSTRACT

The bridge stock management system of the regional metropolis Düsseldorf, located in the west of Germany, is presented.

The central element is a computer aided database system. On the one hand, it includes detailed technical aspects, like the extensive data from the regular inspections, structural checks and recalculations, repairs or strengthenings (Stratman et al. 2008). These data cover about 550 single bridges, tunnels or other engineering structures with a total replacement cost of about 1,4 billion Euro and span back over several decades (Fig. 1). On the other hand, the system interlinks the technical data to efforts in time execution, human resources and expenses. Thus, it is used as a steering tool to achieve a minimised, almost uniform money-invest per year, a suitable distribution of repair works and a reduction of traffic disturbances (Frangopol et al. 2010). To achieve this, the interactive software tool works on multi-levels, namely the level of the single structures, the group level and the level of the total stock. Steering is done on the latter, consistently including the detailed estimations of expenses, time efforts and technical conditions from the lower levels by additive superpositions.

The steering concept as well as the principal technical basics and features of the data system and interlinks are presented and illustrated taken road bridges as examples.

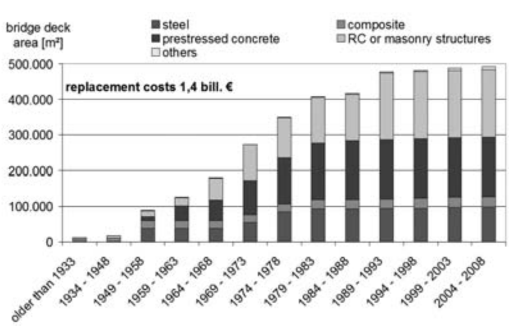

Figure 1. Cumulative replacement costs of 550 single bridges, tunnels and other engineering structures.

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Strengthening of existing bridge decks by additional concrete layers – new research results and design rules

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ABSTRACT

The present contribution focuses on the one hand on experimental investigations carried out at the University of Innsbruck determing the influence of HPC and the joint roughness on the bearing capacity of nonreinforced shear joints. On the other hand a new guideline for the dimensioning of strengthening with additional concrete layers (RVS 15.02.34 – 2011) is presented.

During the life-cycle of a bridge (a bridge normally is designed for a life time of approximately 100 years) on average there is a need for one or two renovations. These renovations are mainly caused by environmental influences but also by demands concerning the structural safety. Due to the fact, that life loads rise constantly, it becomes necessary to in-crease the bearing capacity of existing bridge decks. The use of additional reinforced concrete layers on bridge decks is a quite common method to increase the bearing capacity. The use of anchor plugs combined with cast-in-place concrete supplements leads to an enormous work effort and thus high expenses, so from an economic point of view it is very meaningful to minimize the necessary quantity of anchors. Experimental results show, that the design approach for the bond strength in shear joints due to specific adhesion cannot be described realistically with EN 1992-1-1 (Müller & Zilch 2006). Beside the concrete tensile strength and the surface roughness of the old concrete also other factors – for example the pretreatment of the surface with water or the surface tension of the new concrete – play an important role. The consideration of these influencing factors together with the employment of an overlay concrete with slight shrinkage should – on the one hand – lead to a remarkable increase of quality of the bond strength and – on the other hand – to a reduction of load in the shear joint between old and new concrete. Based on these considerations the test program carried out at the university of Innsbruck consisted of nine specimens. In each test a 6 cm cast-in-place concrete supplement was concreted onto the test slab. Varied test parameters were the magnitude of the reinforcement in the overlay, the type of reinforcement (bars or steel fibres) the roughness of the old surface and the useable area of the shear joint. Before increasing the load until collapse occurred, each specimen had to sustain a special load procedure including a fatigue loading with 2 million load cycles and a thermal loading. In course of this load case a temperature gradient of 15 [K] according to EN 1991-1-5 was applied in the cross section by infrared heating. After that, the specimens were cooled down quickly with cold water.

In no case a collapse in the shear joint occurred (Federal Ministry for Transport 2010). Encouraging values of shear strength due to adhesion could also be observed in course of different other research projects. On the basis of these results, a new guideline (RVS 15.02.34 2011) was published, which, aberrant to ÖNORM EN 1992-2, provides the possibility to consider the part of specific adhesion also in cases of dynamic loading.

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Inspection and maintenance of the orthotropic deck of Avonmouth Bridge

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ABSTRACT

This paper is addressing the fatigue related inspection and maintenance requirements of the orthotropic deck of Avonmouth Bridge, a twin box girder Motorway Bridge close to Bristol, in the Southwest of the UK.

As a result of increased loading due to the introduction of a fourth lane on both carriageways fatigue cracks regularly form in the trough to deck welds. While not immediately threatening the integrity of the deck, because of load redistributions, new cracks are repaired on a 12 monthly basis in line with a risk based inspection and maintenance regime. The weld detail of every repaired crack is improved, new trough to deck connections are carried out as partial penetration welds rather than fillet welds.

The formation of new cracks has slowed down with the installation of a gussasphalt surfacing significantly increasing the stiffness of the steel deck plate, as steel deck and surfacing act compositely.

At critical locations the integrity of the deck plate splices is monitored with fatigue sensors which provide an accurate record of cumulative weld fatigue damage.

The standard access procedure inside the east box girder is to enter via cantilever gantries from the cycle track through hatches in the outer web. From there access to the west box girder is provided by a centre gantry which bridges the gap between both inner webs.

Three spans of Avonmouth Bridge have an orthotropic deck – the river span and the two adjacent spans. The height of the box girders gradually increases from 2.3 m to 7.6 m above the river piers. Therefore mobile scaffolding was required for inspections and repairs of the orthotropic deck. The installation of an elevated walkway and an internal mezzanine floor greatly simplified access. The mezzanine floor is of a lightweight construction consisting of horizontal and vertical cables, profiled steel sheeting and clips for connection.

An external walkway for emergency purposes is under construction. It will consist of two sections running along the inside face of the east box girder and cover the entire length of increased box girder height. The external walkway will provide personnel with safe egress from the centre gantries in emergencies when the underslung section cannot be extended.

Figure 1. Photo of Avonmouth Bridge – looking North.

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Asset management and life-cycle cost optimization for bridges on network, asset and element level

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ABSTRACT

Asset management is not a case by case decision but is based on a comprehensive set of goals and strategies as a cyclic and hierarchical process from budgeting on network level down to individual measures on asset element level. The developed approaches allow long and short term LCC – predictions for bridge assets on all levels.

The paper starts with a short introduction to goals and optimized strategy in road asset management on network level. Following a "top down" approach a short overview on road assets and road expenditures in Austria with special emphasis on bridge rehabilitation is given. A detailed overview of the structure and condition of 3300 bridges on 4920 km of regional roads in Styria together with average expenditures for maintenance and rehabilitation per year completes the analysis on network level.

The presented standardized life-cycle for bridge assets on object level is based on a comprehensive analysis of all phases of a bridge life-cycle. With mean construction costs of 1600 €/m^2 and typical measures for small, medium and major rehabilitation down to reconstruction the cost development during a typical bridge life-cycle is established. The rehabilitation intervals are determined based on an analysis of recent measures as well as the condition development between inspection intervals for the bridge inventory in the bridge database BAUT.

The resulting annual unit costs for bridges amount to 85 €/m^2 based on the verified standard life-cycle on object level. The average annual rehabilitation costs with the verified life-cycle on object level amount to 1.6% to 1.7% of the construction costs and are in line with the findings from literature (e.g. Wicke et al. 2001). Based on the findings a fast forward estimation of necessary budgets on network level as well as a first comparison of investment alternatives on object level are possible.

With a refined version of this LCC – approach on bridge element level the different condition development and behavior of bridge elements on individual bridge objects can be addressed. The paper gives a short overview of the developed approach on bridge element level together with a method to minimize the annual costs during service life under the boundary condition of no failures on critical bridge elements. The necessary failure type and rehabilitation measure catalogues are currently being developed allowing a specific condition assessment and prediction as well as planning and optimization of rehabilitation measures or other investment decisions in the future.

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Analysis of influential factors and association rules for bridge deterioration using national bridge inventory data

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ABSTRACT

Bridge is the hub of a road, playing an important role in transportation. With the age of bridges increasing in recent years, the condition of bridges is deteriorating. Bridge deterioration usually results from multiple factors. Studies in exploring the factors leading to bridge deterioration and its state estimation are booming. However, studies in the relationship between bridge deterioration and external environment by using database are still limited. Since 1992, National Bridge Inventory (NBI) has collected basic and historical information of bridges in every state, a very rich database which should include some valuable knowledge. Hence, through clustering and classifying in data mining, this study explores association rules for bridge deterioration, which is displayed in decision tree with clear routes for reference. Using association rules developed by thus study, bridge maintenance personnel can understand the future state of the bridge under their jurisdiction, respond to risks and costs of bridge deterioration, extend the service life of bridges, and protect the safety of road users.

Based on the methodology previously discussed, this study retrieves bridge information in Florida from NBI database. There were about 15,762 bridges, according to NBI. This study selects 6,948 bridges for analysis based on the principles of filtering. Input factors are existing columns in NBI database. Because columns are divided into qualitative and quantitative ones, discretization must be carried for later analysis. Continuous variables are discretized by two-step algorithms. This study selects 58 columns for later calculation. In two-step algorithm for clustering, the minimum number in a cluster is 3 and the maximum 8. It is assumed that the cluster is a deviator if the number in it is below 5% of the total sample size. Two-step algorithm divides bridges into five clusters. The number of bridges and rule sets in each cluster is shown in table 1.

This study conducts a linear regression to achieve the deterioration trend according to the average of age in the clusters above.

Table 1. Clustering of bridges.

Cluster	Number	Rule sets
1	1,973	4
2	1,244	4
3	897	16
4	612	5
5	620	1

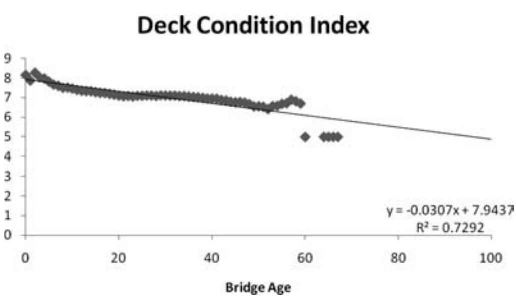

Figure 1. Average deck condition (cluster 1).

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Examples of different strategies of bridge preservation: Part 1

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ABSTRACT

Buildings are subject to an ageing process. The ageing process can be dealt with by applying different design philosophies. The date of repair and its extent result from the chosen design philosophy. If damage has occurred, a decision on the intervention's extent has to be taken considering the requirements for its repair. Damage may be a visible defect, such as a greatly diminished protection of a coating or a concrete cover. It could also be a crack in a tendon. If the required safety level is unverifiable it may also be damage.

It is therefore recommended that the maintenance and repair be planned within the scope of a life-cycle management.

The aim of the life-cycle management is to identify and counteract all ageing processes by using the resources economically and taking the planned lifetime into account.

Maintenance can basically operate in a different manner. Presuming enough time for an extensive repair, a structure can be used within the inacurracy of the forecast for the rest of the life time without further measures (Fig. 1, curve C). However, this can also mean that the economic solution is not optimal.

If sufficient time or means are not available for an extensive repair, the provided rest lifetime is limited to a few years and resources should be used economically. It is possible to combine a "small" repair with monitoring (Fig. 1, curve D). Then the repair is in order to preserve actual conditions or to ameliorate only slightly, so that sufficient security remains for the building.

Different design philosophies are explained in this paper using four examples. Two examples in part 1 and a further two in part 2.

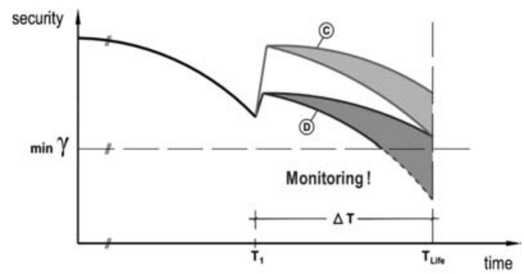

Figure 1. Different strategies of bridge preservation.

The first example deals with an arched bridge that had a new structural performance due to an incorrect repair of war damage. The associated deformation resulted in large cracks. As regards the repair, a decision had to be made as towhether it should be a complex repair to reconstruct the old system, or if it would suffice to merely preserve the new condition.

In a second example, a composite bridge is presented, which did not have the required stability due to technological and conceptual ageing. Both during construction and in a later strengthening, the transverse distribution between the main girders was considered insufficient and lead to an inadequate dimensioning of the transversal girder connections. It was also the case with this repair that the existing distribution of forces within the structural components remained almost the same.

The examples show how an improper construction or incorrect repair reduces the lifetime of a structure considerably. They also show that these defects could sometimes be corrected with small adjustments or extensive effort. This also depends on the chosen design philosophy.

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Examples of different strategies of bridge preservation: Part 2

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ABSTRACT

When considering bridges and their maintenance or preservation, one frequently refers to publicly owned road and railway infrastructure. The big number of aged bridges in this sector is increased by a huge number of privately owned bridges. The two examples discussed in the paper come from the civil engineering of power plants field which is well known for various interesting aspects.

In coal fired power plants, it is a common strategy to ensure a supply of coal by rail, together with coal bunkers for intermediate storage. The coal bunker discussed in the first example is a bridge construction for three railway tracks.

The construction sustained serious damage during day to day operations so that operation of the bunker had to be discontinued. Investigations showed a systemic error. The repair had to be carried out under heavy pressure of time and with proper consideration being given to operational issues. Prolonged closure of the coal bunker would have led to unacceptable economic damage.

The second example is also a bridge construction of a coal bunker. Here, an erroneous detail and an underestimation of the fatigue load led to a failure of the reinforcement in a suspension girder. A durable solution had to be found for the repair in addition to it having to be carried out in a very short period of time.

A detailed description of both of these examples is provided, together with the main dimensions. Occurred damage and the repairs that were carried out are presented in a figurative manner. Figure 1 shows a typical coal bunker.

Figure 1. Longitudinal view of a coal bunker.

The paper shows that bridges are neuralgic elements in the infrastructural network that keeps our economy running. The examples provided in the two parts of the paper show that great effort has to be taken when carrying out repairs under operational conditions. When taking this in mind, it is very obvious that regular bridge inspections should be undertaken in order to detect damage at an early stage. The examples also prove that the responsible principals should give preference to robust constructions with redundancies.

Primarily, redundancies provide safety and secondly, they normally enable engineers who have to design retrofits or repairs under operational conditions to make a choice; thirdly, one should consider that costs resulting from decommissioning are much higher than the normal cost savings that can be achieved by emaciating the construction.

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Research on computational method on the flexural bearing capacity of a box girder strengthened with carbon fiber sheets

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ABSTRACT

Box girder has been widely used in bridge constructions. Unfortunately, the cracks appear to a great extent in either old bridges or newly-built ones constructed with box girders, which shorten bridges' longevity and affect their normal performance in service. For these reasons, the strengthening of a box girder bridge becomes a focused issue. For the features such as high tensile strength and light weight, Carbon Fiber Sheets (CFS) are widely used in bridge's strengthening. However, strengthening box girder with CFS is barely reported and the relevant computing methods are also scarce.

Aiming at computational method and design calculation formula of the flexural bearing capacity of a box girder strengthened with CFS, the paper is to carry out the three-dimensional nonlinear finite element analysis and model experiment study. With the help of parameter language provided by ANSYS, the corresponding finite element calculation program is to be developed, and finite element model of the box girder strengthened with CFS is established for dealing with the whole process simulation calculation, taking the material and geometrical nonlinearity into account. The results obtained by finite element method are in good accord with those by the experimental method, which proves the reliability of the finite element model. Based on the results obtained by both finite element method and experimental method, this paper brings to light the mechanical properties and the normal section destructive mode of the box girder strengthened with CFS when it bears flexural loads. Furthermore, some basic assumptions are put forward and design calculation formula is established for the bearing capacity of the normal section of the box girder strengthened with CFS, which offers a reliable method for strengthening design of the normal section and strength review post strengthening of this kind of bridge.

Four RC box girder specimens strengthened by CFS, without flange plates, are designed and constructed for the experimental study on flexural bearing capacity. Their stirrups and longitudinal reinforcements are made of HPB235 and HRB335, respectively. The measurement value of the stirrup strength f_{yv} is 210 N/mm^2, and the one of longitudinal reinforcement strength f_y is 300 N/mm^2. The grade of concrete design strength is C40, and practical compressive strength of the concrete f_c is 19.1 N/mm^2 and tensile strength ft is 1.71 N/mm^2. The diameter of the stirrups is 6mm. The girder dimensions and the arrangement of its stirrups and longitudinal reinforcements are shown in Figure1. CXS-200 CFS used to strengthen the experimental girders is produced by Jinghua Baolilai CFS manufacturing Co. Ltd, Dezhou, China. At the time of establishing finite element model, 3-D solid element Solid65 is chosen to simulate the concrete, and ordinary reinforcing steel is uniformly dispersed into box girder concrete. For a pre-reinforced concrete box girder, high-strength steel strand is simulated by bar element Link8. The CFS is simulated by the element of Shell41. The various elements are connected together by coupling nodes.

A comprehensive study on the properties of flexural failure whole process of the normal section of a box girder strengthened with CFS by means of both finite element method and model test method is implemented, the authors understand fully its destroying process, the features of its deformation and failure and ultimate bearing capacity, and propose the calculation method and corresponding calculating formula for the normal section strength of the box girder strengthened with CFS. The method, formula and results proposed in the paper provide some references to the strengthening design of a box girder with CFS.

The work described in this paper is supported by Hunan Provincial Natural Science Foundation of China under Grant No. 08JJ3117. The support of China Natural Science Foundation Committee under Grant No. 51008037 is also much appreciated.

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Technical cycle of modern bridge maintenance – an overview

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ABSTRACT

An overview is given on technical steps of modern bridge maintenance highlighting up-to-date developments in research and on site techniques. Road bridges are taken for examples.

The technical steps of modern bridge maintenance can be understood by a closed loop. Figure 1 illustrates the loop (outer circle) that is controlled by the strategic steering of responsible authorities (inner block). It starts from the regular structural assessments, monitoring (cp. Farrar 2007) and (consumptive) standard repairs on site. Serious degradations, enhanced regular or extraordinary live loads or changes in use might give rise to recalculate the structure and analyse damages to evaluate the structural state (cp. Ahrens 2011). Rehabilitations, strengthening or even a (partial) rebuild follow, closing the loop back to the regular inspections. Of course, steering aims of responsible authorities have to regulate the single steps to ensure an optimised treatment. These should underlie a long-term, lifetime oriented strategy to assure sustainable, effective building activities with respect to minimised overall invests and traffic interferences.

Modern ways of structural assessment and damage monitoring are presented using auxiliary mobile platforms and computer aided methods like endoscopy or digital crack monitoring. The latter detects cracks by a standard camera with a tube-like extension from a grey scale analysis of the surface.

Structural evaluations of aged bridges by numerical recalculations are critically considered, as they in particular ask for engineering finesse (cp. Ahrens 2011 & Petryna 2005). Evidence is given to applicable loading combinations and safety margins as well as modelling aspects with respect to higher order calculation methods and redistributions of stresses to activate "hidden reserves" offered by a simplified detailing at the time of construction. Moreover, lifetime oriented design strategies with transient stochastic damage analyses and modern strengthening methods using deviated external tendons are briefly presented. They represent the extensive research activities during the last couple of years at the Ruhr-University of Bochum, Germany.

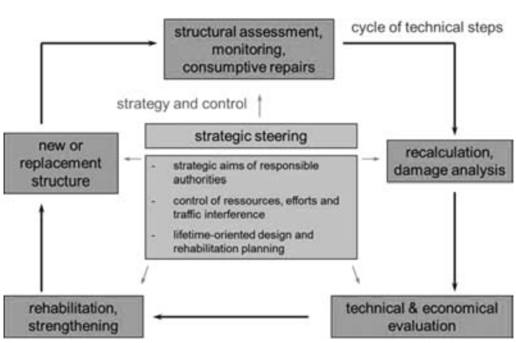

Figure 1. Technical cycle of bridge maintenance and strategic steering, acc. to Mark et al. 2011.

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The application of sacrificial cathodic protection as a corrosion control measure for the protection of reinforced concrete bridges

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ABSTRACT

1 BACKGROUND

Roads and Maritime Services, New South Wales (RMS) has a large number of reinforced concrete bridge structures that are located within an aggressive marine environment. Many of these structures are suffering from reinforcement corrosion that has been initiated by high levels of chloride ingress.

Over the past several years, RMS has primarily relied on Impressed Current Cathodic Protection (ICCP) for long-term durability rehabilitation; ICCP being the only proven technique for long-term protection regardless of the extent of chloride contamination. RMS has installed 12 ICCP systems. Installation costs are high and ongoing monitoring and maintenance is required. Also, recent RMS experience is that a number of these systems have shown signs of premature failure and have required extensive upgrading or re-installation within the first half of their intended design life.

In recent years RMS has been assessing the viability of Sacrificial Cathodic Protection (SCP) as an alternative or interim form of corrosion control. Preliminary Research & Development (R&D) studies commenced in 2007 and the results were presented at the 2007 Australasian Corrosion Association (ACA) conference. More detailed studies and field trials of various anode types continued in 2008/2009 and this work was presented at the 2009 ACA conference.

2 SACRIFICIAL CP SYSTEMS INSTALLED

Due to the favourable outcomes from the SCP R&D studies, RMS has implemented this new technology as a full-scale rehabilitation solution on four bridges. All bridges are located in a coastal environment and in each case corrosion of the reinforcement has been initiated due to chloride attack from environmental salts (salt water splash/spray).

Three different anode types have been installed; discrete anodes, strip anodes and jacket anodes.

Prior to full-scale application, small trial systems were installed for the purpose of:

- Confirming that the proposed anode type would provide adequate corrosion protection;
- Determining the appropriate anode size and spacing.

As part of each SCP system, a number of Monitoring Zones (typically 3–4 per bridge) were installed for the purpose of assessing the performance of the SCP system at these select locations. To measure the level of corrosion protection being provided by the anodes, Ag/AgCl reference electrodes were installed.

Within each Monitoring Zone the performance of the system is assessed through measurements of potential (voltage) and current. Potential measurements provide an assessment of the level of corrosion protection that is being provided. Current measurements provide an assessment of the service life of the SCP system.

3 CONCLUSIONS

- The SCP systems are generally providing a very high level of corrosion protection.
- Of the three sacrificial anode types that have been installed by RMS, the Strip anodes have demonstrated the best performance.
- Overall, SCP would appear to offer significant cost advantages compared with impressed current CP. At this early stage, with only a few SCP projects completed, it is difficult to accurately compare installation costs, however it is estimated that cost savings of up to 50% may be realised. SCP would appear to be particularly cost-effective on smaller projects or where the extent of CP application is confined to a relatively small area.

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Nondestructive evaluation of stress states of steel bars reinforcing concrete structures

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ABSTRACT

Among others the adapting and remodeling of existing buildings has become an essential task. Here, the focus is on the load condition and the reserves of the load capacity of reinforced concrete floors or ceilings. The stress state of the reinforcing bars is important information to evaluate the situation. The present state of the micromagnetic techniques already allow for a nondestructive, on-site application to evaluate the longitudinal stress in a reinforcing bar, and this first studies intend to identify the improvements and adaptations needed for a reliable and economic stress analysis.

Micromagnetic systems need to be calibrated for quantitative stress results. A simple and very accurate calibration is achieved using a sample cut from the bar under test: After the data for stress analysis are taken on the bar of interest a sample is cut from the bar and the calibration is performed in a tensile test experiment using the sample. The previously measured data are converted into stress values by applying the calibration and the evaluated stress is shown as function of the inspected length of the steel bar. This procedure was applied to evaluate the longitudinal stress of a steel bar in a ceiling of a 70 years old building.

A calibration in advance would be advantageous since stress results could be shown at each measuring position, immediately after the measuring data are taken. The evaluation of the stress state of the bar and of the load condition of the floor or ceiling could be performed on-site right after the measurements. In order to minimize the calibration effort for 11 bars of different diameters, steel grades, manufacturers and ages, it is tried to group the bars according to their electromagnetic properties. Group-specific calibrations are performed and tested under laboratory conditions.

The two commercial micromagnetic systems used in this study enable the on-site evaluation of the longitudinal stress of reinforcing bars. The sensors can easily be hand hold or mechanically moved along the trace, also in over heads positions. The measuring systems are portable, and the measurement and the data evaluation is menue guided.

Further applications of both systems will show whether the very easy to perform self-calibration or the use to the group calibration will yield better stress results. It even might be the case that the achieved results happened to be as satisfying as found and the future calibrations have to be performed using a sample cut from the bars under test.

In each case, the measuring accuracy has to be improved. For that purpose, the surface near stress gradient of the bars need to be investigated in a general manner in order to minimize its influence e.g. by appropriate adaptations of the micromagnetic sensors and the data treatment. The shape of the magnetic joke of the sensor has to have the best fit to the bar surface in order to yield a homogeneous magnetization and hence measured data with reduced scatter.

More applications of both systems are required to get a broad data base, needed to identify the best suited system with regard to reliability, system and application costs, as well as to user friendliness for on-site application.

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Bridge management and effective tools to ensure sustainable construction schedules

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ABSTRACT

Bridge construction always fits within the critical path of highway projects. An ideal project performance control system sets the baseline and performance indicators, measures the project performances and compares them with the expected plan. The road construction industry is often judged by analysts to be relatively inefficient compared to performance seen in the manufacturing industry. It is highly unlikely to proceed in all aspects entirely according to plan. Significant variations however may require a revision of the plan to meet the project's objectives. Successful performance monitoring will lead to an appropriate resource management hence to a sustainable development of the project. Through a case study of a complex balanced cantilever viaduct, this paper identifies the potential use of implementing specific management techniques to effectively manage the construction process and produce forecasting assessments for the project completion date.

In project management, it is vital to have adequate means of obtaining information about the progress of a project against a baseline and the anticipated outcome of the project. A project has traditionally been viewed as successful if it was completed on time, within budget and with the specified quality. Earned Value (EV) systems, being a standard method of measuring project performance, have been setup to deal with the complex task of controlling and adjusting the baseline project schedule during execution, taking into account project scope, timed delivery and total project budget. Vanhoucke & Vanvoorde (2006) state that although EV systems have been proven to provide reliable estimates for the follow-up of cost performance within certain project assumptions, it often fails to predict the total duration of the project. Another question frequently asked is how the project managers handle the effects of re-baselining in making their forecasts.

Lipke (2003) proposed the concept of "Earned Schedule" (ES) to address these issues. Rather than just looking at schedule performance using the value of work, earned schedule also looks at when the work was to be completed. ES aims to measure schedule performance using a time-based measure from which time index metrics are derived.

The purpose of the paper is to examine the capability of the methods to represent the schedule performance effectively in a late bridge construction project with inherent complexities and unforeseen events and to adequately forecast the final duration. The three scenarios include the overall time schedule, a re-baselining at early stages and the low-level Work Breakdown Structure (WBS) applied to a single structural element.

Sustainable development is the development that meets the needs of the present without compromising the ability of future generations to meet their own needs. To the concrete bridge community, this definition means designing, constructing, and maintaining context-sensitive bridges with long-term durability, low life-cycle impacts, sensitivity in the selection of materials and methods, and a minimal impact on the environment throughout the bridge's life. Is therefore of paramount importance to implement methods for controlling constructions schedules in a manner that available resources are not exhausted.

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Prediction models for ageing/deterioration

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Selective maintenance planning based on a Markovian approach

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ABSTRACT

The assessment of the life-cycle performance of deteriorating structures can be formulated as a reliability problem where a loss of performance greater than prescribed threshold values is considered as a *failure*. Therefore, when a failure occurs, the system moves from the current state to another one characterized by a lower level of performance. On the other hand, an operative maintenance and/or rehabilitation can improve structural performance. In this case the system may move from the current state to another one characterized by a higher level of performance. Anyway the *failure process* may be defined as a *transition process* through different service states due to environmental attacks and/or maintenance actions. Since the problem is affected by uncertainty, the assessment of the life-cycle performance of a deteriorated structure must base on a probabilistic analysis able to model the time-variant deterioration process over the time. To represent such a transition process, the Markovian system seems to be effective. However, each probabilistic approach requires databases obtained by monitoring actions. Usually these available monitoring databases are limited in time ad extension, facing to compromise the success of the probabilistic approach. Thence the probabilistic approach can be associated to a Monte Carlo simulation implemented with a suitable deterioration modeling. In this way the combined system is able to simulate a huge number of possible damage evolutions of a certain structural typology over the time.

In this paper, the deterioration affects structural members due to aggressive environment, is simulated using the damage model proposed by Biondini *et al.* (2008) and implemented into a computer code based on the Monte Carlo approach.

Using a Markovian approach based on the results obtained by the Monte Carlo simulation, the development of the structural system's state performance over the time is analyzed.

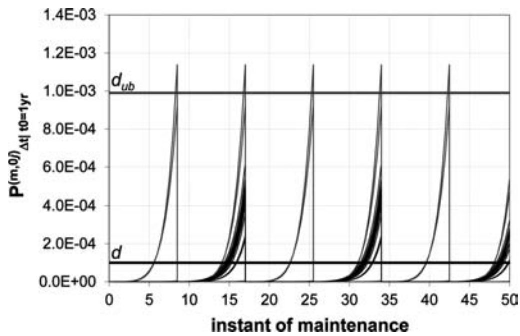

Figure 1. Possible selective maintenance scenario.

Afterwards referring to repair intervention applied just on heavily deteriorated elements, with a high probability of a state transition occurrence, some selective maintenance scenarios are investigated. Studying the effects of selective maintenance, the proposed method is applied on the case study of a steel truss structure.

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Analytical prediction model for concrete cover cracking due to reinforcement corrosion

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ABSTRACT

Current approaches for service life design of concrete structures define the end of the initiation period (depassivation of the reinforcement) as the critical limit state. It would be a significant improvement to specify the occurring damage as a limit state, i.e. the condition when cracking due to reinforcement corrosion appears. However, this is not yet possible since the time interval between the end of the initiation period and the cracking of cover concrete is unknown.

The service life design of reinforced concrete structures requires material models capable of reliably describing both mechanisms of damage and the general progression of damage over time. However, the most advanced models that are currently available only capture the processes of carbonation and chloride penetration into the uncracked concrete that is at the very beginning of the damaging process and leads to depassivation of the reinforcing steel (initiation period), see fib (2006). These models thus disregard the actual damaging phase (propagation period), i.e. the corrosion of the reinforcement. In the case of reinforcement corrosion, months or even decades may pass from the time of depassivation to crack formation and spalling. In the latter case (several decades), repair and upgrading works carried out, for instance, at the end of the initiation period would have been unnecessary if the damage had occurred after the end of the intended service life. Being aware of the time period to the occurrence of damage, which results in a restricted serviceability, is thus of considerable economic significance.

By means of a comprehensive research project the mechanism of fracture and the magnitude of stresses and strains causing cracking of the concrete cover were studied in detail. The influence of the concrete porosity on the corrosion morphology and the behaviour of the concrete cover affected by splitting stresses could be investigated on the basis of miscellaneous experiments.

By combining further novel experimental and numerical investigations it was possible to determine the modulus of elasticity of the corrosion products (Müller & Bohner 2012). The knowledge of the mechanical behaviour of rust is essential for a reliable simulation of the time dependent damage process, which was performed by means of an especially developed numerical model. This modelling approach, involving sophisticated material laws, allowed for the detailed analysis of the stresses, strains and the crack formation within the concrete cover as well as for a realistic prediction of the time development of cover cracking caused by the corrosion of the reinforcement (Bohner et al. 2010, Müller & Bohner 2011).

Based on parameter studies, which are subjected to different corrosive conditions, an analytical prediction model for concrete cover cracking was derived. This model enables the prediction of the time dependent damage process under conditions of practical relevance and serves as a part of a full probabilistic design approach for durability of reinforced concrete structures.

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Numerical analysis of degradation processes in reinforced concrete during life-cycle

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ABSTRACT

Nowadays durability of reinforced concrete structures is affected primarily by different damage processes. In comparison with the load-carrying capacity detection of the durability is very difficult. There are a lot of guidelines and recommendations which the design engineer has to fulfill, but avoidable damages occur repeatedly. Many of these defects result from corrosion of steel reinforcement due to weather and chemical attacks. Experimental studies of the damage mechanism are very complex because of low process velocities. Therefore numerical models are developed which allow a systematic investigation of the damage mechanism and identification of the relevant factors of influence, see Baroghel-Bouny et al. (1999), Gawin et al. (1999), Steffens et al. (2002) & Ostermann (2007).

Here, a numerical model is introduced, which allows the description of the most important processes in concrete in a monolithic algorithm. Therewith the model can be used for predictive numerical analysis of the whole life-cycle. Furthermore the influence of each process on the durability of the structure can be evaluated. The main advantage of the new model is the possibility of coupling different damage mechanism. On the one hand a parallel coupling of processes is possible, which proceed at the same time. On the other hand processes, which run consequently can be coupled in series. Here, the new approach is used to analyze the influence of the hydration of young concrete on the transport behavior of concrete during its life-cycle.

The proposed numerical model comprises three parts, the choice of principal variables, the principal processes to be described and the required constitutive models. The principal variables are variables, which characterize the general state of construction during the whole life-cycle. Examples of these are the deformation, relative humidity, temperature or concentration of toxic substances.

The principal processes describe the development of the principal variables at every time step. Transport, reaction and deformation behavior may be identified as the most important principal processes. The principal processes have to be described by mathematical and physical balance equations which base on the theory of porous media using the conservation laws of mass, energy and linear momentum.

Additionally to the formal treatment of principal variables and principal processes constitutive models dependent on specific variables and material investigated have to be considered which describe the characteristics for example of deformation, of transport behavior or of the chemical reaction kinetics as well as the development of internal variables.

Finally, numerical analysis of the durability of a reinforced concrete bridge is presented. The proposed model allows a detailed analysis of several degradation mechanism as well as the analysis of the durability during the whole life-cycle. Measures for the maintenance can be deduced from the results.

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Optimal next inspection time for bridges based on corrosion deterioration

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ABSTRACT

As the transportation demands grow rapidly, and some vehicle bridges show corrosion degradation, systematic and life-cycle based procedures may help to schedule cost-effective maintenance programs. In this paper a reliability-based formulation is proposed for prediction of the next bridge inspection time on the basis of corrosion evolution prediction. Given unknown characteristics on the corrosion initiation time, the epistemic uncertainty on this parameter is explicitly included. For bridges where little or no follow up has been previously made, the characterization of damage state and the bridge integrity state contains a great deal of epistemic uncertainty. The impact that this epistemic uncertainty has on the identification of corrosion initiation time is appraised in order to estimate the cost/benefit of making additional studies that may contribute to improve the corrosion initiation time and the overall bridge safety assessment. The limit state and structural reliability of the bridge is calculated in terms of a damage index that depends on the bridge corrosion evolution and the consequent reduction on moment capacity. As a consequence, the bridge failure probability evolves. The acceptable failure probability is calculated as a function of the expected losses and it is used as the safety threshold; whenever the bridge failure probability goes below the acceptable value, marks the proper time to perform the next bridge inspection.

The acceptable bridge failure probability is obtained through:

$$P_f = C_2 / \left[\left(C_B + C_H + C_U \right) * PVF + R * PVF' \right] \qquad (1)$$

where:

$C2$ is the cost to reduce the annual failure probability on the order of "*e*", CB the cost of bridge substitution and contents, CH the losses related to human lives, CU the user costs and the loss of income due to deferral of the revenues Rafter the bridge replacement and PVF and PVF' are parameters of the present worth factor required to update future costs to present value.

The curve, for several costs of consequences, is shown in Figure 1.

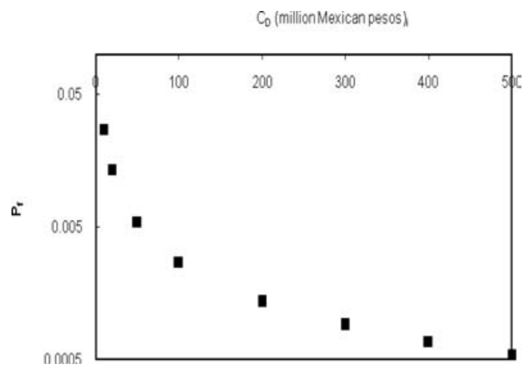

Figure 1. Acceptable failure probability for several consequences costs.

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Modelling of the saturation behaviour of hardened cement paste during freezing and thawing action

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ABSTRACT

Modelling the mechanisms of frost damage is a key element in the life-cycle management of concrete structures (Müller & Vogel 2011). For a realistic description of the concretes behaviour when subjected to a freeze-thaw loading without de-icing salts, the water saturation behaviour of the concrete must be quantified over time and more important with a high spatial resolution especially in the boundary zones of the building member. The reason therefore is to be seen in the fact that every freeze-thaw cycle provokes an increase of the degree of saturation of the concretes pores with water, which by far outweighs the increase in saturation resulting from classical transport processes such as capillary suction. This however only occurs if several unfavourable conditions are met at the same time. These conditions include numerous freeze-thaw cycles with low temperatures in combination with sufficient water influx to the building element and insufficient frost resistance of the concrete. Upon reaching a critical value of pore saturation, structural damage will eventually occur at only one freeze-thaw cycle (Fagerlund 2001).

The water suction caused by the frost attack cannot be described by single classic transport laws such as capillary suction, diffusion or permeation (Kruschwitz 2008). The reasons for this were explained by Setzer with the so-called "micro ice lens model" (Setzer 2009, Setzer 2001).

The challenge of assessing the saturation behaviour of concrete or hardened cement paste is to experimentally obtain reliable information on the water transport processes during the exposure. As the deterioration proceeds unidirectionally from the surface into the concrete and results from a cumulative process, changes in the water content must be measured continuously with high spatial resolution and without destroying the sample. A suitable measurement method for this purpose is the Nuclear Magnetic Resonance imaging method (NMR) based on the strong nuclear magnetization of hydrogen atoms. Basic information on the NMR method is given in (Hardy 2012). For the in situ investigation of moisture profiles in cement stone during the frost-thaw stress a NMR measurement system with a specially adapted sample head which allows for an in situ cooling of the sample in the measurement chamber has been developed.

Based on time- and depth-resolved experimental moisture investigations during a frost attack, a new engineering model approach was developed, in order to predict the saturation behaviour – and therefore the deterioration behaviour – of hardened cement paste during a frost attack. Considering the underlying physical mechanisms the model approach allows for a time- and depth-resolved description of the degree of saturation of hardened cement paste during a freeze-thaw cycle depending on the water/cement ratio w/c and the prevailing environmental conditions during each cycle. The presented model will serve as the basis for a realistic description of the deterioration behaviour of concrete structures exposed to frost attack.

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Seismic intensity parameters as damage potential descriptors for life-cycle analysis of buildings

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ABSTRACT

Overall structural damage indices are a useful tool for Life-Cycle Analysis of Civil Engineering reinforced concrete structures in seismic regions (see e.g. Biondini & Frangopol, 2008, or Strauss et al, 2004) . Relevant earthquake accelerograms (natural and artificial) have inherent information, which can be extracted either directly, like the Peak Ground Acceleration (PGA) and the total duration, or indirectly using a computer supported analysis. The results of such an analysis can be classified in:

- peak parameters, e.g. PGA, Peak Ground Velocity (PGV), Peak Ground Displacement (PGD),
- spectral parameters, e.g. response-, energy-, Fourier-spectra and
- energy parameters, e.g. ARIAS intensity, Strong Motion Duration (SMD) after Trifunac/Brady, power $P_{0.90}$, Root Mean Square (RMS) of the acceleration values.

The definitions of these parameters have been presented in the literature (see e.g Arias 1970, Meskouris 2000, Elenas 2000).

This paper focuses on a methodology to quantify the interrelationship between the seismic parameters and the structural damage. First, a numerical analysis of the used accelerograms provides several peak, spectral and energy seismic parameters. After that, a nonlinear dynamic analysis is carried out to show the structural response for the given seismic excitations. Keeping in mind that most of the seismic parameters are characterized by a single numerical value, single-value indicators have been also selected to represent the structural response. The attention is focused on response parameters which can be used to describe the damages on buildings. For these reasons, two widely used Overall Structure Damage Indices (OSDIs) are selected to represent the structural response. Thus, the modified Park/Ang model (Park & Ang 1985) and the Maximum Inter-Storey Drift Ratio (MISDR) have been used. Next, correlation coefficients after Pearson and after Spearman are evaluated to express the interrelation grade between the seismic acceleration parameters and the examined damage indices. The presented methodology is applied to a multi-storey reinforced concrete frame building, designed according the demands of the recent Eurocodes EC1, EC3 and EC8 (CEN 2004), for a set of natural seismic acceleration records.

In addition, these analyses pinpoint weak spots in the design and ensure that the realistic nonlinear behavior of the structure is as planned, with enough built-in ductility supply to accommodate the seismic ductility demands of the used aseismic design code. Finally, the basic philosophy of recent aseismic codes can be verified. This philosophy is to minimize the probability of total or partial structure collapse during a severe earthquake as well as to minimize the damage level during low and moderate seismic excitations.

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Bridge maintenance education system based on E-learning

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ABSTRACT

Although it has been widely recognized that bridge maintenance is one of emergent and important issues, it is not easy to keep the existing bridges in satisfactory conditions because of the lack of enough budget, sufficient data, and experienced inspectors and maintenance engineers. Many experienced bridge engineers have retired and the number of engineers has been reduced. It is, then, urgent how to inherit their knowledge and experience. The purpose of this study is to develop a support system for learning the fundamental knowledge and recent advanced technologies in bridge maintenance by using E-learning.

In recent years, E-learning has been introduced as an education system based on the Web technology in many educational institutions along with the popularization of Internet. Various educational institutes have introduced E-learning system in order to improve educational method. For example, a class conducted on E-learning system is allowed as a credit in a university. Furthermore, companies have introduced E-learning system. However, E-learning system requires huge costs and much labor in order to perform an effective education. In fact, some universities terminated E-learning due to these reasons. The one of these reasons is the difficulty of production of effective educational contents. (Wada, 2004.)

The E-learning system of SCPD (Stanford Center for Professional Development) is a successful example of its management. In the E-learning site provided by SCPD, students who are employed as a part time produce educational contents. This management performs an effective education at low cost. In addition, SCPD provides contents to not only its student but also companies (Sakamoto, 2004.). Also it is guessed that this is a success factor.

In the example described above, the educational capacity of SCPD is considered as the biggest factor to the production of high-quality educational contents at low cost. However, SCPD is not general; there is generally a trade-off between quality of educational content and management cost. Therefore, E-learning system requires for a support to productions of educational contents.

In E-learning, various media such as text, audio and video are adopted to educational contents. In this study, an attempt is made to develop a support system to production of educational video. Some videos used as educational contents are very long or includes various topics. This is because a long video which was taken is applied to content as it is. A video which includes parts unrelated to learning may discourage the learning motivation. To overcome this problem, the cutting work for a video along with its content is generally required. However, the longer a video gets, the longer time this work spends. Therefore, in this study, the system which makes chapters along with speech words in a video automatically is proposed. In the proposed system, making chapters automatically is expected to reduce the time for the production of content and to improve its educational effectiveness. The proposed system adopts summarization method based on important sentence extraction by using Maximal Marginal Relevance (MMR: Carbonell, & Goldstein, 1998) in order to make chapters automatically. Experimental result is presented to demonstrate the applicability and efficiency of the proposed system.

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Structural, economic and environmental performance of fiber reinforced wood profiles vs. solutions made of steel and concrete

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ABSTRACT

The purpose of this paper is to evaluate the ecological performance of moulded fibre-reinforced wood profiles and to compare them with other building materials. The estimation is based on the concept of environmental Life-Cycle Costing (eLCC).

The moulded wood profile is a new product, the manufacturing of which is done in a therm-hydro-mechanical regime (Haller 2007).

Fibre-reinforced wood profiles consist of such products reinforced by filament winding. The reinforcement copes with the drawbacks of wood such as low strength and durability and thus makes the product more applicable and reliable. Although wood has been proved to be a sustainable material in various researches, the use of resin and fibres has a negative effect on the environment. Therefore, the ecological performance of the entire compound is assessed.

The load-bearing capacity, which is tested in compression, serves as the functional unit. Finally, the concept of Life-Cycle Assessment (LCA) is applied to evaluate the ecological performance. Here, the evaluation utlizes the Eco-Invent database to establish the flow diagram and the material inventory of the investigated object. Two methods, Eco-Indicator 99 (I) and CML, are adopted to create the LCA.

Profiles from steel and concrete with the same load bearing capacity are designed according to the building code.

Results of the LCA for moulded fibre-reinforced wood profiles are presented and compared to gluelam, steel and concrete.

Data is taken from the Eco-Invent database. With the results from life-cycle impact analysis environmentally sound products and processes can be engineered.

Compared to Eco-indicator CML seems to be more applicable for products.

With the existing knowledge and tools, it is possible to find the best environmental and economical solution for a specific building. In practice, however, these two criteria may be satisfied by different materials and solutions. It is important that design for life time becomes an accepted approach despite of the problems that are rather obvious. LCA will improve environmental awareness and will enhance innovation in the construction sector.

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A stochastic prediction model of degrading process for tunnel management systems

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ABSTRACT

A resurgence of attention to and interest in system identification techniques has recently been observed among engineers in the field of structural engineering in conjunction with the rehabilitation of existing structures possibly damaged by past earthquakes and other loads. The load resisting capacity of these structures may also degrade due to aging. As a structure deteriorates or approaches its design life, the existing condition may be quite different from that of the original system. In this regard, the field of system identification has special significance in the connection with the asset management of the existing structure.

Especially, tunnel concrete degradation has become serious social problem since tips of concretes fell off in concrete structures. In this regard, much effort has been paid for the inspection work. So far, the monitoring and maintenance of concrete structure has been done by visual inspections. Then, system identification techniques must exhibit analytical stability and numerical efficiency in identifying significant parameters indicative of deteriorating process of tunnel concrete.

This paper propose a probabilistic model for describing temporal variation of damage accumulation in tunnel lining concrete under the effect of low temperature and snow in cold regions. The proposed model is based upon a stochastic differential equation driven by a Poisson white noise, whose solution process represents a damage accumulation.

Numerical demonstrations of the proposed model are indicated in Figs. 1 to 3, in which parameters of the model are identified by using inspected data obtained from existing tunnels.

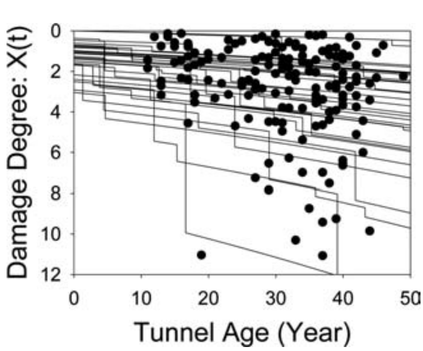

Figure 1. Reproduced samples by Poisson Model ($\lambda = 0.05$).

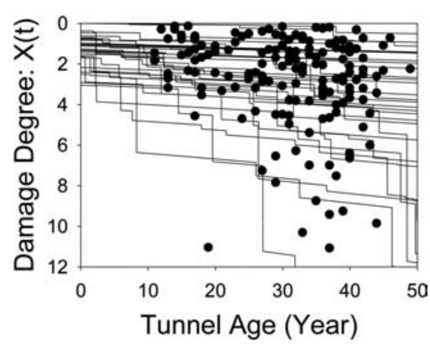

Figure 2. Reproduced samples by Poisson Model ($\lambda = 0.1$).

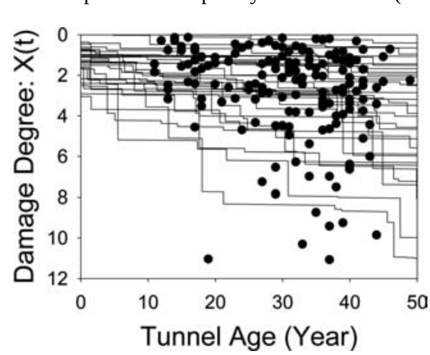

Figure 3. Reproduced samples by Poisson Model ($\lambda = 0.15$).

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Remaining life prediction of an aged bridge based on field inspections

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ABSTRACT

Because the major part of bridge assessment which is the kernel of practical maintenance is to develop a method of remaining life prediction based on field inspections such as approaching visual inspection, concrete core test, etc., the author have been developing a Bridge Management System (J-BMS) which is able to predict the deterioration process of existing bridge members.

This paper describes a remaining life prediction method of concrete bridges in service based on field inspections and concrete core tests. The prediction of remaining life of a target bridge which called KT bridge (old bridge), a reinforced concrete simply supported T-beam bridge (total length of 364 m with 28 spans) can be quantitatively estimated by applying the Bridge Rating Expert system (BREX), a subsystem of the Bridge Management System (J-BMS), with field inspection data.

The main conclusions obtained in this paper can be summarized as follows:

1) As a result of diagnosis (soundness check and remaining life prediction) by the BREX system, it was found that both the main girder and concrete deck had a remaining life not exceeding a decade. In view of this finding, the decision based on the results of diagnosis to dismantle, remove and replace the old KT Bridge was determined to be valid.
2) Existing bridges are generally checked mainly through visual inspections. Inspection data collected even by experienced professional engineers sometimes vary because of the diversity of the type of deterioration or damage, and oversight due to varying levels of knowledge and experience of engineers. That affects the reliability of subsequent diagnosis and selection of repair and retrofit measures. In this study, it was revealed that short-time virtual experience using two-way dialogue by the "bridge visual inspection support system using virtual reality" that enables virtual experience of damage to and deterioration of a bridge and of the changes thereof with time is effective for reducing the variations of inspection data collected by experienced professional engineers.

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Gamma processes for the degradation analysis of engineering structures

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ABSTRACT

In maintenance management of infrastructural systems where the safety level of the subjected concrete structures continuously decreases as a result of aging process, more and more stochastic processes, like Gamma process, are being taken in to account. Gamma Process approach is applied as a tool in quantification of deterioration and assessment of the lifetime aging of a structure which result from the environmentally dependent mechanical and chemical loads. This approach considers temporal uncertainty associated with the evolution or progression of deterioration over the lifetime of a structure which cannot be captured by a random variable model. Hence temporal uncertainties associated with the deterioration in lifetime are expressed by a continuous time stochastic process that allows stationary independent jump of increment. Gamma Process approaches can be applied in structures depending on classical information from visual inspections, like for instance crack formation, bending, but also development of near surface strain (stress), and allow the development of forecasting models as an effective basis of decision making for optimization of inspection intervals and maintenance measures. An effort of making inspection and examination data efficiently available for infrastructure owner is persuasive, on one hand because of the bulky nature of inspection records which are not very suitable for decision making on the other hand limited information from the main inspection of for instance an older structure. The current inspection techniques in reference to the main examination have not shown significant changes in the last decades and have limited ability to capture the initial degradation process with a higher probability. This reality justifies also the necessity of a stochastic process modeling. Therefore development of effective methods to set up appropriate models for structures that are subjected to inspection; serves to improve and adapt the inspection routines and as a result for temporal and systematic optimization of the maintenance and management systems. In the lifecycle assessment; preventive maintenance incorporates characterization of initial degradation, determination of inspection techniques and interval for the given deterioration, execution of inspections, adaptation of degradation profile based on inspection outcomes as well as selection of inspection techniques and examination intervals. This paper shows an approach for an effective adaptive maintenance management employing the Gamma process with incorporation of additional information from inspection, simulation of deterioration process etc. Furthermore this adaptive method facilitates temporally and systematically optimized maintenance action; hence it is an essential tool in decision making. This particular study focuses on the degradation process which took place in one already damaged bridge and simulated by the application of a Cellular Automata approach compared with the Gamma process approach. Specifically the Gamma process approach considered structural behavior, like crack formation, bending, and surface strain (stress development), which can be captured by traditional inspection and/or monitoring method. Owing to material analysis after the demolition of the examined Neumarkt-bridge in south Tyrol, Austria, the degradation process due to corrosion and carbonation of pre-stressing could be closely researched. Thus in this contribution among others depending on material characteristics during and after demolition of pre-stressed concrete bridge, the possibility of the Gamma process approach (in relation to visual inspections) to capture the internal mechanical changes for example due to pre-stressing steel corrosion processes is conducted. A pronounced correlation between the Gamma Process approach and the internal mechanical properties of structure are bases for a (a) well quantitatively ascertained remaining service life, (b) optimization of inspection periods, (c) identification of critical structural components for the overall condition, (d) consequently cost-efficient maintenance.

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Bridge condition assessment based on long-term strain and vehicle monitoring

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ABSTRACT

Bridge structures in service may be subject to environmental erosion, material aging, fatigue, and many other detrimental factors and inevitably suffer from damage accumulation and load capacity descending. These deficiencies may result in fatal incidents and huge loss of life and properties. On such an occasion, effective countermeasures to ensure the safety, durability and serviceability have great importance. Structural Health Monitoring (SHM) provides a useful tool for detecting the evolution of damage and estimating performance deterioration of brides, and attracts worldwide research interests. In the past decade, fruitful achievements in all aspects of structural health monitoring have been paid back for researchers' great efforts, including the progresses in advanced smart sensors, data acquisition and processing systems, damage detection, model updating, safety evolution, design methods of SHM systems, implementation to practical civil infrastructure and strategic decision-making etc.

However, urgent needs and challenges for the development of SHM system in bridge structures still open. One of the existing challenges lays in the structural condition assessment based on the monitoring data. The health condition is a key issue for structural safety assessment and decision making. Without accurate bridge structural health evaluation, the functions of the SHM system are limited. The difficulties for the health condition evaluation mainly come from the following three factors: (1) slight condition changes in a complex structural system are difficult to be recognized with the insufficient information gathered by limited sensors; (2) changes in structural behavior could be the consequence of at least two factors: structural deterioration and environmental influences. And it is highly possible that changes due to normal environment variation mask that due to structural deterioration. (3) long-term monitoring data exhibit strong randomness and uncertainties.

In an effort to solve the mentioned problems, several solutions are proposed in this paper: (1) in the condition of limited funding, try the best to obtain more monitoring data; (2) distinguish the structural influences from various environmental factors or put forward an environment insensitive solution for structural behavior evaluation; (3) use statistical methods to find the evolution rules of the monitored parameter and the interaction between various monitored parameters.

In this study, the features of the operational loads including temperature, wind and traffic loads were analyzed. A damage detection method based on the influence line of the vehicle load was proposed. The proposed method was validated by numerical simulation and demonstrated by using the SHM data of The Changjiang Bridge, China. The research conducted indicates that the methodology proposed is qualified for structural condition assessment so far as the following respects are concerned: (a) capability of revealing structural deterioration; (b) immunity to the influence of environmental change; (c) adaptability to the random characteristic exhibited by long-term monitoring data.

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A consideration on the deterioration of tunnel lining based on actual inspection data

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ABSTRACT

This paper proposes deterioration methods based upon actual inspection data in order to carry out strategic maintenance and to rationalize life-cycle cost analysis for tunnel structures.

The resistance of deteriorating tunnel structures is non-stationary stochastic processes, and reliability problems of such structures are essentially time dependent reliability problems.

While the forecasting of deterioration is one of the most important things in infrastructural asset management, it is often the cases where are few data stocks available for estimating the deterioration forecasting model (See Figure 1).

Firstly, using deterioration rates, the methodology of predicting of deterioration is discussed to model the deterioration of tunnel lining concrete. The present problem is the estimation and/or identification of the degrading process of the tunnel lining concrete represented by the Ito stochastic differential equation as follows (Baxter and Rennie 1996).

$$dX(t) = \beta X(t)dt + \sigma X(t)dW_1(t) \tag{1}$$

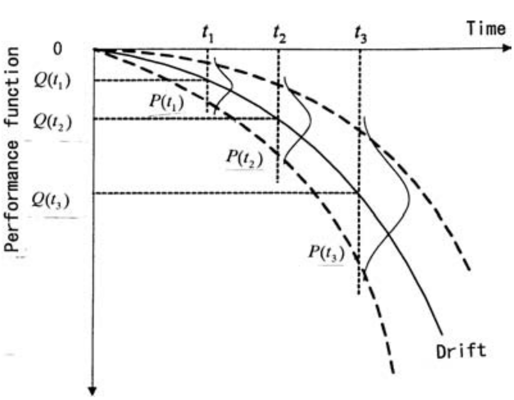

Figure 1. Degrading process of the tunnel lining.

where, β: The constant drift parameter, σ: The constant volatility parameter, $W_1(t)$: The Wiener process.

And, assume that the resistance recovers immediately after each repair process, so that $Z_i(t)$ becomes discontinuous and returns to vertically at a repair time (Madanat, S. 1997).

$$dZ(t) = \beta Z(t)dt + \sigma Z(t)dW_1(t) + \sum_{i>1}\{Z_1^* - Z_2^*\}l(t - t_1^*) \tag{2}$$

where, l: The Dirac measure.

In the secondly, for the prediction of the individual structure of tunnel lining concrete, a probabilistic approach using the distribution of deterioration rates and its own historical inspection data is proposed. The validity of these methods is verified through the actual visual inspection data of tunnel lining concrete. And, the applicability of the methodology presented in this paper is examined against the real data concerning the deterioration on the road tunnel lining concrete in Hokkaido.

In addition, the average deterioration curves, variance and distribution density of time history, are obtained using the visual inspection data of tunnel lining concrete, which was considered the repaired process of the each tunnel lining (Nishi et al, 2011).

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Reliability of deterioration prediction with Markov model for mooring facilities

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ABSTRACT

To discuss the influence of variability in the rates of deterioration on the lifetime prediction, transition probabilities in the Markov model are estimated at first with referring to the overall condition grades of 335 mooring facilities nationwide in Japan. The facilities include gravity quay walls, open-piled marginal wharves, and sheet pile quay walls.

It is found that the distributions of transition probabilities are almost the same regardless of facility types. The mean of the transition probabilities are 0.081 for gravity quay walls, 0.094 for open-piled marginal wharves, and 0.080 for sheet pile quay walls. Based on the Kolmogorov-Smirnov test, it is concluded that the distributions of transition probabilities conform with the log-normal distribution. Following the log-normal distributions, the probability density function and the cumulative density function are determined and are applied for the lifetime predictions.

Figure 1 shows the distribution of lifetime (elapsed years after starting the service to require preventive maintenance) predicted. The modes and medians of lifetime predicted are 18 and 25 years for gravity quay walls, 15 and 22 years for open-piled marginal wharves, and 19 and 26 years for sheet pile quay walls respectively. Approximately more than a half of the mooring facilities have the lifetime of 25 years or shorter. Moreover, it can be predicted that almost all the mooring facilities require some preventive interventions at least once during 50 years.

Figure 2 shows average errors in lifetime predictions. The error decreases as an increase in the transition probability. As shown in the figure, an equation is proposed to approximate the error depending on the transition probability. It is found that the errors are 2.96 years for gravity quay walls, 3.55 years for open-piled marginal wharves, and 3.00 years for sheet pile quay walls. These values may be used for considering the variations and errors in lifetime predictions.

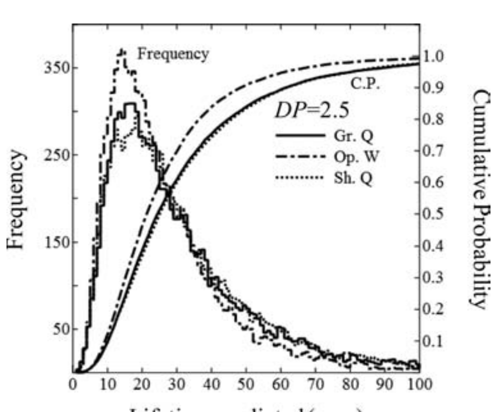

Figure 1. Frequency and cumulative probability of lifetime predictions. Gr.Q = gravity quay walls, Op.W = open-piled marginal wharves and Sh.Q = sheet pile quay walls.

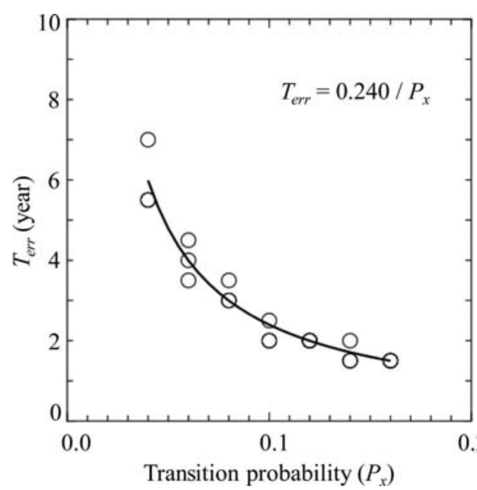

Figure 2. Average error in lifetime predictions (T_{err}).

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SPECIAL SESSIONS

Performance based evaluation of corrosion in reinforced and pre-stressed concrete structures

Organizer: U. Schneck

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Corrosion-induced cracking evolution and reliability prediction of aging RC structures

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ABSTRACT

The serviceability and durability of concrete structures may be seriously affected by the corrosion of steel reinforcement in structures that are exposed to aggressive environments, such as motorway bridges, car parks and marine structures. Reinforcement corrosion consumes original steel rebar, generates much lighter rust products and creates expansive layer at the interface between the reinforcement and the surrounding concrete cover. As corrosion progresses, the expansive displacement at the interface generated by accumulating rust products causes tensile stress in the hoop direction within the concrete cover, leading to radial splitting cracks in the concrete. The cracking and eventually spalling of the concrete cover significantly affect the bond strength between the rebar and the surrounding concrete cover and consequently influence the durability and resistance of reinforced concrete structures. Therefore, correct predictions of the evolution of cracking in cover concrete and evaluations of residual strength of the cracked concrete are of great importance to estimate the remaining life and prevent the premature failure of reinforced concrete structures.

Reliability analysis associated with limit states of a structure is often utilized for assessing the safety of the structure at design stage. This approach however displays some limitations for the reliability analysis of existing structural systems, e.g. it usually ignores additional knowledge available from health monitoring; it often neglects the performance deterioration over time; and it is unable to predict the safety and performance in the future. New investigations are therefore required on reliability-based reassessment of structural systems focusing on updating structural reliability from monitored data. The symptom-based reliability is more appropriate than the traditional time-based reliability for existing structural members and system as the monitoring process can provide useful data (symptoms) for further assessing current condition and predicting future performance.

The paper presents an approach for evaluating crack development in the cover concrete due to reinforcement corrosion and predicting the structural reliability and remaining life on the basis of the concrete cracking development. The proposed approach estimates analytically the crack width in concrete cover over time, where the thick-walled cylinder model for the concrete cover and the realistic bilinear softening curve for the cracked concrete are considered. The analytical estimates of cracking development are then examined by existing experimental data. The crack width is therefore adopted as a representative symptom associated with structural performance deterioration. By assuming that the life time of a structural system is a random variable, the structural reliability can be estimated in relation to a hazard function. The reliability function directly links to the chosen hazard function and is expressed as a function of the representative symptom. Therefore, the symptom-based approach can be used for evaluating the current and future conditions and predicting the remaining lifetime during the whole service life. Finally, a case study is utilized to demonstrate the applicability of the proposed approach for predicting the reliability of aging reinforced concrete structures affected by reinforcement corrosion.

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Durability of cooling tower constructions and methods of their repair and reinforcement

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ABSTRACT

The paper discusses the problem of the durability and rehabilitation of reinforced concrete cooling towers in power plants. During long operation time of a cooling tower (after twenty to thirty years) there may occur serious damages of construction elements. In the paper authors analyze most frequent reasons leading to failures in the construction of cooling tower shell and load-bearing supports. Possible reasons for the failures usually stem from a wrong structural analysis and a poor workmanship, both resulting in inadequate concrete reinforcement of shell or even inadequate shell thickness itself. Another reason may be a harmful influence of a coal power plant environment causing concrete degradation and concrete reinforcement corrosion. All these mentioned factors eventually lead to a catastrophe. The authors study such failures giving examples along with the evaluation of a technical state and testing of a cooling tower after a long operating time. Furthermore, the authors discuss the methods of construction repair and reinforcement that increase both cooling tower durability and safe and sound operation time.

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Probabilistic assessment for structural performance of port RC structure

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ABSTRACT

To achieve the strategic maintenance of port facilities, the authors have tried to establish simplified assessment procedures for port Reinforced Concrete (RC) structures by focusing on the relationship between the deterioration grade and structural performance aiming at achievement of strategic maintenance. In this paper, total of 40 RC slabs extracted from existing port structures was tested to evaluate the relationship between visually judged deterioration grades and load carrying capacities. Wide variations were observed in the relationship show in Figure 1, however, the load carrying capacities tended to be smaller than the design expectations when the symptom of deterioration appeared on the surface of concrete.

The tested results implied that the probabilistic approach is required to appropriately evaluate the structural performance with considering the variation observed in the existing structure efficiently. Therefore, based on the statistical analysis on the relationship between deterioration grades and load carrying capacities, a probabilistic relationship between structural deterioration grades and load carrying capacities ratio was proposed as shown in Figure 2. For example, the load carrying capacity ratio of RC slab becomes 1.0 or more with the probability of about 70% for deterioration grade *c*, about 60% for deterioration grade *b*, and about 20% for deterioration grade *a*, respectively. For another example, the minimum load carrying capacity ratio with the probability of 95% can be estimated as 0.72 for deterioration grade *c*, as 0.65 for deterioration grade *b*, and as 0.35 for deterioration grade *a*.

By utilizing proposed method, the priority of repair work to RC decks was examined based on estimated probabilistic structural performance for 7 existing wharves as listed in Table 1. For achievement of the strategic maintenance of large numbers of existing port facilities, the probabilistic structural performance obtained by the proposed method is expected to become an effective evidence for the prioritization of countermeasures.

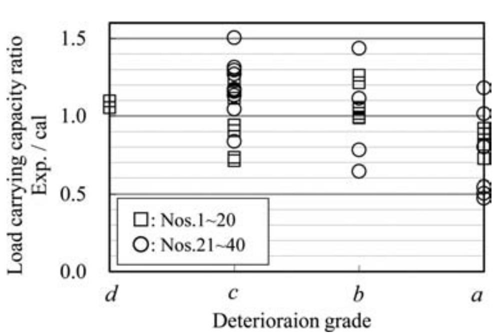

Figure 1. Deterioration grades vs. normalized maximum loads.

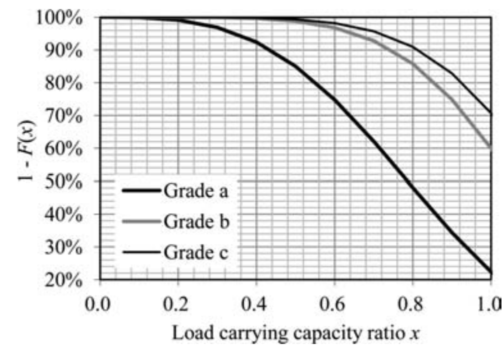

Figure 2. Cumulative distribution with which the minimum load carrying capacity ratio becomes *x* or more.

Table 1. Probabilistic structural performance.

	RDG*		Ave. of whole wharf		Min. of BLs in a wharf	
W.	Beam	Slab	Beam	Slab	Beam	Slab
A	*c*	*c*	0.716	0.758	0.709	0.684
B	*c*	*c*	0.875	0.885	0.790	0.797
C	*c*	*c*	0.697	0.710	0.674	0.696
D	*c*	*c*	0.714	0.707	0.703	0.687
E	*c*	*c*	0.709	0.705	0.680	0.684
F	*c*	*c*	0.695	0.700	0.658	0.678
G	*c*	*c*	0.700	0.709	0.663	0.678

*Representative deterioration grade.

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The impact of resistivity on potential mapping

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ABSTRACT

The focus of the presented research is the influence of the concrete resistivity on potential mapping. Potential mapping is a non-destructive measurement method that provides information about the corrosion state of a reinforced concrete member at time of inspection. A real structural part (2 m × 1.2 m) from a 40 year old building (exposure class XD3) was used for the investigation. The aim of the research was investigating potential mapping on a real sized and realistically exposed specimen.

The specimen is stored unsheltered. Potential mapping was executed frequently and after long rain and dry periods. Beside the influence of the precipitation potential differences were measured at high and low temperatures. The aim was to cover all environmental situations which have a high impact on concrete resistivity. In addition to the changing environment it was taken also into account different pre-wetting conditions.

The next diagram (figure 1) summarizes all measurement data which was gained with the same pre-wetting condition in 2011 according to Guideline B3.

All measurement results show comparable outcome even comparable gradients and the values are in a comparable range. The curve with the most positive values is from December 2nd 2011. This measurement is characterized by low temperatures and an exceptional long dry period of six weeks. High concrete resistivity is expected during the measurement. The curve with the lowest values is from June 27th 2011. Before this measurement high temperatures were combined with high precipitation. In this case low concrete resistivity is assumed.

The concrete resistivity has a major influence on measured potential differences. Changes of moisture content from "dry" to "wet" shift potential differences to more negative values. The influence of temperature is not pronounced. Local potential minima and potential gradients do not change. Potential mapping is a very robust measurement method.

This results show very clearly that corroding and non-corroding areas cannot be divided using absolute values.

Furthermore, not only the impact of resistivity on potential mapping has to be investigated but also the interaction between pre-wetting and the expansion of the macrocell corrosion process.

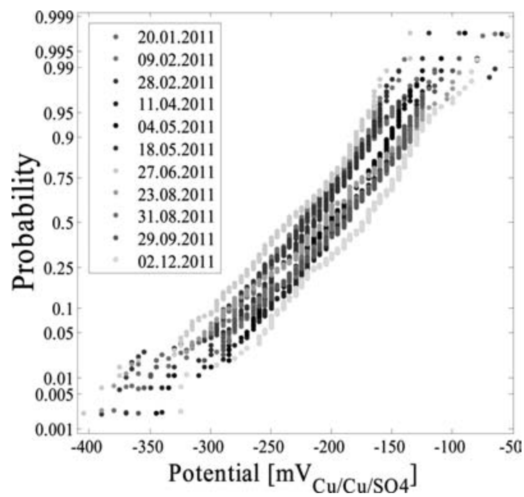

Figure 1. Probability plots of all measurements in 2011 with the same pre-wetting condition after Guideline B3.

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Non-destructive corrosion surveys: Methods and opportunities

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ABSTRACT

The corrosion of reinforcement is one of the most important threats during the lifetime of a reinforced or pre-stressed concrete structure; mostly caused by chloride, which may penetrate into the concrete as de-icing salt, as spray water or from chloride bearing ground water in costal regions.

After a longer period of chloride ingress through the concrete cover and damage propagation, corrosion and loss of rebar cross-section only become visible and auidble with spalling and delamenating concrete. Often local concrete repair fails, because new corrosion elements will be created between repair patches and adjacent reinforcement, still wrapped in chloride containing concrete. Sometimes, heavy rust staining or other visual signs seem to indicate severe corrosion problems; a major repair or even a replacment of the structure is planned, and after hydrojetting the concrete the reinforcement appears to be in a good shape.

Non-destructive, electrochemical measurement methods give the opportunity to assess corrosion activity long before concrete will be damaged and to determine exactly corrosion active and corrosion endangered areas. With that knowledge, maintenance and repair actions can be planned exactly, in the correct extent and for a long durability of the structure – a truly performance based approach.

But smart corrosion surveys need a sound understanding: Different to most of the other measurements used in civil engineering, corrosion measurement methods have no easy interpretation – there are no definite values which allow a direct determination into “good” and “poor”. The data assessment and the understanding of the possible corrosion processes within a concrete structure have to be concluded from many different investigations, to be successful. Starting with the current density – potential plot of iron, the base and background of all electrochemical corrosion measurement methods will be explained briefly.

The following corrosion measurement methods, their general properties, merits and limitations will be presented:

– rest potential measurement: easy to do, but sophisticated to interpret – some common sources of misunderstanding
– surface resistivity measurement: chloride and carbonation cause problems when having concentrated in reinforcement vicinity, but they migrate from the concrete surface,
– reinforcement detection: different corrosion activity due to potential measurements can be better understood when correlating this to the concrete cover within an investigated survey
– galvanostatic pulse measurement: an enhanced electrochemical method to know if the reinforcement is active or not
– polarization measurement: qualified information about the corrosion current is possible, but needs careful interpretation
– additional: chloride and concrete humidity analysis from drill samples: how these data help to understand the NDT measurements.

Figure 1. Corrosion case or mainly visual effect? A corrosion survey helps to understand and to decide correctly.

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Approaches for non-destructive corrosion surveys of bridge decks from the soffit and under traffic: A case study

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ABSTRACT

As part of a larger test program, corrosion problems on both reinforcement and post-tensioned steel were evaluated on the superstructure of a bridge with a non-destructive corrosion survey. A comparative measurement of rest potentials and surface resistivity on the deck surface and on the soffit of the 80 cm thick bridge deck was used to establish how reliable corrosion measurements can be done from the soffit and under traffic for identifying corrosion problems on the deck surface side. Following data were measured:

– rest potentials and surface resistivity on upper side and soffit in a grid system of 60×60 cm
– concrete cover on the top side
– chloride and concrete humidity profiles in sampling depths of 20/40/60/80 mm in locations that were suspect of corrosion due to the results of the non-destructive measurements
– concrete damage, rebound values and other corrosion relevant data

Figure 1. View on the 80 cm thick superstructure.

In summary, following conclusions could be made from the corrosion survey results:

– It has been shown, once again, that non-destructive, full scale corrosion surveys are possible without traffic disruptions. According to the measurement results locations of possible damage can be inspected precisely for safe, object related data evaluation.
– Even at thick cross sections rest potentials measured at the soffit are similar to the respective values measured at the deck surface – only shifted by an offset value. Hence it is possible to assess corrosion activity on the top side also by measurements on the soffit. The decision whether possible damaged areas are exhibiting real corrosion problems must be made by supporting investigations. However, area with no corrosion activity can be clearly identified.
– There was no chloride induced corrosion activity on the reinforcement and the pre-stressing steel
– The extent and appropriate timing of repairs can be determined exactly at the stage of design and planning and can be concentrated on necessary areas. This raises the cost efficiency and durability of repairs to a significantly improved level.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Life-cycle design of concrete structures under consideration of advancing reinforcement corrosion

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ABSTRACT

Structures must be reliable throughout their whole life-cycle. Due to their exposure to varying environmental influences, their reliability decreases with time. Hence, life-cycle design must consider durability aspects.

One major durability challenge of reinforced concrete structures is the corrosion of the reinforcement. Reinforcing steel is generally protected against corrosion by a passive layer. However, its depassivation is possible due to carbonation or chloride ingress. If the reinforcing steel is depassivated, corrosion can occur. The basic consequence of reinforcement corrosion is the loss of cross section which directly affects the structure's bearing capacity.

At the time being, in Germany, the durability of concrete structures in regard to reinforcement corrosion is generally verified by complying with descriptive rules which are based on operating experience and expert knowledge according to DIN 1045-1 (2008). Moreover, in some major projects the durability design is performed with an additional limit state 'depassivation of the reinforcing steel' according to DAfStb (2008). For both concepts the reliability of the structure's bearing capacity at the end of its lifespan is unknown.

In this paper an approach to validate the durability of reinforced concrete structures in regard to reinforcement corrosion is shown which is based on the ultimate limit state. The proposed approach for a life-cycle design improves the currently used durability concepts as the reliability of the bearing capacity at the end of the structure's lifespan can be quantified. Moreover, the approach offers an opportunity for material optimization that does not only consider the initiation stage but also the propagation stage of reinforcement corrosion.

The idea of the approach is to implement the time-dependent deterioration process into the commonly used ultimate limit state functions of structural safety design. For the description of the deterioration process, predictive probabilistic models are used so that the result is a time-dependent reliability of the concrete structure. According to the proposed concept the structure's durability is verified if the reliability of the bearing capacity at the end of the structure's lifespan lies above the target reliability. The corresponding limit state function is:

$$G = R(t_l) - E \tag{1}$$

where $R(t_l)$ = bearing capacity at the end of the lifespan; E = load effect.

An example of a life-cycle design according to the described approach is demonstrated in the paper. The exemplary concrete structure is a concrete column of a noise protection wall next to a highway. Wind forces are exerted on the structure and it is exposed to deicing salts from the highway area.

The results of the time-dependent reliability are illustrated in Figure 1.

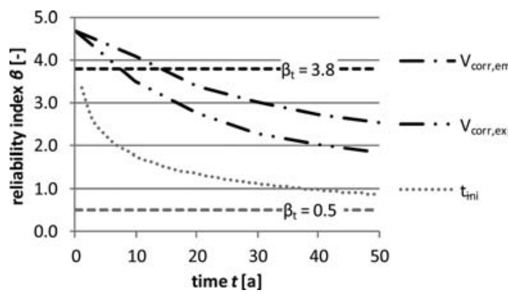

Figure 1. Time-dependent reliability index β.

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Prediction of remaining service life of cracking concrete box girder bridges

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ABSTRACT

The service lifes of bridges refer to the service time of bridges with predetermined functions under the condition of the normal use and regular maintenance. Remaining service life is the difference time of the service life minus the practical serviced life.

The predetermined functions and function failure state must be given when the bridges are designed or assessed. So far, there were some evaluation criteria about the remaining service life prediction of bridges, such as life criterion of reinforced bar preliminary rust (Niu et al. 1995 & Liu et al. 2004), life criterion of steel bar rust expansion and concrete cracking (Tuutti 1982), limited criterion of concrete crack width and steel corrosion volume (Hui 1997), life criterion of load carrying capacity (Zhang 2001) etc. Correspondingly, the evaluation methods of remaining service life may mainly divided into experience-based prediction method, comparison-based prediction method, accelerated test-based prediction method, mathematical model-based prediction method and random statistics-based prediction method, etc. In recent 15 years, many the researchers in the world focus on predicting and evaluating the remaining service life of bridges, such as Frangopol, Dan, Liu (2007) & Van (2004) etc. But, how to reasonable predict remaining service life of concrete box bridges is still to be solved problem.

This paper analyzed the influence of cracks and crack width in the box girder bridges on remaining service life. A subjection function of cracking impact on the structural remaining service life was proposed. The modified method of evaluating the remaining service life of cracking concrete box bridges was given based on Liu (2007) & Van et al. (2004) work. The remaining service life of a practical cracking concrete box girder bridge was predicted by adopting the presented modified method. The results show that the box concrete cracking will lead to decreasing the structural service life by about 30 years. The investigated concrete box bridges with cracking require urgently the crack growth to be controlled and maintained.

ACKNOWLEDGEMENTS

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Life-cycle and reliability assessment of anchorage systems in concrete, masonry and steel construction

Organizers: R. Mihala & J. Hofmann

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Aspects of long-term behaviour of power-actuated fastenings made to steel and concrete

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ABSTRACT

Power-actuated fasteners are pins or threaded studs made from high strength carbon or stainless steel. They are driven directly into the base material steel or concrete in a one step operation using a power-actuated fastening tool.

Figure 1 shows two typical examples of fixtures made to steel and concrete: fastening of profiled sheet metal in a roof construction or fastening of hangers for suspended ceilings.

This paper provides a survey of power-actuated fastening technology in the context of life-cycle assessment and long-term behaviour. The following aspects, which are also covered by the criteria for European technical approvals, are discussed and complemented with test data: Durability, sensitivity to brittle fracture, effect of concrete relaxation and carbonation.

Further aspects potentially affecting life-cycle of the connection are reviewed: long-term static and dynamic loading on fasteners as well as the effect of base metal corrosion and stress. These topics concern on the one hand the robustness of the connection of the power-actuated fasteners in the substrate. On the other hand the presence of fasteners affects the fatigue strength (Niessner & Seeger 1999) and the inelastic cyclic behaviour of the base steel. Those topics concern reliability and life-cycle of the base construction.

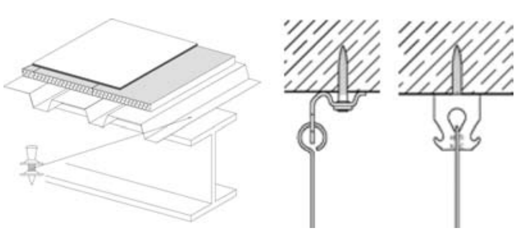

Figure 1. Anchorages with power-actuated fasteners.

Anchorages with power-actuated fasteners represent reliable connections with a high robustness against long-term load effects. Prerequisite is that the fasteners are used within their approved range of application and are driven correctly into the substrate. The durability is ensured through definition of the appropriate environmental conditions for the respective fastener. As carbon steel zinc plated power-actuated fasteners are made from high strength material, their use is limited to dry indoor conditions. Additionally the required fastener ductility needs to be continuously verified within the factory production control.

In case the approved application range is left, for instance when using fasteners for shallow embedments or when applying new corrosion protection systems promising fastener use in moist atmospheres, fundamental research is needed in order to reliably justify the corresponding uses. As no standardized procedure is available, this life-cycle design has to consider all aspects of long-term behaviour relevant for the specific anchorage and application under consideration.

Current research on concrete base material is dealing with the use of fasteners with shallow embedments and the effect of concrete carbonation on pullout resistance (Stipetić 2011). In case of steel base material research is on-going on the use of power-actuated fastener in protected zones of steel frames, which are designed to react as plastic hinges in the seismic event.

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Time-to-failure behavior of epoxy based bonded anchor systems

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ABSTRACT

Bonded anchors are widely used products in the construction industry for connecting any kind of structural building elements. Because of an extensive approval process (e.g. according to ETAG 001) their resistance against nearly any kind of short-term load effect is well known. In contrast, the knowledge about the long-term behavior is low. Based on the examination of the time-to-failure behavior of bonded anchors under static loads, the objective of a joint research project at the IWB, University of Stuttgart and the University of Florida, was to evaluate the existing behavior under long-term loading.

For this purpose, a test method was developed which objective was to predict the available lifetime of bonded anchoring systems for any load level by extrapolating time-to-failure values that were obtained from creep-to-failure tests at high load levels (see Figure 1). Figure 2 shows a hypothetical result of this time-to-failure method using a power regression as time-to-failure function. Similar extrapolation approaches are used in the material testing e.g. for the evaluation of the lifetime of plastic pipes under pressure acc. to EN ISO 9080.

However in contrast to this, in the testing of bonded anchors not only the material (mortar) itself is tested, but the whole load bearing mechanism of the system (anchor, mortar and concrete). Because of the complexity of this mechanical system, it is supposed that the time-to-failure behavior of a bonded anchoring system is not consistent over the entire load range. In fact, the results of the examination of two, epoxy based anchoring systems confirm this assumption. They reveal an unexpected time-to-failure behavior that can't be clearly predicted using the current time-to-failure method.

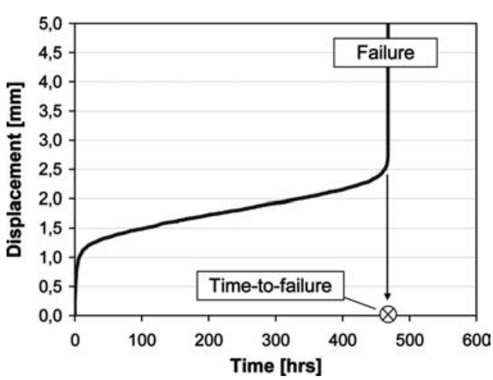

Figure 1. Creep-to-failure curve of an epoxy based bonded anchor system under a static load.

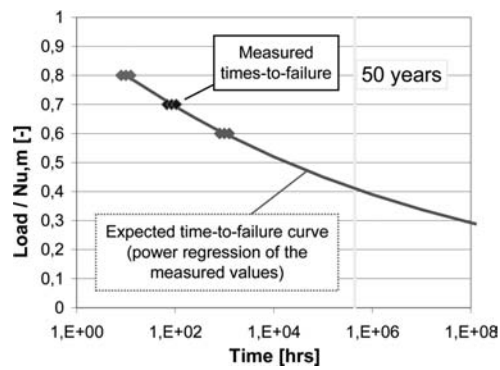

Figure 2. Hypothetical time-to-failure curve with an extrapolation based on a power regression analysis of measured times-to-failure at high load levels.

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Sustained load performance of adhesive anchor systems in concrete

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ABSTRACT

Adhesive (polymer) bonded anchors have been used in many countries for many years. These types of concrete anchor installations have been shown to be very sensitive to sustained loads and will very likely fail during the design life of the structure that they are installed in if their sensitivity to sustained loads is not taken into consideration in the initial design of the structure. The sustained load effect has been included in product evaluation procedures in both the USA and Europe but questions still remain.

Significant research work has been undertaken in both the United States at the University of Florida and in Germany at the University of Stuttgart in a joint effort to try to understand sustained load effects when adhesive anchors are installed under various conditions (e.g., damp concrete, improperly cleaned holes, installation direction, etc.).

Adhesive anchors subjected to sustained loads have longer service lives under lower stress levels. If stress level is plotted against time-to-failure, this relationship can be modeled by a power or logarithmic relationship.

This stress versus time-to-failure approach has been shown to be a useful tool for both designers and researchers in the use and evaluation of adhesive anchors systems under sustained load. Designers can use a stress versus time-to-failure relationship to determine a safe stress level for an adhesive anchor system given a structure lifetime. Researchers can use the stress versus time-to-failure relationship to evaluate the effect of various parameters on the long-term performance of adhesive anchors by comparing the reduction in stress level at given time-to-failures against projected reduction in stress level assuming no long-term effects.

This paper presents the results of this international research effort to determine the sustained load reliability of adhesive anchor systems under various types of installation and in-service conditions. Only preliminary data was available at time of publication and final conclusions on which parameters do affect the long-term performance of adhesive anchors must be held until the completion of the project.

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Service life design for bonded anchors – a rheological approach

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ABSTRACT

Bonded anchors exhibit nonlinear viscoelastic material behavior. While the short-term performance of bonded anchors has been intensively investigated, insufficient attention has been paid to their creep behavior. An exact prediction of nonlinear creep up to failure and the time-to-failure of bonded anchors are imperative for service life estimation. In this contribution, a rheological model is proposed for this purpose.

The new model is based on the Burgers model which is well-established for linear viscoelastic materials. To enable a precise prediction of the short-term and the long-term performance, the model was modified to include nonlinear effects. The level and duration of the applied load as well as material damage processes were taken into account. Three tests are necessary to determine the model input parameters for a particular bonded anchor. 1) a tensile test to determine the initial stiffness which depends nonlinearly on load, 2) a creep test over 1000 h to determine the nonlinear time-dependent evolution of viscosity and 3) a test to determine stiffness loss of the bonded anchor due to structural changes and degradation processes in the polymer based adhesive and at the interfaces anchor/concrete and adhesive/concrete.

This contribution focuses on the evaluation of stiffness degradation which is indispensable for service life design of bonded anchors because it enables the prediction of creep-failure and the time-to-failure with the modified Burgers model.

A special degradation test was developed to determine loss of stiffness in dependence of anchor displacement. The degradation test is similar to a test performed by, for example, Bolle (1999) who investigated the stiffness degradation of reinforced concrete. It consists of multi-repeated cycles of load application, constant load and load removal at different load levels until failure to determine a damage index. To evaluate the damage index and thus the stiffness degradation, a technique developed by Spooner & Dougill (1975) was adopted. The technique is based on the assumption that the deformation energy dissipated during loading and unloading corresponds to material degradation which results in a change in the initial Young's modulus.

For the bonded anchors investigated, the stiffness degradation can be described by a power function of anchor displacement.

Creep tests taking up to 3.5 years without creep-failure were carried out at different loads to test the ability of the model to predict the nonlinear long-term creep behavior of bonded anchors. Creep tests were also carried out at high sustained loads in order to achieve creep-failure due to ongoing structural changes and degradation thus testing the ability of the model to predict creep-failure. The model enables accurate prediction of the time-dependent displacement for loads occurring in practice as well as the creep behavior at high loads up to failure. The time-to-failure is precisely predicted by the model.

In contrast to current standards which yield pass/fail criteria, the new model predicts the real performance of bonded anchors under sustained load. It is therefore possible to assess the quality of different bonded anchors according to time-to-failure (Cook et al. 2009). Thus the model permits a precise service life design for the bonded anchors. Bonded anchors can thereby be selected with regard to a particular quality for a specified application in practice.

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Gluing to masonry for efficient and sustainable anchoring

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ABSTRACT

Low heat transmission coefficients are required for saving energy in structural engineering. Therefore especially in masonry structures, the base material becomes more and more porous. Masonry stones with thin division bars and low strength are used. In such masonry stones it is difficult to apply loads using anchorage systems.

Newly developed gluing technologies make the appliance of loads in a more efficient and sustainable way possible without drilling holes and damaging single stones or whole constructions. In automobile and aircraft construction gluing is already state of the art. But there is only little experience on gluing technologies with mineral construction materials.

Gluing as a method for applying loads to masonry means that the steel plate is not fixed by bonded anchors that are embedded in with mortar filled bore holes or anchor rods. To transfer high loads to masonry normally bonded anchors according to ETAG 029 are used. A bonded anchor is a system consisting of a threaded rod, a sleeve and a bonding agent. The system is installed by drilling a hole in the base material and injecting the bond agent into the hole. Afterwards the threaded rod is placed in the mortar filled borehole. After a hardening time of 6 to 24 hours the anchor can be loaded and the steel plate can be fixed.

In the used fixing method the epoxy mortar is applied directly to the surface of the masonry stone, so that the load plate is fixed by adhesion. To show the dependence of load bearing capacity from material properties, different kinds of masonry stones are examined considering physical and chemical and especially environmental aspects.

Not only compression strength and tension strength are decisive for load bearing capacity but also the geometry of the stones (full blocks or perforated blocks, geometry of division bars in perforated blocks) (Meyer 2005). Subsequently, different kinds of stones and environmental influences are tested. All tests are carried out using epoxy mortar as preliminary tests.

The different aspects and problems coming up using gluing as an efficient and sustainable anchoring method especially due to environmental influences, durability and long term behavior are shown up.

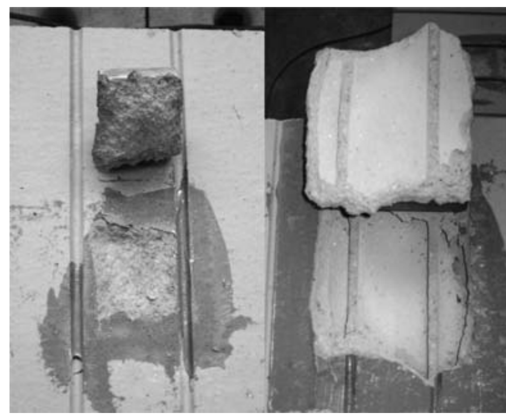

Figure 1. Stone break-out on tests with vertically perforated bricks.

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Implementation of the basic requirement "sustainability" on construction works according to the construction products regulation in the field of anchor technology

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ABSTRACT

On 1 July 2013 the new European Construction Products Regulation will replace the current European Construction Products Directive completely. In Annex 1 of the Regulation a new "basic requirement for construction works" (essential requirement) will be introduced – "Sustainable use of natural resources". For implementation of this basic requirement, there are not yet any available harmonised Application Documents; one possibility would be the use of Environmental Products Declarations (EPDs). The Author gives an overview on the current situation and expectations of assessment with respect to sustainability in the field of Anchorages and Fastenings.

In this paper the attempt is made to compare the activities of the section Anchorages and fastenings of DIBt with the requirement of sustainability.

Currently there are two German certification systems for sustainable buildings. One is required for governmental buildings by the Ministry of Transport, Building and Urban Development (more Information: www.nachhaltigesbauen.de). Another certification system is established by the DGNB ("Deutsche Gesellschaft für Nachhaltiges Bauen", www.dgnb.de) for all other types of buildings.

The CEN/TC 350 deals with "Sustainability of construction works – Integrated assessment of building". The EN 15804 is a European EPD Standard for construction products for harmonised Environmental Product Declarations (EPD). Finland, France, UK, Italy, The Netherlands, Norway, Portugal, Sweden, Spain and Germany cooperate in the ECO (Environmental Construction products Organisation). The basis for the assessment of the products is EN 15804. Currently in Germany those EPDs are granted by the "Institut für Bauen und Umwelt" (IBU).

Currently DIBt is in the beginning of assessment of sustainability of construction products with the aim to integrate product declaration for environmental in addition to the other product characteristic in the national approvals.

The activities in the Working Group are based on the mandate of the European Commission. The Essential Requirements which are entitled in the mandate for ETAGs for anchors are:

- ER 1 Mechanical resistance and stability
- ER 2 Safety in case of fire
- ER 3 Hygiene, health and the environment
- ER 4 Safety in use

The current Guidelines for European Technical Approvals cover three types of anchors with different material and working principles:

- Metal (steel) anchors for use in concrete (ETAG 001)
- Plastic anchors for use in concrete and masonry (mostly a plastic sleeve with a nail or screw made of steel or plastic material) (ETAG 014, ETAG 020)
- Chemical anchors for use in concrete or masonry (ETAG 029).

The current activities of DIBt in the field of anchorages are reflected under the aspect of the new basic requirement Sustainable use of natural resources.

REFERENCE

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Durability of temporary anchors in rock

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ABSTRACT

In tunnel engineering specific anchors are used for local stabilization of the rock immediately after excavation. Since the structural inner shell usually is realized within a short period after excavation, the anchors are designed for a very limited service life. The durability of the anchors depends strongly on the local conditions of the surrounding rock i.e. presence of water, chemical composition of the water and geological/geotechnical uncertainties related with the rock mass. The 1.8 km long access tunnel "Mules" on the Italian side of the Brenner Base Tunnel has been excavated by drill and blast and is supported by shotcrete, reinforcement mesh and friction expansion anchors. The design life of the system has been set to 10 years.

In order to assure the durability of the anchors in a potentially corrosive medium for a design life of more than 2 years EN 1537 requires a corrosion protection of anchorages taking into account the aggressiveness of the ground environment. Therefore a protective bitumen based coating has been applied. Since the coating may be damaged during transport and/or installation in the tunnel and the internal part of the anchor tube is not protected, corrosive effects during design life can't be excluded. Therefore the time dependent safety and reliability of the anchorage is calculated by Monte Carlo simulation taking into account various corrosion scenarios.

In Fig. 1 the time dependent number of failing anchors per year and kilometer n_f is plotted for the various assumed coefficient of variation of the rock stress in the case of corrosion on both sides of the anchor.

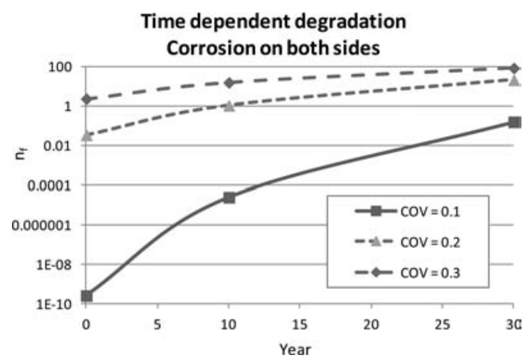

Figure 1. Influence of COV of rock stress.

The scatter of the rock stress governs the safety of the system decisively. A moderate COV of 0.20 may be assumed to be realistic for the investigated rock mass. An extension to 30 years yields a much higher probability of failure of more than one order of magnitude. This may be of interest if the design life of the temporary tunnel shell should be increased for specific reasons.

A monitoring program is developed in order to ensure the safety and reliability of the temporary anchors in the time interval between excavation of the tunnel and realization of the inner shell. The monitoring activities consist in periodic visual inspection of the tunnel shell (once per year), in-situ pull-out tests with a specified number of anchors and periodic convergence measures of the tunnel shell at fixed distances. The distance is reduced in sections with enhanced geologic/geotechnical risk.

In the case that critical parameters (e.g. convergence, cracks in shotcrete) are exceeded, rehabilitation measures are performed immediately. Such measures may consist in the application of additional anchors and/or additional shotcrete.

REFERENCE

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Durability proof of steel metal screws in accordance with European technical approvals

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ABSTRACT

Since the first European Technical Approvals for steel metal screws have been issued in summer 2010, there is an uncertainty about the use of the screws in external environment.

The ETAs are valid in all Countries of the European Union, but common practice is different in different Countries. In Germany, for example, it's mandatory that fasteners exposed to weather or similar conditions have to be made of stainless steel material. In contrast to the German approach the regulations in France require Kesternich tests for the fasteners according to national standards (DTU). Stainless steel material for the fasteners is not mandatory. With all these different national standards, regulations and common practice harmonized requirements have to be established in the industry.

Within the ETA approvals it is stated, that: *"screws exposed to external weather or similar conditions are made of stainless steel or are protected against corrosion"*.

More details about required tests are given in CUAP 06.02/07 (Common Understanding of Assessment Procedure) where the ETA's are based on: *"when the paint or coating combination is not presented in the standard EN ISO 12944-5, the testing according to EN ISO 12944-6 shall take place with adequate results."*

EN ISO 12944 gives a clear advice about testing and evaluating coatings on steel plates. Unfortunately this procedure is not directly applicable for small Screw fasteners. Screws have no plain surfaces and thick coating can't be applied, because the functionality of the fastener would be detrimental effected by the coating.

Another question is what are the real environmental conditions within a typical wall or roof construction? The structural physical conditions can be determined with the help of finite element calculation tools to allocate the actual conditions to corresponding corrosivity categories according to EN ISO 12944-2 (C2 to C5-I).

With that the general procedure to determine the corrosion resistance of fasteners is defined. Still the test details for small screw fasteners and the assessment of these tests have to be defined. Corrosion and construction experts, approval body's and screw manufacturer have to work closely together to adjust the provisions of EN ISO 12944 to be applicable for small screw fasteners. If these provisions and sufficient proprietary coatings are available, corresponding screw fasteners can be implemented into an ETA where all required technical data are given and Engineers can rely on.

Actual situation at this point is that no ETA approved proprietary coated screw fastener is available on the European market. As long as no documented evidence of conformity is given for coated screws in outdoor or similar conditions, the requirements are keeping unchanged to the current situation.

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Serviceability and ultimate limit states for anchorages in concrete – effects of assembly tolerances

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ABSTRACT

In bolted assemblies between steel and concrete sufficient installation tolerances have to be considered in order to facilitate the connection of components at the building site. However, existence of tolerances may also lead to adverse structural, functional and aesthetic effects. Apart from that, the definition of ultimate and serviceability limit states for the two different building disciplines (i.e. steel and concrete construction) may be based on different criteria (e.g. cracking or deformation limit values). An approach to couple the varying criteria is attempted focusing on the structural performance of fastenings of steel elements on concrete structures particularly prone to concrete related damage modes as described in (Mallée & Pusill-Wachtsmuth 2007) and (Spyridis et al. 2010). The probabilistic aspects in the assessment of structural impacts due to assembly tolerances are addressed and a probability-based module for the estimation of ultimate and serviceability limit states for fastenings to concrete is proposed. For anchor groups with hole clearance loaded in shear toward a free edge of the concrete member, definition of the various load bearing limit states comes to particular interest. The present paper attempts to provide feedback on this topic based on literature and recent studies in the Institute of Structural Engineering, BOKU Vienna.

Scope of the paper lies in the behavior of shear loaded anchorages located close to a free edge of normal-strength concrete structural components, while the anchoring system is comprised of two anchors or rows of anchors arranged perpendicular to the edge. Influence from either a second edge (i.e. corner situation) or small member thickness are not considered. The investigated anchorages are realized through grouping of post-installed anchors (or fasteners) by means of a steel base plate, which often requires that for the connection of the individual anchors, adequate assembly tolerances should be provided in the fixture. The load in the investigated cases is applied with direction towards the free edge, while the area of the investigations is restricted to concrete edge failure modes. Particular interest of this work lies in the influence of assembly tolerances on the distribution of the shear load to the individual anchors of the group and thus on the group's actual bearing history. This encompasses different loading levels reached throughout the life-cycle of an anchorage and their corresponding limit states.

Varying limit states should always be determined in any type of structure in order to define the structural response for different levels of actions and for the classification of various possible unfavorable occurrences. Operability and fitness of the structure for its intended use can therefore be better controlled. Crop of such approaches is the educated decisions with respect to the economy and efficiency of the structure's construction and maintenance in various life-cycle stages. This framework should not exclude the fastenings to concrete through post-installed anchors.

Objectives of the paper are (a) to brief the reader on the structural behavior of fastening systems loaded in shear towards the edge, (b) to discuss possible approaches on the definition of limit states for such anchorages, (c) to present recent reliability studies relevant to limit state definition, and (d) to provide feedback for potential improvements in codified design. These are discussed and applicability of proposed modes for the Ultimate and Serviceability Limit States (ULS and SLS respectively) are defined.

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The functionality of fasteners with small embedment depth in carbonated concrete

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ABSTRACT

For concrete structures post installed fasteners are gaining popularity for transferring high loads into structural material. Within the last decades the design of fasteners was strongly improved with the focus to increase the load that can be transferred by such fasteners. However, these fasteners are more and more sophisticated and sensitive to changes during the life-cycle of minimum of fifty years.

In most cases power actuated fasteners as well as plugs for suspended ceilings are installed within a short time after concreting the base structure. With time the concrete properties will change especially near the concrete surface e.g. by carbonating. Changing material properties of the concrete cover normally do not influence the behavior of fastener because the embedment depth is large enough. For anchor nails and power actuated fasteners the embedment depth can be smaller than 30 mm.

Currently no tests and analyses are available to estimate the influence of changing properties on the failure load and displacement behavior of such fasteners. The aim of the tests presented in this paper is to show the influence of the changing concrete cover properties due to carbonation on the failure load and the displacement of fasteners with small embedment depths. The following paper shows in detail how the properties of the concrete surface are influenced with time and which effect on the safety of fasteners must be considered to ensure a life time of fifty years.

The pH value of non-carbonated concrete is approximately 12.6. During carbonation concrete is neutralized from the outside to inside, causing the pH to fall below 9.

Accelerated carbonation was carried out for three months to obtain carbonated concrete. The test chamber with the concrete specimens to be carbonated was filled with 3% of CO_2. The constant humidity of 65% was regulated by a saturated solution of ammonium nitrate. The test chamber was not air-conditioned, but placed in temperature-controlled room, with a temperature between 20–22°C.

The carbonation depth was tested by 1% phenolphthalein in ethanol solution.

The carbonation depth in carbonated concrete was always larger than 3 cm and therefore powder actuated fasteners with an effective embedment lower than 30 mm could be tested in non-cracked concrete.

Each anchor was tested to four different conditions. Altogether 320 tests with four different nails and four different test procedures were performed.

The compressive strength of concrete at the beginning and the end of the carbonation was tested on concrete cubes (150 mm × 150 mm).

To determine the influence of carbonation to concrete cover, tensile strength tests were made before and after carbonation.

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Life-cycle engineering tools for risk-based decision under uncertainty

Organizers: C.F. Cremona & A.D. Orcesi

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Vulnerability analysis of a park of small dams to natural hazards through a GIS

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ABSTRACT

The important development of small dams in irrigation in Algeria is linked to the need to store water to protect against seasonal or interannual variability of floods. In the eighties, an extensive program of dams (numbering 83) was made by the Direction de l'Hydraulique of Tizi Ouzou (Algeria). These small dams are not managed by a management agency and for the most part, their use is left to farmers. They were exposed to the phenomenon of aging by the degradation of embankment dams and ancillary structures. Today, thirty years later it is time to review their condition, because the mastery of the aging phenomenon is a major economic challenge.

The small dams of the department have known a number of incidents and disorder. Mechanisms degradation that can affect the embankment and ancillary structures are revealed by surface movement or crash land, erosion, sliding, scour … etc. Visual inspection is the best way to identify such indices and is essential to prevent the breakdown of these structures and consider comfortably fitted. We will give through a particular exemple, briefly the mechanisms of degradation in our region of study.

We develop through our paper, a simple and practical method that allows to a professional of civil engineering to quickly assess the presumption of vulnerability to natural hazards (snow, wind, earthquake … etc) of a small dam, by guiding them in their diagnosis.

The park of small dams that is the subject of our study is in the department of Tizi Ouzou (Algeria), classified medium seismicity zone (zone IIa) according to the Algerian seismic code. According to the Algerian Snow and wind code 99, the area is classified as snow zone A and wind zone I. All small dams expertised are in local materials (compacted embankment). The small dams are located as points objects geo-referenced using coordinates (X, Y), measured by GPS, using a UTM (Universal Transverse Mercator) (WGS84).

In order to collect geographic data and attribute of our study area and to organize them so as to extract information for making management decisions, we use the Geographic Information System (GIS). The extracted data allow us to make thematic analysis and classify the expertised small dams according to their vulnerability index. The information is stored in a workbook "Excel©" and the small dams are located on the GIS software "MapInfo Professional©" on the topographic map, in order to geocod the small dams. The GIS will allow us to store, view, edit and analyze all the details of the small dam valuated. From the available informations, we include the calculation of the vulnerability index able to assess their condition. From all collected informations, multi-criterias thematic analysis were made.

The final product made is a considerable contribution and efficient decision tool for expert engineers and managers that are in charge of a wide small dams park.

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A probabilistic approach for life-cycle environmental analysis of motorway bridges

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ABSTRACT

The construction, operation and ultimately the demolition of the bridge have a significant impact on the environment. A Life Environmental Cycle Analysis (LCEA) aims to quantify all environmental burdens from all stages over the bridge life-cycle, from raw material production to the end-of-life stage.

However, in LCEA uncertainties are unavoidable. In order to enhance the quality of information provided by LCEA, uncertainties should be properly addressed in the analysis. Therefore, it is the purpose of this paper to present a framework for LCEA addressing different types of uncertainties in different stages of the analysis, by means of a probabilistic approach. The probabilistic approach is applied to a case study: a bridge crossing a motorway.

Three main types of uncertainties are taken into account, which are indicated in Table 1: (i) the uncertainty in the input data; (ii) the uncertainty in the choices; and (iii) the uncertainty in the mathematical models.

Moreover, the probabilistic approach presented in this paper aims to assess the simultaneously effect of the different uncertainties listed in Table 1 and the propagation of uncertainty throughout the life-cycle analysis. The techniques for quantifying uncertainty include Monte Carlo numerical techniques of simple random sampling and bootstrap analysis.

The evaluation of the environmental criteria follows the guidance from ISO standards ISO 14040:2006 and 14044:2006. The life-cycle environmental results of the simulation for the impact categories of Abiotic depletion, Acidification, Eutrophication and Global warming are represented in Figure 1 by the respective 90% confidence interval of the mean value.

Similar results were obtained for the remaining impact categories.

In a deterministic life-cycle analysis single values are obtained for each impact category. On the other hand, in a probabilistic life-cycle analysis, a probabilistic distribution is obtained for each indicator. Both analyses provided important information, however, the range of values provided by the probabilistic approach enable more reliable conclusions. This aspect is particularly important in case the purpose of the LCEA is to aid in a decision making process.

Table 1. Types of uncertainties in life-cycle analysis.

Stage of analysis	Types of uncertainties	Tool to deal with uncertainties
Inventory stage	Uncertainty in data; Uncertainty in the allocation	Probabilistic analysis (MCS[1] & LHS[2]), Scenario analysis
Impact Assessment	Parameter uncertainty in models; Space variability; Temporal var.	Scenario analysis, Probabilistic analysis (MCS[1], LHS[2] & Bootstrap)
Normalisation	Uncertainty in data; Uncertainty in choices	Probabilistic Analysis (MCS[1] & LHS[2])
Weighting	Uncertainty about preferences	Probabilistic Analysis (MCS[1] & LHS[2])

[1]Monte Carlo Simulation; [2]Latin-Hypercube Simulation

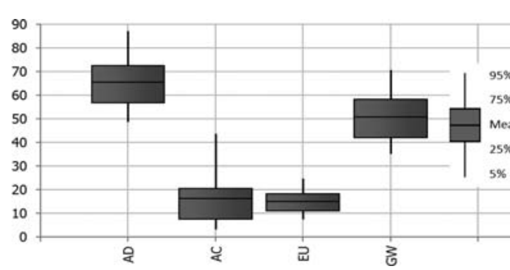

Figure 1. Results of the simulations for Abiotic depletion, Acidification, Eutrophication and Global warming.

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Life-cycle design of concrete bridges

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ABSTRACT

In recent years, rapid development can be observed in two areas of life-cycle design of civil engineering structures. One direction is the *service life design*, which focuses mainly on durability issues and related design approaches to extend the design working life of structures up to 100 years or even longer. The other direction is the *Life-Cycle Cost Analysis* (LCCA) that analyses the cost effectiveness of structures and finds design strategies, by which the total life-cycle cost of a structure can be minimized during its service life or a defined service period.

Majority of structures, to which the application of life-cycle design is desirable, are parts of the traffic infrastructure. These structures are required to work on acceptable technical level during a relatively long service life and need relatively high costs during this period.

This paper focuses on the applicability of High Performance Concrete (HPC) to load carrying superstructure of concrete highway bridges. The research suggests that the maintenance cost of a bridge depends first of all on the durability of the pavement system. Consequently, the life-cycle cost-effectiveness of concrete bridges is based mainly on the proper combination of the load carrying superstructure and the pavement system.

Structural re-design with the consideration of three alternative pavement systems and following partial life-cycle cost analysis have been carried out on three existing, "prototype" concrete bridge decks. The considered pavements were:

I. separated 150 mm thick asphalt-based pavement system built on waterproofed deck slab,
II. separated 295 mm concrete-based pavement system built on waterproofed deck slab,
III. HPC pavement integrated with deck slab without waterproofing (no separated pavement).

The main goals of the paper were

- to analyze the applicability of the integrated HPC deck-pavement system (HPC system) as bridge superstructure from structural point of view in comparison with traditional decks made of normal performance concrete and combined with structurally separated asphalt- or concrete-based pavement systems (NPC systems);
- to perform an economic comparison between integrated and separated (asphalt-based and concrete-based) deck-pavement systems by estimating their life-cycle costs.

In comparing HPC systems with NPC systems the following conclusions were made:

- The application of HPC system helps in reducing the dead load of the superstructure but makes necessary the consideration of more sever durability and, consequently, structural requirements in design in comparison with NPC systems.
- The improved durability and structural requirements can be fulfilled by special concrete technological measures and structural solutions that results in definitely higher construction cost for the HPC system compared to the NPC systems.
- However, due to the significantly less maintenance cost for the HPC system, the saving, which can be achieved in the life-cycle cost during 100 years of service life, is on average 25–35% compared to NPC decks with asphalt-based pavement and 10–20% compared to NPC decks with concrete-based pavement.
- The pay-out period of the HPC system has obtained as about 33 years against NPC decks with concrete-based pavement and about 28 years against NPC decks with asphalt-based pavement.

The applied "partial" life-cycle cost analysis demonstrated the long-term economic advantage of the investment strategy for concrete bridge decks, which focuses on improved durability design in order to significantly reduce the necessary maintenance effort despite the higher construction cost of the structure.

The results also underline the exploitable structural benefits of the application of the integrated HPC deck-pavement systems.

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Relative performance concepts: A new approach in life-cycle management of concrete structures

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ABSTRACT

Service life predictions can be seen as a key element in life-cycle management of structures. For this purpose, several prognosis models have been developed in recent years including deterministic as well as stochastic models in order to describe time-dependent degradation processes of concrete offering the possibility of service life predictions of concrete structures. However, the application of these predictive service life models in practice is subject to significant uncertainties (Lohaus & Gerlach 2011). These uncertainties arise, among other reasons, from an insufficient degree of knowledge, lack of understanding and a limited number of observations. As a consequence, predictions can only be relatively accurate for a relatively short period of time and should not exceed 10 years (Vu & Stewart 2000). One efficient way to increase the confidence of prediction models and thereby the reliability of service life estimations is to incorporate additional information gained through inspections and monitoring during different stages of the structure's service life.

Following these objectives, a newly developed methodological approach is the concept of relative performance. This practice-orientated approach consists of two major elements (i) the data acquisition and (ii) the data interpretation and integration (see Figure 1).

On the one hand, the data acquisition is carried out by placing concrete dummies at strategic positions at the structure. Hence, concrete is used as sensor allowing to 'readout' the value of degradation directly from the dummy and additional information on the degradation process can be gained. On the other hand, additional information is gained through comparative laboratory tests. In order to interpret and integrate the additional data, a Bayesian framework is chosen. This enables to consider statistical and modeling uncertainty besides the underlying physical uncertainty.

Considering the test results gathered through comparative laboratory test, a relationship between the environmental load conditions at the structure and the laboratory conditions can be derived. Consequently a measure of the load intensity can be determined. This relative measures allow a better interpretation of prevailing field conditions and, hence, a better transferability of laboratory conditions. As a consequence, the time-dependent development of the degradation can be estimated with more precision and uncertainties in the environmental conditions can be reduced.

Furthermore, the relative performance concept offers the possibility to establish an early warning system. This can be achieved by placing comparable but less resistant concrete dummies at a structure. These dummies can be used as an indicator for performance development of the considered structure.

This proactive monitoring leads to an additional safety margin and lead time which can be used for decision-making purposes and maintenance planning. Additionally, the information gained with these dummies can be incorporated in the updating process during service life.

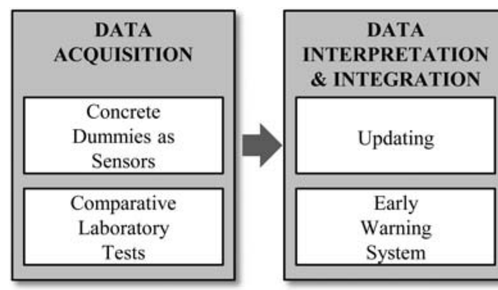

Figure 1. Basic elements of the relative performance concept.

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A stochastic aging model for life-cycle assessment

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ABSTRACT

This paper presents a simple approach already used in practice. It is mainly a simulation method where a complete asset of bridges, tunnels or whatever can be treated and investigated by various strategies. The objects are put into cohorts regarding to their actual condition rating. The process simulates time steps where some of the objects may migrate from one cohort to the next. At the cohorts represented bad condition objects are taken out and maintained. They are transferred to a cohort representing good rating like renewal theory assumes. The whole approach is based on stochastic modelling of appropriate hazard functions. It works at element level and considers a set of measures reflecting maintenance work in practice. Improvements like consideration of short term and long term measure will be presented by a multi objective decision tree approach.

The central idea behind the employed CSM (Cohort Survival Method) is to describe the deterioration only by condition rating of objects or their elements and not focusing on to their age. The maintenance measures are simplified and related to those cohorts where the most deteriorated objects or their elements are hosted.

The simulation model based on the cohort survival method has been proven to be applicable for various types of objects like bridge, tunnel, floodgate and road pavement. It considers actual inspection results for elements of a structure and combines them to an indicator value by means of best scalarization approach. The model treats every object and uses a maximum likelyhood approach fitting paramters for the stochastic model of the degradation process. The process itself is described by migration of objects from one cohort to the next as it would happen in nature when inspection will take place and all the observed shortcomings will lead to a worse condition rating as before. A major step of improvement has been done, by introducing a multi objective decision tree. This model is used to select more realistic maintenance strategies of objects during simulation which have reached an upper cohort class and have to be renewed. An ongoing research project will implement the network view into the model. In reality the decisions are taken by considering parts of a road network in order to ensure availability of it. This approach will take into account spatial dependencies between objects and may necessitate higher budgets. In the actual solution the target budget will be optimally used, which is somehow an ideal situation not observed in practice.

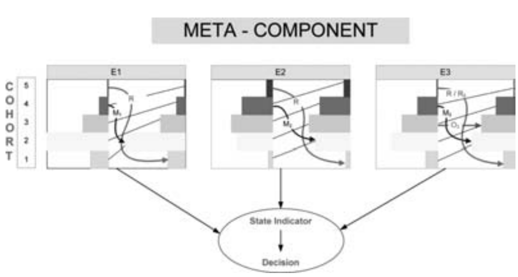

Figure 1. Simulation model.

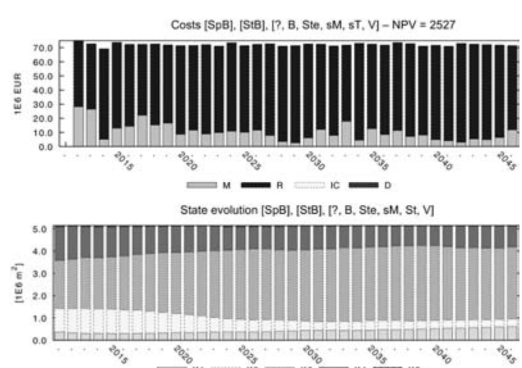

Figure 2. Cost and development for “Limited Budget”.

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Stochastic models for degrading infrastructure systems

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ABSTRACT

Modeling deterioration is central for describing the time-dependence of the basic infrastructure systems' properties. It is also important for reliability assessments, maintenance and cost-based optimization problems. A comprehensive deterioration model should take into consideration the combination of both progressive and sudden damaging events (i.e., shocks), which are random in nature. This paper presents an overview of structural deterioration models (graceful and shock-based) and provides a discussion about the associated uncertainties.

Progressive (graceful) degradation results in capacity/resistance continuously being removed from the structure's initial condition state. Most progressive degradation models available in the literature assume that the *form* of the degradation process is known, but the parameters are uncertain. The solution to this problem conveys to a parameter estimation problem. Progressive degradation can be handled also in terms of a degradation rate i.e., $\delta_p(t), t \geq 0$ that may or may not change (randomly) over time. However, frequently, it is not possible to evaluate its time-dependent nature (Kiessler et al. (2002)). A third group of models focus on evaluating progressive deterioration discretely. In these models, the loss of remaining life (i.e., deterioration) occurs only at discrete times and, therefore, they are limiting solutions of shock models. Two shock-based approaches to graceful deterioration were presented. The first one is based on the *gamma* process, where shocks are equally spaced but the shock size distributions vary with time. The second assumes random *iid* shocks spaced according to a predefined deterioration trend.

On the other hand, shocks can be defined as events that cause a significant change on a system's physical property in a small time interval. Then, Shock-based degradation occurs when a fixed amount of capacity/resistance is removed from the structure at discrete points in time (Sánchez-Silva et al. (2011)). Shock models were classified in three groups. the first group comprises shock models constructed based on the *Poisson process* (homogeneous and non-homogeneous), or as an extension of these models; e.g., gamma process. The second group includes models based on *Markov processes* (Barlow & Proschan 1965). Finally, the third group includes models that attempt a full stochastic description of the process. In these models, time intervals are not necessarily exponential and shock distributions may be also arbitrary (see Sánchez-Silva et al. (2011)).

Finally, the paper presents a model of the combined action of both progressive and shock-based degradation; and study the case of structures subjected to successive reconstruction. Separate and combined deterioration models were illustrated with practical examples. The results clearly show the impact of deterioration on the failure probability and suggest that most existing models underestimate it.

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A comparative life-cycle cost analysis of steel-concrete composite bridges

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ABSTRACT

One goal of life-cycle cost analysis is to enable comparative cost assessments over a specified period of time, taking into account all relevant economic factors from initial costs to future operation and maintenance costs until the destruction of the structure. The total Life-Cycle Costs (LCC) include not only construction costs but also other costs such as inspection, maintenance/rehabilitation, end-of-life and user costs. It is taken for granted that service life costs (operation and maintenance) represent a significant portion of the total life-cycle cost of a structure (Flanagan & Norman 1989). This is why the traditional design concepts are now put into question to make a shift to a life-cycle level and optimize life-cycle costs. In this context, one important motivation to use LCCA is to balance the decrease of operation and maintenance costs with a possible increase of initial costs (Kirk & Dell'Isola 1995). Several researchers have developed methodologies for structural design and management systems based on lifetime costs and benefits (Tao et al. 1995, Hearn & Shim 1998). Many researchers and practitioners (Estes et al. 1997, Vorster et al. 1991, Markow et al. 1993, Frangopol et al. 1998, Yanev 1998) have proposed optimal maintenance strategies for critical structural elements. In general, a Life-Cycle Cost Analysis (LCCA) enables to create a more comprehensive view of the various costs that occur in the different phases of the structure service life.

This paper illustrates the application of LCCA to steel-concrete composite bridges. The proposed approach aims at assessing life-cycle economic impacts of the design choice made at the conception stage. The suggested approach takes into account economic considerations during the total life-cycle of structures. The proposed framework finally enables to optimize the choice of the design solution by considering economic constraints.

The aim of this paper is to apply the LCCA on several design solutions of a composite bridge to illustrate the life-cycle optimization approach for bridge design. The main motivation is to show that a solution more expensive at the construction stage can finally be more attractive when considering the overall service life of the structure. Some bridge case studies are considered in this paper to illustrate the comparison between different design solutions.

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DIOGEN: Environmental impact database for life-cycle assessment of civil engineering structures

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ABSTRACT

Life-Cycle assessment (LCA, also known as life-cycle analysis) is an analytical framework (as part of the ISO 14000 environmental management standards: ISO 14040:2006 and 14044:2006) for measuring environmental and social impacts of a product system or technology, including raw material acquisition, production, use, final disposal or recycling and the transportation between these phases. Often, LCA elucidates unseen environmental and social burdens incurred over a product lifetime. Modelling the complete life-cycle of a single product or process is therefore complex and data-intensive.

The recent life-cycle analyses applied on civil engineering structures highlight the importance of environmental impacts of the materials production stage. As environmental criteria are more and more pregnant for awarding public works contracts, it is nowadays crucial to correctly assess the project variants to retain the most efficient solutions.

The input data concerning the environmental impacts of the produced materials must fulfil with two essential criteria: reliability and representativeness.

Reliability requires to identify data sources and to make available the details of the Life-Cycle Inventory (LCI). Data must also be representative of the studied structure and of the production technologies of materials. It is therefore necessary to know the nature of the materials and the standards criteria allowing their use.

Conducted under the auspices of the french association of civil engineering (AFGC), the DIOGEN project aims to build an environmental database dedicated to the production of materials ("from cradle-to-gate"): it identifies and gathers existing data that can be used to assess the environmental impacts of civil engineering structures. DIOGEN project also provides a methodology for qualifying data in order to determine the confidence to place in the values given by producers of materials and databases.

This methodology is based on the requirements defined by ISO 14044 and on the PEDIGREE matrix of the ECOINVENT database. After scoring data based on various criteria, the methodology defines a confidence index that highlights the reliability and the representativeness of the data within its scope (civil engineering in a French context).

From a designer point of view, the confidence index has for objective to assess the ability of the data to be used without constraint or otherwise with reservation in accordance with the materials' influence in the study. It also allows stakeholders to ensure the validity of the elements of the proposed environmental assessment.

This paper presents the principal components of the DIOGEN project, its features and some potential applications.

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Model for negotiation of refinancing gain from public-private partnership

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ABSTRACT

Due to the long contract period, the uncertainty and risk level are very high at the very beginning of the project. However, when the project is successful on completion and operation, the uncertainty and risk level are greatly reduced. The private sector will be more willing to refinance the project on better financial terms and benefit a lot from it. Meanwhile, the public sector will require sharing the refinancing gains. Due to the conflict position and no stable theoretic sharing mechanism, there will be negotiation between the public sector and the private sector.

Based on Rubinstein's (1992) bargaining model and "Take-It-or-Leave-It" principle, finite-stage bargaining game model and infinite-horizon bargaining game model for sharing negotiation between the two sectors have been established. The Nash Equilibriums of these two models also have been found.

Considering the infinite-horizon bargaining game, the common formula of the distribution factor θ when the public sector makes offer first is generated in Eq. (1).

$$\theta = \frac{1-\delta+\delta^2-\delta^3+\delta^4\ldots-\delta^{n-2}}{1-\delta^{n-1}} = \frac{1}{1+\delta} > \frac{1}{2} \qquad (1)$$

The common formula of the distribution factor θ when the private sector makes offer first is generated in Eq. (2).

$$\lim_{n\to\infty}\theta = \frac{\delta+\delta^3+\delta^5+\ldots+\delta^{n-3}}{1+\delta+\delta^2+\delta^3+\ldots+\delta^{n-3}+\delta^{n-2}} = \frac{\delta}{1+\delta} < \frac{1}{2} \qquad (2)$$

Based on the discount factor space [0.8, 1), the distribution factor θ derived from infinite-horizon bargaining game is plotted in Figure 3.

There are four important conclusions have been drawn. (1) The public sector will gain more profit when they make offer first. (2) Each party can obtain more profit percentage by shorten the time period between each stage when the counterparty makes offer first. (3) The public sector can properly increase the bargaining stage when the private sector makes offer first. (4) The proper value space of the distribution factor θ is [1/3, 1/2] for the short-term refinancing gain bargaining game.

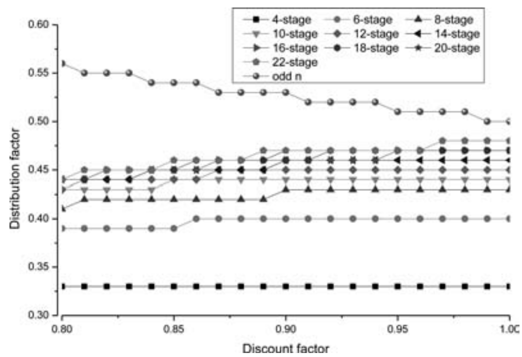

Figure 1. Value of distribution factor θ.

A case study of the successful refinancing project, Fazakerley PFI prison project in UK, has been done. The results of this paper may provide theoretic foundation and thinking logic for the public sector to negotiate with the private sector when refinancing happens.

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Industrial risk reduction system

Organizer: H. Wenzel

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Reliability of decisions based on lifetime functions and monitoring data

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ABSTRACT

The problem of managing existing infrastructure and constructed facilities is becoming of greater and greater economic importance in modern countries. Safety and usability, user security and sustainability are amongst the issues that already developed countries like Europe, United States and Japan have become to face.

In particular, safety and usability of existing structures actually depend on the level of degradation and obsolescence due to ageing. As concerning safety, the problem has often been formulated in terms of time-dependent reliability.

Although the discussion can be generalized to all type of structural systems, concrete structures are considered as a paradigmatic case.

Concrete structures indeed represent the majority of engineered components of infrastructure, like bridges, tunnels, piers, etc. For concrete structures, the time-dependent reliability problem for ageing structures was discussed by several authors.

In view of the application of the above concept to infrastructure management, several reliability-based maintenance strategies, able to keep a global measure of safety and effectiveness above acceptable limits, have been subsequently proposed in terms of lifetime functions and life-cycle-cost optimization.

Lifetime functions for concrete structures can be constructed from theoretical/experimental degradation models and, if a Structural Health Monitoring system is installed on the structure, at any required time, the actual state of degradation can also be determined by applying a damage identification process, allowing updating of the lifetime functions.

With reference to a real case of permanent static health monitoring systems, the paper is aimed at discussing the influence of the uncertainties affecting the damage identification process on the construction and updating of the lifetime function representing the resistance versus time of reinforced concrete beams.

In the first part of the paper, the phenomena of concrete degradation and corrosion of reinforcing steel are discussed in a probabilistic framework with the aim of defining the lifetime functions.

The discussion points out that the phenomena are highly dependent on the environmental characteristics in which the structure is located. Monitoring of the real development of the degradation mechanisms is therefore essential in any condition determination process.

Based on the characteristics of real monitoring data, the performance estimate of damage detection algorithms for a given detection threshold, are then discussed through the Receiver Operating Characteristic (ROC) curve.

A bayesian modeling of monitoring results is then introduced by defining the probability of detecting damage conditional to an actual existing damage (Probability of Detection, PoD) and the probability of detecting damage conditional to non existing actual damage (Probability of False Alarm, PFA).

The process performance can be completely described by the couple (PoD, PFA). Looking for the best detection performances, the probability of detection should always take larger values than the probability of false alarm (low noise sensitivity).

Special emphasis is given to the influence of the environmental conditions on the probability of false alarm, because this latter depends on the noise level and the chosen threshold only. On the contrary, the probability of detection depends on the damage extension.

The difficulties in obtaining a good performance curve in presence of strong temperature variations (noise) and small levels of damage is also discussed and the possibility of improvement of the detection process through post-processing of acquired data is illustrated.

The modeling of uncertainties in lifetime models update and the corresponding effects on engineering decisions is also addressed.

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Evaluation of structural behaviour of a fire damaged highway bridge in Lagos-Nigeria with BRIMOS® structural health monitoring

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ABSTRACT

The Eko Bridge is located in Ijora, Lagos and connects Lagos Mainland with Lagos Island. The prestressed concrete structure consists of two separate load-bearing structures – one for each driving direction. The Main Bridge East has a total length of 190 m and was opened to traffic in the early 1970ies. The cross-section of the superstructures comprises of a three cellular box girder and has a width of 13.8 m.

On the 11th of July in 2008 a fire caused extensive damage to the underside of the superstructure and the piers of the Eko Bridge. Because of the fire the concrete cover at the superstructure's underside failed and the exposed reinforcement bars were partly buckled (Fig. 1). Furthermore at several piers concrete has broken off up to the depth of the first reinforcement layers. In those areas where fire caused the most excessive damage, additional temporary supports were erected – surrounding certain piers (Julius Berger Nigeria PLC 2008). The Site Inspection Report stated that without immediate investigations the safety and stability of the bridges could not be reviewed.

In order to broaden the insight on structural integrity and load bearing capacity a dynamic bridge monitoring campaign was undertaken. Along with the conventional bridge assessment this investigation supports the determination and localization of critical areas as well as the evaluation of already evident problematic zones based on the measured vibration behaviour of the structure (= Performance Assessment) (Wenzel 2009).

For this purpose the monitoring campaign aimed to identify the following key performance indicators of the structure with regard to their relevance for civil engineering issues:

- The bridge structure's relevant eigenfrequencies and corresponding mode-shapes providing information about the load bearing capacity and operability, the distribution of the global and local dynamic structural stiffness in the bridge's lengthwise and transversal directionand furthermore enabling the evaluation of the bearings
- Sensitivity analysis to investigate the progression, the character, the stability and probable changes in the energy content of the relevant eigenfrequencies to evaluate the load bearing capacity and operability
- Energy dissipation path in the structure's lengthwise direction (dissipation of the induced vibration energy) to localize problematic sections
- Vibration intensity at the entire bridge deck to detect weak points with regard to fatigue threat

Based on these detailed investigations judgements and evaluations to what extent the fire has caused serious damage and tailored recommendations regarding possible retrofit and maintenance interventions were given to support the decision process of the bridge owner.

Figure 1. Visual impression of fire damaged bridge sections.

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Monitoring and evaluation of an arch bridge over the Traun River – Austria affected by the blasting of the adjacent highway bridge

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ABSTRACT

The bridge object W4-Steyrermühl is part of the Austrian highway A1 and was constructed in 1959. The arch bridge is composed of 18-spans (reinforced concrete), has a total length of 240.45 m and consists of two separate superstructures (Fig. 1).

As one bridge structure – related with the driving direction Salzburg – was removed (blown) in August 2010, the remaining structure (driving direction Vienna) had to be monitored in order to evaluate the impact of the blasting on its structural safety and operability. For that purpose the BRIMOS® Structural Health Monitoring technology was applied (offering a successful project track record worldwide – e.g. Veit-Egerer 2007, Veit-Egerer & Wenzel 2008, Wenzel 2009, Wenzel et al. 2009). The comprehensive dynamic analysis focused on the primary load-bearing structure (arch).

Due to the fact that an initial dynamic measurement with BRIMOS® has been conducted already in 2005, the prevailing investigation (measurement 2010) could be made more efficiently. The assessment was based on the following time series of the measured bridge behaviour:

– Phase I: Regular traffic 2005
– Phase II: Regular traffic 2010 before the blasting
– Phase III & IV: Closed bridge immediately before and after the blasting
– Phase V: Regular traffic 2010 after the blasting

This direct comparison over 5 years of structural service life enabled to extract the progression of measured structural resistance over time. In addition the bridge's life-cycle curve was calculated showing the bridge arch under the influence of regular traffic load on the one hand and the bridge blasting on the other hand.

The on-site assessment was of crucial importance for the decision to re-open the bridge to traffic after the blasting of the adjacent structure. In a subsequent stage possible additional effects of the blasting were analysed in further detail – coming up with results which broadened the on-site findings.

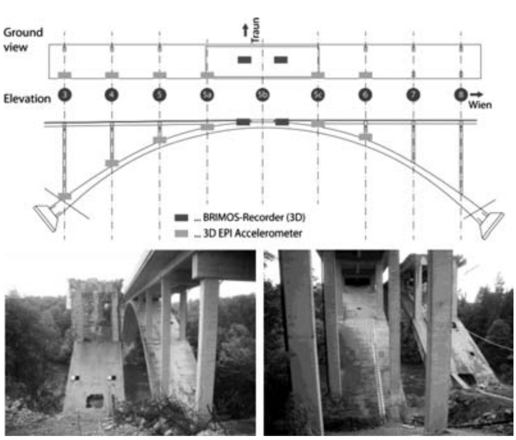

Figure 1. Sensorlayout (elevation and ground view) for the dynamic measurement of the arch and photo documentations.

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Management concept for highway infrastructure based on life-cycle analysis providing heavy maintenance instructions and cross asset harmonisation

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ABSTRACT

The paper describes a case study elaborated for the Austrian Federal Highway Company ASFINAG in 2011. A 25 km long highway section (see Fig. 1) comprising 102 structures on the Austrian S6 highway was analysed in order to derive a maintenance concept for the upcoming 30 years.

This maintenance concept was intended to give a long-term outlook for maintenance measures (heavy and routine maintenance) during the upcoming service life of the analysed structures.

To derive tailored maintenance plans a life-cycle model was developed which utilizes state-of-the-art information from literature as well as VCE's experience from performing monitoring and bridge inspections worldwide. The multi-level procedure is based on the entire life-cycle of a structure and considers all available information gained from visual inspections, the applied design concept, field measurements and the relevant loading exposure (Veit-Egerer & Widmann 2010).

Probabilistic methods are used for the service life calculations of the whole structure as well as for individual items delivering lower and upper bounds of life expectancy.

The calculated lifetime prognosis represents estimations at the time of investigation. This means it is necessary to update the incorporated ageing curves periodically, using the latest knowledge from on-site inspections based on the Austrian RVS-regulations (FSV 1995) and assessment in further succession.

Figure 1. The S6 highway and the given analysis section.

Table 1. Analysed structures.

Structural Type	Amount
Bridges	76
Tunnels	8
Gantries	18
Total	102

Based on the results of the life-cycle analysis maintenance instructions were elaborated for every structure and structural member in order to ensure the demanded structural service life and operability.

In times of reduced maintenance budgets cost optimization is an important issue in the field of infrastructure management. In the present case study the cost and availability optimization considering the existing pavement management concept was one of the key demands. Furthermore the case study includes proper maintenance concepts tailored to different budget scenarios given by the client. The result can be used as a basis for decision making in the long run. As the analysed structures consist of bridges, culverts, flyovers, access ramps, gantries and tunnels (see Tab. 1) cross asset harmonization was another task to be considered.

In a final step a parameter study was conducted comparing different asset specifications in order to detect possible correlations between certain structural characteristics and life expectancy on the one hand and life-cycle costs on the other hand.

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Ultrasonic monitoring of high temperature pipes in power plants using wave guides

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ABSTRACT

In order to reduce the CO2-emissions and to increase the energy efficiency, the operating temperatures of power plants will be increased up to 720°C. This demands for novel high-performance steels in the piping systems.

Higher temperatures lead to a higher risk of damage and have a direct impact on the structure stability and the deposition structure which have a large influence on the long-term stability under load and the damage behavior. Adequately trusted results for the prediction of the residual service life of those high strength steels are not available so far. This causes security problems that avoid the application of those steels. To overcome these problems the implementation of a non-destructive online monitoring system in addition to periodic testing is needed.

Wave propagation dynamics predictions for damage detection AcoustoUltrasonics (AU) uses ultrasonic methods in a frequency range between 10 and 500 kHz. Ultrasonic waves are reflected by surfaces and interfaces, attenuated by dispersion and absorption, and undergo mode changes during reflection and transmission. These effects depend strongly on the frequency of the wave, its direction of propagation, its initial mode, and the location and orientation of surfaces and damage. When damage has occurred to a structure, changes in the signal and therefore the transfer function indicate the type of damage like cracks and wall thickness reductions due to corrosion. By precalculating the expected changes in the signal from given types and degrees of damage, the damage can be evaluated from AU measurements. This kind of measurements is repeated according to the expected damage velocity. Using e. g. hourly measurement intervals, the growing of the damaged can be described with high time resolution. A high spatial resolution is achieved by using high frequencies with the disadvantage of shorter possible travel paths. An initial situation (baseline) must be measured to describe the undamaged situation at different load levels since the damage might be load dependent.

RWE operates the lignite power plant Neurath. In Block E of this power plant the bypass is installed and operated. All test and research activities have to be checked regarding their safety and have to be coordinated with the business operation of the plant. In order to investigate the noise level and the influences of the steam flow in the pipe, an extra bypass was established for this research. This made the investigations independent from the power plant operating. The bypass is made of high temperature steel with 9% Chrome. In order to protect the actuators and sensors from the heat radiated from the pipe, waveguides were welded to the bypass. 12 measurement locations out of the 18 waveguide positions were chosen for installation at the bypass. The waveguides are located in circles of three waveguides around the pipe at 6 pipe locations.

The stability measurements with no steam flow show good correlations. The amplitude and phase variation of the direct wave represented by the first wave packet are low. The signals can therefore be regarded as reproducible and stable.

The second measurements with varying steam parameters indicate that the data has to be normalized regarding the parameters temperature and pressure to separate influences of environmental parameter and damages to the signal.

The data are currently being processed in order to develop and algorithm for the elimination of the environmental parameters. After applying this algorithm, all changes in the signals are caused by structural changes and defects in the area of the signal path. The next step will be the interpretation of the changes regarding the evaluation of the defects.

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Contribution to US Long-term bridge performance program with regard to life-cycle investigations – reference bridge New Jersey

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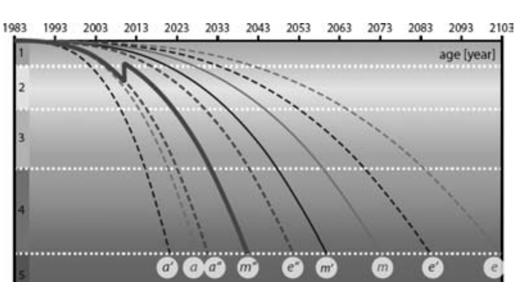
ABSTRACT

A representative composite bridge has been selected by the Federal Highway Administration (FHWA) for the international collaboration between the Long-Term Bridge Performance Project (LTBP) initiated in the USA and the European FP7 Project IRIS. Various teams from the USA, Japan, Korea and EU have performed separate assessment routines to demonstrate their approach on bridge inspection, structural assessment (monitoring) and evaluation.

The following critical questions had to be answered:

– Does the bridge have any strength/capacity issues? If so, what are they?
– How can technology be used to identify, quantify and understand these issues?
– What is the root cause of any deficiencies in the bridge?
– What – if any – retrofits or interventions would you recommend?
– Maintenance and repair investigations for a 75-year life-cycle?



Residual life expectancy [years]	Initial range		Adaptation		Enhanced prediction	Prediction due to maintenance	
Lower bound	a	18	a'	11	12	a*	21
Mean	m	63	m'	50	21	m*	31
Upper bound	e	93	e'	76	30	e*	41

Figure 1. Derived life-cycle curve incl. maintenance interventions – reference bridge New Jersey.

In the course of the IRIS project a strongly life-cycle oriented approach has been developed and standardized in order to clearly address the stated problems. The multi-level procedure is based on the entire life-cycle of the structure and considers certain structural characteristics gained from:

– a visual inspection of the critical elements following the Austrian RVS 13.03.11 regulations,
– specifications from design and loading,
– a monitoring campaign using the BRIMOS® wireless methodology.

In further consequence the BRIMOS® Life-Cycle Methodology enables proper maintenance planning for engineering structures over a long time period and provides a tool that allows the optimization of mitigation measures and financial optimization. From certain key parameters the structures' so called Health Indices are calculated which represent the reference state for life expectancy calculations. For a refined prediction any additional information which is able to contribute to a better understanding of a structure is incorporated. A parameter study was carried out – analysing the effect of different maintenance measures and their impact on residual lifetime and life-cycle costs.

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International investigation on the earthquake damaged Chi Lu Cable Stayed Bridge after repair and several years of operation

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ABSTRACT

Chi-Lu Bridge, located at Nantou Country, Taiwan, is a pre-stressed concrete cable-stayed bridge, crossing the Juosheui River. The bridge has a 58 m high single pylon, two rows of harped cables, and a streamline-shape double box girder. On September 21, 1999, near the final construction stage of Chi-Lu Bridge, a significant earthquake (Chi-Chi Earthquake) with 7.3 ML magnitude seriously struck the central part of Taiwan. For only three kilometers away from the epicentre, Chi-Lu Bridge underwent very strong ground motion and several of its structure members were heavily damaged. The bridge became the first earthquake damaged cable-stayed bridge in the world.

While the mending of the concrete structure had been finished in 2001, the repair of the cable system lasted until 2005 because the status of this bridge was not clear and the ways to repair the cable system were not determined.

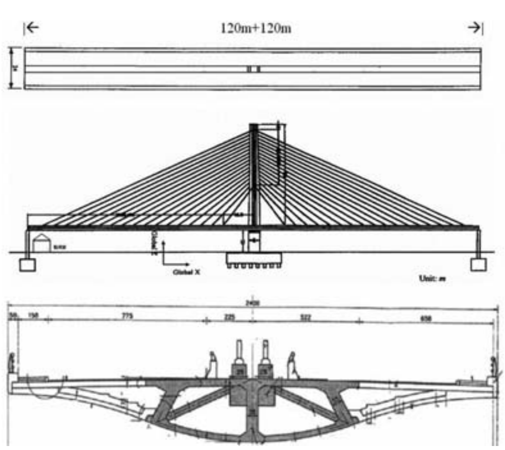

Figure 1. Plan, elevation and cross-section of the Chi Lu Bridge at Nantou Country, Taiwan.

Since 2001 collaboration exists between European research projects and the Taiwanese National Centre of Research on Earthquake-Engineering (NCREE). Furthermore IRIS beneficiary VCE has been involved in the design and construction supervision of the Chi Lu Bridge. This in combination with the unique earthquake history of the structure provides a spectacular opportunity for collaboration.

VCE has done a basic measurement on the cables in 1999. In May 2011 a follow-up monitoring campaign (Bridge Deck & Cables) was performed focusing on the following major points of interest:

- Substantial change in structural properties (cable & deck) is to be considered
- Are there traces of stress-history introduced into the structure by the earthquake?
- Comparison with the NCREE monitoring activities (started in 2001)
- Utilization of available results and findings for integral Lifecycle Assessment (LCA) based on the Observation Period 2004–2011.

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Seepage analysis of a gate dam with a layered deep deposit foundation

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ABSTRACT

Failure of a dam by subsurface erosion ranks among the most serious accidents in civil engineering. The seepage field and the risk of subsoil erosion of the foundation of Ying-liang-bao gate dam are analyzed.

As showing in Figure 1, the foundation consists of 5 inter-bed layers with strong permeable gravel layers and relatively weak permeable sand layers with a total depth up to 126.5 m. The seepage coefficient of gravel layers is more than 100 times that of sand layers as shown in Table 1. Concrete cut-off wall up to 80 meters depth is used in the first design scheme to control the seepage. Although the cross water head of the dam is only 29.8 m, the hydraulic gradient in layer ② and ④, as showing in Figure 2, are very large, thus the internal erosion risk of the sand layers is very high.

The reduction of flux is not effective via the depth increase of the suspended cutoff wall, unless the covering layers of the foundation are closed to cut off completely. Not only the internal erosion risk in the relatively impermeable layers is difficult to reduce via increasing the depth of a suspended cutoff wall, but also the risk in the soils beneath the suspended wall increase greatly and thus jeopardizes the structure safety of the cutoff wall. The scheme with a closed cutoff wall is erosion safety and it is recommended to and adopted by the preliminary design.

A row of size fixed elements is used to represent any desired width of elements via the adjustment of the tensor of permeability according to Darcy's law. If the represented size in the direction of axis 2 is n times the size of the scale reduced element, a transformation of the values of the permeability tensor of the scale reduced elements with an equivalent flux can be achieved by the following transformation.

$$k_{2j} = nk'_{2j}, k_{1j} = k'_{1j} / n, k_{3j} = k'_{3j} / n \quad (1)$$

where the superscript sign in the equation represents the value of the ordinary element.

The appropriate position of the cutout boundary in the covering layers is evaluated. It shows that the position of an imperious cutout boundary in covering layers has a significant influence to the total flux of the foundation in the model of the project.

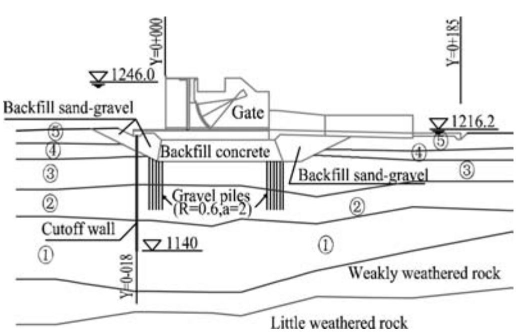

Figure 1. Typical cross section of the gate.

Table 1. Infiltration feature of the covering layers

Layer	Dry density g/cm^3	Permeability m/s	Permissive hydraulic gradient
⑤	1.9∼2.0	$5 \times 10^{-5} \sim 1 \times 10^{-4}$	0.12∼0.15
④	2.0∼2.05	$2 \times 10^{-8} \sim 1 \times 10^{-7}$	0.50∼0.60
③	2.05∼2.1	$1 \times 10^{-5} \sim 1 \times 10^{-4}$	0.15∼0.18
②	1.4∼1.6	$5 \times 10^{-8} \sim 5 \times 10^{-7}$	0.40∼0.50
①	2.05∼2.1	$1 \times 10^{-5} \sim 1 \times 10^{-4}$	0.15∼0.20

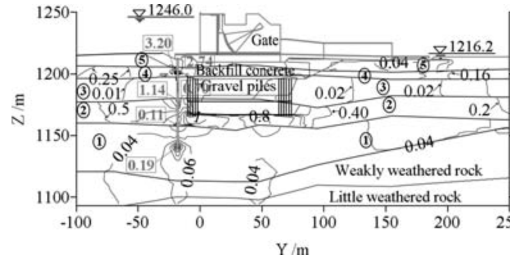

Figure 2. Hydraulic gradient contour at the section of the gate.

Actions and interventions upon existing structures

Organizers: R. Caspeele, S. Matthews & G. Mancini

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Decision making tool for seismic retrofit of existing structures based on marginal costs and actualized costs of the retrofitting operations

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ABSTRACT

The loss estimation analysis of a structure subjected to seismic hazard is used to determine the expected costs to be sustained due to seismic damages in the structure life-time. In this work a procedure is developed and applied to an existing structure in L'Aquila, Italy damaged by the April 2009 earthquake, in its as-built and retrofitted configuration. The method consists in a seismic hazard analysis, damage analysis and loss estimation. The loss evaluation is performed considering the probabilistic distribution of construction costs in Italy. The structure is divided into eight assemblies: foundations, external partitions, doors/windows, plumbing system, structural elements elevation, floors, retrofit system. Four limit state for each assembly are considered {very light, light, moderate, severe}.Then, the procedure is reiterated for different increasing retrofitted configurations of the structure, generating increasing initial costs, but a reduction of the expected costs. The results are elaborated in terms of actualized expected costs to face during the lifetime of the structure and marginal cost of the retrofit interventions.

The expected actualized costs have a decreasing trend; the drop in the costs to sustain is due to the retrofitted configuration.What we want to inquire is whether the retrofit is economically convenient. To answer this question we have to introduce the neoclassical economic concept of "marginal cost": it is the variation of total costs due to one unit of increase of the quantity produced.

The concept describing the retrofit "marginal cost" behavior involve more complex aspects:

1. a "behavioural aspect", i.e. the marginal benefit experimented by the reinforced structure;
2. a "saving aspect", i.e. the costs saved thanks to the retrofit component, due to the better performance of the reinforced structure.

In our specific case we show that the addition of a retrofit component will be twofold: on one side it will have the effect of raising the total repair costs but, on the other hand, the total costs to be sustained in the lifetime of the building will be lowered.

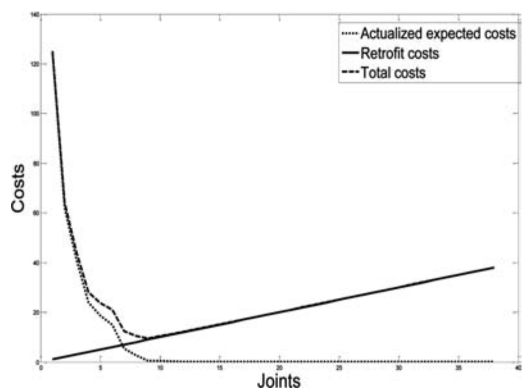

Figure 1. Marginal cost of retrofit and optimal reinforcement joints – costs in k€.

In figure 1 we show the total actualized costs, the retrofit costs and their summation vs. the number of the retrofitted joints; it is clear that, increasing the retrofitted joints, the actualized costs reduce but, obviously the retrofit costs increase. The minimum of the summation of these two curve (corresponding to 7 joints) represents the optimal amount of retrofitted joints, i.e. the optimal retrofitted configuration, from the economic point of view.

It is economically reasonable to reinforce till the seventh joint; the neoclassical concept of "diminishing return to scale" suggests that it is convenient to raise the production inputs until it produces a positive output: from a certain point on, by adding more and more workers in a small land, the costs will increase but from a point on, the output will increase less than inputs, meaning that costs have begun larger than benefits.

The concepts of marginal cost and diminishing return to scale have been adapted for our specific aims.

In conclusion, the minimum of the summation curve represents the minimum cost to undertake corresponding to the optimal retrofitted configuration.

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Rehabilitation design methodology for public hospital using seismic monitoring throughout structural life-cycle

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ABSTRACT

The paper presents the time behavior analysis of a regional hospital structure dedicated to the Romanian railways employees, located in Iasi Municipa-lity. The building underwent the action of three major earthquakes of magnitudes higher than 6 on Richter scale, which have unfavorably influenced the safety level of the structure. The building was designed in 1970, using the Romanian design code seismic safety in force at early 70's. This building is located in an area of increased seismic hazard with design acceleration of $a_g = 0.20$ m/seconds^2 and corner period of $T_c = 0.7$ seconds in accordance with P100-1 (2006), being founded on a heavy loessial clay, sensitive to humidity, of 11 meters thickness. From among the most conspicuous structural design errors, identified in Murarasu (2009), which influenced the time behavior to seismic load action, mention has been made of:

- adopting a foundation system made up of isolated blocks without any interplay that would enable the taking over of potential differential settlings, NP-112 (2004);
- adopting a frame structure with an uneven arrangement of vertical structural elements which lead the structure in the 2nd vibration mode, at general torsion, identified in Finite Element Modal Analysis using Chopra (2006) and the software AxisVM9 (2009);
- insufficient thickness of the floor which is unable to ensure the diaphragm effect in its plan, essentially important in ensuring the interaction when the seismic load is taken over and an inadequate longitudinal confining and reinforcing of the potentially plastic zones in the structure;
- insufficient lateral stiffness of the structure.

The building displays a series of damage states in the structural and non-structural members, amplified by technologic errors in making up the reinforced concrete structural members (local segregations of concrete in sections corners and column base, insufficient concrete coverings leading to prejudicial corrosion of the transverse and longitudinal reinforcements in structural elements of the basement, affecting the building safety level as recommended in Eurocode 8 (2006).

In order to improve the building's behavior to seismic action, without substantially intervening on the structure, the authors have proposed the use of shear panels disposed in the grid mesh of the frame structure, by enhancing the use of the existing brick masonry interior walls with increasing their mechanical resistance by local confining. A redistribution of the shear stresses in the structure will be thus achieved, an important amount of the seismic energy being absorbed by the shear panels.

Another proposal was made to build some balancing beams that would take over potential differential settlings and ensure an increased degree of interaction, in order to improve the behavior of the founding system.

By adopting these structural retrofitting measures, the level of assurance to seismic loads will be improved, according to design requirements of code P100-1 (2006).

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Evaluation of Bayesian updated partial factors for material properties in existing concrete structures

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ABSTRACT

In contrast to the design of new structures, the assessment of existing structures often relies on the subjective judgment of the investigating engineer. However, in a previous analysis the authors have showed that an objective verification format for existing structures is feasible and they proposed a suitable Bayesian semi-probabilistic partial factor method for existing structures, enabling a rather simple and straightforward, but objective and coherent safety evaluation of existing concrete structures by practitioners. The proposed framework is compatible with the current Eurocodes for the design of new structures, but additionally enables to incorporate alternative values for the target reliability level β_t, alternative values for the remaining working life n and also updated information based on e.g. on-site inspection data and data from testing, as these all considerably influence the partial factors in the structural reliability assessment of existing structures.

The basic philosophy of the Adjusted Partial Factor Method consists of calculating an adjusted partial factor $\gamma_{X,exist}$ for existing structures (considering alternative values for n, β_t and COV δ_X) for a given variable X by simply multiplying the regular partial factor γ_X as provided in the Eurocodes for new structures by an adjustment factor ω_γ according to:

$$\gamma_{X,exist} = \omega_\gamma(n, \beta_t, \delta_X) \cdot \gamma_X \tag{1}$$

In case of material properties, the adjustment factor can be derived as:

$$exp\left(\alpha_R \beta' \delta'_X \left(\frac{\beta''}{\beta'}\frac{\delta''_X}{\delta'_X} - 1\right) - 1.645\delta'_X\left(\frac{\delta''_X}{\delta'_X} - 1\right)\right) \tag{2}$$

where the characteristics with respect to new structures are designated with $'$ and those for existing structures with $''$ (as in case of the COV of material properties this relates to posterior information based on Bayesian updating).

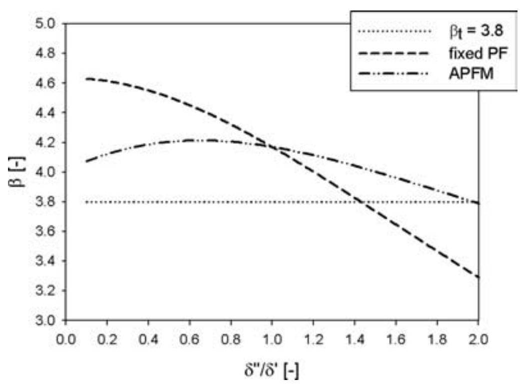

Figure 1. Comparison of the achieved reliability index β of a concrete column in function of the Bayesian updated ratio δ''/δ' in case of concrete ($\beta_t = 3.8$, $t_r = n = 50$y, $\chi = 0.5$, $\rho = 1\%$).

In case lognormal-gamma distributions are considered, the Bayesian updated ratio δ''_X/δ'_X can be calculated as follows, according to the prior and posterior hyperparameters of the distribution of the material property under consideration:

$$\frac{\delta''_X}{\delta'_X} = \frac{s''_{ln\,X}\sqrt{\dfrac{n''}{n''-1}}\sqrt{\dfrac{\nu''}{\nu''-2}}}{s'_{ln\,X}\sqrt{\dfrac{n'}{n'-1}}\sqrt{\dfrac{\nu'}{\nu'-2}}} \tag{3}$$

A FORM based analysis was performed in order to calculate the reliability index in case of fixed or adjusted partial factors, especially with respect to a Bayesian updated COV for concrete strength.

Although slightly conservative, the simplicity of the proposed adjustment method does not compromise the achieved reliability level and moreover has the benefit that it is generally applicable for assessment, repair or rehabilitation of existing concrete structures.

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Verification of older prestressed concrete road bridges according to the German structural assessment provisions

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ABSTRACT

Over the last decades a considerable increase in traffic volume occurred, which has been taken into account by introducing new traffic load models. Additionally, the design and detailing rules for new structures have constantly been improved over the course of time. Hence, many of the existing older German road bridges were not dimensioned to carry the current design traffic loads and were not built according to the latest code generations. This is the reason for the recent particular focus on the assessment of older road bridges.

An assessment of the structural performance of older prestressed concrete bridges according to today's standards will lead to not complying with several SLS and ULS checks in most cases. To enable a more accurate assessment of these existing bridges the German Structural Assessment Provisions for Older Road Bridges (BMVBS 2011) were developed by a working group over the last three years and published in May 2011. The provisions are based on the current German bridge design codes but allow for additional verification measures without leaving the reliability level required by EN 1990 (DIN 2010).

When current design codes are used in the re-design of older German prestressed concrete road bridges, some typical defects can be observed. Amongst others, insufficient shear reinforcement close to supports, failure to fulfill the decompression limit and low fatigue resistance in coupling joints of superstructures cast in multiple construction stages are common results. Another problem characteristic of German prestressed concrete bridges from a certain period of time is the unintended but numerous use of prestressing steels prone to stress corrosion cracking even after grouting of the ducts.

To take into account these particularities of existing road bridge structures the German Structural Assessment Provisions for Older Road Bridges feature a stepwise approach (level 1–level 4) during which more and more prior information about the structure can be included in the assessment and more sophisticated verification models than used in current codes can be applied. A level 1 structural assessment is equal to a re-design according to current DIN-Fachbericht 102 (DIN 2009), while a level 2 assessment allows for the use of more precise models and an adjusted safety concept not contained in the code. A level 3 structural assessment corresponds to a level 2 assessment under consideration of measurements made at the structure. In a level 4 assessment, scientific methods like a probabilistic analysis or a nonlinear 3D finite element analysis can be used. The provisions contain regulations on the assessment of reinforced and prestressed concrete, steel, composite and masonry bridges.

First applications of the Structural Assessment Provisions in the evaluation process of real structures have brought up several unsettled questions and future challenges that have to be dealt with in the enhancement and updating process. In addition to further develop more precise verification models and perform thorough research on an adjusted safety concept for existing structures, it is of high importance to discuss and determine exactly the responsibilities of all parties in the assessment process. To facilitate the evaluation of a certain bridge in comparison with other bridge structures, it might be useful to introduce certain standard examination scenarios (e.g. ratio of corroded steel) and to perform a mandatory sensitivity analysis.

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Target reliability levels for the assessment of existing structures – case study

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ABSTRACT

Existing structures are mostly verified using conservative deterministic procedures. More realistic verification of actual performance of existing structures can be achieved by probabilistic methods for which specification of the target reliability levels is required. In addition the target reliabilities can be used to modify the partial factors for a deterministic assessment. It has been recognised that it is uneconomical to specify for all existing structures the same reliability levels as for new structures.

ISO 13822 indicates a possibility to specify the target reliability levels for existing structures by optimisation of the total cost related to an assumed remaining working life. The paper attempts to apply this approach in conjunction with the criteria for safety of people in accordance with ISO 2394.

From an economic point of view the objective is to minimize the total working-life cost that may include costs of inspections, maintenance, upgrades and costs related to structural failure.

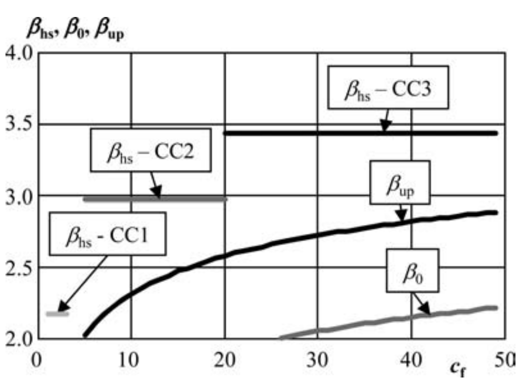

Figure 1. Variation of the target reliability indices with the failure consequences c_f ($t_{ref} = 15$ years).

The cost optimisation is aimed at finding an optimum decision from the perspective of an owner of the structure. However, society commonly establishes limits at human safety. General guidelines for human safety criteria provided in ISO 2394 are accepted here and applied in the case study.

Application of the considered procedures is illustrated by the example of reliability assessment of a generic structural member with the remaining working life of 15 years.

Figure 1 shows variation of the target reliability indices with failure consequences c_f for different Consequence Classes CC ($\beta_0 =$ minimum level below which structure should be upgraded; $\beta_{up} =$ level indicating optimum upgrade strategy). Apparently human safety is decisive for the target reliabilities.

The present study indicates that:

- Decisions in the assessment can result in the acceptance of an actual state or in the upgrade of a structure; in principle two target reliability levels are thus needed – the minimum level below which the structure is unreliable and should be upgraded, and the level indicating an optimum upgrade strategy,
- Economic criteria yield lower optimum reliabilities than the criteria for human safety,
- The following annual target reliability indices might be accepted: $\beta \approx 3.1$ (CC1), $\beta \approx 3.7$ (CC2), and $\beta \approx 4.1$ (CC3).

Further research should be aimed at improvements of the background for the human safety criteria.

ACKNOWLEDGEMENTS

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Verification of existing reinforced concrete structures using the design value method

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ABSTRACT

Upgrades of existing structures including design of adequate construction interventions are becoming an important issue. Decisions about the interventions should be always a part of the complex assessment of a structure, considering relevant input data including information on actual material properties.

At present existing structures are mostly verified using simplified deterministic procedures based on the partial factor method. However, such assessments are often conservative and may lead to expensive repairs. More realistic verification of actual performance of existing structures can be achieved by the design value method according to EN 1990 and ISO 2394. The submitted study intends to clarify applications of this method in verifications of existing reinforced concrete structures.

Conventional probabilistic models for basic variables influencing structural reliability (model uncertainties, material properties and permanent and variable actions) are proposed. Using probabilistic methods, partial factors of the basic variables are then derived for different target reliability levels. In addition a reference period for the reliability verification is considered in the assessment of partial factors of variable actions.

Application of the described procedures is illustrated in the example of reliability analysis of an existing reinforced concrete beam and short column exposed to a permanent and snow load. The target reliability index $\beta_t = 3.1$ and reference period (equal to a remaining working life) $t_{ref} = 15$ y. are considered. Figure 1 shows the variation of the reliability index β with the load ratio χ (characteristic variable to total actions). In addition the reliability index for the partial factors recommended for new structures, independent of χ and t_{ref}, is plotted for the beam.

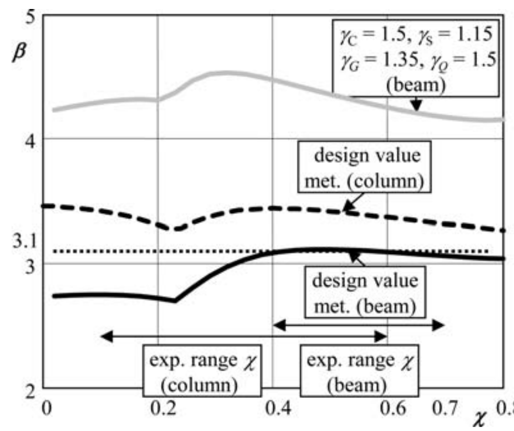

Figure 1. Variation of reliability index β with the load ratio χ.

The study indicates that reliability of existing reinforced concrete structures can be efficiently verified using partial factors obtained by the design value method. Numerical example reveals that:

– The design value approach captures well random properties of the basic variables. For the expected ranges of the load ratio, the obtained reliability indices are reasonably close to the target level for different reference periods.
– Influences of the target reliability and of information obtained from the measurements are not adequately covered when the partial factors recommended for new structures are considered.

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Load-carrying capacity and refurbishment of a historic RC Vierendeel bridge

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ABSTRACT

The Pontweg bridge is located near Ghent and connects the city suburbs to a residential area. However, lorries seem to use this road as well and the bridge is considered to carry a consistent amount of heavy traffic. The bridge is of the Vierendeel arch type and has a typical form. The nodes are heavily reinforced and evidently they match the flow of bending moments. The opening in the nodes is a specific characteristic. In addition, the arch springs show a change of curvature, evidently outside the intersection of the arch and lower chord centerlines. The Pontweg bridge was built in 1926. Because of its relatively rare concept, the specific details and historic value, this bridge is now listed as a monument, meaning that no alterations can be done. Recent inspections detected rather serious damage of the concrete and corrosion of rebars, due to carbonation. However, the damage is concentrated at a limited number of locations.

During dynamic testing no particular difference in the natural frequencies of the unharmed and the damaged edge beam of the bridge has been found. This was confirmed by the numerical model and is due to redistribution of bending moments and tensile normal force towards other members of the concrete bridge deck, as a central stiffening beam and the edge beam of the walkway.

The load-carrying capacity of this bridge was verified for the ultimate limit state with LM 1, for which condition the structure was never designed. To assess the remaining load-carrying capacity, the fatigue load models of EN 1990 were used, both for verifying serviceability state stresses as for determining the failure probability. The latter was compared to the conditional probability for loss of human life which equals $3 \cdot 10^{-2}$. Apart from the present situation of the bridge, the owner also wanted to know what would be the conditions after repair and moreover, what could be the consequence of removing the cobblestone pavement and replacing it by asphalt, thus reducing the dead load on the bridge. The results show that some lorries introduce larger stresses for the asphalt alternative. This applies only to the central beam, since steel stresses in all other members are about 10% lower. Obviously, the asphalt pavement being thinner, there is a poorer distribution of knife loads.

Clearly, the maximum stress method and the failure probability method render different results. For the damaged condition, the stress method detects critical situations for the central beam, the walkway beam and even for the closest hanger, and the failure probability method renders identical results. Referring to the condition with the concrete repaired, without any change in the rebar sections, the stress method still detects all three members to be critical, as well as the repaired edge beam. The failure probability method detects that only the central beam shows insufficient strength to satisfy conditional probability for human loss criterion. Hence, both methods can result in rather diverging results and the conclusions are difficult for the owner to decide on repair actions.

A refurbishment of this historic structure has been proposed. Further carbonation can be prevented by decreasing capillary porosity and refining the pore structure of the concrete. Sealing with fine granular mortar is expected to efficiently improve the concrete condition. The refurbishment also includes mechanical cleaning of the remaining rebars and passivity by cold galvanizing. However, on the basis of stress analysis many members would require additional reinforcement. This is not confirmed by the conditional probability method and the evident conclusion has been to reinforce the central beam and the damaged edge beam. For this purpose CFRP-laminate has been proposed, which can be glued to the repaired concrete surface. In addition, a partial repair procedure for the steel roller bearings has been worked out.

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Simulation technique for service life assessment of façade refurbishment

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ABSTRACT

The EU Research Project SUSREF proposes as its outcome sustainable concepts and technologies for the refurbishment of building facades and external walls. One of the tasks in the SUSREF project was to predict the performance and service life of the proposed refurbishment concepts.

A simulation software developed at VTT in the 1990s was updated and used to evaluate possible degradation in refurbished concrete facades. The software is able to emulate the temperature and moisture content in a cross-section of a sandwich wall and to apply temperature and moisture sensitive degradation models at critical points of the wall throughout its service life. Five different refurbishment concepts of building facades were analyzed for predicting possible moisture and degradation problems. The analyses were conducted in various European climates and with various materials as thermal insulation and outer core.

The results show that the risk of mould growth and continued corrosion of reinforcement should be considered when using dense insulation materials, such as expanded polystyrene and polyurethane. The concepts where the original outer core and thermal insulation are replaced by new ones seem to be relatively risk free. In addition, in cold weather countries the risk of frost attack should always be considered with concretes and mortars. The use of dense coatings on original outer core or rendering may increase the mould risk.

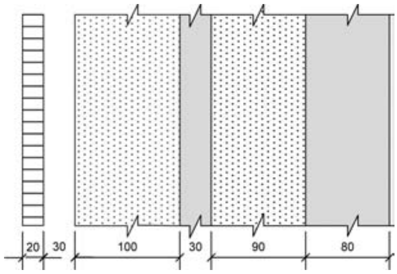

Figure 1. Refurbishment concept E2.

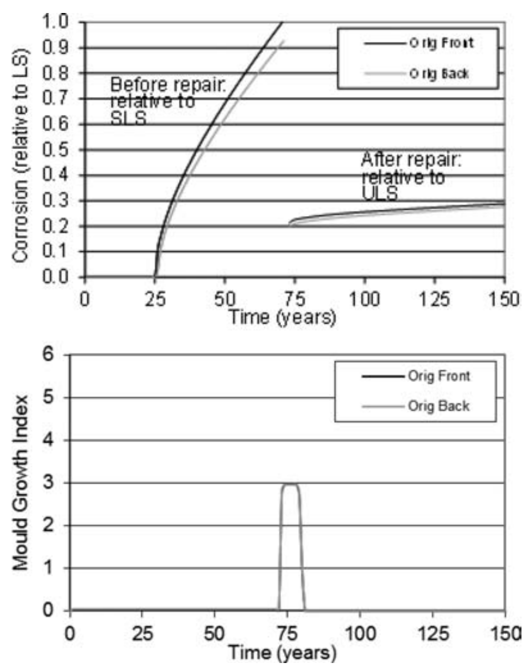

Figure 2. Results for the refurbished façade with concept E2 and mineral wool option. (a) Corrosion of reinforcement, and (b) mould growth in the front and back side of the original outer core before and after refurbishment.

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Girder shear resistance assessment – applications of SIA 269/2

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ABSTRACT

In the beginning of 2011, the Swiss Society of Engineers and Architects (SIA) edited the new code series SIA 269 for existing structures.

The code series SIA 269 has a very strong link to the code series SIA 260 for new structures. In particular for structural safety assessment of existing structures, no new resistance models are introduced; rather, simplifications initially introduced for the resistance models for new structures are eliminated or detailed to allow for more precise actualizations of structural resistances. Furthermore, particularities of older existing structures are dealt with.

Assessment of girder shear resistance according to SIA 269/2 (2011)

A particular issue in the preparation of SIA 269/2 (2011) was the formulation of code rules for the assessment of shear resistance of concrete girders with transverse reinforcement at Ultimate Limit State (ULS). Such types of elements are fundamental for concrete structures, and can be found in almost every construction works.

The new Swiss code SIA 269/2 (2011) contains only very few regulations for the determination of the shear resistance of existing concrete girders with transverse reinforcement at ULS. This is essentially related to the fact that basically the same resistance model as for new structures according to SIA 262 (2003) is applied, but that more brittle shear failure modes often have to be exploited for existing girders than in the structural design of new girders.

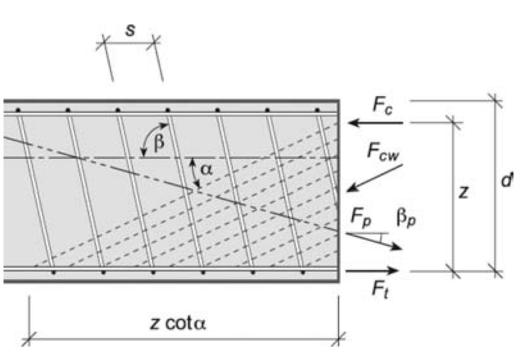

Figure 1. Stress field in girder web with variable inclination of compression fields, according to SIA 262 (2003).

The intention behind this approach is that the structural engineer in charge of the structural assessment has to be very much aware of what he is doing and that he needs profound knowledge on the resistance models and structural behavior of concrete girders in shear – more refined und thus more precise assessment of structural resistances has to be "bought" with more effort in analysis and education.

The full paper discusses the background to the few directives given in SIA 269/2 (2011), i.e.

– the applied shear resistance model corresponding to a variable angle truss model (Fig. above), and what shear resistance can be maximally achieved;
– the influence of the state of strains in the web concrete on shear resistance, in particular on the web's effective compressive strength, the coherence of the proposal of SIA 269/2 (2011) with SIA 262 (2003) for new concrete structures and comparing it to other proposals from literature;
– the impacts of the ductility properties of shear reinforcement and of bond properties between reinforcement and web concrete, and how they influence the choice of allowable compression field inclinations at ULS.

The full paper is concluded by some exemplary applications of the directives of SIA 269/2 (2011) to typical shear reinforcement types, and how these directives could be applied in daily practice.

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Probabilistic durability assessment of concrete structures

Organizers: B. Teplý & D. Novák

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Life-cycle assessment of RC structures in Czech regional conditions

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ABSTRACT

First draft version of a tool for life-cycle assessment of reinforced concrete structures was developed based on general methodology (standards) and tested for implementation in Czech regional conditions. The programme enables to compare and evaluate various types of structures from various kinds of concrete from the perspective of impacts on environment within the whole life-cycle of the structure. A complex LCA analysis of three alternatives of RC floor structures and one timber floor structure is presented and environmental impacts are compared and discussed in the paper. Relevant complex LCA is based on local environmental data collected within the inventory phase of the LCA procedure.

The complex Life-Cycle Analysis (LCA) was performed for four various floor structures: V1 reference full RC slab, V2 timber floor structure with sub-floor from OSB boards (glued), V3 timber-concrete floor structure C30/37 (nailed) and V4 timber-concrete floor structure HPC140 (glued).

Alternatives V2–V4 were designed for mass/unit area of 250 kg/m^2(due to acoustics requirements), same deflection L/300 and equal axial beam spacing a = 625 mm. Furthermore, dead load of 3.0 kN/m^2 and live load of 1.5 kN/m^2 were considered, in total with self-weigh 7.0 kN/m^2. Designed span was 4.4 m.

Figure 2 shows aggregated impact data of entire life-cycle. It is apparent that consumption of primary raw materials of V2–V4 alternatives is from 40 to 50% lower in comparison with V1. Water consumption and photo-chemical ozone creation potential are almost equal for all four variants. The remaining environmental indicators show the same trend, the lowest values for V2, followed by V4, V3 and V1. The results proved that timber floor structure based on renewable natural materials is the most environmental friendly alternative from four assessed floor structures. The second best is the timber-HPC floor structure. The results show that the high quality of mechanical and environmental performance creates the potential for wider application of High Performance Concrete in building construction.

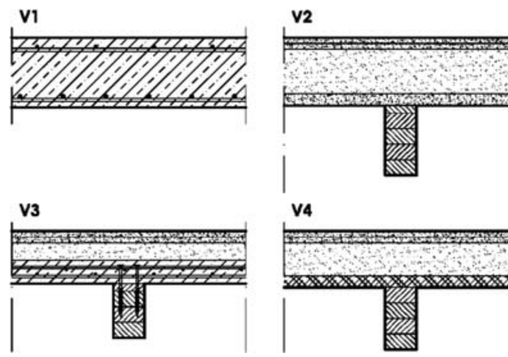

Figure 1. Schematic sections of floor structures alternatives.

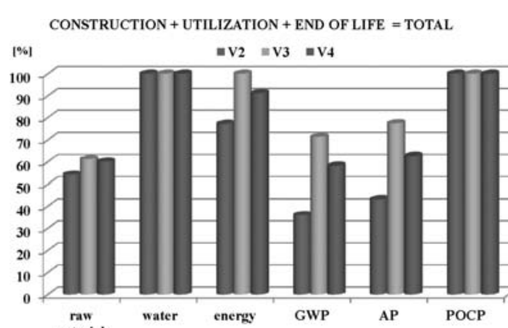

Figure 2. Aggregated impact data of entire life-cycle. 100% is represented by V1 full RC slab.

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Time-dependent reliability analysis of reinforced concrete beam under different failure modes

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ABSTRACT

Reinforced concrete bridges in service, resistance of structure affected by various factors, such as concrete carbonization, cumulative damage under loading, chloride penetration, alkali-aggregate reaction of the concrete and so on. Corrosion of steel bar due to the main factors of concrete carbonization and chloride penetration is lead to influence of resistance. The cover corrosion cracking due to concrete carbonization can be neglected when concrete cover thickness reach 45 mm. It is showed that the failure probability due to chloride penetration is much larger than concrete carbonization. Therefore, the reliability analysis of reinforcement concrete beams caused by corrosion of steel bar due to chloride penetration is necessary.

Concrete bridge has some failure modes due to chloride penetration. But for reinforcement concrete beams, flexural and shear failure are the two main failure modes. Research shows that it is necessary to thinking about the space of the pit rust variability and the influence of the structure degradation by loading for the bending time-variant reliability analysis of corroded reinforced concrete beams. Material degradation due to chloride penetration has a significant influence on shear strength of RC beams. Failure mode will be from flexural into shear with service time growth of RC beams and the consequences of shear failure is much more serious than flexural. Numerical example shows that calculating the reliable index of flexural and shear strength, the bearing capacity of inclined section is the weakest section. Val (2007) shows that corrosion of stirrups, especially pitting corrosion, has a significant influence on the reliability of reinforced concrete beams.

The two failure modes are separate considered from most of the existing research results and considering the two failure modes of the reliability analysis of concrete structural system is very few and the correlation of resistance between them is not considered, it may in some degree underestimated the reliability of structure system. In addition, the time-dependent reliability analysis of flexural strength is much more than shear strength. Therefore, the authors try to taking into account the correlation of resistance of time-dependent reliability analysis of RC beams under shear and bending moment. The Monte Carlo method is used to evaluate the time-dependent reliability index due to chloride-induced corrosion under different failure modes. The correlation coefficient of different failure modes is discussed. Then the time-dependent reliability index of the girder is calculated based on the theory of Ditevsen. The results show that the flexural failure is the main failure under the slight corrosion, while the shear failure is dominated under severe corrosion; the time-dependent reliability index changed very little for the correlation of flexural and shear failure mode is small. Based on the theory of Ditevsen, the boundary between lower limit and upper limit is very small under the low correlation of failure modes.

The financial supports by China Natural Science Foundation Committee under Grant No.50878031 and 51178060 were much appreciated.

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Probabilistic deterioration models of reinforced concrete structures under complex environments

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ABSTRACT

Corrosion of steel rebar is an important factor which affects durability of concrete. The chloride in concrete structure is the main factor which leads to the corrosion of reinforced concrete in chloride environment and it is an important index that affects the service life of RC structure. Recently, a lot of researchers focused on the diffusion of chloride, and the theoretical model of Fick's second law of diffusion. However the Fick's second law of diffusion has many limitations on itself, it should satisfy the following assumptions: (1) Chlorine does not react with materials; (2) The diffusion coefficient is constant; (3) The material should be homogeneous. Actually, concrete is a porous structure of the mixture, and part of chloride in concrete are solidified by cement. The recent studies have shown that only free chloride causes corrosion of steel rebar, and the diffusion speed of chloride is variable over time. It is not a constant, and it is relevant to porosity of concrete, water to cement ratio and age of concrete.

Inspection and repair of deteriorating structures are scheduled based on assessment of time-varying structural reliability. Recent researches mainly focused on the ultimate limit state, but few academic work has been done on durability limit state, especially on assessment of the time-varying structural reliability under multi-factor corrosion induced by chloride. The main reason is lacking of a model that chloride ion corrosion under multi- factor corrosion is considered comprehensively. Based on the Fick's second law, a multi-factor model for chloride ion corrosion is developed. In this model, the influences of time, chloride ion binding, temperature, deterioration of concrete, prestress, cover thickness and chloride convective region are taken into accounts. Finally, the mathematic solution is given to the developed model.

Based on this improved model, a sensitivity analysis for the corrosion initiation time of the steel rebar was conducted, 1000000 of the random variable samples are generated by using random number generator of the Matlab program, and then they are taken into the sensitivity analysis function. The initiation time of corrosion is obviously reduced as the cover thickness depth increases. The degradation of resistance caused by chloride is directly proportional to the square of the cover thickness of concrete. A small increase of cover thickness can lead to delay corrosion initiation time largely and extend the service life of structure. So the increase of cover thickness depth could prolong corrosion initiation time of steel rebar effectively. Time of the steel rebar out of passivation will be prolonged as the critical chloride ion concentration increases. The corrosion initiation time will be delayed. It can be seen that the corrosion initiation time will be in advance a little when humidity has increased 20%, which means that the increasing of humidity could accerelate corrosion initiation time, but the effect is not obvious. An increase of water-cement ratio leads to the void age of concrete becomes larger. The speed which chloride moves into concrete depends on not only diffusion but also adsorption and penetration. It was found that temperature is the most sensitive factor for corrosion induced by chloride ion when compared with humidity and quantity of mixed slag.

Probability of initiation time and its reliability of an illustrative bridge are predicted based on the revised model using Monte-Carlo simulation. Then failure probability of the bridge is 0.4012 when it has been 11a of service. Since $\beta = \Phi - 1(1 - \mathrm{Pf})$, $\beta = 0.2502$. This means that the speed of chloride diffusion in improved model is slightly less than that in traditional model when the time-varying of chloride diffusion coefficient and other influence factors were considered.

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The role of modeling in the probabilistic durability assessment of concrete structures

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ABSTRACT

One of the methods of carrying out the performance-related design of concrete structures is the use of predictive models needed to specify the performance characteristics of the material used and to estimate how its resistance will change over time. Performance requirements are established by means of performance criteria and the associated constraints related to service life and reliability. Several current standards advocate probabilistic approaches, the utilization of mathematical models and the design of structures for durability, i.e. a time-dependent Limit State approach (LS) with service life consideration: ISO 13823 and the *fib*-Model Code.

In this respect it is important to develop a relevant assessment method, evaluation procedures and service life prognoses for concrete structures. In this area, mathematical modeling is often a useful tool together with the relevant limit states for the accomplishment of such tasks, utilizing stochastics.

The main criteria involved in selecting a suitable degradation model for each specific use with regard to the above-mentioned tasks are listed below:

- type of relevant limit state and exposure conditions;
- type of concrete (HSC, FRC, . . .);
- relevance of the model to the required design accuracy level and representation (1D, 2D or 3D; spatial and/or temporal variability; dependency of inputs; type of outputs);
- level of physical sophistication (macro-, meso- or micro-level);
- level of mathematical involvement;
- feasibility of model combination or their conditionality;
- availability of model data and their statistical characteristics, and/or the availability of relevant testing methods;
- labor and/or time consumption;
- level of model validation and calibration;
- availability of effective software tools.

The present paper comments on the aspect of time introduced together with the probabilistic safety format. The readiness of effective models for both the initiation as well as the propagation period may enable the verification of the SLS and/or ULS for concrete structures in different time steps considering the change of performance potential in time due to degradation. The point in space representation (1D) vs. spatial representation is briefly discussed.

A description of a software tool encompassing 32 models is provided. Some numerical examples are presented which consider the initiation period: (i) carbonation depth was analyzed for a cooling tower by four model variants and for the Olympic Tower (Gehlen & Sodeikat, 2002) by two models; the results were compared with on-site measurements, and, (ii) an example of the verification of models concerned with the effect of de-icing salts on a concrete bridge is shown. Computed chloride profiles using five different models are compared with profiles derived from experiments.

It may be concluded that the usefulness of effective degradation modeling as a method of assessment for durability, may bring positive economical and sustainability impacts; a broader choice of models is advantageous. Thus, the effective software tool described in this contribution may serve to facilitate the realistic decision making of designers and clients.

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Time-dependent reliability analysis of RC bridge under incomplete information

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ABSTRACT

Bridge engineering is a key component of infrastructure and their long-term performance in service is clearly of great social and economic importance. Evaluation of reliability and residual life of Reinforced Concrete (RC) bridges have attracted much attention in recent years as many of the existing structures have aged over years. Deterioration is considered as one of the major factors to affect structures. It is well known that the deterioration rate not only depends on materials compositions and construction processes, but also relies on the environment during the service time. Furthermore, truck loads are not constant with time. So the reliability level of structures is time-variant during the lifetime.

The influencing information of structural resistance cannot be exactly inspected due to limitation of the experimental means, time and space. The statistic parameters such as concrete cover, chloride concentration at concrete surface, threshold content of chloride ions, diffusion coefficient, corrosion initiation time and corrosion rate for reinforcement of structural resistance are not easy to be acquired completely. Reliability of existing RC bridges could be less accurately evaluated due to the lack of experimental means, the limited specimen, the simplified assumption, the lack of knowledge and the artificial mistakes. To overcome this problem, the evaluation of the reliability must be conducted under incomplete information. The incomplete information can brought the uncertainty consisting of fuzziness, randomness, and faultiness of knowledge. The faultiness of knowledge is the weak uncertainty, and can be incorporated into fuzziness and randomness.

In this paper, a novel time-dependent reliability analysis method is developed to consider fuzzy and random information. The probabilistic degradation models under fuzziness and randomness of reinforcement area are established. The relationship between area corrosion rate and yield strength is presented based on the experimental investigation on mechanical property of corroded reinforcement. The probabilistic degradation model of reinforcement yield strength is built in this paper.

A time-dependent reliability analysis model regarding corrosion-induced resistance degradation subject to fuzziness and randomness is developed. The total analytical method and process are demonstrated by an example of a RC bridge. According to aforementioned principle of the improved JC method in this paper, the computing program is developed based on language of Matlab. The FRTD reliability indices are obtained. Consider the curves in Figure 6, the FRTD reliability indices are in fuzzy state inside infimum and supremum. It is a particular example as conventional time-dependent reliability theory when $\alpha = 1$. At the any time, its fuzzy zone is increasing with decrease of the threshold value α. Between the initial 10 years, the FRTD reliability indices increase slightly due to increasing concrete strength and better reinforcing bar. However, the FRTD reliability indices decrease 10 years later.

During the course of the prediction of resistance level or time-variant reliability of the RC bridge, test facility, simplify hypothesis, human error, and knowledge faultiness can bring the uncertainty including fuzziness and randomness. Reported work presents a novel concept to study the RC bridge member resistance degradation, in which incomplete information are considered.

The probabilistic simulation shows that the effect of incomplete information on resistance degradation is significant. This method is very useful to incorporate fuzziness and randomness into time-variant reliability computations, to better predict the service-life of RC bridges, and to develop optimal lifetime reliability-based maintenance strategies for these bridges.

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Probabilistic analysis on shear resistance degradation of reinforced concrete beams induced by corrosion

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ABSTRACT

The resistance of RC structure will be deteriorated with time. Compared to flexural failure, the shear failure is more serious because it belongs to brittle failure. However, at present, relatively few academic works have been done on shear resistance degradation. Meanwhile, the existed resistance models for shear degradation are without consideration of uncertainty of parameters and so they only would be suitable for limited scope.

Thus, it is needed to focus on the probabilistic model of shear resistance deterioration. In this study, the deterioration of shear capacity is analyzed for corroded RC beam in chloride ion environment. The probabilistic models of shear resistance degradation for corroded RC beam are developed based on the Chinese and American Codes. A comparison of shear models between two codes is conducted. The values for the main descriptors of shear deterioration are inferred from the Monte Carlo numerical simulation. In the model, the contributions of concrete, stirrup and diagonal reinforcement are considered, and the time-variant effect of corrosion of steel rebar is incorporated. The sensitivity of concrete cover and chloride ion concentration is analyzed. The results show that the effect of computational models from various specifications on shear resistance is not needed to be considered. The influence of various factors on shear degradation is significant. For the RC member with the same stirrups level at ends and midspan, the effect of corrosion of diagonal reinforcements on shear degradation is more sensitive than that of stirrups.

The objective of this paper was to develop the probabilistic model of shear resistance degradation for corroded RC beam. This article considered the time-variant effect of various steel rebars in chloride environment and incorporated different calculation formulas in Chinese and American codes. Several important conclusions can be drawn from this study:

1) The calculation results of probabilistic models based on two Codes have little diversity for resistance degradation, and the max difference does not exceed 3%.
2) In chloride ion environment, the influence of corrosion of steel rebars on flexural capacity is more serious than that of shear capacity.
3) The various factors have different influences on shear resistance deterioration. The depth of stirrup cover and equilibrium chloride concentration are more sensitive to the initial time of resistance deterioration; and the depth of diagonal bar cover and equilibrium chloride concentration are more sensitive to the rate of resistance deterioration.
4) Compared with corrosion of stirrup, corrosion of diagonal reinforcement is more sensitive to the resistance deterioration for RC bridge with the same stirrups level at ends and midspan. Then, avoiding diagonal reinforcement bar corrosion is more important to defer the resistance deterioration.
5) In this paper, the probabilistic models of shear resistance are developed based on the current codes. The developed models can be used for maintenance and strengthenment for RC bridges subjected to corrosion in chloride environment.
6) The future work has been identified as follow: analyzing the resistance deterioration based on the calculation formula that the interaction of corroded steel bars and concrete are incorporated.

The financial support of the Natural Science Foundation of China under Grant No. 50878031 and 51178060 is greatly appreciated.

Probabilistic lifetime assessment of concrete structures under combined environmental attack

Organizers: R. Caspeele, N.D. Belie, C. Gehlen & S. Kessler

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Influence of sulphates on chloride diffusion and the effect of this on service life prediction of concrete in a submerged marine environment

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ABSTRACT

A lot of damage is reported for constructions in marine environments (e.g. bridge pillars, piers, wharfs, foundations, ...) due to the aggressiveness of sea water, containing chlorides and sulphates. A commonly used material for such structures is reinforced concrete. However, chlorides affect the durability of concrete by initiating corrosion of the reinforcement steel, and sulphates by deteriorating the concrete itself. In this research, single-ion attack by chlorides and multi-ion attack by chlorides and sulphates were compared with regard to the full probabilistic service life prediction of concrete structures, according to the Model code for service life design (*fib* bulletin 34 2006).

Four concrete mixtures were tested, two Portland cement concretes containing Ordinary Portland Cement (OPC) on the one hand and High Sulphate Resistant cement (HSR) on the other hand, and two Blast-Furnace Slag (BFS) concretes with 50% and 70% BFS as cement replacement, respectively. These mixtures were subjected to migration tests at five ages, according to NT Build 492 (1999) since this test provides us with the necessary input parameters for a full probabilistic service life prediction. Besides, a non-steady state diffusion test based on NT Build 443 (1995) was performed at 28 days to study the influence of combined environmental attack by chlorides and sulphates. Therefore, two extra test solutions were made next to the prescribed test solution. These test solutions contained 165 g/l NaCl as well, but the sulphate content amounted to 27.5 g/l Na_2SO_4 (18.6 g/l SO_4^{2-}) and 55 g/l Na_2SO_4 (37.2 g/l SO_4^{2-}). The results were used to modify the subfunction calculating the apparent diffusion coefficient in the *fib* bulletin 34 (2006) service life prediction model.

First, the ageing factor a was estimated based on the obtained migration coefficients. This factor considers the influence of ageing on chloride resistance. From the results, it is clear that the ageing factors for OPC, S50 and S70 are not significantly different. As a result, this parameter is equaled to the average value, namely 0.179 ± 0.012. ageing of HSR concrete has a smaller impact on chloride resistance. The ageing factor for HSR amounts to 0.072 ± 0.059.

Next, the transfer parameter k_t which takes into account the difference in migration coefficient and natural diffusion coefficient was estimated based on the experimental results. A constant k_t value was assumed for the Portland cement concretes as well as for the BFS concretes. The values amount to 0.487 and 0.172, respectively.

In the end, the influence of the sulphate content was translated into the service life prediction model by adding an extra parameter k_S. In order to find an appropriate parameter, the relation between free chloride diffusion coefficients and the sulphate content of the test solution was estimated:

$$\frac{D_{nssd,*}}{D_{nssd,0}} = k_S = 0.0065(SO_4^{2-}) + 1 \tag{1}$$

where $D_{nssd,*}$ is the free chloride diffusion coefficient after immersion in a combined solution with a certain amount of sulphates, $D_{nssd,0}$ is the reference diffusion coefficient after immersion in a 165 g/l NaCl solution and SO_4^{2-} is the sulphate content expressed in g/l solution.

Using the modified model, BFS concrete has a lower probability of failure at an age of 100 years than Portland cement concrete however it is more sensitive to combined attack of chlorides and sulphates.

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Effect of internal freeze-thaw deterioration on chloride migration in concrete

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ABSTRACT

Depending on the specific exposure conditions, reinforced concrete structures are simultaneously subjected to different physical and chemical loads. However, service life prediction of concrete structures usually focuses on only one damage mechanism. Therefore, the present contribution investigates the effect of freeze-thaw induced inner damage on chloride migration. Four different concrete compositions were exposed either to a defined freeze-thaw attack or stored in moist environment. Afterwards, the chloride migration coefficient of the specimens which were both exposed to freeze-thaw-cycles with 3% NaCl and not, was determined with a modified rapid chloride migration test, Figure 1.

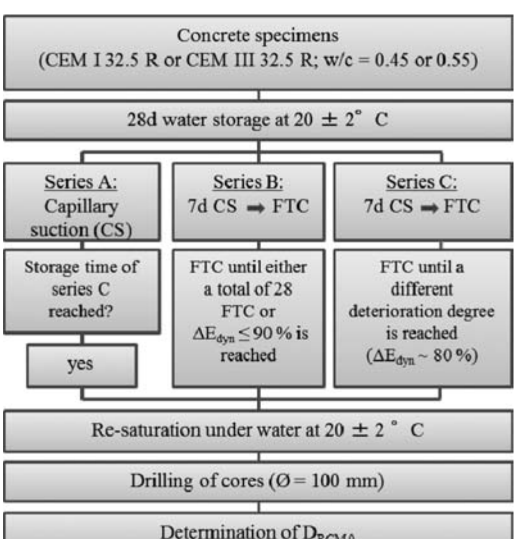

Figure 1. Overview of experimental procedure. Here, FTC is the Freeze-Thaw Cycle and $D_{RCM,0}$ is the chloride migration coefficient. To accelerate freeze-thaw damage no air entrainment was used in three of four mixtures.

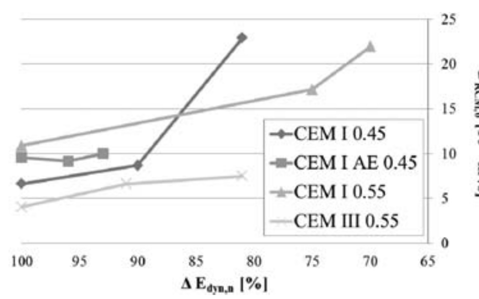

Figure 2. Mean value of the rapid chloride migration coefficient $D_{RCM,0}$ depending on the $\Delta E_{dyn,n}$ of damaged and intact concrete specimens, reference age 56 days.

The results clearly show the impact of freeze-thaw induced damage on the $D_{RCM,0}$, Figure 2.

The best performance under combined attack has concrete with blast furnace slag cement and low w/c ratio. However, minor changes on the chloride migration coefficient before and after freeze-thaw exposure were determined for concrete containing ordinary Portland cement and having a sufficient air void system. This was basically due to the high freeze-thaw deicing salt resistance of concrete containing air entraining agent ("CEM I 0.45 AE") which was therefore insignificantly damaged during freeze-thaw exposure. For concrete with ordinary Portland cement and a w/c ratio of 0.55, the $D_{RCM,0}$ increases with increasing damage by a factor of 3.5.

For field conditions, the initiation phase of chloride induced corrosion may be reduced considerably due to freeze-thaw induced damage. In addition, frost suction is the most effective transport of chloride ions and is so far neglected in current service life prediction models. The effect of combined attacks could be taken into account by implementing a further "damage factor" into current models. However, to quantify such a factor further research is needed.

Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Chloride diffusion tests as experimental basis for full probabilistic service life prediction and life-cycle assessment of concrete with fly ash in a submerged marine environment

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ABSTRACT

Nowadays, policymakers stimulate architects, engineers and contractors to use more environmentally friendly materials when designing and constructing buildings, bridges, etc. Since a lot of the currently available building materials are claimed to be 'green', this seems fairly easy. Often though, little quantitative proof of this actual environmental benefit is being provided by the manufacturers.

For instance, partial replacement of Ordinary Portland Cement (OPC) in concrete with Fly Ash (FA), an industrial by-product, is considered as a more than valid strategy to reduce cement related greenhouse gas emissions. As the amount of CO_2 emitted per ton cement produced is well-known today, a precise environmental impact calculation of concrete with a given (reduced) cement content seems perfectly possible. However, this simple estimation method, misses two key ingredients of the Life-Cycle Assessment (LCA) methodology: (i) an adequate Functional Unit (FU) choice and (ii) a justified allocation method for the by-product impact.

(i) The FU should include all relevant concrete aspects, being its strength, its durability and to some extent its workability. To take into account both strength and durability, it should comprise the concrete amount needed to manufacture a structural element with a given mechanical load and a predefined service life. The concrete's life span should be evaluated in relation to the latter, using probabilistic service life prediction models based on experimental durability tests representative for the application field (Van den Heede & De Belie, 2012).
(ii) A certain impact of the main product of FA, i.e. electricity produced by a coal fired power plant, needs to be allocated to FA. This effect cannot simply be neglected in the environmental evaluation of this by-product since the amount of CO_2 emitted per kWh of electricity is around 1.022 kg (Spath, 1999) and 1 kg FA corresponds with about 19.2 kWh of electricity (Van den Heede & De Belie, 2012). An allocation coefficient based on the economic (instead of the mass) value ratio between FA and electricity is preferred in order to guarantee the enduring use of FA as cement replacing material (Chen et al., 2010).

Both aspects (i) en (ii) were taken into account in this paper. It presents the results of a full probabilistic service life prediction of concrete with various amounts of fly ash (0%, 15% and 50%) assumed to be located in a permanently submerged marine environment. The limit state function used was similar to the one proposed by the Fib Bulletin 34 (2006). The experimental input to the model was provided by diffusion coefficients from accelerated (cf. NT Build 443, 1995) and more realistic chloride diffusion tests. The obtained service life estimates together with the strength classes of the studied concrete mixes were used as LCA input to quantify the Global Warming Potential (GWP) of the concrete amount needed to construct and maintain a centrically loaded column in a submerged marine environment for 100 years with an appropriate impact assigned to the FA by means of economic allocation. GWPs for concrete compositions with 15–50% FA were found to be at least 45–48% less than for traditional concrete.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Probabilistic lifetime assessment of concrete structures in consideration of combined deterioration mechanisms and singular risks

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ABSTRACT

With regard to sustainability it is essential to manage the lifetime of a civil structure based on an effective life-cycle management. Therefore in practice it is necessary to monitor and to manage the whole lifetime of a civil structure by means of elaborated tools to avoid cost-intensive maintenance measures and corresponding downtimes. In particular, for an effective life-cycle management it is important to predict the time dependent material degradation which results from different and complex deterioration mechanisms. However, while the knowledge about mechanisms of individual exposures is well developed there are substantial lacks in understanding the effects of combined deterioration mechanisms which occur in practice.

The procedure for durability design as given in the standards (DIN EN 206-1 2001, DIN 1045-2 2008) and in the Model Code for Service Life Design (*fib* bulletin 34 2006) is focused on the most decisive deterioration mechanism of concrete structures which are treated individually. The knowledge and the design methods established so far have to be extended from considering interacting instead of single actions and therefore to deterioration models which incorporate related effects.

Against this background a new approach for the realisation of a realistic probabilistic lifetime assessment was developed in consideration of combined deterioration mechanism related to chloride and carbonation induced corrosion. This approach involves time variant deterioration models concerning the durability of concrete structures as well as singular structural risks (e.g. cracks). In the course of numeric simulations (reliability analysis) the interaction of the combined working deterioration mechanisms are taken into account by implementing a scaling factor η which represents the modification of the material characteristics as a result of another exposure (Müller & Vogel 2011 and Müller, Vogel & Neumann 2011).

The computer based investigations proves the feasibility of the new approach. By this introduced method a first step is made towards an interaction model that is suitable, flexible and gives the possibility to be extended. The level of detail can be increased if necessary. Furthermore, the service life prediction can be extended by singular failure risks caused by structural safety problems.

Although the feasibility of the developed concept has been verified by a sophisticated computer based example, it becomes clear, that there is a high need for extended research. In addition to the probabilistic methods also the material technological aspects of interactions have to be subject of further research. For example, a quantitative specification of the characteristic values for the factor η can be derived from laboratory investigations to achieve the relevant material properties concerning to the chloride and carbonation induced corrosion.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Life time prediction for concrete repair measures

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ABSTRACT

Periodical repair and maintenance of concrete structure and buildings still gain increasing importance in practice. For repaired structure the prediction of the maintenance cycle length needs to take into account the condition of the original structure as well as the condition of already repaired parts. For reinforced concrete many prediction models are established. In contrast to that, models for rehabilitation measures are not available today.

The contribution shows an approach to describe the degradation of the rehabilitation measures repair mortar and surface coating and their failure modes. As a first step, the modes are described as independent of one another.

For repair mortar load-oriented and transport-oriented failure modes can be distinguished. Transport-oriented failure modes include damages caused by carbonation or chloride ingress for example. Failure of surface coatings covers the modes loss of adhesion, loss of layer thickness and loss of effectiveness. Loss of effectiveness is related to the transport-oriented failure of repair mortar. It describes how long coatings can protect the concrete from the ingress of harmful substances like chloride ions or carbon dioxide.

Chloride induced rebar corrosion or corrosion caused by carbonation of the concrete is described in models for reinforced concrete in (Fib 2006). The influence of rehabilitation measures can be integrated into these models.

For repair mortar the modified material behavior has to be taken into account. It affects the ingress into the concrete as well as the transport of the substances inside the material. As it is not possible to give a general statement about the influence of repair mortar on chloride and carbon dioxide ingress and transport testing is necessary. The Accelerated Carbonation Test (ACC) and the Rapid Chloride Migration method (RCM) can be used to classify the behavior of repair mortar.

Coatings influence the concentration of harmful substances on the concrete surface. A long-term test helps to describe the coatings' degradation. Specimens are exposed to natural weathering and their diffusion resistance is tested regularly.

The testing on both rehabilitation measures can help to summarize the materials into groups so that the influences of each group can be generalized. An aim in the future is also to consider the data for the models probabilistically due to the uncertainty of the material itself and the testing methods.

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Influence of combined mechanical and environmental loads on service life of reinforced concrete structures

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ABSTRACT

Reinforced concrete structures are generally exposed to mechanical loads and simultaneously to permanent or temporary environmental loads. So far in most applications all different actions have been considered separately. In this contribution first an overview on possible load combinations as shown in the Table is presented.

Then selected load combinations are considered in more detail. It will be shown that mechanically induced damage will facilitate capillary absorption and chloride penetration. The influence of an applied load on chloride diffusion will be outlined. It will be shown that both compressive and tensile stress have a significant influence on carbonation. Finally it will be shown that damage induced by frost action will accelerate carbonation and chloride penetration.

CONCLUSIONS

Based on the results described, the following conclusions can be drawn:

- The composite structure of concrete can be damaged by the application of tensile and compressive stresses. Micro-cracks are formed, which facilitate capillary absorption of liquids and penetration of gases.
- Mechanically induced damage accelerates diffusion of ions dissolved in the pore liquid. The apparent diffusion coefficient increases.
- Micro-cracks are also generated in the cement-based matrix of concrete by frost action. Exposure to freeze-thaw cycles facilitates ingress of chloride and increases the rate of carbonation.
- Carbonation of hardened cement paste coarsens the pore structure. This leads to an accelerated penetration of chloride.
- Service life of reinforced concrete structures will be significantly overestimated if the synergetic effects of different actions are not taken into consideration adequately.

Table 1. Different actions, which may act individually or in combination on reinforced concrete structures.

Mechanical load	Migration processes	Chemical reactions	Thermal effects	Hygral effects
Compression	Ion migration by convection	Carbonation	Heat of hydration	Drying
Tension	Ion migration by diffusion	Chloride in HCP	Thermal gradients	Capillary absorption
Shear	Osmosis	Sulphate in HCP	Thermal decomposition of HCP	Moisture diffusion
Torsion	Electro- osmosis	Ammonium in HCP	Frost action	Shrinkage and swelling
Sustained	Leaching	Hydrolysis	Freeze-thaw cycles	
Cyclic	Electrophoresis	Alkali-aggregate reaction		

Life-cycle cost optimization in cross asset management (ERA-NET road projects)

Organizers: A.J. O'Connor & S. Deix

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Bayesian identification of uncertainties in chloride ingress modeling into reinforced concrete structures

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ABSTRACT

Chloride penetration into Reinforced Concrete (RC) induces corrosion when a threshold concentration of chlorides reaches the reinforcement. Modeling chloride penetration into concrete is crucial for optimizing maintenance interventions of these structures (Duracrete, 2000). Chloride penetration is a very complex process and its modeling should be based on experimental measurements in order to make relevant lifecycle predictions and maintenance recommendations (Bastidas-Arteaga et al., 2011).

This paper proposes a Bayesian probabilistic to identify the input parameters for chloride penetration models. For instance, the European Union project (Duracrete, 2000) proposes the following equation to compute the chloride content C at depth x and time t:

$$C(x,t) = C_{s,D}\left[1 - erf\left(\frac{x}{2\sqrt{k_e k_t k_c D_o \left(t_o/t\right)^{n_D} t}}\right)\right] \tag{1}$$

where $C_{s,D}$ is the chloride surface content computed for this model, k_e is an environmental factor, k_t is a factor that considers the method used to determine the diffusion coefficient D_o, k_c is a curing time factor, t_o is the time for which D_o has been measured and n_D is the aging factor. Some parameters of eq. (1) could be considered as constant taking into account the characteristics of the structure – e.g., exposure zone, method to estimate D_o and curing time. Then, k_e, k_t and k_c could be considered as constant and there are three random variables to identify: D_o, $C_{s,D}$ and n_D. Assuming that D_o, $C_{s,D}$ and n_D are independent random variables, the probability of assessment of a chloride concentration at a point x and a given time t, $p(C(x,t))$ becomes:

$$p(C(x,t)) = \sum_{D_o, C_{s,D}, n_D} p\left(C(x,t) \middle| D_o, C_{s,D}, n_D\right) p\left(D_o, C_{s,D}, n_D\right) \tag{2}$$

where $p(C(x,t)|D_0,C_{s,D},n_D)$ is a conditional probability that must already be known. The determination of this conditional probability is carried out by using the Netica® software. Once $p(C(x,t))$ is determined, *a posteriori* distributions for D_o, $C_{s,D}$ and n_D can be calculated from a set of measurements of chloride profiles.

The final part of the paper presented the application of Bayesian inference to the identification of random variables based on measurements on the Ferrycarrig Bridge in Ireland. This structure, built in 1980, is located in a marine environment and several chloride profiles were measured in 2007. These results were used for Bayesian identification. Table 1 shows the *a posteriori* mean and standard deviation of $C_{s,D}$. It is observed that after five iterations, the mean and standard deviation gradually lead to constant values. However, the results will be more accurate if the number of iterations increases. Similar results were found for D_o and n_D. The type of distribution was determined taking into account both physical considerations and the log-likelihood estimated after 5 iterations. It was found that $C_{s,D}$ followed a lognormal distribution, D_o could be represented by gamma or lognormal distributions and n_D followed a beta distribution.

Table 1. *A posteriori* results for $C_{s,D}$.

Variable	Iteration	Mean	Std. dev
$C_{s,D}$ (% per wt. of concrete)	0	0.0809	0.01
	⋮	⋮	⋮
	4	0.0801	0.0064
	5	0.0802	0.0064

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From road asset management to cross asset optimization procedures

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ABSTRACT

Cross Asset Management in Transport: Asset management is an important aspect of effectively maintaining road infrastructure's value and keeping it in safe and reliable condition. It comprises optimized coordination of all maintenance activities on the different sub-assets according to the expectations and requirements of road users, road operators, road owners and all affected stakeholders. It is a complex process which needs flexible and adaptable methods, the experience from the road owners and operators as well as a clear definition of the stakeholders' requirements.

An innovative approach is the development of optimized procedures for cross asset management of the total road infrastructure considering all asset types jointly. This is somehow different to the traditional approach in asset management where models, monitoring and measurement data are used to assess condition levels for each sub-asset more or less separately. Overall life-cycle costs, performance and asset value are of secondary importance within many procedures.

This paper presents an asset management approach considering all influencing parameters (e.g. age, environment, materials, deterioration processes, loadings, maintenance policies, etc.) and lessons learned from practical experiences. Different sub-assets (e.g. pavements, tunnels, bridges, culverts, walls, noise barriers, variable message signs, drainage systems, etc.) are proposed for a combined cross asset framework. The main benefit of introducing such a holistic road asset scheme is to save monetary and non-monetary resources and to minimize negative impacts from socio-economic, technical and environmental points of view.

If you can't measure it, you can't manage it! – is a famous quote (origin unknown) often used to describe the necessity for measurable indicators in a management process. And this holds true for asset management in general and for cross asset management in particular. In common practice each single group of assets is measured, monitored and managed individually. For this purpose specific management systems, like PMS, BMS, etc. were developed. Those systems enable a selection of appropriate maintenance solutions/strategies by using different analysis methods (e.g. prioritization, Life-Cycle Analysis (LCA), Life-Cycle Cost Analysis (LCCA), etc.). The method used to define a recommended maintenance treatment is strongly dependent on the availability of technical data and the predictability of those characteristics which describe the deterioration of the asset or elements. But the separation of asset management into different sub-management systems (often held under the responsibility of different management departments) might be seen as the source for all difficulties in identifying appropriate cross asset management methods. However, the fact that indicators for each sub-asset are available helps in setting up a cross asset management system. In general, two approaches were identified:

1. A strategic, guided decision-making to optimize maintenance planning on the network level (top-down approach).
2. Coordination of maintenance activities planned for each sub-asset on a road segment to avoid prolonged user disturbance and other adverse effects (bottom-up approach).

Both approaches are valid and consistent in finding an optimum solution based on the preconditions (i.e. strategic requirements, regulatory and legal framework). The difference merely lies in the way the optimum solution is identified. The two approaches are used to visualize the different concepts and to help road operators and road authorities identify the appropriate concept fitting their requirements. The authors assume that these appropriate concepts are merely located in between the bottom-up and top-down approach.

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Development of procedures for cross-asset management for road infrastructure

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ABSTRACT

Cross-asset management is not a 'one-stop-shop' solution, but rather a best practice, robust methodology through which the entire road transportation network may be maintained and operated in a safe and efficient fashion with an emphasis on cost minimisation. The term 'cost' as employed here, does not necessarily mean the liquidity at any point of time and covers a broader financial aspect. Some specific characteristics of cross-asset management include high-quality information on asset inventory, the condition of such assets, the management strategies of such assets and customer perceptions. Additionally, unlike a standard management system of a certain type of sub-asset (e.g. a Bridge Management System), cross-asset management explicitly encourages cost-effective data collection, monitoring and target oriented asset appraisal. Two key tasks in the definition of cross asset management protocols lie (i) in the provision of monitoring requirements and (ii) in the development of the procedures for cross asset management themselves. The former rely on the ability of the protocols to translate, in a broadly understandable format, typical monitored parameters into performance indicators, and upon agreement concerning appropriate intervention levels. Whilst, the latter relies upon detailed consideration of (a) the organisational requirements of road administrations, (b) the technical requirements of sub-asset management and cross-asset interdependencies, (c) the availability of, and agreement on, Key Performance Indicators (KPI's) and their associated monitoring requirements as well as (d) stakeholders' expectations. Only when these two key tasks are completed is it possible to truly provide procedures for cross asset maintenance optimisation. The aim of this paper, is to detail how these two tasks are being addressed in the context of the EraNet road funded project PROCROSS.

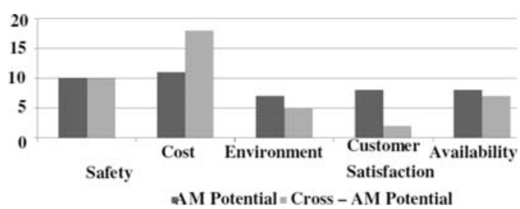

Figure 1. Relevance of stakeholder objectives for Asset Management (AM) and potential for Cross – Asset Management (Cross-AM).

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A holistic life-cycle approach for traffic infrastructure

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ABSTRACT

UN climate summits and Intergovernmental Panel Of Climate Change alerts the world to reduce greenhouse gas emissions and to increase environmental awareness. Therefore, scientific institutions and engineering practices are aimed to push life-cycle engineering. In the past many research projects focused on sustainability of building structures but not of infrastructure projects and related structures.

Infrastructure planning is a long-term process which should be accompanied by a holistic assessment considering all aspects of sustainability in the whole life-cycle. The phases of a life-cycle of infrastructure projects include planning, construction, operation, surveillance, maintenance and demolition.

We have started to assess the CO_2-Equivalents (CO_2E) of processes of the planning and construction of roads, bridges and tunnels. The influences of longitudinal gradients, cross sections and construction methods have been considered.

Moreover every project impacts on traffic. Traffic interferences lead to an increase of emission, so called indirect effects. These indirect effects have to be taken into account over the whole life-cycle. As an example an existing alignment and different alternatives to same have been analysed. To allow a better understanding of the relevance of CO_2E-emissions of construction processes, the CO_2E-emissions have been evaluated monetarily using the typically value of 70€/t-CO_2E according to the Federal Environment Agency (Umweltbundesamt) [UBA, 2007].

Initial results show a small influence of CO_2E-emission costs of production processes in comparison to life-cycle costs (Fig. 1). Because of different cross sections and construction options the life-cycle costs have dispersion. Indirect effects causing huge CO_2E-emission costs have a large range of dispersion according to technical quality of construction, process quality and traffic flow.

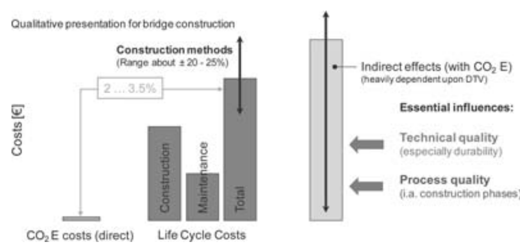

Figure 1. CO_2E-emission costs for bridges.

To get sustainable road, tunnel and bridge sections we have to look at the whole life-cycle. But most decisions have to be done at an early stage of the project. With this background, the aim is to get an evaluation concept for a holistic life-cycle approach.

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Cross asset management procedures in practice

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ABSTRACT

The Asset Management System of the Austrian motorway company ASFiNAG is used for a net-wide objective maintenance planning process in consideration of different aspects and demands on different decision levels (project-level, network-level). It is applied on the whole motorway- and expressway-network with a total length of more than 2,200 km.

The primary aim of this Asset Management System is to allocate a comprehensive basis for an objective maintenance planning process of pavements, bridges, tunnels, galleries, culverts, noise protection walls, retaining walls and gantries. To guarantee a high availability and road user-orientation the different management tasks are coordinated in an optimized process which is based on different asset specific maintenance management procedures but also on procedures for a holistic cross asset management (see Figure 1).

Cross Asset Management needs to combine the different maintenance needs of the single assets and the general, strategic requirements on network level. Thus, it is necessary to implement and work with procedures, which enable on the one hand a clear definition of technical maintenance needs of each single asset and on the other hand to assess the consequences and effects in doing maintenance in relation to the given strategic requirements (budget, availability, safety, etc.). From the ASFiNAG point of view it is essential to blend the specific technical maintenance needs of single assets or sub-assets (Bottom-up approach) with the general strategic targets of the whole network (Top-down approach) as described in Deix et al. 2012. Both approaches, “Bottom-up” and “Top-down”, reflect the different requirements from the object (technical or asset) level but also from the strategic (policy) level (see Figure 2).

Figure 1. ASFINAG Asset Management Cycle (Weninger-Vycudil A. et al. 2010).

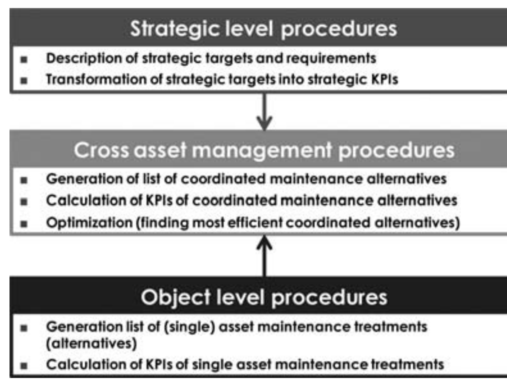

Figure 2. Cross asset management procedures within the asset management approach (PROCROSS 2012).

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Cost effective maintenance to supply end user value: Visionary or utopian

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ABSTRACT

Maintenance managers of civil infrastructures are facing the problem to deliver, and also to account for, End User Service Levels (EUSL) defined at an abstract level; e.g. availability, comfort, safety, while their span of influence involves maintenance measures, which are at a much more technical level. In ASCAM, a project of the ERANET Road Program; 'Effective asset management meeting future challenges' a framework was developed in which the effect of maintenance scenarios on the technical level on the EUSL are quantified.

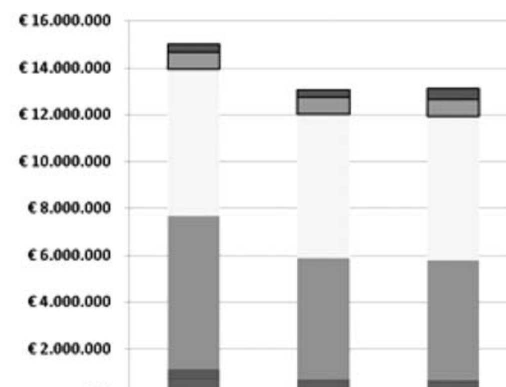
To do so, two types of relationships need to be developed. The first is the relationship between measures and the asset condition and asset condition degradation. The second relation is between the asset condition and EUSL. With modern societal cost benefit models next, network value can be quantified and compared to the costs of the measures taken. In principle this framework thus makes it possible to perform cross asset maintenance optimization in terms of end user values. The framework will enable policy makers, maintenance managers and their specialists to communicate on different levels and to overcome the boundaries between different fields of knowledge.

Furthermore current maintenance practice for pavement, structures and road equipment were assessed from a technical point of view. Principles were identified and data on condition, measures, costs and EUSL were gathered.

From a small cross section of this information and the constructed EUSL, a numerical implementation of the framework was made as a demonstrator, a screenshot of possible demonstrator output is shown in figure 1.

The demonstrator enables the comparison of different maintenance strategies for cross-asset management on a mutual scale taking EUSL costs into account in relation to the intervention costs.

Comparison with current asset management practices shows that data and principles used in



	scenario 1	scenario 2	scenario 3
measures (I)	358104	335387	464398
measures (D)	717026	718762	748356
noise	3152	1202	1107
emission	6262194	6153057	6142174
safety	6575131	5175638	5124637
accessibility	389613	42839	56496
comfort	724416	650026	596823

Figure 1. Summary of realized costs over 40 years; Costs decomposed per EUSL and intervention cost per scenario.

maintenance management are applicable in combination with the framework.

Using the principles of the framework will also enable cross-asset optimization, enabling asset managers to prioritize between the three types of assets (pavement, construction, re), because the effects of maintenance of the three types of assets can be seen on EUSL.

This paper demonstrates that cost effective maintenance to supply end user value is a vision. Important to come to such maintenance is a strong belief in bringing value based thinking and method to our civil infrastructure community.

Life-cycle cost analyses

Organizers: H.G. Jodl, P. Veit, J. Glatzl & T. Simandl

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)
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Life-cycle cost analysis considerations for accelerated bridge construction

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ABSTRACT

The ongoing growth in the traffic coupled with the demand of public for uninterrupted travel and improved safety, led to the evolution of the state-of-the-art bridge construction technology known as Accelerated Bridge Construction (ABC). Among other benefits, ABC possesses improved safety, improved construction quality, and reduced construction times, compared to conventional construction methods that required months of restricted traffic movements. The decision of utilizing the ABC technologies is primarily based on funding availability due to the increased initial costs. The decision of utilizing the ABC technologies is constrained due to the increased initial costs. Several ongoing research efforts are focused on formalizing the decision-making process to justify ABC technology selection in the US. Yet, these processes do not include the life-cycle cost calculations, rather, they anticipate the life-cycle cost of bridge system built using ABC technologies to be less than the bridge system built using conventional construction techniques. During our study we identified that the durability performance of some of the recently constructed bridge structural systems using the state-of-the-art ABC technologies is below expectation. This is a concern as the qualitative anticipation of the life-cycle costs in decision-making process may often lead to inferior decisions.

Our research developed an informed decision-making model that can amalgamate both qualitative and quantitative parameters, and help the decision makers with site specific information to make consistent judgments. The model evaluates among the ABC and conventional construction techniques for a specific site, to select an optimal construction alternative. Analytical hierarchy process coupled with a legitimate mathematical formulation has been used in the model. The model estimates some of the quantitative parameters to assist the decision makers during their judgment process. Among the quantitative parameters is the life-cycle cost parameter. For calculating the life-cycle cost for the contending alternatives, various life-cycle models were reviewed (Ehlen & Marshall 1996, Walls & Smith 1998, Furtner & Veit-Egerer 2010).

The *deterministic* and *probabilistic* methods of life-cycle cost analysis were implemented with information from literature and few conjectures. The other *state-of-the-art* life-cycle cost analysis model by Furtner & Veit-Egerer (2010) was found impractical, at this time, for implementation due to lack of performance data of bridge systems constructed using ABC technologies. Our study acknowledged database requirements for implementing the available life-cycle analysis models on the ABC technologies.

Evaluation of life-cycle cost using deterministic method generated discrete life-cycle cost values and provided less opportunity to the decision makers for contemplating their decision. The probabilistic method provided statistical inferences for the life-cycle cost of contending alternatives along with associated probability. This facilitated the decision maker with a vast arena of possible inferences during his/her judgment process.

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Life-cycle financial modelling of long term infrastructure projects "PPP-BOT Projects" under uncertainty and risk

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ABSTRACT

Economic modelling and risk analysis are important tools in the appraisal of long term infrastructure and revenue-generating PPP-BOT projects. Analysis of a project is usually performed by considering a number of different scenarios and risk assumptions. However, proposed method and available software for this purpose is frequently project specific and provides only a summarized level of detail which relies mainly on using time- and/or quantity-related costs/revenues. In the face of increased project complexities and diverse estimating tools (and/or models) such approaches appear to be less effective, hence the requirement for a generalized model (and/or software). The paper will attempt to develop a generalized model for analyzing life-cycle financial modelling of PPP-BOT projects, which will provide the much required level of detail. In addition to sensitivity analysis of various risks at any specific time of the project, the proposed model is expected to provide their suitable management and control mechanisms. Various performance measures thus formulated, will together build the general economic model for PPP-BOT projects. Furthermore some new criteria more over the common criteria for financial evaluation will be introduced and developed for PPP-BOT projects in this model. Dealing with uncertainty and risk in the lifespan of these projects is the last part of this paper which will be addressed by using decision making under uncertainty and risk methods via a case study.

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Life-cycle cost management for newly constructed infrastructure

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ABSTRACT

Life-Cycle Costing (LCC) is a methodology for assessing the total cost performance of an asset over time. Life-Cycle Cost Management (LCCM) stands for the management of cost effects over the life-cycle in processes en decision procedures within an organisation. This paper summarises opportunities for the use of LCCM in construction processes and describes how these are currently implemented at the Dutch Ministry of Infrastructure and Environment (Rijkswaterstaat). By implementing Life-Cycle Cost Management (LCCM) Rijkswaterstaat strives to deliver a desired performance for minimal cost over the life-cycle.

Traditionally design processes are focussed at creating new infrastructure for lowest building cost. However, the cost of mobility are determined by both founding cost and exploitation cost. A focus at only the founding cost leads to ineffective investments in mobility. LCCM can provide a solution to achieve a better value for money.

The greatest benefit of LCCM can be obtained in the initiation phase of building projects. Benefits gradually decrease with each step towards the exploitation phase (Fig. 1). Nevertheless, LCC is possible in all phases of the life-cycle of infrastructure. In most cases simple, deterministic LCC calculations are sufficient to achieve a significant optimisation. Therefore LCC can, and should be part of every day life of employees of public entities that have to decide and justify spending taxpayer's money. In order to achieve a more optimal value for money understanding is needed about the underlying economical principles and technical properties, but also about human behaviour and organisational aspects.

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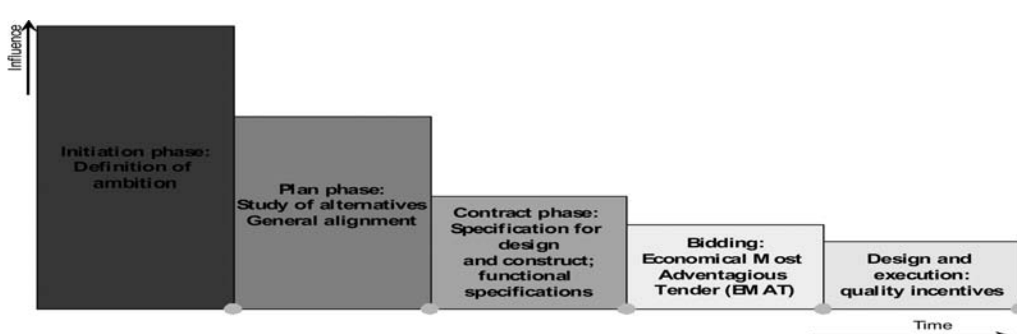

Figure 1. (Indicative) figure of the influence on LCC in subsequent project phases.

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Monitoring of an LCC-oriented maintenance of rail routes

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ABSTRACT

The Republic of Austria, represented by the Federal Ministry of Transport, Innovation and Technology (BMVIT), concluded a multi-annual financing contract with the infrastructure manager ÖBB-Infrastruktur AG defining the quantity, quality and cost-effectiveness of the rail infrastructure to be operated.

One of the aims is to ensure an asset management optimized throughout the whole life-cycle. In order to enable supervision of the adherence to this aim, SCHIG mbH has developed a system for the monitoring of an LCC-oriented maintenance of rail routes for that group of assets which is essential for the use of rail infrastructure, i.e. the rail routes (= permanent way consisting of subgrade, ballast, sleepers and rails with fastening material).

The monitoring system reveals in which phase of the life-cycle the track of the respective line of the framework planning is and allows conclusions as to the LCC development: High repair expenses, for example, resulting from short repair intervals, which do not lead to a sustainable improvement of track bed quality, concurrently with an old age of the permanent way suggest that a reinvestment for the respective asset will be necessary. If a reinvestment was made, this should lead to a significant decrease in repair expenses, in connection with a significant and sustainable increase in track bed quality.

Moreover, the revenues generated by ÖBB-Infrastruktur AG from marketing the products for the respective framework planning line can be opposed to the expenses incurred by that. In connection with the rail transport services provided on this line, these pieces of information allow an assessment of the cost-effectiveness of the relevant rail infrastructure. In addition, a benchmark can be established for those parts of the rail infrastructure operated by ÖBB-Infrastruktur AG which are similar concerning their load category and thus comparable.

Such a system for the monitoring of an LCC-oriented maintenance shall put the public provider of funds in the position to assess the management of the infrastructure manager not exclusively on the basis of the annual financial statements to be drawn up according to the legal requirements but, beyond that, to render apparent if the management of the infrastructure manager has taken the right measures at the right time in terms of a sustainable LCC-optimized corrective maintenance.

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Challenges for RAMS/LCC analysis of railway rails

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ABSTRACT

Railway infrastructure is a complex system with high investment costs. After the installation it is very difficult and expensive to modify the initial concept. Thus a design decision should be done in an early stage of the planning process. After the investment for track construction and components, the track has to be maintained during operation. These actions are also important cost factors, thus applied technologies and strategies should be conceived to realize a reliable and safe track operation. Therefore it must be the aim of the railroader to choose track components which are Reliable (R), ensure maximum track Availability (A), are easy to Maintain (M) and make a Safe (S) and cost efficient track operation possible. This paper describes the principle of RAMS and LCC analysis for the central track component "rail". Finally an example for RAMS analysis is presented, that makes a comparison between two rail grades R260 and R350HT.

The reliability of the system rail depends on the inspection and maintenance intervals and also from the time of replacement. The total service life of rails is not time-dependent but it also depends on axle load, on the number of cycles and on the loading conditions: For the RAMS calculation the Mean Time Between Failures (MTBF) for different types of failures is indispensable.

An exact calculation of the MTBF for the system rail is only possible to a limited extent, because the progress of any occurring damage is determined by the quality of the entire track properties.

Based on extensive field experience and expert estimates, maintenance data (MTTM, MTTR) and failure rate for different failure mode are generated for the steel grade R260 and R350HT. Head hardened rails R350HT show in average 3 times higher resistance against wear and RCF damages than the R260 steel grade. This leads to less maintenance activities, especially grinding expenditure (Girsch 2004, Heyder 2005). These data are used in order to make a comparison between the RAMS parameter of the grade R260 and heat treated grade R350HT.

Table 1. RAMS parameters for rail grades R260 and R350HT.

Category	Item	Unit	R260	R350HT
Reliability	TFR****	failures/h	100	71
Reliability	MTBF	years	100	141
Availability	Availability	%	99.49	99.67
Maintainability	Preventive DT*	h/year	100	55
Maintainability	Corrective	DT h/year	100	87
FMECA**	max. RPN***	[–]	100	80

*DT: Downtime.
**FMECA: Failure Mode, Effects and Criticality Analysis.
***RPN: Risk Priority Number.
****TFR: Total Failure Rate.

Typical European conditions for loads, maintenance strategies and track characteristics were used for this current RAMS calculation as ex. The results, which were normalized to the grade R260 rails representing 100% are shown in Table 1. The significantly reduced requirement for preventive maintenance shows one of the relevant advantages of the head hardened rails R350HT (Heat Treated) compared to the steel grade R260. The other RAMS parameters show smaller differences, but they are still significant. The availability for both steel grades is similar.

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Life-cycle costs of tunnels

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ABSTRACT

The requirement of sustainability has increasingly come to characterize our personal and professional lives in recent years. The fundamental concept of sustainability is based on the notion that the current needs of society are required to be met only to the extent that future generations need not suffer, and that our present needs should be fulfilled. Along with ecological and social issues, there are also the economic points to be considered. Up until now, these sustainability principles have only been taken into account in the real estate industry. For infrastructure or engineering projects, these considerations are more of an exception, or still in the stages of development.

The present paper is concerned with the economical principles of sustainability. It is not sufficient merely to use the planning and construction costs incurred up until the initial use of a structure in the evaluation. The costs arising from the utilization of a structure can even exceed the original construction costs, and are therefore an essential part of Life-Cycle-Cost (LCC)-calculations. In the case of tunnels, there has hardly been a cost report that has given a full account of a project's complete life-cycle costs in its calculations. The cost-optimizations that have been carried out have therefore merely focused on the area of initial costs, while the resulting follow-up costs have largely been neglected. Necessary decisions were made on the basis of construction costs in the absence of further knowledge concerning the evaluation and quantification of additional decision criteria.

This paper describes the necessary foundations for the LCC-calculations of tunnels and provides a flow diagram of the strategy for doing so.

A consistent and structured acquisition of initial and follow-up costs is not yet possible in the absence of binding rules and guidelines. The first step must therefore be the creation of such cost structures. In terms of the construction costs, this means that temporary costs such as building site facilities and tunnel drilling can be distinguished from the permanent costs. The permanent (structure related) costs are to be differentiated based on the life span of the individual components. In the case of the utilization costs, the operating and maintenance costs – which repeat themselves in a regular cycle – and the repair and modernization costs – which are included as singular expenditures in the calculation – are to be recorded.

After the cost structure has been determined, the costs of existing constructions in terms of production and operation can be evaluated. The first step will take road tunnels into account, while other uses such as for railway traffic and utility tunnels will be examined at a later phase. The goal is to determine comparable characteristic cost values. The result is a tool with which the effects of current investment decisions on future costs can be determined.

This paper shows how the previously used concepts of life-cycle calculations, which have been used primarily in the area of residential and commercial construction, can be transferred to the evaluation of tunnels. The aim is to develop a generally applicable cost structure and characteristic cost values that will allow for carrying out the life-cycle costs calculations in an easy and comprehensible manner.

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Utilization times of railway bridges

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ABSTRACT

The utilization times of bridges are relevant for life-cycle analysis, for determining the redemption fees in case of change of the bridge owner and/or operator and for the budgetary planning of the infrastructure operator. The values for the theoretic utilization time given in relevant literature vary over a wide range; the basis of these values could not be identified. The same applies to the maintenance cycles which are the basis for the theoretic utilization time. In this study a base value of the actual utilization time is determined on the basis of the renewal plan of the ÖBB railway bridges for certain superstructures and substructures. Therefore, it was particularly necessary to know the exact reason for the renewal of the bridge.

During these basis value determinations and because of practical experience it showed that certain parameters influence the bridge utilization time. The parameters which affect the utilization time were studied and – when relationships could be detected – evaluated. These parameters are the network category, the type of deck for steel plate girders and steel framework structures, the age difference between superstructure and substructure and the drive-through use of the bridge for the substructure. The building materials technology and environmental changes were also considered with the factor for the construction period.

The theoretic utilization time of the superstructures determined in this work is calculated according to Formula 1:

$$m = (m_{basis} * k_{network} * k_{deck} * k_{year}) \tag{1}$$

where m = theoretic utilization time of new bridges; m_{basis} = basic value of the utilization time; $k_{network}$ = factor for the consideration of the network category; k_{deck} = factor for the consideration of the type of deck of steel superstructure; and k_{year} = factor for consideration of the development of building materials and environmental conditions at the present time.

The resulting theoretic utilization times of new bridges in the core network are:

- 105 years for reinforced concrete superstructures
- 110 years for composite superstructures
- 120 for steel plate girders with closed decks
- 125 for steel framework structures with closed decks

The remaining utilization time of superstructures is a function of the calculated theoretic utilization time, the age and the current state of maintenance of the superstructure as seen in Formula 2:

$$n = f(m_{bth}, A_{TW} \text{ and } ZK) \tag{2}$$

where m_{bth} = calculated theoretic utilization time of the existing structure; A_{TW} = the age of the superstructure; and ZK = the actual maintenance condition classes of the structure. The calculated theoretic utilization time results from Formula 3:

$$m_{bth} = (m_{basis} * k_{network} * k_{deck} * k_{diff} * k_{year}) \tag{3}$$

where k_{diff} = factor for the consideration of the age different between superstructure and substructure.

This study also points out that for an LCC analysis not only the utilization time of individual components is essential – superstructures, substructure and equipment – but these must also be matched to each other. Only then an optimal LCC solution is possible.

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Stochastic cost estimation for large infrastructure projects: A computational framework

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ABSTRACT

The realisation of large infrastructure projects appears as a challenging venture, not only from a technical point of view, but also in terms of methodical and effective cost evaluation. Numerous decisions throughout the phases of planning and construction, as well as during the project's service life, are firmly related to efforts to prevent, mitigate or address possible adverse events. The twofold requirement to quantify subjective perceptions of risk and to elabourate the resulting data in a mathematically consistent fashion may prove to be extremely demanding. The primary aim of the present study is the representation of risk factors as individual cost elements, along with a computational setup wherein these elements can be effectively combined in order to yield reliable total cost estimates. Emphasis is drawn on quantitative decision-making under conditions of high uncertainty, complexity and partial information. The total cost is not viewed as a mere sum of fixed and independent values, but rather as the outcome of a stochastic framework, where risk dependencies are also considered. The conceptual model is inspired from the work conducted on the risk analysis of the Brenner Base Tunnel, one of the most important construction projects nowadays in Europe (Bergmeister, 2011).

Estimating the cost of a complex infrastructure project requires the consideration of a multitude of factors. The total cost *TC* can be schematically expressed (ÖGG Guidline, 2005) as the sum of three principal terms, namely the base cost *B*, the risks part *R*, and a term *F* encompassing financial issues (inflation, exchange rates, present value adjustments, etc.):

$$TC = B + R + F \tag{1}$$

The present study aims to clarify basic aspects pertaining to the calculation of the risk term *R*. The base cost *B* expresses the cost that will occur "if all goes as expected" (Reilly & Brown, 2004). Obviously, this definition cannot account for planned activities with variable cost; assuming fixed cost values for such events is not the expected, but rather a favourable extreme. As the analyst attempts to account for variability, the borderline between base cost and risk cost becomes blurred, and the issue of a formalised contingency consideration policy begins to flesh out.

The beta distribution is employed for the stochastic representation of risks. For each individual risk, the minimum, the most likely and the maximum cost impact value, are obtained through expert judgement. An additional parameter is used, aiming to account for the assessment's uncertainty, by means of dispersion considerations. Associations between risks are defined upon sequential relations, causal considerations, logical assumptions, historical data and expert opinions. To that end, a value for the correlation coefficient *r* is assessed, when appropriate, in order to represent weak, moderate and strong dependence (Yang, 2006). As a representative risk impact metric, the 95 percentile of the total cost, frequently referred to in Financial Engineering as Value-at-Risk (Connor et al., 2010) is calculated.

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The calculation of life-cycle costs for road tunnels under the influence of uncertainties

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ABSTRACT

Since the mid 1970s, a couple of Western European countries have intensively started to integrate road tunnels into their road network. At that time, the road network in most countries has largely been completed, so that tunnels either help to upgrade the effectiveness of the existing road infrastructure or to eliminate local traffic bottlenecks.

Peil & Hosser (2007) predict that the operation of facilities will be faced with a fundamental change. The reason for this development will firstly be the replacement of aged existing structures, and secondly the advancement of technical standards. The increase in the total numbers of tunnels on one hand as well as the need for re-investments in existing tunnels justify a philosophy change towards the life-cycle concept for road tunnels. Addressees of such a life-cycle concept are public authorities as well as private companies participating in Public Private Partnership projects.

If the operation of a tunnel is based on the life-cycle concept, decisive key issues have to be stressed in very early project stages. While some factors either affect initial or follow-up costs, others influence both portions, initial as well as follow-up costs. If the planning of a project is at the very beginning, a substitution of initial by follow-up costs – and vice versa – results in cost savings. In contrast, if a project is in an advanced stage, the set of mutual dependencies is usually limited to aspects concerning operation, repair and maintenance.

A life-cycle cost tool for road tunnels, which is based on a spreadsheet analysis, has been developed by the authors. The approach makes use of the cost breakdown structure by applying full cost accounting. Costs associated with all materials and components necessary for the operation of a tunnel have to be assigned to their appropriate time of occurrence.

The entire service life of the tunnel is usually governed by its structural integrity and the magnitude might be in the range of one century. In contrast to that, the useful lifetime of technical components is commonly less than 20 years (ABBV, 2010).

The design and the operation of a road tunnel are characterized by process-specific properties and require the application of various materials and technical components. All materials and components that are to be processed during the construction phase can be divided into two main groups: The first group comprises components and materials for assembling the shell construction of the tunnel, whereas operational components and technical devices are to be assigned to the second group.

From the mentioned aspects, it can be concluded that several parameters in a life-cycle cost analysis come with uncertainties. Uncertain parameters can be identified within those groups:

- Group 1 considers the theoretical service life-span of materials and components,
- Group 2 contains cost estimates for installing and maintaining materials and components,
- Group 3 encompasses the boundary conditions concerning investment appraisal.

The procedure for calculating the life-cycle costs of tunnels is based on a standardized routine, which has to be repeated as often until all necessary components have been considered. All costs to be implemented in the tool are valid for one specific reference date. Usually the reference date is similar to the beginning of the life-cycle analysis. This date mirrors either the completion of a tunnel after construction or the beginning of a philosophy-change for an existing tunnel.

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Management of existing building stocks

Organizer: C. Bahr

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Seismic insurance market for the Italian building stock

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ABSTRACT

The life-cycle cost can be regarded as a benchmark variable in decision making problems involving insurance policy making for existing structures in seismic risk prone areas. The problem of evaluating the life-cycle cost involves uncertainties in both hazard and structural modeling parameters. The present study is a preliminary study aiming to calculate the expected insurance premium for RC building stock in Italy subjected to seismic hazard in its service lifetime based on probabilistic loss estimation. A methodology is presented that takes into account the uncertainty in the occurrence of future events due to seismic hazard. The expected insurance premium can then be evaluated based on the time-dependent probabilities that the structure exceeds a set of discrete limit state thresholds. It can be argued that the evaluated insurance premium can be influenced by possible seismic upgrading operations undergone by the structure. Further, the choice of upgrading the structure allows the insurer to discriminate amongst structures (citizens), and, consequentially, to alleviate adverse selection and moral hazard problems. Finally, the methodology is implemented in an illustrative numerical example which considers the Italian portfolio of RC (Reinforced Concrete) structures discretized in 2 structural typologies and in 103 areas, corresponding to the Italian provinces; then the insurance premium is evaluated for each structural typology in each Italian province. It is demonstrated how the evaluated premium can be affected by the decision to upgrade the structure.

In fact, the obtained results showed a very different insurance premium among the different Italian provinces as a result of the different seismic hazard.

Table 1. Insurance premium per structural typology in the main provinces.

	Specific insurance premium p [€/m^2]		Insurance premium for the average residential property unit [€]	
Provinces	Gravity	Seismic	Gravity	Seismic
Milano	0.18	0.13	16.26	12.16
Venezia	0.72	0.54	66.25	49.73
Udine	15.13	10.31	1,392.32	948.51
Trieste	3.93	2.86	361.69	263.56
Roma	4.48	3.30	411.97	303.47
L'Aquila	26.94	18.04	2,478.86	1,659.80
Isernia	25.49	16.91	2,344.78	1,555.52
Campobasso	19.30	12.96	1,775.48	1,191.92
Napoli	8.92	6.26	820.38	575.70
Avellino	12.89	8.88	1,186.16	817.18
Potenza	14.49	9.91	1,333.17	911.42
Cosenza	26.92	17.79	2,476.79	1,636.87
Catanzaro	22.09	14.73	2,032.72	1,355.09
V. Valentia	25.79	17.05	2,372.41	1,568.30
R. Calabria	26.22	17.31	2,412.05	1,592.69
Palermo	9.96	6.92	916.57	637.03
Messina	22.34	14.83	2,055.45	1,364.07
Catania	16.48	11.19	1,516.30	1,029.68
Bari	0.69	0.52	63.75	47.86

Furthermore, in each province, a significant difference between the considered structural typologies was observed, as a result of the different fragility functions/seismic vulnerability. In particular, tone can appreciate the influence of retrofitting operations in reducing the expected loss and as a consequence, the insurance premium to be paid in order to have covered the economic loss due to seismic events.

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The challenge of existing building stocks

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ABSTRACT

Large property owners, like the public authorities, urgently require appropriate means to face the challenges of successful building maintenance in the future. To this end, the Department for Facility Management, University of Karlsruhe (TH), has developed appropriate tools and tailor-made strategies to enable decision-makers to maintain their building portfolio in an optimal way.

The research work was done in a quantitative way, based primarily on the analysis of empiric real estate data provided by the owners of existing properties. The analysis comprised the real data of a total of 50 buildings including schools, office and administrative buildings, as well as convention centres and churches. The research project lead to the development of an innovative calculation method for maintenance budgets: The PABI method (Practical Adaptive Budgeting of maintenance measures) is the first calculation method to differentiate between regular and extraordinary maintenance measures. Also, its modular structure sets it apart from other methods.

The method uses a basic PABI module which can easily be adapted to different analytical needs using a number of parameters. For regular as well as for extraordinary maintenance measures, a constant percentage and a standard calculation base (the replacement value) were determined. These can be modified, depending on the influencing parameters, using different weighting factors. Thus, the PABI method can easily be extended or adapted, if need be. The formula below shows the basic module of the PABI method:

$$B_{IH} = \sum_{i=1}^{n} \underbrace{1{,}2\,\% \cdot WBW_i \cdot KF_{I,W,IS,i}}_{\text{regular measures}} + \sum_{i=1}^{n} \underbrace{4{,}4\,\% \cdot WBW_i \cdot KF_{V,i}}_{\text{extraordinary measures}} \quad (1)$$

where B_{IH} = Maintenance Budget; WBW = Replacement Value; I = building index; n = Number of Buildings; KF = Correction factor to include other influencing factors.

Table 1. Overview of the influencing factors.

<table>
<tr><th>post-war buildings</th><th>old buildings</th><th>church buildings</th><th>factory and laboratory buildings</th></tr>
<tr><td>-</td><td colspan="2">architectural style</td><td rowspan="8">under examination</td></tr>
<tr><td>-</td><td colspan="2">preservation order</td></tr>
<tr><td colspan="2">technology level</td><td>-</td></tr>
<tr><td colspan="2">type of use</td><td>-</td></tr>
<tr><td colspan="2">business competition</td><td>-</td></tr>
<tr><td colspan="3">building age</td></tr>
<tr><td colspan="3">building geometry</td></tr>
<tr><td colspan="3">quality of building planning</td></tr>
</table>

Over time, several module variants to the PABI method have been developed: for post-war buildings (Bahr, 2008) and for historic buildings (Bossmann, 2008) of different types as well as for church buildings (Bossmann, 2011). Currently, a module for factory and laboratory buildings is being developed. Depending on their building portfolio, budgeting experts can calculate their maintenance budgets choosing the most suitable variant.

These variants differ primarily regarding the influencing factors which need to be considered.

Irrespective of the building type, all PABI modules determine the maintenance costs using a constant base for recurring maintenance measures and a cycle of 30–40 years for exceptional measures with the influencing factors varying depending on the type of measure.

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MultiMap: A tool for strategic analysis of building portfolios

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ABSTRACT

Due to a huge backlog of maintenance in the public sector, a tool for mapping technical condition, MultiMap, was developed in 1997 in cooperation with Oslo Municipality who had a building portfolio of app 4 million m^2. In the past years there has been an increasing focus on how buildings affect the core business effectiveness over time. Changes in organization and changing needs in the core business will leed to new requirements of the building. The buildings, with their physical limitations, are a deciding factor for continuous efficient operation of the core business.

The objective of this paper is to show how this innovative model has developed (through R&D projects and real life projects) from surveying only technical condition to also include other modules such as adaptability, usability for the core business, possible future use etc. Today the model has been widely tested and proved to be an efficient method and tool for strategic analysis of building portfolios.

The MultiMap model is module based and can be used for several purposes such as portfolio and management strategies, long term development plans, documentation of technical values, space costs and accumulated need for maintenance, future use of buildings and identifying which ones are survivors.

In order to carry out a strategic building analysis it is necessary to use data and knowledge already present in the actual organization. This gives quick and cost efficient access to information at required level of accuracy.

The model has so far been used for strategic portfolio analysis of approx 25 million m^2, mostly hospitals and buildings in the municipality sector. In addition it has also been adapted to cover other types of infrastructure, such as roads and nautical installations along the total coastline of Norway.

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Cost-benefits and environmental impact of seismic retrofit for low-rise reinforced concrete buildings

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ABSTRACT

On the basis of investigations made after several major earthquakes occurred in Taiwan, e.g., Ruei-Li earthquake (July 17, 1998), Chi-Chi earthquake (September 21, 1999), and Chia-Yi earthquake (October 22, 1999), a number of buildings in the schools have suffered damages of various degrees. Especially in Chi-Chi earthquake, nearly half of the school buildings in the central area of Taiwan collapsed or were damaged seriously. Even in Taipei City, which is about 150 km far away from the epicenter, there were 67 school buildings damaged. According to the report published by Ministry of Education of Taiwan, a total of 656 elementary and secondary school buildings, which are almost categorized into low-rise Reinforced Concrete (RC) buildings, were damaged in Chi-Chi earthquake. Additionally, because school buildings are usually required to act as emergency shelters soon after a disastrous earthquake, seismic upgrading of existing elementary and secondary school buildings is a serious issue. In Taiwan, there are 3,497 elementary and secondary schools. 3419 schools of them (about 98%) completed the simple survey. Based on the assumptions for the simple survey, e.g. common structural types, seismic resistance, possible failure modes and available experimental data, the seismic performance of the school buildings were scored; then, about 55% of the school buildings were evaluated to be with insufficient seismic performance. However, Ministry of Education of Taiwan is constricted by the amount of resources available for immediate investment on seismic retrofit, yet costs and benefits of seismic retrofitting strategies in the remaining service periods are of increasing interest.

This study presents a novel estimating method for assessing seismic damage to RC buildings by using the hazard curve of response spectral acceleration. Furthermore, the occurrence of an earthquake is assumed to follow a Poisson process when analyzing the occurrence probability of a specified damage state in a specified service period and expected costs induced by the seismic damage. Besides of costs, the CO_2 emission induced by the seismic retrofit and repairs is also considered as the environmental impact herein to build an estimating method for the payback period of the seismic retrofitting investment based on economic and environmental aspects. Finally, for understanding the financial and environmental payback periods of the seismic retrofitting investment in Taipei, Taiwan, sixteen practical design projects for the seismic retrofit of RC school buildings in Taipei are subjected to the cost-benefit analysis and the CO_2 emission analysis in the case study.

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The Bridge-Management-System (BMS) in Germany as a basis for life-cycle considerations

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ABSTRACT

Due to the increasing globalization, the traffic grows in the world and steadily in Europe. This also means that the existing infrastructure is increasingly under pressure and exposed that with increasing age.

The financial difficulties, particularly in Europe make it clear that participation in this globalization leads to higher spending for the states, for example to be able to keep pace with the increasingly rapid development in the communications world. At the same time the infrastructure and as an important part the transport infrastructure, must be able to withstand this development.

This means on the one hand, that significant financial and human resources must be invested in the maintenance and upgrading of the road infrastructure.

This means also that a sustained systematic maintenance planning is increasingly necessary. It offers the possibility to use the available financial resources optimally and sustainably. This can provide an important contribution to this life-cycle considerations, which makes it possible to consider not only the lowest construction costs, but also a part or the whole service life of a building.

The following article will outline the possibilities of the German Bridge Management System (BMS) for life-cycle aspects.

The German BMS is designed to optimize both at the network level, as well as at the object level. The network layer will mainly be used for budget planning with the use of maintenance objectives as input, such as the requirements for mean values of condition indices. The object level allows the calculation and presentation of different maintenance strategies and their impact on direct and indirect costs.

It will be shown for bridges the future behaviour by the use of certain behaviour models. The impact on the environment is detected by the impact on traffic and covered with costs.

The actual approach of BMS is that bridges, which show damages, are considered. The implemented algorithms allow considering a bridge from the beginning of its life time.

Using these algorithms and deposited catalogues it’s possible to calculate for different strategies life-cycle considerations also for decades. Thus the road authorities’ get a basis for decisions for the choice of preservation strategy and where the bridge was not built for the planning and erection phase of the bridge.

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LCC as a decision tool for strategic development of the public building portfolio: A Norwegian study

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ABSTRACT

The paper explores the status of use of LCC and whole life costing methods in Norwegian municipalities today. The material is a questionnaire to leaders of municipal boards and chief municipal executives, and also interviews with senior managers in some municipal FM organization units. The study includes questions on the status for use of LCC today, interest and knowledge for operation and maintenance costs among the decision making politicians, and the value of extensive use of LCC in improvement processes.

Experience with and use of LCC in Norwegian municipalities today varies partly according to the size of the municipalities. In accordance to previous studies our material indicates that larger municipalities have the most expertise and practice in LCC-based planning.

Methods in use for existing portfolio varies, and includes historical accounting and use of national key performance indicators. Also benchmarking for best practice among similar municipalities and computer assisted FM systems are in use. For improvement and optimizing work cleaning and energy get more attention than maintenance.

The value of LCC depends partly on the usability for optimizing processes and partly on the ability to communicate to the political decision makers the long term implications of alternative priorities.

Norwegian municipalities manage a building portfolio which represents a considerable share of the national capital assets. Attention has been drawn to a maintenance backlog and the long term consequences of reduced technical quality of existing building portfolio. During the last decade there have been initiatives to improve knowledge of life-cycle cost considerations.

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Decision aiding & multi criteria optimization for existing buildings holistic retrofit

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ABSTRACT

Under our latitudes, existing buildings energy consumptions – related to heating, cooling, ventilation, domestic hot water, and lighting – are responsible for significant environmental impacts. In France, buildings share in national final energy consumptions represent about 43%, in 2010. Moreover, the annual replacement rate for existing buildings is inferior to 1%, in most of developed countries. Consequently, existing stock retrofit represents a major lever to reach national and international commitments on climate change and non renewable energy consumption mitigation (IEA, 2008). However, the identification of optimal sustainable retrofit programs, including actions planning over a time period, is still a difficult task for professionals. Most operational approaches are based on iterative building simulations guided by experience (Alanne, 2004).

The present paper is a contribution to decision aiding through optimal energy retrofit programs identification. Considering the mathematical nature of the considered problem – combinatorial, discrete variables, implicit non linear objective functions – various multi criteria optimization techniques could be involved (Colette, 2002). In the present paper, a multi criteria genetic algorithm (NSGA-II) (Deb, 2002) (Deb, 2000) is used to optimize solutions – retrofit programs – on both their content and planning.

Each solution is represented by a pair of chromosomes; one coding the retrofit measures to implement, the other representing the time sequence. These retrofit measures address building envelopes (external walls, top and bottom floors thermal insulation, windows replacement, windows to wall ratios), and the replacement of equipments for ventilation, heating and DHW production. For each of these retrofit measures, various options are considered.

The potential solutions are evaluated on a multi criteria and life-cycle basis. The objective functions considered target environmental impacts (i.e. climate change potential, primary energy consumption, abiotic resources depletion, air acidification, etc.), financial indicators (i.e. investment cost, global cost) and occupants well-being (thermal comfort indicator). Life-cycle assessment and life-cycle cost models, using building dynamic thermal simulation for heating load and thermal comfort evaluation, are implemented to assess solutions performances.

The methods and tool developed have been used to identify promising sequential energy retrofit programs on a building case study. The construction considered for this case study is a multi family building, located in Paris suburban area, built before 1974. The optimization gives access to promising solutions and suggests strong correlations in between some criteria.

These methods and tools contribute to decision aiding; identifying Pareto non dominated holistic retrofit programs, at a building scale, on a multi criteria basis, over life-cycle.

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Structural health monitoring of civil infrastructures in a life-cycle analysis

Organizers: C.F. Cremona & A.D. Orcesi

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High precision structural health monitoring system using wireless sensor networks

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ABSTRACT

Some advances in extensive bridge monitoring using low cost dynamic characterization Structural Health Monitoring (SHM) systems have excellent potential to improve the regular operation and maintenance of the structures. Wireless Sensor Networks (WSN) have been used to avoid high cost of traditional generic wired systems. The most important limitations in SHM wireless systems are time synchronization accuracy, scalability and reliability.

In this article, a complete wireless system for structural identification under environmental load is presented. Our contribution ranges from the hardware to the graphical front-end. The system is paying attention in avoid-ing the main limitations of WSN for SHM specially focused in reliability, scalability, and synchronization.

In order to validate the system we have carry out different tests: check synchronization at the lab, compare our operational modal analysis over a real bridge with a numeric analysis and analyze the results of a comparison between couples of signals which belong the same position over the bridge. Obtained results have been excellent and the system has been validated.

This work makes three main contributions to SHM systems. The first one is to fulfill the require-ments needed to obtain high quality data to be used for current and future operational modal analysis. With this purpose, a wireless acquisition system with high-frequency sampling together with a very reliable time synchronization accuracy and low jitter, not provided by previous works, has been devel-oped. Spatial jitter has been reduced to 125 ns, far below the 120 μs required for high-precision acqui-sition systems. The second contribution is a system designed with the ability to scale to a large number of nodes. This way, a dense sensor coverage grid of real world structures becomes possible. Finally, this network has been tested in a real world structure solving a myriad of problems encountered in a real deployment in difficult conditions.

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In-service inspection of reinforced concrete cooling towers – EDF's feedback

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ABSTRACT

In France, EDF operates a fleet of 28 reinforced concrete towers, which are very large hyperboloid structures that can be up to 180 meters tall and 80 meters in diameter, as depicted in Kratzig et al. (2000) or shown at Figure 1. These structures are not safety related, but considering the investment and their role in the plant operation, EDF aims at anticipating the effects of the ageing phenomena. Since the beginning of operation life, structural survey comprising in service inspection and monitoring have been implemented to feed analysis supporting maintenance management processes, as recommended in IAEA standards (IAEA 2009).

As a result of the cooling towers collapses occurred within the years 1960 and 1980, and although these structures are not safety related, EDF decided to inspect all the shells in the aim of mapping all defects and deformations and to record and follow up the adverse ageing effects. Then, EDF follows a detailed procedure for theses inspections, involving:

– measuring the differential settlement and potential tilting of the structure using traditional topography equipment;
– concrete surface to map the shape and size of the cracks;
– mapping the real shape of the shell by planimetry at different levels and by photogrammetric measurements or by using a 3D laser scan survey.

In case of observed significant distress, specific analysis on core samples or non destructive evaluation could be carried out.

A synthesis report is issued every year for the entire cooling towers fleet. The document is concluded by a ranking of all the towers. This ranking is based on experts' judgments.

In order to get a more objective assessment, EDF decided to experiment the use of Symbolic Data Analysis (Afonso et al., 2010), which enables to extract useful information from complex databases, by gathering and comparing heterogeneous objects (images, measurements, calculations outputs…).

Besides, technical developments are carried out to improve the management of maintenance for these structures, in the field of instrumentation (embedded sensor for new built projects, fiber optics, corrosion monitoring) and in the field of monitoring data processing.

Figure 1. Natural draft cooling towers.

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Automated geomatic system for monitoring historical buildings during tunneling in Roma, Italy

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ABSTRACT

The present work is focused on the preliminary results obtained through the geomatic integrated monitoring system that is currently running in some relevant test sites of the central archeological area of Rome.

This system it is based on the innovative use of integrated surveying techniques, and it has been designed in order to monitor relevant archaeological buildings and sites before, during and after the tunneling for the new metro line, presently under construction.

Nowadays, the monitoring system is installed and running in a beta version in three test sites: the Basilica of Maxentius and Constantine in the Roman Forum, Aurelian Walls near Porta Asinaria and the Monument to Vittorio Emanuele (Vittoriano). It includes different high precision geomatic sensors: automatic total stations, GNSS geodetic class receivers (Fastellini et al. 2011), bi-axial inclinometers and meteorological stations that acquire continuously at high rate and are controlled by a suited management system, presently under refinement.

During the present stage of construction, since the tunnelling has not started yet under the monitored monuments, the main goal of the monitoring system is the evaluation of the standard daily/seasonal displacements which structures undergo. This aim is obviously of crucial importance, for filtering out these displacements from the total ones detected during the tunnelling, and therefore to be able to highlight possible critical structural deformations and provide early warnings when given thresholds are exceeded.

The geomatic sensors (Dominici et al. 2008) (Mat et al. 2010) included in the monitoring system are installed in proper monitoring stations, where they are utilized for different aims: the automatic total stations are the main sensors, devoted to the direct control of the structures; bi-axial inclinometers and GNSS receivers behave as auxiliary sensors, aimed to complement the monitoring of the short (daily) and long (seasonal and more) period stability of the monitoring stations, which is routinely carried out by the automatic total stations themselves through suited external reference benchmarks; finally, meteorological sensors are devoted to supply the climate information to be cross-correlated with the detected displacements in order to highlight the daily/seasonal standard structural behavior.

We started to develop a joint analysis of all the available data, using a rigorous estimation approach, which takes advantage of the redundant information. In this way we pursued a twofold goal: to carry out the preliminary accuracy assessment of each sensor and calibrations between sensors, particularly for monitoring the short period stability of the monitoring stations; to define a methodology useful for future massive data processing.

The first results are presented and discussed showing the effectiveness of the designed system.

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A vibration-based framework for structural health monitoring of railway bridges

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ABSTRACT

One main objective of Structural Health Monitoring (SHM) is to assess the performance of structures. The detection of structural damage at an early stage is essential for civil engineering infrastructures to prevent the occurrence of catastrophic failures and to reduce costs concerning maintenance. A long-term Structural Health Monitoring (SHM) is usually considered for monitoring the structural performance level and for damage detection. This method has been widely considered for aircrafts (Baker et al. 2009), ships (Stull et al. 2011), offshore platforms (Nichols 2003), buildings (Zhao et al. 2009), as well as for bridges (Cury et al. 2010, Cury & Cremona 2012a, b). In this paper, the authors focus on the damage assessment problem based on a vibration-based detection approach specifically designed for a bridge in real environment and traffic conditions. For this purpose, a cluster-based approach is proposed to discriminate abnormal changes from normal changes in the structural behavior. Besides, Symbolic Data Analysis (SDA) is introduced to process and analyze large amounts of data. At the same time, some novelty detection strategies using original symbolic objects and Principal Component Analysis (PCA) are proposed to extract useful information related to structures from large amounts of data. The efficiency and reliability of this approach is checked using both simulation and real data for a road-rail bridge, the Adour Bridge (Fig. 1), which is a steel road-rail bridge located in Bayonne, France, and managed by SNCF (National Corporation of French Railways). A long-term Structural Health Monitoring (SHM) program of the Adour Bridge was decided until its demolition, to analyze the structural dynamic behavior and detect abnormal structural response under the current traffic loads.

Figure 1. View of the Adour Bridge.

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Force monitoring with contact free elasto-magnetic sensors on single strands for multi strand anchorages

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ABSTRACT

Structural health monitoring in civil construction is becoming increasingly important due to client and authority requirements and due to cost saving issues during service and maintenance. The DYNA® Force EM (Elasto-Magnetic) sensor system can measure the force that is applied on a single seven-wire prestressing steel strand of a multistrand anchorage such as the DYNA Grip® system without any contact. The DYNA® Force system basically consists of three components: Firstly, the sensor (see Figure 1), pushed over the strand. Secondly, the read out unit, which supplies the sensor with current and voltage and displays the force. Thirdly, the multiplexer, which permits the connection of more than one sensor to the read out box.

The DYNA® Force system does not only monitor the force applied on the whole anchorage, as is mostly the case with load cell applications. It is possible to detect the stress in single strands on the anchorage and therefore to provide a higher resolution and more detailed result for service and maintenance. The system makes use of the fact that the permeability of steel changes with stress. This permeability change can be used for measuring the current stress after the system has been calibrated. The system can be adjusted to various requirements. It is possible to configure an economic and basic system that is easy to operate for a simple, manually conducted measurement on only a few sensors using a portable read out unit. Alternatively, the system can automatically measure several sensors connected together by a permanently installed multiplexer and a read out unit that is located on site. The DYNA® Force system has recently been installed in a football stadium located in Lille, France (see Figure 2). The system will be presented based on this project.

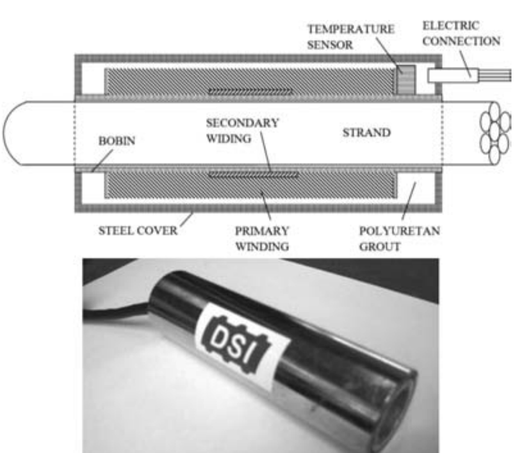

Figure 1. Design of the sensor.

Figure 2. Sensor position on anchorage.

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A SHM framework comprising real time data validation

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ABSTRACT

Structural Health Monitoring (SHM) can provide real time valuable data for life-cycle analysis however some issues are seldom addressed in SHM works such as, (i) real time automated validation of data, (ii) concurrent access of automated software and authorized users, (iii) data independence, both from physical location and software and (iv) long-term data preservation. Moreover the amount of data generated by monitoring systems can make their validation terribly time consuming, especially for workgroups responsible for monitoring a significant amount of structures.

To address these issues a SHM data management framework was developed in LNEC and is described in the present paper. Besides the description of the spatially distributed hardware and the architecture of the software its most important features are described and their advantages in the SHM process referred.

Attention is paid to a novel relational metadata model. Metadata can be defined as the discipline of data which is designed to critically deal with original data (Sen 2004). Its architecture is briefly described in terms of the definition of entities and relationships and the influence of their definition in the automatic validation of data in what concerns consistency and integrity of the acquired data.

Moreover, emphasis is given to a four step validation procedure which combines the use of the Database Management Systems' (DBMS) intrinsic functionalities and two distinct robust statistical procedures carried out, respectively, by the software component installed in each monitored structure (the AMoS monitoring system) and the database manager and data validation software (SHMR Manager), both developed in LNEC by the authors. While the first procedure has been used in previous works the second consists of a original statistical approach which iteratively combines a multivariate robust indicator, the MVE (Rousseuw 1985), and two types of statistical tests, (i) a χ^2 mean statistical test to define hyper ellipsoids around a multivariate data set and (ii) the Kolmogorov-Smirnov test (Massey 1951) to assess if the data's distribution before and after the outlier removal according to the hype volume of the ellipsoids is independent. From the p-values provided by the later test the SHM framework decides upon the validity of data and tags it in the database as valid or invalid.

A case study is presented consisting in the International bridge over river Guadiana, in the South of the Iberian Peninsula. The application of the framework is presented and an example of the data validation is carried out comprising static data "corrupted" with values resulting from traffic and wind action and also outliers related to a maintenance action in two of the eight bi-axial tilt meters installed in the bridge.

The example allowed to conclude about the good performance of the validation method, not only in detecting the abnormal values related to sensor maintenance but also in detecting values related to the referred dynamic effects, which are less distant from the true data distribution. Besides concluding about the success of the novel validation method the authors also emphasize the good performance of the described framework and the architecture of the database which validates and preserves data from multiple structures regardless of their location and type.

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Analysis of rehabilitation needs and maintenance strategies

Organizers: M. Hastak & Y. Yoon

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Management system for infrastructures at waterways

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ABSTRACT

The Wasser- und Schifffahrtsverwaltung of Germany, WSV (Federal Waterways and Shipping Administration of Germany), is responsible for a huge number of infrastructure facilities such as locks, weirs, culverts, canal bridges and lighthouses. A management system for the maintenance of these waterway infrastructures is currently worked out by Bundesanstalt für Wasserbau, BAW (Federal Waterways Engineering and Research Institute). The components and procedures are described with focus on the statistic forecasting methods of the facility condition. The aim and the benefits of this system for Waterway Infrastructure can be achieved by reliable predictions for maintenance and renewal activities, cost transparency, planning security and increasing sustainability of investments and traffic safety.

The management system is based on the actual condition of the construction. The information of the condition is gathered by regular inspections and described it-based in a standardized way. To ensure the quality and the standardization of the damage classification, a guideline for damage classification was prepared, (BAW 2009).

The deterioration of the construction is described with stochastic models. For detected damages Markov chains are used. Some restrictions are used to simplify the procedure and reduce the number of needed parameters. The deterioration of construction parts which are not damaged is described with survival functions, which are derived from the population statistics. The parameters for both models are taken from a Delphi-interview which was performed with experts.

To estimate the latest point of time for maintenance or renewal action a fixed accepted grade of deterioration is defined.

Beside the condition of the structure other parameters as must be taken into account. A first step to include the importance of the waterway is demonstrated.

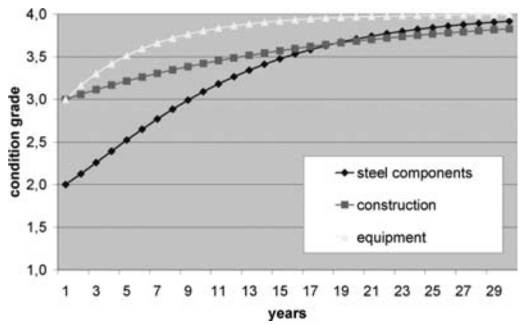

Figure 1. Examples of deterioration.

Finally it is necessary to link the construction parts with possible maintenance or renewing activities to take the financial budget into account. A survey in the German waterways administration helped to find realistic charges for different maintenance actions.

Using the real data of the German waterways network the modus operandi is shown and results are presented.

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Life-cycle considerations in bridge deck rehabilitation strategy

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ABSTRACT

The infrastructure deficit in Canada is growing and it is estimated that 40% of all bridges are older than 30 years. Managing the aging infrastructure with the limited funds available is challenging and demands a comprehensive approach to evaluating the condition of structures, selecting optimum rehabilitation strategies based on a life-cycle cost analysis. Current bridge management systems often base decisions solely on the results of visual inspections, frequently disregarding the benefit of employing non-visual techniques to gain further valuable information. Visual inspections supplemented by chain dragging often overestimate or do not correctly identify the full areas that require repair. This can have a significant impact on the life-cycle cost as needed repairs are not always addressed immediately and needless repairs are enacted.

In this case study, visual inspections were supported by a non-destructive (radar survey) condition assessment, detailed petrographic analysis, and chemical analyses. The results of the non-destructive radar survey indicated that the delaminations identified by chain dragging may overestimate the actual area of surface deficiencies and not identify other critical areas.

This comprehensive inspection approach provided sufficient information to informed decisions on the rehabilitation. Deck rehabilitation options were recommended based on the study and detailed specifications for the construction, including rehabilitation methodology and materials selection, were developed. Construction monitoring and strict quality control was followed by the warranty inspection and the remaining service life of the structure was re-evaluated.

This “cradle-to-grave” approach combined with the tight timing of the bridge deck condition assessment and the rehabilitation works delivered the lowest present value cost to extend the remaining service life of the structure.

Post rehabilitation radar survey of the bridge deck was recommended for a continuous monitoring of deck anomalies and to define the rate of changes in performance and provide an early warning system before the service life is undermined.

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Strategies and methods to increase the life-cycle of RC buildings in seismic prone areas

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ABSTRACT

In earthquake prone areas, much of the existing building stock has been constructed using past outdated knowledge, old code regulations, a lack of earthquake data and an insufficient prediction of the expected seismic action. Therefore, such buildings should be considered as low life-cycle structures, since they may suffer considerable damage during a strong earthquake. In order to increase the life-cycle of such structures, seismic retrofitting is essential. From the techno-scientific side of the problem, redesigning existing buildings to increase their life-cycle is incomparably more difficult and complex than that of designing new structures. In the whole redesign procedure the following three main stages can be recognised: In the first stage, the seismic capacity of the existing structure is assessed through an inspection of the structure and the identification and documentation of the structural system. In this stage, the weaknesses and the pathology of the structure are recognised and the expected life-cycle can be evaluated for a design earthquake. The second stage deals with the procedure of decision making, which concerns an investigation to find the most appropriate intervention strategy. The third stage involves the design of the intervention for a pre-selected performance level. With regard to structures of reinforced concrete, strategies increasing life-cycle under seismic actions can be distinguished in the following three main categories (Fig. 1). The first strategy aims mainly to enhance the ductility while simultaneously rectifying recognised local weaknesses, the second strategy aims at a medium increase in stiffness and strength with a simultaneous enhancement of ductility and finally, the third strategy results in a high increase in stiffness and strength. The specific method that will be selected for the intervention follows the decided strategy taking into account the desired levels of strength, stiffness and deformation of the structure. A variety of methods and techniques should be considered as possible solutions for the selected strategy. The strengthening of weak elements by various techniques according to the recognised local weaknesses, the incorporation of bracing systems, the construction of wing walls and the addition of infilled and new external walls are described. Technical aspects for the implementation of each method and technique, followed by recommendations for modelling and design are presented and discussed.

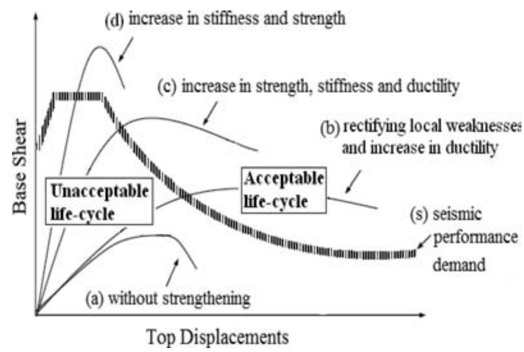

Figure 1. Strategies increasing life-cycle for seismic actions.

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Optimal allocation of resources in MR&R planning for heterogeneous bridge networks

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ABSTRACT

Highway bridges undergo rapid deterioration in both condition and safety under the influence of aggressive environmental factors and increasing traffic loads. Timely and adequate maintenance interventions are therefore crucial to ensure the functionality of existing bridges in a network. Under budget constraints, it is important to prioritize maintenance needs to bridges that are most significant to the functionality of the entire network. In this paper, the network-level bridge maintenance-planning problem is formulated as a combinatorial optimization problem and an algorithm is developed to select and allocate maintenance interventions of different types among networked bridges over a specified time horizon under the assumption of annual inspections. The objective function considered is the network probability of failure as expressed by the time-dependent reliability of connectivity between the origin and the destination locations, which is maximized while keeping the total maintenance costs under a budget constraint. A variety of maintenance actions, which differ in unit costs as well as in their effects on bridge performance, are used in the optimization. As an illustration example, the optimization procedure is applied to a heterogeneous 5-bridge network. The results show that the proposed maintenance planning procedure has the capability of prioritizing scarce maintenance needs to deteriorating bridges that are most crucial to the network performance.

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Life-cycle cost and life-cycle carbon dioxide analyses of the new/renewable energy systems and the energy-saving measures in the elementary school facilities in South Korea

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ABSTRACT

With the serious effects of global warming, international efforts to reduce Greenhouse Gas (GHG) emissions have been exerted since the 1992 United Nations Framework Convention on Climate Change. In South Korea, a variety of policies to reduce GHG emissions have been implemented, and in line with this, many efforts have been exerted to reduce the GHG emissions of buildings. Among the various types of buildings, improvements are urgently required for school buildings due to the high percentage of building deterioration.

This research aims to analyze the effects of the introduction of ESMs and NRE systems for educational facilities, and through the result, the optimal improvement scenario is to be selected from a variety of scenarios combining ESMs and NRE system.

The measures for reducing GHG emissions are mainly classified as follows: (i) increasing the efficiency of the energy production; (ii) using energy resources with lower or no CO_2 emission; and (iii) reduction of GHG emission through the application of the carbon capture and storage technology.

Among these, the measures that can be applied to buildings are application of ESMs and NRE system. As the national budget that is allocated for the improvement of school facilities is limited, the issue of whether to apply ESMs and NRE should be analyzed in the economic and environmental aspects. In this study, the following processes were applied: (i) the selection of school as standard model for a case study regarding the application of ESMs and NRE; (ii) the analysis of the effective energy savings and CO_2 reduction through the application of ESMs and NRE, using energy simulation; and (iii) the selection of the optimal improvement scenario for school facilities through LCC and $LCCO_2$ analyses.

Table 1 shows the three selected ESMs and the seven scenarios combining these factors. And Table 2 shows the energy-cost-savings, and SIR value as well as the rank of SIR by scenario. The highest SIR value appeared in a combined scenario in which scenario #3 and PV 28.6 kW were applied. This is because the initial cost of the zero standby power outlet, the ESM applied in scenario #3, was very low, i.e., a large amount of budget was invested for the PV system.

Table 1. Energy-saving measures and scenarios.

Energy saving measure	S#1	S#2	S#3	S#4	S#5	S#6	S#7
6-12-6 double low-E glazing & UPVC frame	O			O	O		O
22W LED lighting		O		O		O	O
zero standby power outlet			O		O	O	O

S = Scenario, e.g., S#1 = Scenario#1.

Table 2. SIR index of ESM and PV combined scenarios.

	Energy cost saving (present value)				
	US$				
Scenario	ESM	PV	Total	SIR	Rank
PV only (30.1kW)	0	178,599	178,599	0.931	4
S#1 & PV (14.6kW)	27,021	86,629	113,650	0.593	8
S#2 & PV (11.1kW)	108,977	65,863	174,839	0.912	5
S#3 & PV (28.6kW)	57,534	169,695	227,229	1.185	1
S#4 & PV (0.0kW)	135,891	0	135,891	0.708	7
S#5 & PV (12.2kW)	84,805	72,388	157,193	0.820	6
S#6 & PV (9.6kW)	166,156	56,962	223,118	1.163	2
S#7 only (0.0kW)	193,693	0	193,693	1.010	3

KRW/USD exchange rate = 1,133.80 (Jan. 20, 2012.)

It is expected that the research results will be utilized to establish the maintenance strategies for the educational facilities that needs rehabilitation. Also, it can be extended to the various types of facilities.

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Overall environmental impact for structural polymers: A material selection process

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ABSTRACT

The effect of the production of materials on the environment is a wide area of research. While trying to use polymers as structural members there can be a vast number of polymer materials which show more or less the same mechanical and even thermal properties, but yet have great and significant differences in production process. This fact intensifies the role of environmental impact in the material selection for the case of structural polymers.

For the case of structural polymers the environmental impact of the selected material can hardly be confined by a single significant indicator like CO2 footprint or energy content. Most designers and engineers, when designing for the environment, want to assess the overall environmental effect of both production and processing of a single material or a combination of materials. Considering that the production process of these materials can provide dozens of outputs to the environment, plotting charts for the emission quantities gathered from the production processes of different polymer materials would overwhelm the designer with large amount of data.

In order to provide a better understanding of the environmental impact of the polymeric materials, API (Air Pollution Index) and WPI (Water Pollution Index) are generated for the polymers commonly used as structural members. Assuming that these indexes demonstrate the overall impact of the production of the material on air and water, they are used as material selection tools for the structural polymers.

Given the fact that embodied CO_2 and energy content are not always the best factors to compare environmental effects of the production of materials new approach was used to compare the effect of production of different materials.

In the new approach it was attempted to use all the data from the emissions due to production of the material. Using the "the European commission decision on the implementation of the European Pollutant Register (EPER)", Air and Water Pollution Indices were calculated for a group of polymeric materials that are used as structural members. Table 1, shows the results of the API and WPI for a set of Polymers used as structural members and components in buildings.

Table 1. Air and Water Pollution Indices for different groups of polymeric materials.

Material	Total API	Total WPI
HDPE	0.2762228	0.0989640
ABS	0.6018439	0.3976300
POLYAMID	2.0005745	0.4970251
PET	0.7360931	0.0392336
PMMA	1.0209231	0.4005280
PVC	3.5157280	0.1072852
Polycarbonate	1.4230446	5.3646503

Considering that the API and WPI represent the overall environmental impact of the production of materials it was shown that High-Density-Polyethylene (HDPE) by far has the lowest environmental impact on both the atmosphere and water, while the results from the embodied CO_2 and the energy presented PVC with having the least impact on the environment.

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Life-cycle assessment of historical structures towards sustainable architectural heritage in Kosovo

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ABSTRACT

The aim of the paper work is to provide a research activity and information regarding the basis for the Life-Cycle Assessment and its importance for Historical Buildings in general (Frey, 2007) with intention to suggest the direction and to encourage decision makers, responsible authorities, preservationists, architects and environmentalists and to provide them with information about evaluated LCA and its usage in context of Kosovo. In particular the research presents environmental performance assessment (ATHENA® Impact Estimator for Building) of three historic buildings – the Residences Xhafer Deva and Mehmet Pasha Gjinolli in Mitrovica and Vushtri, and Building of Architecture Department, University of Prishtina. Their environmental performance emphasizes the importance of applying the LCA of preserving historic buildings towards sustainable architectural heritage in Kosovo, the newborn state in Europe.

The basic observations are done in order to identify the international and/or national LCA methods and tools (COST Action 025, 2011), thus their contiguity to historic buildings, as follows: the environmental merit of retaining and conserving older buildings, including heritage buildings, may seem obvious. The inclusion of social and cultural criteria would acknowledge heritage conservation as a sustainable action while deciding about whether to keep or demolish a building which often revolves only around cost considerations without taking into account the environmental implications (Halliday, 2008).

Basically, the suggestions for further understanding of LCA rating system in context of Kosovo, is to analyze and adopt the rating system which will be realized using underlines: (a) easy-to-use – simple rating system; (b) efficient – not consuming time; (c) frugality – cost effective, rescued certification cost; (d) consistent – improved reliability & consistency; (e) transparent – enhanced clarity & transparency; (f) accuracy – rigorous, continued market-leadership; (g) innovative – updated, new rating tools.

The Sustainability Assessment of Constructions (SAC), Building Environmental Performance Assessment, in particular the LCA rating system (Kibert, 2005) reflect to historical buildings, thus require an urgent nexus between governments' recommended system for new buildings and preservation of old too. The developed model, based on the case studies is outlined in basic principles and phases, such as: 1) goals and scope definition; 2) inventory analysis; 3) impact assessment and 4) life-cycle interpretation of case studies; which recommends the value of heritage buildings over their life-cycle, in general to suggested and agree for the importance of a re-evaluate tools to be used and the policies to be meet into today's sustainability goals.

Outcome of this research and its publication will increase better understanding of overall benefits why implementing LCA in Kosovo, e.g. the method of rating with checklists in comprehensive measures for sustainability of historical buildings during design stage, reconstruction and maintain stage, performance of architectural heritage in economic, social, and environmental aims.

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Fibrecement recycling as raw material for Portland clinker production

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ABSTRACT

The sustainable production of cement and clinker will dominate the construction industry in the following years. This paper aims to examine the use of fibrecement waste as an alternative raw material for clinker kilns taking into account the possible limitations. Fibrecement is the generic term given to a wide variety of composite materials consisting of Portland cement, inert and/or reactive mineral fillers and a mixture of several types of organic fibres. Within the context of the present paper, only fibrecement products produced by Hatschek technology are considered. Numerical simulations were carried out to maximise the use of fibrecement waste in alternative clinkers. Three reference clinker factories, CBR Antoing and CBR Lixhe in Belgium and ENCI Maastricht in the Netherlands, all belonging to the Heidelberg Benelux group were simulated in line with their chemical and mineralogical requirements listed in Table 1 and Farag (1994). Lab clinkers were produced and analysed according to numerical simulations and in function of realistic dosages like presented in table 2. It was demonstrated that the chemistry and mineralogy of the final clinkers were not influenced significantly by the use of the fibrecement materials. By using the fibrecement waste in clinker production, a reduction of the ecological impact of the clinker production was measured. It was shown that compared to a situation where pure limestone is used in the cold clinker meal, an inorganic CO_2 emission reduction as well as a decarbonation energy gain was measured like demonstrated in figure 1. The possible energy gain by using an alternative versus a reference clinker meal coming from the exothermal degradation of organic fillers minus the estimated energy consumption needed for the liberation of chemically bound H_2O was investigated. The use of fibrecement waste as an alternative raw material was found to be realistic without compromising on physical, chemical or mineralogical properties of the clinker.

Table 1. Chemical and mineralogical limits on the final clinker.

Limits (%)	CBR Antoing	CBR Lixhe	CBR Maastricht
Cl	x < 0.08	x < 0.08	x < 0.08
SO_3	x < 1.2	x < 1.2	x < 1.2
Na_2Oeq	x < 1.2	x < 1.2	x < 1.2
MgO	x < 4.0	x < 4.0	x < 4.0
DoS-level	80.0 < x < 120.0	80.0 < x < 120.0	80.0 < x < 120.0
LSF_MgO	98.5 < x < 98.5	98.5 < x < 98.5	98.5 < x < 98.5
C_3A	7.4 < x < 7.4	6.7 < x < 6.7	7.3 < x < 7.3
LiqSimple	19.2 < x < 19.2	22.7 < x < 22.7	23.0 < x < 23.0

DoS-level (Degree of Sulfatisation) = $77.41 \cdot SO_3/(Na_2O + K_2O \cdot 0.658)$

Table 2. Compositions of the alternative clinker meals (%).

Raw materials	CBR Antoing	CBR Lixhe	CBR Maastricht
Poor limestone	0	–	–
Rich limestone	79.08	–	–
Tufa limestone	–	38.52	–
Marl limestone	–	–	84.38
Loam	–	0	–
Sabulous clay	–	–	0
Fly Ash	7.03	9.32	10.33
Iron Carrier	0.61	0.53	1.77
Fibrecement	13.28	51.63	6.01

"0" indicates the presence of the raw material in the reference clinkers

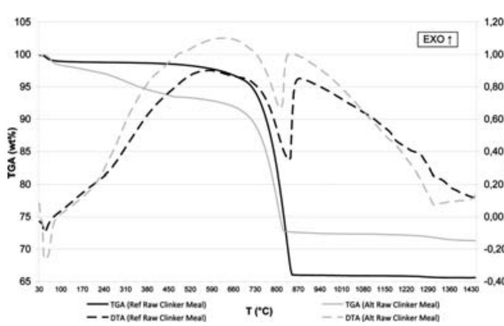

Figure 1. TGA/DTA analysis of ref. and alt. raw clinker meal *(solid line: Reference, dashed line: Alternative).*

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Reducing CO_2-emission by using CEM V eco-cements

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ABSTRACT

CO_2 concentration in the air is rising constantly. Globally, cement companies are emitting nearly two billion tons/year of CO_2 (or around 6 to 7% of the planet's total CO_2 emissions) by producing Portland cement clinker. At this pace, by 2025 the cement industry will be emitting CO_2 at a rate of 3.5 billion tons/year causing enormous environmental damage (Shi et al., 2011; Janotka et al., in press).

This evokes pressures to reduce the cement consumption through the use of industrial by-products and supplementary cementing materials substituting the Portland cement in concrete. Another possibility to reduce the required energy during cement production and thus the emission of CO_2 is the usage of eco-cements or so called blended cements.

Hence new CO_2-saving eco-cement types are gaining in importance. In these cement types the energy-consuming Portland cement clinker is partially replaced by latent hydraulic additives such as blast furnace slag, fly ash or natural puzzolana (e.g. zeolite). These hydraulic additives do not need to be fired in the rotary furnace. They only need to be pulverized to the required grain size and added to the ground Portland cement. Consequently energy is saved by skipping the energy-consuming firing process, in addition there is no CO_2-degassing from the raw material as there is in the case of lime burning.

In due consideration of ecological and economical interest in reducing the required energy input, a research project between Austria and Slovakia, funded by the EU (Project ENVIZEO), was initiated in 2010. The main goal of this project is to develop new CEM V eco-compositions and certificate them for common usage. CEM V is a Portland clinker saving cement type that allows the reduction of clinker to 40–64% for CEM V/A and 20–39% for CEM V/B respectively by the input of blast furnace slag, puzzolana and fly ash (according to standard EN 197-1).

In this context four new CEM V kinds have been created, two Austrian (AT) types based on slag and fly ash, and two Slovak (SK) types, one based on slag and fly ash, the other on slag and natural puzzolana.

It was possible to reduce the cement clinker content to a level of 45.1% (CEM V/A-AT), 52.9% (CEM V/A-SK), 26.9% (CEM V/B-AT) and 30.9% (CEM V/B-SK) respectively. Additionally a Slovak CEM I 32.5 R cement containing 95% of clinker was created for comparison purpose. CEM V cement and concrete properties were extensively investigated and compared to standard CEM I.

Basic cement properties (normal consistency, initial and final setting, volume stability) and fresh mortar properties (workability, volume density, air content) as well as strength parameters using standard mortars with w/c of 0.5 were determined. Compressive strength tests of both CEM V/A cements (SK and AT) show similar or even higher strength than that of CEM I. The CaO content of 35.1 to 43.3% of all CEM V in comparison with 60.8% for CEM I indicates improved resistance against chemical attack. pH values of all cement water extracts between 12.35 to 12.47 suggest a satisfactory high alkalinity necessary for steel passivation.

CEM V concrete specimens show comparable and in case of CEM V/A even higher compressive strength versus CEM I. Concrete analysis using ESPI (Electronic speckle Pattern Interferometry) also suggests good concrete characteristics.

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Multiyear infrastructure rehabilitation strategy within the context of MR&R

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ABSTRACT

Constant monitoring, maintenance and repair activities are needed in order to maintain the infrastructure systems in good condition and extend their service lives. However, maintenance and repair activities alone cannot prevent continuous deterioration of infrastructure systems so that a rehabilitation strategy should be developed. There have been many efforts to determine the optimal rehabilitation time for infrastructure systems using the optimization model (e.g., project level, network level, or a combination of both). In particular, the optimization models at the network and combined levels require the prioritization approaches (e.g., single year, multiyear, or yearly-based multiyear analysis) to prioritize and screen rehabilitation projects within the available budgets.

Recent research and practice focus on developing the optimization mode in terms of integrating the project and network level analysis over a long-term budgetary goal. However, they still have limitations as a rehabilitation planning model due to the two challenges in infrastructure management: 1) no considerations of non-selected rehabilitation projects within the multiyear prioritization approaches due to limited budget and 2) incompatibility of the purpose of a multiyear budget approach with the required annual rehabilitation cost.

This paper suggests a new paradigm for efficient infrastructure management to overcome those challenges. That is, infrastructure management strategy should be established in the consideration of the deferment of rehabilitation projects and rehabilitation time float. This paper proposes the optimal rehabilitation strategy for infrastructure systems within the context of maintenance, repair, and rehabilitation (MR&R) over a multiyear period in a new paradigm. The rehabilitation time float is defined as the acceptable time frame within which a rehabilitation project can be accommodated to develop a leveled annual rehabilitation requirement cost over a multiyear analysis period.

Therefore, this paper first presents the limitations which remain in the current optimization models for rehabilitation strategies. Also, it briefly reviews the purpose of a multiyear budget program for infrastructure management. Then, this paper introduces a new paradigm that suggests the future direction to deal with the challenges in the state-of-the-art rehabilitation strategies over a multiyear period. Finally this paper illustrates the new paradigm with a hypothetical example of concrete bridge deck systems.

The research methodology consists of four processes: 1) identification of an optimal MR&R strategy at the project level, 2) leveling of annual rehabilitation costs for all rehabilitation projects needed at the network level using rehabilitation time float, 3) multiyear analysis to provide candidate rehabilitation projects to public agencies for planning of multiyear capital investment, and 4) annual reanalysis for the selection of final rehabilitation projects at a target fiscal year. To demonstrate the research methodology, this paper uses concrete bridge decks as a hypothetical example in the state of Indiana. Those data can be obtained from the National Bridge Inventory (NBI) database.

The suggested multiyear infrastructure rehabilitation strategy using the new paradigm could help public agencies do the following:

- Identify leveled annual rehabilitation costs which they should dedicate to keep their infrastructure systems in good condition.
- Establish a long-term and steady budgetary goal for rehabilitation programs.
- Make a more reliable decision for a multiyear rehabilitation program.

Structural retrofitting for maintenance and rehabilitation

Organizer: A. Unterweger

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Strengthening and repair of damaged structural elements of revitalized apartment, public service and industrial buildings from the turn of the 19th and 20th century in Poland

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ABSTRACT

A lot of old residential, public service and industrial buildings which to the present time still create a substantial part of many of the Polish cities were erected at the turn of the 19th and 20th century (Berkowski et al. 2004, Berkowski et al. 2010).

Evaluation of the technical state of these old residential, public and post-industrial buildings is a very important task for structural engineer and is one of the key elements in the design of revitalization process. One of the aims of this work is to present some aspects of verification of constructional and material suitability of these buildings for the revitalization design. Set of specific procedures and also nature and range of examinations and calculations that must be conducted during this process are defined to guarantee the best evaluation of structural technical state of these old buildings. Considerable range of work related to the assessment of technical state of such buildings should precede architectural conceptual work, which, from the other side, should be strictly carried out in close connection with the results of the evaluation of structural technical state. Depending on the degree of structural deterioration, building is qualified for repair, major repair or demolition. Economical aspects of revitalization process are also taken into account, although other factors as social, maintenance or functional ones may also have an impact on decision-making. Following this procedure the appropriate methods of strengthening and repair are selected for these structural elements which do not ensure strength, service and overall safety requirements.

All structures are subjected with time to processes of deterioration which finally leads to a situation in which they became not able to grant the purpose they were built for. However, rate of destruction depends on various factors, taking into consideration the age of building at the first place. Among the most important can also be specified: quality of workmanship and materials, way of the use of building, ongoing repairs and maintenance procedures. Basing on these elements can be defined the mean time of different building durability (the service life), taking into account proper maintenance and lack of exceptional cases (natural disasters, wars etc.).

Figure 1. View of old apartment building's façade before and after renovation.

Carrying out the proper assessment of technical condition of old buildings, as well residential, commercial and post industrial ones, is an important task for structural engineers. Their task is to determine the structural suitability of these facilities for the newly designed purpose of revitalization. System of the appropriate standard procedures can ensure the correctness of the technical assessments. A significant range of these activities must precede any action of architectural design, which, in turn, must fully take into account the results of the assessment of technical condition of reconstructed objects (Fig. 1).

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The functional-structural rehabilitation of a building belonging to the archaeological industrial heritage

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ABSTRACT

The study building was built in the early 1900 in Naples (Italy), in the site of Bagnoli, where the siderurgical company ITALSIDER, now dismantled, was located. At present the Executive Urbanistic Plan, by Bagnolifutura Company, aims at enhancing the exceptional cultural-tourist vocation of the area.

The building was originally a thermo-electrical power plant. It had a former steel structure, typical of the industrial buildings (Fig. 1), which underwent some modifications along time, due to 2nd world war induced partial collapses, extensions, reductions, as well as changes of use related to siderurgical site needs. As a result, the current configuration of the building (Fig. 2) presents a main single-story nave, a secondary two stories nave, having an overall rectangular plan 35x48m wide and a 20 m ridge height.

Nowadays the building is degraded because unused and abandoned since many years (Fig. 2): the west façade is collapsed, some members are lacking, steel is widely deteriorated due to corrosion, some connections failed, façades walls partially crumbled.

The rehabilitation is devoted on one hand to retrofit the existing steel structure, by preserving the original typology, on the other hand to integrate it with a new construction made in part of steel and in part of reinforced concrete, aiming at the change of destination of use of the building (Fig. 3). The new construction should host multisports activities, such as swimming and fencing. From the structural point of view, three separate constructions can be identified: 1) the original steel structure; 2) the new main structure; 3) the structure of the semi-Olympic pool. The structural design is performed according to the Italian seismic code (CS.LL.PP. 2008 & 2009). The restoration of the historical building structure enforces the disassembly, revision and final repositioning of either restored or new elements. The substitution or integration with new members are required for meeting the strength, stability and deformability demands.

The paper depicts the actual state of the construction, evidencing the degradations, it presents a brief review of the structural-functional modification and illustrates the rehabilitation intervention.

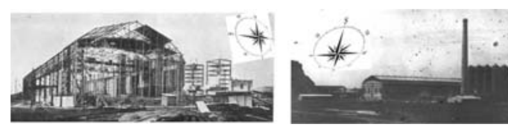

Figure 1. The original building (SIUMR 1928).

Figure 2. The building in the current state.

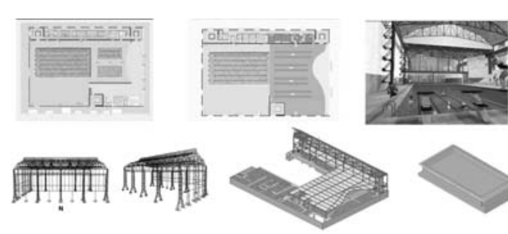

Figure 3. The functional-structural rehabilitation design.

The activity is developed by the authors and eng. Matteo Esposto, as consultants engineers for structures, within the group coordinated by eng. Claudio Rossi of the PSE s.r.l. Company (Naples, Italy).

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Construction rating attempt under life-cycle design

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ABSTRACT

The paper is dealing with the practical problem of an existing steel industrial hall with crane, located in Timisoara/Romania, for which the investor was requiring the conversion into a multistory office building. The geometrical dimensions of the existing hall were: span = 21.0 m, bay = 9.0 m, length = 3 × 9.0 m = 27.0 m, eaves height = 18.0 m. The capacity of the crane was of 12.5 t. The existing steel elements of the hall were made with built-up cross-sections of welded steel plates and the analysis has found elements of class 4 or at most class 3 which makes them quite inappropriate to provide structural ductility in case of a severe earthquake.

On the other side, the demolition of the existing structure and replacement with another one was not allowed by the authorities. In order to fulfill the requirements, a number of structural transformations were operated on the steel structure by inserting some supplementary transverse frames built of hot rolled European profiles at mid-bay and also of 5 levels of beams to obtain the required ground floor + 4 story building. As a result of the performed structural conversion, the existing industrial hall with crane, already obsolete in the central area of the city, was transformed into a five storey office building, having a much better environmental impact.

An attempt of construction rating (not frequent in our country at present time) in relation with life-cycle analysis principles is made in the paper.

In the first phase a simulation was performed using LEED, in order to understand the strengths and weaknesses of the project and the measure that could improve the building performance from the perspective of environmental pillar.

The second simulation used a multi-criteria decision based methodology that allows the integrated assessment of all pillars of sustainability. There are 12 criteria and 42 parameters used in this sustainability framework, grouped in four main chapters: environmental quality, economic quality, social and functional

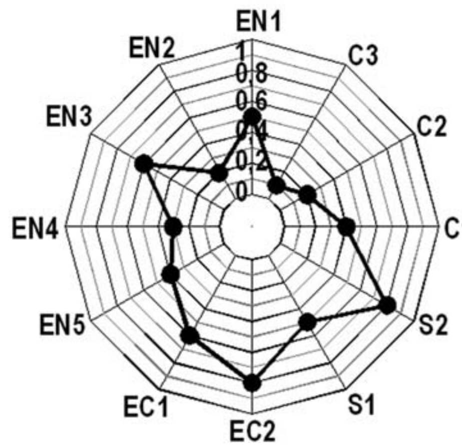

Figure 1. The final result for the rating presented in for of a radar diagram.

quality, cultural and institutional quality. The radar diagram shows how the project scores in different areas and where is place for improvement.

The conclusions should help to the choice of a proper construction rating system to be proposed for extensive use at national scale especially in the project phase.

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Tension stiffening of RC members subject to biaxial tensile stresses

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ABSTRACT

This paper discusses the contribution of strengthening with Near Surface Mounted (NSM) carbon strips on the tension stiffening and cracking behavior of concrete tension members. The experimental program involves ten uni-axial tests to study the tension stiffening and cracking behavior of reinforced concrete members strengthened with NSM carbon strips subjected to cyclic tensile load. The strengthening with NSM strips significantly increases ductility and tension stiffening of RC members strengthened with carbon strips. In addition, the strengthening with NSM carbon strips significantly reduces the crack spacing and crack width in RC members.

INTRODUCTION

The tension stiffening of reinforced concrete members strengthened using near-surface mounted CFRP technique has not been studied extensively until now [1][2].This technique is often able to mobilize a greater proportion of the strength of the FRP because of superior bond characteristics that help to prevent debonding failures [3] [4]. The influence of the reinforcing ratio [A_{CFRP}/A_{Steel}] on the tension stiffening of reinforced concrete members strengthened with near-surface mounted has not been sufficiently investigated [5]. A summary of the typical response of an RC member without strengthening and effects of tension stiffening is given, by (CEB) [6], (Mitchell) [7], and (Kishi) [8]. Ten uni-axial tensile tests are carried out. Three specimens without CFRP strips are used as reference specimens, and seven specimens are strengthened with CFRP strips. The influence of the reinforcement ratio [A_{CFRP}/A_{Steel}] and the type of load [static and cyclic] on the tension stiffening, crack width and crack spacing of RC members strengthened with near-surface mounted CFRP strips is studied.

CONCLUSIONS

The main conclusions can be summarized as follows:

1. The reinforcing ratio (A_F/A_S) has a significant effect on increasing the tension stiffening in specimens with small bar sizes.
2. The potential for forming splitting cracks increases as the bar diameter increases (i.e., for the larger bar sizes) and the beneficial influence of strengthening with CFRP strips on tension stiffening is reduced.
3. After yielding of the reinforcing bar, only those specimens strengthened with CFRP strips showed tension stiffening.
4. The specimens strengthened with CFRP strips exhibited larger amounts of tension stiffening than similar unstrengthened specimens.
5. The influence of the cyclic loading [Until 400.000 cycles] on tension stiffening was insignificant.

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Behaviour of interfaces in repaired/strengthened RC elements subjected to cyclic actions: Experiments and modelling

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ABSTRACT

In several seismic repair and/or strengthening techniques of Reinforced Concrete (RC) elements, a new concrete bearing element is added to the existing structural member. The behaviour of the interface between existing and new concrete, including the steel dowels that are present to enhance integrity, may become critical under cyclic actions, causing substantial degradation of the resistance of this interface thereby reducing the effectiveness of the connection to transfer load. On the other hand, when designing a reinforced interface, one cannot superimpose the maximum resistance of the two main mechanisms (shear friction and dowel action). Actually, both the interaction between the mechanisms and the fact that their maximum resistance is not mobilized for the same shear slip value should be accounted for.

The available experimental data regard mainly interfaces under monotonic actions and the information is not sufficient for the design in the case of RC structures subjected to earthquake excitations. Cyclic tests simulate various cases of interfaces, such as construction joints, connections between precast elements (Soudki et al., 1995), natural cracks (Maksoud, 2002), etc. Data regarding the behavior of reinforced interfaces simulating the interfaces between old and new concrete in repaired/strengthened elements, subjected to cyclic shear slip are rather scarce (Bass et al., 1989, Valluvan et al., 1999).

A series of research programs has been carried out at the Laboratory of RC Structures, NTUA, for the systematic investigation of RC interfaces within repaired or strengthened elements. Artificially roughened concrete interfaces crossed by reinforcing bars in the form of dowels (the percentage of the reinforcement, the embedment length of bars and the concrete strength being among the investigated parameters) are subjected to cyclic imposed shear slips of varying amplitude (± 0.1 mm to ± 3.0 mm). The test setup and instrumentation allowed for the resistance of the interface and for the force-response degradation to be measured.

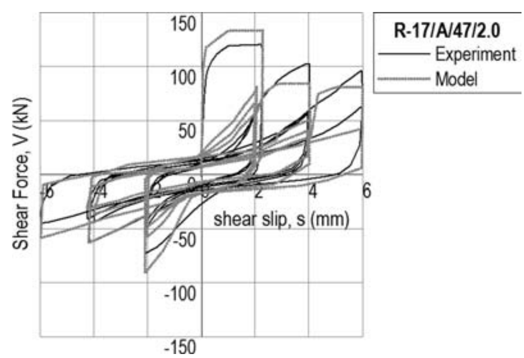

Figure 1. Typical hysteresis loop for specimen. Comparison between experimental results and model predictions.

In the present paper a summary of the experimental results is presented regarding interfaces reinforced with 8 mm diameter bars, as well as the results of an analytical study undertaken with the aim to model the behavior of the RC interfaces under imposed cyclic excitation. The ability of the model to predict experimental response is demonstrated (Fig. 1).

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Damage due to traffic before and after rehabilitation of a reinforced concrete bridge

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ABSTRACT

An investigation of the damage caused by road traffic was performed on a reinforced concrete bridge called the Weizbach Bridge (Figure 1) in the province of Styria in Austria. This study was undertaken with a focus on the determination of the damage induced by heavy vehicles in relation to the damage caused by average everyday traffic. A damage model based on fatigue of reinforcement bars was employed.

The Weizbach Bridge is a three-span span reinforced structure constructed in 1969 with spans of $10 + 20 + 10$ m and a double-Tee cross-section carrying two lanes. This bridge will be due for rehabilitation in mid-2012, two years after completion of the described study.

A monitoring program over more than seven weeks was undertaken. Measurement data was collected in order to serve as input for a damage model giving an indication of damage induced by traffic. Traffic across this bridge included special transport vehicles that haul loads in excess of 100 tons.

It was found that the damage caused by a typical special transport vehicle passing over the bridge can be compared to the damage caused by all traffic during one average work day.

Dynamic effects caused by a depression near the bridge joint were shown to contribute significantly to the overall damage caused by traffic.

Figure 1. Weizbach Bridge.

The rehabilitation brief had several objectives, one of them being the reduction of ongoing accumulating damage due to heavy traffic. It was decided to implement stressed bars on each side of the webs of the Tee-beams. These bars will run horizontally at about the level of the centroids in the side spans, and increase their lever arms in order to counteract bending moments in the mid-span. Bars will be stressed to ensure decompression under permanent loading, thus closing the existing bending cracks. These bending cracks are expected to re-open under heavy traffic loading. However, crack widths, and therefore stresses in the reinforcement bars, are expected to be much smaller than before rehabilitation. Due to the double-logarithmic nature of the S-N curve this reduction in steel stresses will lead to disproportionally high reductions in damage.

The monitoring program will be repeated upon completion of the rehabilitation in order to evaluate its effects.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)
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Rehabilitation of existing structures by optimal placement of viscous dampers

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ABSTRACT

The aim of the present article is threefold. First, it aims to test an optimal damper placement algorithm for the seismic conditions of pulse like earthquakes, such as the ones generated by the Vrancea source. Secondly, it studies the influence of specific pulse like earthquakes on internal forces in the elements of the structure. Thirdly, it analyses the extent to which the viscous dampers increase the performance objectives of a structure.

Although there are studies on the optimal viscous damper placement, most of them, similar to Martinez and Romero's (2003), use a series of trial and error time history analyses to establish an optimal distribution. This article aims to test a method developed by Takewaki (2009) to the mentioned particular seismic conditions. The method promises to provide a more direct, theoretical approach of assessing an optimal damper distribution. The algorithm aims to minimize the sum of mean square response of the interstorey displacement while accounting for certain design restrictions.

The algorithm is applied to a six storey concrete building designed for an earthquake with a 100 year mean return period (Sf = 1), which needs to be rehabilitated in order to ensure design requirements for a 475 mean return period (Sf = 1.5). The possibility of using viscous dampers to rehabilitate the building is studied using Incremental Dynamic Analysis (IDA). In order to perform the IDA, 5 accelerograms are used, 1 recorded earthquake from the Vrancea source and 4 generated spectrum compatible accelerograms using the Vanmarke (1976) algorithm. For each accelerogram 4 scaling levels of the PGA are used (Sf = {0.6, 1, 1.5, 2}).

The nonlinear behavior of the structure is modeled using plastic hinges at the end of each member. A uniform distribution which has the same sum of damping coefficients as the optimal distribution is also studied.

The results indicate that the optimal distribution algorithm works for the particular conditions considered.

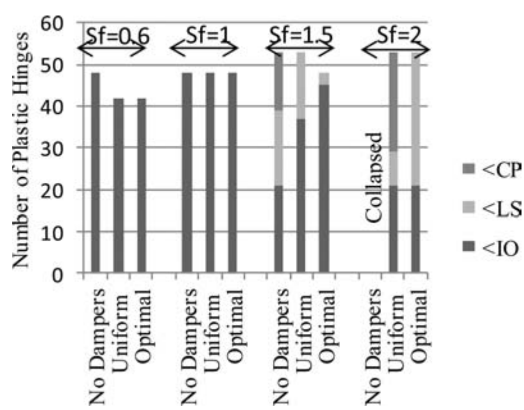

Figure 1. Number of plastic hinges and corresponding performance criteria.

The dampers are more effective for higher levels of seismic action, for a Sf = 2 the decrease in displacement is 50%, for the optimal distribution. The structure without dampers does not meet the Life Safety (LS) requirement for a Sf = 1.5 or the Collapse Prevention (CP) requirement for a Sf = 2. Both of these requirements are met by the structures outfitted with an optimal distribution of viscous dampers.

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Experimental and numerical investigation of model tests strengthened by overlays

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ABSTRACT

The present contribution focuses on the experimental evaluation and the numerical simulation of the effects of drying shrinkage on the behavior of concrete structures strengthened by overlays.

Drying shrinkage of concrete is characterized by the time-dependent volume decrease due to moisture migration and moisture transfer to the environment caused by a decrease in ambient relative humidity (Wittmann 1985).

In order to investigate the impact of drying shrinkage of concrete overlays on the behavior of strengthened concrete structures in the first part of the contribution a comprehensive laboratory test program is presented. By means of shrinkage tests on thin concrete slices the water desorption isotherm as well as the drying shrinkage strains in terms of ambient relative humidity are determined. Depth dependent moisture distribution profiles were determined by measuring electrolytic resistances by means of Multi-Ring-Sensors, placed in concrete prisms. In addition to the lab tests for determining the hygral and mechanical properties of the employed concrete, tests on larger brick-shaped specimens supplemented by concrete overlays were performed. Multi-Ring-Sensors were placed in the near-surface region of the brick-shaped concrete specimen for monitoring the depth dependent moisture distribution during drying for more than two years, during roughening and wetting of the interface and after placing the concrete overlay. In addition, moisture migration and the shrinkage strains were recorded in the concrete overlay during hardening and drying.

For commonly encountered values of relative humidity the physical origin of drying shrinkage is related to the increase of the capillary pressure in the porous concrete during the drying process (Baroghel-Bouny et al. 1999). Hence, a physically based model of drying shrinkage relies on a multi-phase formulation, in which concrete is considered as a porous material, consisting of a solid skeleton and voids, filled by liquid water and gas (Gawin et al. 2006).The multi-phase concrete model as well as the determination of the respective material parameters for drying shrinkage from measurement data are described in the second part of the contribution.

Finally in the last part of the contribution the application of the multi-phase concrete model to the numerical simulation of the laboratory tests, aiming at the investigation of the effects of drying shrinkage of concrete overlays, is presented. The computed response compared to available experimental data demonstrates the capabilities of the numerical model.

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Post installed fastenings at retrofitting systems in Japan

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ABSTRACT

The last events in Japan have shown the importance of earthquake safe construction. Social important buildings like schools, government buildings and ample apartment houses have been strengthened in the last decades till now, mainly by use of post installed anchors. Most of the buildings have widely withstood the Pacific earthquake, but some of those buildings have been already on the limit of their resistance before retrofitting, what shows once more the importance of the professional and efficient application.

Post installed anchorage in concrete is applied frequently because of its flexible use in all kinds of connections in concrete structures. It finds widespread application for the fastening of nonstructural elements to structures and is frequently used to connect new structural elements to existing structures in earthquake retrofit schemes. The deformation of such a structure (steel brace or shear wall) is shown in Figure 1. First of all the deformed structure shows a diagonal compression strut which is introduced into the existing structure. The other diagonal under tension produces gaps which have to be minimized through the use of anchorage as connectors. In addition to the shear transfer the transfer of tension forces plays a serious impact on the global behavior.

Mainly these connectors are located close to the edge, and without proper guidelines it is a difficult situation for the design engineers to estimate the proper way of installation. The current Japanese guideline is compared with the corresponding state of the art and the case of tension load is focused in this paper.

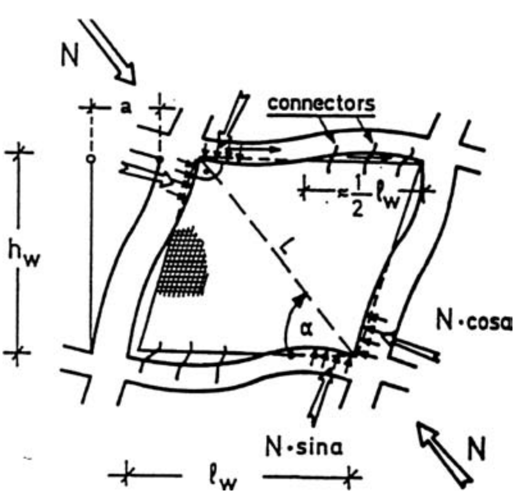

Figure 1. Deformed post installed shear wall at earthquake loading (Guidelines for Seismic Retrofit, 2001).

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Strengthening of historical stone masonry buildings: Experimental testing and modeling of a 2-storey plain masonry building

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ABSTRACT

Under the framework of NIKER EU-FP7 project, the seismic behaviour of three leaf stone masonry substructures and building models before and after strengthening is investigated. Within this paper, a summary of selected experimental and analytical results of a 2-storey (three-leaf) rubble stone masonry building with timber floors is presented.

The specimen (Fig. 1) was tested on the earthquake simulator in the facility of Laboratory of Earthquake Engineering/NTUA. The 1:2 scaled model simulates typical historical buildings in Europe. The model was tested twice (as-built and after repair and strengthening). The model was subjected to several accelerograms (with gradually increasing maximum acceleration). Prior to the performance of the shaking table tests, the dynamic characteristics of the specimen were estimated. The techniques applied to improve the seismic behaviour of the model are a) grouting of masonries (using a hydraulic-lime based grout developed within NIKER) and b) enhancement of the diaphragm action of floors with a technique used during the testing campaign at University of Padova (Valluzzi et al. 2010).

The damages observed in the as-built model have confirmed the vulnerability of the three-leaf masonry structures, as well as the negative effect of flexible floors. Significant improvement of the seismic behaviour of the building was achieved (in terms of maximum imposed acceleration and in terms of reduction of their vulnerability to separation of the leaves and to out-of-plane bending) after the strengthening of the model. Therefore, the previously mentioned techniques seem able to significantly enhance the overall seismic behaviour of such structures.

The numerical part of this research is being ongoing. Thus, only some preliminary results regarding the as-built model are presented in this paper. The as-built stone masonry building was modeled using the finite element software Abaqus 6.10. The analytical results obtained seem to confirm the observed behaviour of the model.

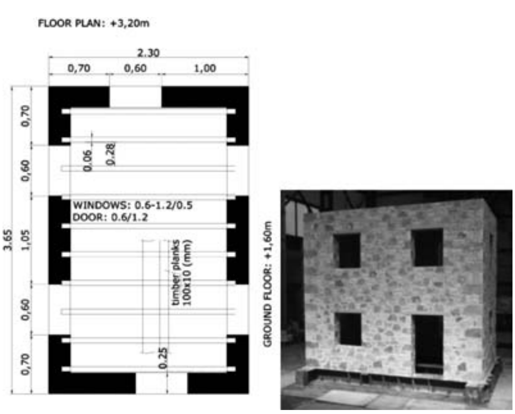

Figure 1. Plan and picture of the two storey building model.

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Fatigue of concrete – experiments, models, applications

Organizers: S. Seitl, Z. Keršner, R. Pukl & D. Pryl

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Experimental investigation of transitional size effect and crack length effect in concrete fracture

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ABSTRACT

The effects of structure size and crack size at quasibrittle failure are an important aspect of life-cycle analysis of concrete structures, they influence the brittleness of failure. The size effect, which consists of a dependence of the strength of geometrically scaled structures on the structure size, has been widely investigated for (i) structures with a large notch or structures that develop a large crack prior to reaching the maximum load, which gives the 'Type 2' size effect, and (ii) for structures having neither a notch nor pre-existing crack, which give the 'Type 1' size effect. These two size effects are quite different. In practice, however, there are also structures where the crack at maximum load or pre-existing crack is neither large nor negligible compared to the structure dimensions. Therefore, the transition of the nominal strength as well as the post-peak behavior between Type 1 and Type 2 size effects is also important. For a life-cycle analysis, it is essential to know the structure capacity at a given point in time. Determining the strength from equations that do not consider the size effect can lead to insufficient maintenance plans, higher costs and potentially unsafe structures. To determine this transition, a large-scale experimental program has been undertaken. More than 120 geometrically similar three-point bend beams of 4 sizes, with the size range of 1:12.5, as well as 24 cylinders for standard compression strength tests and 12 beams for ASTM flexural strength tests, were cast from the same batch of concrete to ensure minimal random variability of material properties. One-sided notches of four different depths, ranging from 0.025 to 0.3 times the beam depth, were cut at mid-length, additional beams were left with no notch. All the specimens were moist cured until the time of testing. At the time of paper submission, only partial results are available. The results on the effect of notch length varying from 0 to large values show that the 'boundary element model' previously expounded by Hu, Duan and Wittmann is invalid. A detailed journal article is in preparation.

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Experimental study on fatigue durability of RC road bridge decks subjected to chloride induced deterioration

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ABSTRACT

Recently, lots of reinforced concrete road bridge decks in cold regions in Japan are being severely damaged by fatigue due to vehicle loadings and chloride induced deterioration due to deicing salt. The aim of this study is to investigate the fatigue durability of RC road bridge decks subjected to chloride induced deterioration experimentally.

The size of the specimen is 3000 mm long × 2000 mm wide × 160 mm thick. The specimen was designed according to Japanese specifications for highway bridges specified in 1964. In order to perform the accelerated corrosion test, three types of experimental conditions were pre-pared, Type A: Adding salt of 10 kg/m^3 to fresh concrete at mixing, Type B: Immersing specimens in salt water (10% NaCl solution) for 3.5 day and drying them up for 3.5 days repeatedly, and Type C: Spraying salt water (10% NaCl solution) on the upper surface of specimens once a week. Additionally, the benchmark specimen without the accelerated corrosion test was prepared to compare to the specimens with corrosion. Then, the fatigue durability of these specimens under travelling wheel-type loads was investigated with a wheel running machine, which was developed by Dr. Shigeyuki Matsui (See Figure 1).

As shown in Figure 2, the experimental results revealed that the fatigue durability of RC bridge decks was remarkably degraded by chloride induced deterioration, where the equivalent number of loadings at 98 kN was calculated by the equation suggested in the previous work. Further, it was clarified that the fatigue life of RC bridge decks subjected to chloride induced deterioration depends on the corrosion of the upper reinforcement in compression rather than that of the lower reinforcement in tension. The reason is due to the fact that corrosion of the upper reinforcement in Type C as shown in Figure 3 results in the lateral cracks along the upper reinforcement and the cracks lead to rapid punching shear failure of the RC bridge deck.

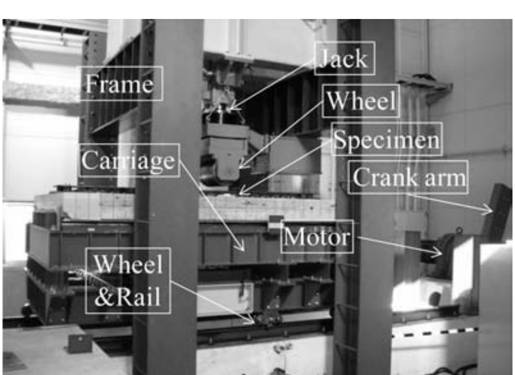

Figure 1. Wheel load traveling test equipment.

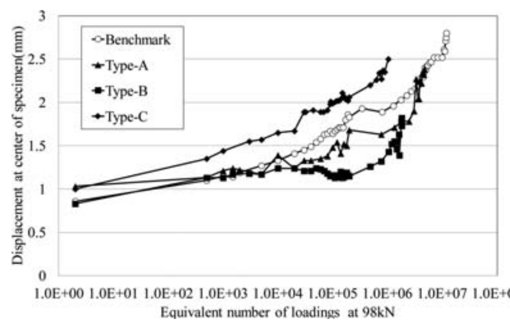

Figure 2. Relationship between displacement center of specimen and equivalent number of loadings.

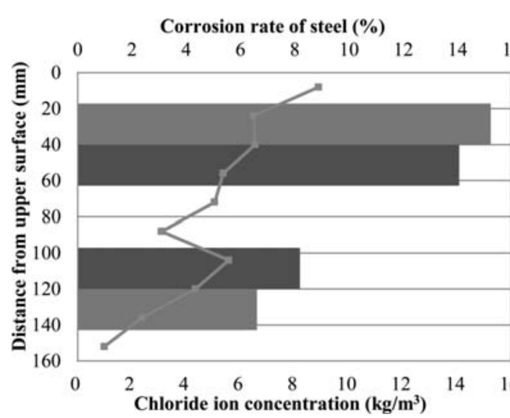

Figure 3. Distribution of chloride ion concentration and corrosion rate of reinforcement for Type C.

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Numerical modeling of crack growth in quasibrittle structures under compressive fatigue

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ABSTRACT

Many engineering structures, such as buildings, infrastructure, aircraft, ships, and biomedical implants, are often subjected to compressive cyclic loading. This paper investigates the fracture behavior of structures under compressive fatigue. Attention is limited to the structures that are made of quasibrittle materials, which are brittle heterogeneous in nature exemplified by concrete, fiber composites, ceramics, mortar, rocks, etc.

This study focuses on the mode-I fracture under compressive cyclic loading, which has been observed in many experiments. The simplest way to study such a fatigue behavior is to employ a cyclic cohesive element model, where the nonlinear behavior is lumped into a line of elements along the crack ligament. The corresponding constitutive behavior of cohesive elements is formulated for both tensile and compressive regimes. A plastic-type model is used for the compressive regime, where no damage could occur. By contrast, a strain-softening damage model is adopted for the tensile regime. Fig. 1 shows the simulated crack extension, which qualitatively matches the experimental observations (Ewart & Suresh 1986).

Based on a Fracture Process Zone (FPZ) equivalence principle, a crack growth rate equation for compressive fatigue is proposed by extending the well-established fracture kinetics equation for tensile fatigue, e.g. (Paris & Erdogan 1963, Priddle 1976). It is shown that the proposed crack growth rate equation agrees well with the simulation results.

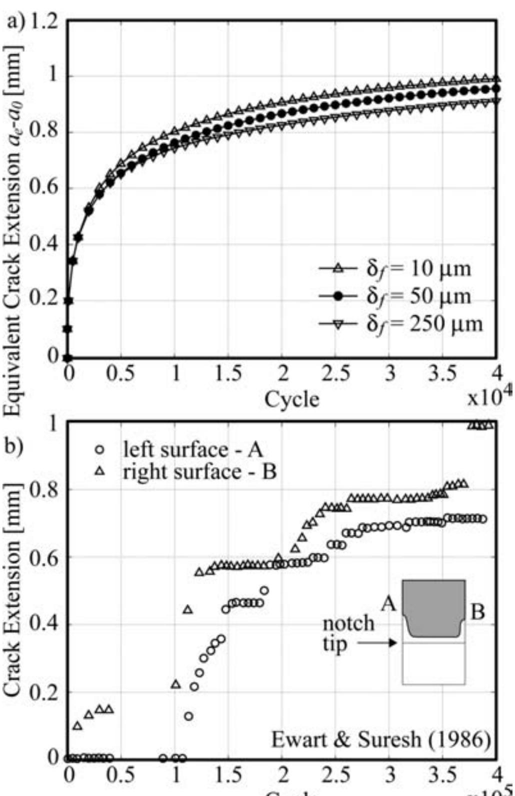

Figure 1. Crack extension a) simulated and b) experimentally observed.

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Modelling high-cycle fatigue of concrete specimens in three point bending

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ABSTRACT

A model is presented for fatigue crack propagation within the framework of the finite element smeared crack analysis. This model has been initially developed for fatigue life predictions of railroad concrete sleepers. As such it mainly concentrates on modelling fatigue behaviour of concrete under tensile load. It has been implemented in the ATENA Finite Element software package. The fatigue material model is based on the existing three-dimensional fracture-plastic material model.

Two fatigue contributions are considered: 1. crack initiation as a result of cyclic stress and 2. development of existing cracks based on crack opening and closing during the cycles. The damage is introduced into the material in the form of maximum reached fracturing strain.

The model has been used to model experiments of high-cycle loads applied to three point bending specimens of class C30/37 and C45/55 concretes tested in Brno. The static concrete material parameters were determined from monotonically loaded specimens and used for the models under cyclic load. Analysis results are compared to the measurements. The analysis results are in good agreement with the experimental data for both concrete classes tested. Some important aspects of damage introduction and stress and crack redistribution during high-cycle loading are also discussed.

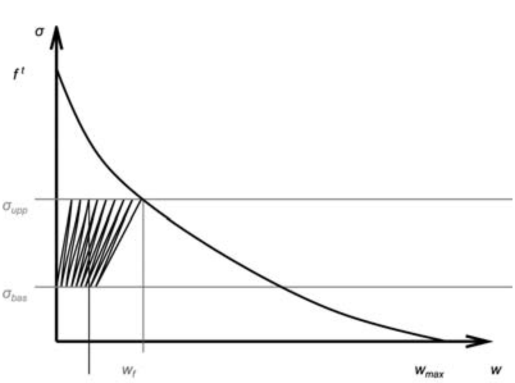

Figure 1. Softening law vs. crack opening displacement and fatigue damage.

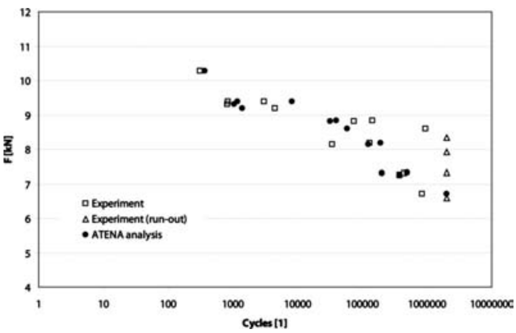

Figure 2. Comparison of measured and calculated results for C45/55 class concrete.

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Damage evolution in concrete under high compressive cyclic loadings

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ABSTRACT

The aim of this project is to get a deeper insight into the fatigue behaviour of concrete under cyclic compressive loading. It focuses on the evaluation and modelling of the entire damage process during life-cycle by means of various non-destructive measuring techniques. In numerous tests on cylinders h/d = 30/10 cm under different cyclic loading conditions, the crack development and the damage evolution are monitored using acoustic emission analysis and ultrasonic velocity measurement. The final aim will be the definition of a damage descriptor D based on experimental observations, which can be implemented into a numerical model on a meso-mechanical level.

The damage evolution of concrete under cyclic compressive loadings is a continuous process starting with the first loadings and going on during the entire lifetime until final failure. The nonlinear course of the material degradation in three principal phases (Fig. 1) has been confirmed by all types of non-destructive measurements. The material degradation results of the formation and expansion of micro-cracks spread over the entire specimen. Beginning at about 90% of the lifetime, the formation of macro-cracks leads to localized failure.

The existing normative regulations for fatigue failure regarding only the number of loadings cycles under defined stress conditions may be applicable for design purposes due to an appropriate safety margin. However, a realistic life-cycle prediction suitable f.e. for risk and return on investment considerations or the definition of maintenance strategies require other qualified parameters.

The results obtained in this project using different experimental techniques like strain and elastic modulus measurements, ultrasonic velocity and acoustic emission analysis reveal a clear potential to describe the three-phase fatigue behavior of concrete with some differences in the information value. While the acoustic emission analyses is most useful in defining the initiation of failure, the ultrasonic velocity is suitable to describe the stiffness reduction during life-cycle. In combination with the elastic modulus, which due to scientific experience has the greatest potential to take into account relevant material parameters like concrete microstructure, the life-cycle behavior of concrete under fatigue loading will be possible to describe qualitatively in laboratory tests. However, all techniques require at least an initial value at the beginning of lifetime and knowledge about the dependence of the decrease on the material parameters. The investigation of these dependencies will be the next stage of the project.

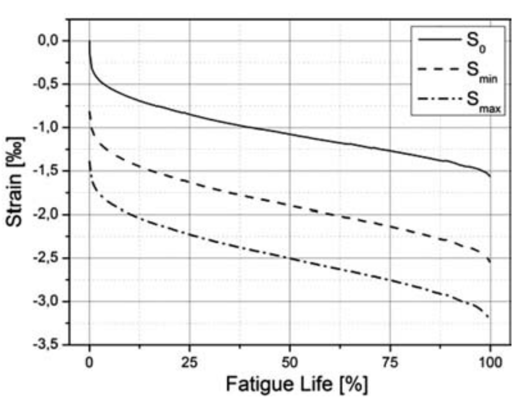

Figure 1. Longitudinal strains during life-cycle on different load levels (exemplary).

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Fatigue crack growth in cement based composites: Experimental aspects

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ABSTRACT

The evolution/growing of cracks in concrete structure requires reasonable modeling to obtain more reliable predictions of structural response to safe civil structure like bridges, concrete pavements, cooling and wind tower buildings against earthquake, traffic loads, environmental changes, and various other severe loads. For concrete the knowledge of fatigue fracture is still limited, see article by Lee & Barr (2004) for review. Fatigue fracture has previously been experimentally studied for normal strength concrete Bažant & Xu (1991), Bažant & Schell (1993) – size effect, Seitl et al. (2009) – glass fiber reinforced concrete. The purpose of this work is determination the material parameters that describe the fatigue fracture of C30/37 concrete.

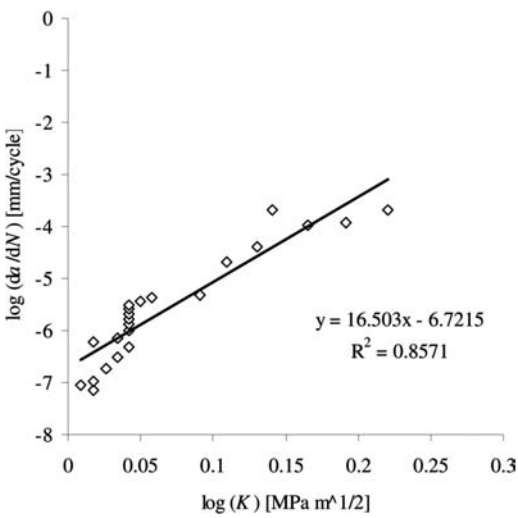

Figure 1. Relation between measured da/dN and K for specimen made from C30/37 concrete.

Such parameters are needed for predicting the growth of cracks in concrete structures under large repeated loads due to traffic, wind, etc.

The basic fatigue fracture mechanics parameters of C30/37 were measured. The measurement was based at new testing procedure to assess crack propagation performance of composites. The procedure has been applied successfully for the studied concrete, but for the data analysis some modifications were necessary.

The duration of the crack propagation phase was relatively short for all specimens. This may be improved with a shorter notch length (for example 5 mm) or sharpened shape of notch.

The results do not fully agree with field experience. The time dependence of change material properties during the test program may partly cause some discrepancy and should be studied further.

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Fatigue and cumulative damage of concrete grain silos

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ABSTRACT

As a rule, the concept of fatigue of reinforced concrete structures is related to the reduced steel and concrete resistance under large number of loading cycles.

In accordance with the EN 1992-1-1code, typical structures or structural elements for which a fatigue verification should be carried out, are those "subjected to regular load cycles (e.g. crane-rails, bridges exposed to high traffic loads)". The code procedures applies to cases where the number of cycles is assumed to be $N = 10^6$ and the compression force is assumed to act only in one direction.

Still, there are situations when the structures are subjected to a small number of cycles of large intensities resulting in so called "low-cycle fatigue". Usually, this type of fatigue is produced by the seismic actions, but may be encountered in constructions, such as silos and reservoirs, which have a few cycles of filling every year and for which the design loads are frequently attained. To make things worse, these structures have thin walls, constructed by slip forming and are subjected to a very unfavourable stress combination: vertical compression and horizontal tension.

Between 1966 and 1990 an important number of large storage capacity grain silos, using the IPCMC II (Fig. 1) design were erected in Romania. They are constructed by slip forming, a technology that requires optimal condition for providing continuous casting of the concrete and highly qualified workers. The stored material produces dynamic, pulsatory pressures with maximum values upon cell emptying producing a low-cycle fatigue of the concrete. These two factors led to a poor behavior of concrete bins with large horizontal cracks. Such damages develop in time, the cracks become wider and the reinforcement bars are buckling, showing the settlement of concrete. Damages develop with every cycle of loading-unloading of the cell because of material fatigue 'concrete and steel' and once started they have an exponential development and, in time, can lead to partial or overall collapse of a battery. The damages and the rehabilitation solution of the Traian-Sat silo is presented.

Figure 1. IPCPC II Traian-Sat Silo. Overall view.

In conclusion, some proposals for improving the design of the large concrete silos are made.

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Study on wind-induced fatigue of transmission tower-line in hilly terrain wind field

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ABSTRACT

Ultra-High Voltage (UHV) transmission towers were usually built on higher slopes or higher top of the mountain on both sides of rivers, which made them always in the wind field influenced by the mountain terrain during the whole serving. Therefore, based on the wind tunnel test, we studied the hilly terrain wind field with different slopes, heights and distances in this paper. Meanwhile, the calculation method about wind-induced fatigue of transmission tower were presented based on the theory of linear fatigue damage, which could be used to analyze the wind-induced fatigue of UHV transmission tower-line coupled system in various circumstances.

The different experimental conditions of three types of mountain terrain means are as follows: (1) the gradient variation of single mountain are S1-S8, that is 1.00, 0.75, 0.60, 0.50, 0.43, 0.38, 0.33, 0.30; (2) the height variation of single mountain are H1-H3, namely 100 mm, 150 mm and 200 mm; (3) the distance change between two mountains is L1-L5, namely 0, 200 mm, 400 mm, 600 mm and 800 mm. Mean wind speed is 15 *m*/*s* under all conditions, wind speed time series of wind profiles are collected on five key positions of the ridge along-wind and typical positions on leeward direction.

The gradient of the slope has small influence on the mean and fluctuating wind of the windward and hilltop. And the near ground fluctuating wind great changes with the change of the height of mountain. On the contrary, the height of mountain hardly influence the mean wind at the windward side and hilltop.

The fatigue life of the most unfavorable forced poles of each tower-line system is calculated in the upward, hilltop and leeside under different conditions. Thus, the fatigue life of the transmission tower-line system could be obtained under each wind field conditions, as shown in the Table 1.

The mountainous terrain has obvious influence on the wind field of the leeward side at the foot of hill in three types of mountainous terrain. The fatigue life of transmission tower is minimum in this area, but it increases with distance rising after the leeward. The fatigue life of the tower is less influenced at the windward side and hilltop.

Table 1. Fatigue life of key members under different hilly terrain wind field (one year).

Terrain	Working conditions	Upward side Location 01	Upward side Location 02	Hilltop Location 03	Leeside Location 05	Leeside Location 10
Gradient	0.3	148	152	145	136	148
	0.5	138	141	133	133	146
	1.0	120	133	124	126	147
Height	100	151	151	122	127	151
	150	132	124	125	119	148
	200	114	115	114	111	142
Distance	100	123	127	133	119	143
	150	130	132	136	122	144
	200	133	140	140	126	148

Increasing durability of concrete structures

Organizer: J.L. Vítek

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Design of concrete structures for durability

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ABSTRACT

Design of concrete structures is currently focused especially on the effects of direct actions. Increasingly the consequences of indirect, accidental and environmental actions are manifesting. Long term exposure to environmental actions (chemical, biological and physical effects of the environment), causes deterioration of concrete and reinforcement. When considering the reliability of structures all types of actions should be taken into account. This holistic approach to the design and verification of structures shall be applied to all constructions, especially civil engineering works, because of their large ratio between the area exposed to the surrounding environment and cross-section dimensions as well as longer design life.

One of the most predominant factors responsible for the structural deterioration in concrete structures is identified as a corrosion of reinforcement, which may result in damage of the structures in the form of expansion, cracking and eventually spalling of the cover concrete.

The corrosion of the reinforcement is caused by the carbonation of concrete and/or the chlorides penetration into the concrete. A probabilistic approach is very appropriate to deal with problems with uncertainty and randomness which is the nature of the corrosion process and its effect on structural response. The probability of corrosion initiation at a given time follows $t, P_{f,i}(t)$ due to carbonation may be written as

$$P_{f,i}(t) = P\{a - x_c(t) \leq 0\} \leq P_{target} \tag{1}$$

where $a =$ concrete cover; $x_c =$ depth of carbonation at time.

The durability of concrete structures may be adversely affected by excessive cracking. The corrosion of reinforcement results in the formation of various corrosion products due to the process of oxidation and causing an increase in the volume. Based on this process, the probability of the serviceability failure due to corrosion induced concrete cracking, $P_{f,c}(t)$, can be determined from Equation 2

$$P_{f,c}(t) = P\{\sigma_t(t) \geq f_{ct}\} \leq P_{target} \tag{2}$$

Another response related to structural failures is concrete spalling. For given degrees of corrosion rate the risk for cracking, spalling and bond strength decrease depends mainly on the geometry of the cross section and the confinement.

The reduction in confinement on cracking of the cover will lead to a progressive reduction in bond strength.

The last phase of service life is the period of time from loss of serviceability to final collapse of the structure. Also, failure can occur in many modes, such as loss of both flexural strength and shear strength. The recommended target reliability indices β for ULS verification, related to specific reference periods and consequences of failures given in (fib 2010b). The target reliability level for the existing structures may be chosen lower than for new structures.

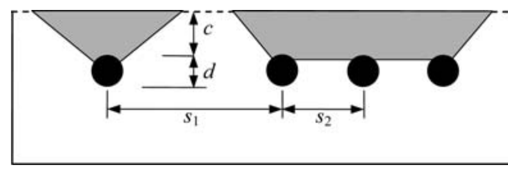

Figure 1. The geometric parameters affecting the spalling (Hunkeler 2006).

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Durability design of structural cover concrete based on bleeding rate

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ABSTRACT

In this research, the relationship between the results of the non-destructive surface air permeability test and the carbonation rate of concrete was examined in order to facilitate smooth information processing from the design stage to the construction, inspection, and maintenance stages. Corrosion of steel bar due to carbonation was set as the cause of deterioration of the concrete structure.

Through many experimental results, the following outcomes were obtained.

The relationship between carbonation rate and surface air permeability can be expressed by the following equation.

$$\alpha = a_i \log(kTi) + b_i$$

where i = age of concrete; a, b = experimental constant. The difference between the kT values at the same W/C can be simply expressed using the amount of bleeding, as the actual bleeding affects not only the concrete surface but also the inner concrete.

$$\log(kT28) = A \times W/C + B \times Bl + C$$

where Bl = amount of bleeding (cm^3/cm^2); A, B, C = experimental constant.

The change in surface air permeability may be governed by both the bleeding of concrete and consolidation due to the weight of concrete. The new index, which gives the amount of bleeding times the height, was introduced to express the change in the surface air permeability due to these influences.

$$\frac{kT28}{kT28s} = 1.0 (Blh \le I)$$

$$\frac{kT28}{kT28s} = II \times Blh + III (Blh > I)$$

where $kT28s$ = surface air permeability of the standard specimen; Blh = bleeding index (cm^3/cm); I, II, III = experimental constant.

A case study in the steel bar corrosion due to carbonation from the design stage to the maintenance stage was conducted according to the proposed method. Table 1 shows the design conditions. The combinations of W/C and the amount of bleeding will be calculated for satisfying the design conditions based on the examination result of this research.

Table 1. Design condition.

Item	Value
Designed service life of structure (year)	50
Design value of concrete cover (mm)	70
Remaining non-carbonated cover thickness (mm)	10
Design value of carbonation depth (mm)	<60
γ_{cb}	1.15
Design carbonation rate (mm/$\sqrt{\text{year}}$)	<7.38
Relative humidity (%)	60
Rainy days	133

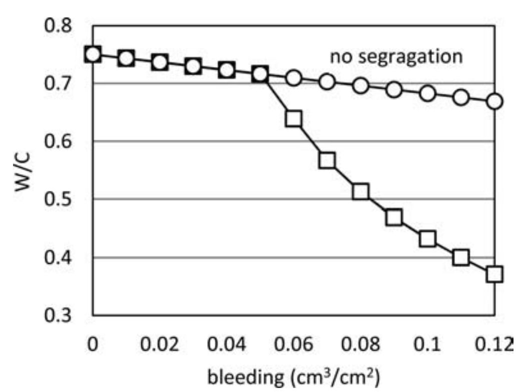

Figure 1. Combinations satisfying the design conditions.

The calculation results are shown in Figure 1. The W/C satisfying the design conditions decreases as the amount of bleeding increases. For instance, the amount of bleeding should be less than 0.083 cm^3/cm^2 when the W/C is 0.5.

When the standard construction works are applied as recommended by the standard specification for concrete structures in Japan, the coefficient of air permeability of surface concrete can be formulated as a function of the water-cement ratio, amount of bleeding water, and placement height per layer. The mix design, inspection of durability performance, and inspection of initial performance of structural concrete can be done based on the proposed method.

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Improvement of performance of concrete precast elements using FRC

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ABSTRACT

The aim was improvement of function and performance of some members by using FRC instead of ordinary reinforced concrete and to transfer innovative technologies from laboratory in academic sphere into real industrial production which is cost-effective and bring about savings of labour and material.

For groups of precast concrete members intended for application it was necessary to choose different efficient type of fiber concrete with suitable mix composition. The reasons were differences in loadings and conditions acting on members during their service life. The result of the study was that concrete with synthetic or steel fibers were better solution for most elements and concrete with steel fibers was the more suitable material for various members due to its greater toughness and increased tensile strength in comparison to plain concrete. All selected fiber reinforced concrete materials have good ductility needed for elements exposed to severe conditions on bridges or on roads concerning loads and changes of temperature and humidity including influence of deicing means.

Performed tests, calculations and simulations show that prestressed columns have higher resistance than the reinforced concrete columns; at the same time the failure mode is acceptable and safe. For the tested length of columns are the prestressed columns reliable even without additional shear reinforcement or dispersed reinforcement. Yet the fibre reinforcement is assumed to enhance toughness, reliability and resistance to damage during transport and manipulation and also durability in severe conditions. Innovation consisted in a transition from current concrete to fiber reinforced concrete when an optimization procedure is necessary: starting from selection of proper type of fiber reinforced concrete mixture, adjusting shape and thickness of the member and experimental verification of structural behaviour of the new member accompanying with sets of additional tests including long-term

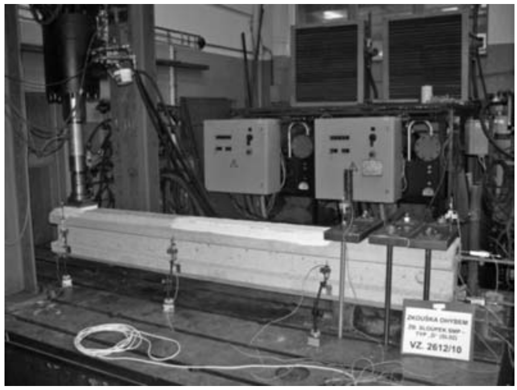

Figure 1. Set up of the full-scale test in a laboratory.

behaviour modelling and simulation. The procedure included optimization of amount of fibers so that it was ensured that the member would not fail or be damaged during function or transport under conditions prescribed for precast elements of common type and common reinforcement. The manufacturing in the real factory conditions was verified and successful production of members was started. The product was given the Innovation of the year Award by Association of innovative entrepreneurship of the Czech Republic that regularly appreciates an improved product, technology or service effectively located into the market.

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Increasing concrete resistance to deicing chemicals by using metakaolin

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ABSTRACT

This paper describes the influence of metakaolin on resistance of concrete to deicing chemicals. Metakaolin is an example of pozzolan with main component metakaolinite $Al_2O_3 \cdot 2SiO_2$ and is produced by calcination at temperatures around 600–700°C. After the calcination process metakaolin is grinded to high fineness of 5–50 μm. Addition of metakaolin to concrete or partial replacement of cement by metakaolin can change and in most cases even improve mortar and concrete properties. Metakaolin positively influences kinetics of hydration creep and porosity, reduces autogenous shrinkage, increases strength, and improves durability.

This paper describes the influence of metakaolin addition into air-entrained concrete to enhance its resistance to deicing chemicals called scaling. It is defined as superficial damage caused by freezing a saline solution on the surface of the concrete specimen (Valenza & Scherer 2007). The degree of damage is usually progressive and proportional to the number of freeze-thaw cycles and consists of removal of small chips or flakes of material. Concrete scaling does not decrease the flexural strength or structural integrity of the specimen. Four types of concrete mixtures were tested for scaling resistance. The mixture proportions are presented in Table 2.

Table 2. Mix proportion.

	kg/m^3			
Ingredient	REF	AE	MK5	MK9
CEM I 42.5	440	440	418	400
Metakaolin	0.0	0.0	22	40
Sand 0/4	795	795	795	795
Aggregate 4/8	315	315	315	315
Aggregate 8/16	670	670	670	670
Plasticizer	5.50	4.18	4.18	4.18
Stabilizer	1.32	1.32	1.32	1.32
Air-entrainer	0.0	0.33	0.33	0.33
Water	155	155	155	155

It was found that the Air-Entrained concretes (AE, MK5, MK9) have lower compressive strengths compared to concrete without air-entrainment (REF). However metakaolin helped to increase strength of concretes MK5 and MK9 by 30% in comparison to AE concrete. All scaling tests presented in this paper were performed according to CSN 731326/Z1 – method C. A significant influence of air-entrainment on scaling properties of concrete was measured during the experimental work. All concretes containing air-entraining admixture (AE, MK5, MK9) performed after 144 F/T cycles 4 to 10 times better than reference concrete (REF).

It was found that metakaolin does not significantly improve scaling results of AE concrete but it is very beneficial in increasing its strength. Therefore metakaolin can be used in applications were both high scaling resistance and compressive strength are required. The results were also compared to previous work conducted by the authors and it was found that metakaolin improves scaling resitance by nearly 40% in cases were no air-entraining admixture is used. This can be utilized in many applications. For instance in prestressed concrete, air-entraining increases creep of hardened concrete which is not desired. However in case of bridges, high durability is required. These both requirements can be satisfied in case of high strength concrete by replacing 5 to 10% of cement by metakaolin.

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Impact of cracks on the durability and on the service-life of fiber reinforced ultra high performance concrete – evaluation, modeling and structural measures

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ABSTRACT

Besides its high strength one of the main characteristics of Ultra-High Performance Concretes are a perfectly dense microstructure being widely free of capillary pores. Thus it is practically impermeable to harmful substances like chlorides from deicing salt or to gases like O_2 and C_{O2} both reducing the corrosion resistance of the steel reinforcement in structures made of ordinary and thus porous concrete. The rate of steel corrosion depends on the accessibility of oxygen and water near the steel-concrete interface, and is accordingly a function of the concrete permeability [Gowripalan et al. 1998]. Due to the brittleness of the matrix UHPC as a rule contains steel fibers to improve the ductility and to provide it with a certain post-cracking load bearing capacity both in compression and in tension. That means that the fibers become active when the adjoining matrix is already cracked. If the crack spacing is above a certain limit the cracks may give way for the intrusion of water, chlorides and carbonation despite of the facts that the matrix itself is still impermeable. The question is in how far the crack opening must and can be restricted to an uncritical value preventing the loadbearing fibers from corrosion. In a comprehensive study based on both laboratory tests and theoretical considerations validated by tests on UHPC-elements which has been in service for several years physically and chemically based algorithms has been developed to calculate the diffusion and the time depending rate of corrosion of steel fibers in UHPC. Implemented in a computer program it is a valuable tool for the designer to restrict the deformation of elements made of UHPFRC in a way that the fiber induced load bearing capacity lasts for the whole service-life the structure being designed for.

In this way, a comprehensive finite difference model for predicting the corrosion current as well as steel cross section loss in cracked concrete exposed to chloride ions has been developed. The model includes the effects of changes in exposure conditions, temperature, concrete age, crack width, concrete microstructure, and concrete cover on the corrosion behavior. The significant agents that contribute to the rate and amount of steel corrosion such as temperature, moisture, chloride, oxygen movement and distribution, have been modeled by the second order explicit finite difference method. The corrosion current and steel dissolution amount are predicted based on the potential distribution around the concrete-steel interface, which is obtained by solving the Laplace's equation [Burkan Isogar et al. 2006].

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Influential variabilities in reliability of reinforced concrete pipes

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ABSTRACT

In practice of the structural design of buried reinforced concrete pipes, the pipes strength are specified from several tests, being the three edge bearing test the most used one (El Debs, 2003).

This paper verifies the reliability of circular reinforced concrete pipes considering the quality control of the production. The pipes have geometry in spigot and socket and it is subjected to three edge bearing test. The pipes are evaluated in the ultimate limit state using experimental and numerical procedures.

The pipes are evaluated in the ultimate limit state for experimental and numerical procedures. In the experimental part, 24 pipes were tested from which 12 pipes were 800 mm nominal diameter and the other 1200 mm nominal diameter. According to procedure of test, only loadings applied in the tests were measured. However, in order to obtain more information to better understand of mechanical behavior of pipes, displacements were also measured using displacements transducers installed on the models.

In numerical part of this research, circular reinforced concrete pipes were analyzed according to a mechanical model based on finite element method developed for framed structures. The behavior of the pipes was predicted by stress and strain based model that allows to consider the physical nonlinearity. The geometrical nonlinearity was based on the large displacements and strains theory, in which the stiffness matrix of each element is found on a local corrotational coordinate system.

The probabilistic method to evaluate the reliability of circular reinforced concrete pipes in the context of non-linear analysis is developed by coupling a nonlinear finite element model with the Response Surface Method (*RSM*). The *RSM* appears associated with the well-known First Order Reliability Methods (*FORM*), or Second Order Reliability Methods (*SORM*), to estimate the structure failure probability of the structure. The structural safety is evaluation in terms of the reliability index (Ang and Tang, 1984; Soares, 2001).

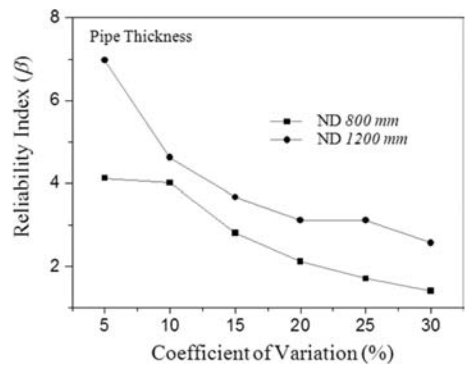

Figure 1. Parametric analysis.

A sensitivity analysis to determine the most important parameters in the reliability was performed. Among the most influential variabilities in evaluating of reliability index, the concrete compressive strength, the pipe thickness and the position of the reinforcement in the pipe can be highlighted. Also, it was observed that analyzed pipes presented reliability index greater than 4, to meet the diametrical compression test.

A parametric analysis was performed in the Figure 1, the coefficient of variation of pipe thickness (h) was modified for values of 5%, 10%, 15%, 20%, 25% and 30%.

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Life-cycle considerations for reinforced concrete structures in case of fire with respect to spalling

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ABSTRACT

Structural fire safety has become an important engineering discipline during the last years. Within buildings, the devastation caused by fire can result in failure of structural elements up to loss of life. Although fire usually occurs only once or twice during a buildings life-cycle, the understanding of the behaviour of buildings during fire becomes more and more essential. It is absolutely necessary to get more and more experiences in prediction of the buildings performance in fire. This allows designers to obtain more economical solutions and to identify any weak points within the structural system, ensuring a robust and safe design. Prescriptive rules deal with many simplifications and do not reflect the real performance of a building. The main weakness of these rules is the consideration of single elements only. The realistic behaviour and in particular the interaction with adjacent elements can not be taken into account. Well known fire tests have shown that fire resistance increases due to these effects [4]. Furthermore, the change of material properties during the life-cycle is neglected in most cases. Especially in case of fire, there are some special effects which should be considered. Some significant observations were made in concrete buildings in fire. In such buildings, structural elements were weakened in case of fire due to spalling. Spalling means an explosive blasting of parts of the structural element due to high water vapor pressures which is strongly dominated by the rate of moisture (Figure 1).

The aforementioned effects have significant influence on the fire safety and should definitely be considered. A disregarding of these effects leads to a overestimated fire resistance and the structure might fail in worst case.

In this paper, an approach to assess and predict the load bearing capacity of a representative building with regard to the fire safety is introduced. Sequential coupled thermal-stress analyses are presented in consideration of spalling effects. The effects of spalling

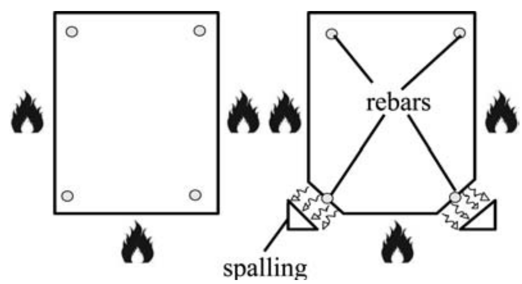

Figure 1. Cross-section with and without consideration of spalling effects.

during a buildings life-cycle have been taken into account with a coupled thermal-hygral and mechanical model [1,2]. The numerical coupled model allows an assessment of spalling time and spalling location from the mechanical and chemical damage of the structure.

The structural analyses of the total system have been performed including these effects. Finally, comparisons of the load bearing capacity, deflection and internal forces allow an assessment of the regarded system in case of fire.

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High performance concrete for the Troja Bridge in Prague

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ABSTRACT

The new network arch bridge is just being built in Prague. The paper describes advanced concrete types used during construction.

The bridge has two spans. The main span 200.4 m long crosses the Vltava River and the side span 40.4 m long is designed for the case of flooding. The bridge carries 2 tram tracks, 4 lanes for the road traffic and 2 pedestrian lanes. The bridge is about 35 m wide. The steel arch is very flat; its rise is 20 m, which is only 1/10 of the span. It represents a main load carrying element of the main span supporting the relatively thin bridge deck stiffened by transversal precast concrete beams. The stays connecting the arch and the composite steel concrete tie of the bridge deck are made of inclined steel bars with fork and pin connections at the ends. The side span has two longitudinal prestressed beams and cast in situ transversal beams supporting the concrete slab. The complete deck of the main, as well as of the side span, is longitudinally and transversally prestressed.

The precast transversal beams supporting the slab of the main span are made of concrete C70/85 XF2. A great attention had to be paid to the casting process. The transversal beams are only 500 mm wide and almost 30 m long. They are prestressed by 2 bonded cables, each including 9 strands 15.7 mm in diameter. Two anchors are located at the ends, while the other two anchors are passive inside the beam. The dense reinforcement not only in the anchorage area required to use the maximum aggregate size 16 mm. The total number of 47 transversal beams were produced in the precasting plant of SMP CZ, the subcontractor of Metrostav.

The side span and the slab of the main span were produced from concrete C50/60. Since the deck is rather thin (280–314 mm) and some parts, e.g. end cross-beams are in contrary very massive, the very robust concrete had to be designed. In order to prevent early cracking of the surface, PE fibres were added into the concrete mix. When the massive end cross-beams were cast, the cooling of fresh concrete using liquid nitrogen had to be applied.

The composite tie of the arch is located above the level of the bridge deck. It is exposed to the direct attack of the de-icing salts from the road. Therefore the concrete had to resist the agressivity of environment classified as XF4. The complex shape of the tie required a field testing. The test showed the possible difficulties when filling the formwork of the tie. Due the inclined surface in transversal and longitudinal direction (in some parts exceeding 10°) the self-compacting concrete could not be used. The so called easy-crete was designed which was able to fill the complete section with a limited compaction.

Very high forces must be transferred from the arch to the bridge deck and to the tie of the arch at the ends of the bridge. At these locations the arch footings are embedded into a bridge deck. In order to reduce the number of steel stiffeners inside the arch footings, their steel structure was filled with High Strength Self-Compacting-Concrete (HSSCC) C80/95. It allowed for a smooth transfer of forces between the deck and the arch and also the transfer of stresses at the anchorage area of 6 cables composed of 37 strands. The HSSCC was designed in the laboratory conditions and then tested also in field tests. It was necessary to verify, whether the HSSCC is able to fill a very complex shape between the stiffeners of the arch footing. The model of the most complex part of the footing was made of wood and after hardening of concrete it was cut and the cores were drilled. Finally the experiment showed that the designed technology is feasible and can be used in practice. At the time of writing the paper, the two footings were completed successfully.

ACKNOWLEDGMENT

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Numerical modelling of long-term behaviour of concrete structures using B3 model

Organizers: L. Vráblík & V. Kristek

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Influence of repeated variable load on long-term behavior of concrete elements

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ABSTRACT

The objective of this paper is to study the influence of repeated variable action on long-term behavior of concrete structural elements using quasi-permanent combination of actions for the assessment of long-term effects (e.g. effects due to creep and shrinkage in concrete structures), as it is proposed in Eurocodes. Extensive experimental program and analytical research using model B3 (Bazant & Baweja 2000) and AAEM method (Bazant 1972) was performed in order to define quasi-permanent coefficient ψ_2 for two specific loading histories. These loading histories were consist of long-term permanent action "G" and repeated variable action "Q". The variable action was applied in cycles of loading/unloading for 24 hours and 48 hours in period of 400 days appropriately for two series of concrete elements "D" and "E". 24 reinforced concrete beams, dimensions 15/28/300 cm, were tested. 12 beams were made of concrete class C30/37 and 12 of concrete class C60/75.

Experimental results from testing of drying shrinkage, autogenous shrinkage and creep of concrete were analytically verified using Model B3 and updated B3 on the basis of long-term measurements. For improvement of Model B3, update parameter p_1 and p_2 for creep compliance and p_6 for drying shrinkage were used obtained on the basis of experimental results. Analytical analysis of the autogenous shrinkage was done in accordance to model B3 proposing following improved formula based on experimental results:

$$\varepsilon_a(t) = \varepsilon_{a\infty}(0.99 - h_{a\infty})^{\frac{1}{\sqrt{t-t_s}}} \cdot S_a(t) \tag{1}$$

Typical diagram for development of deflection in the middle of the beam span during time "a – t" is given in Figure 1. Presented are experimental results and analytical results obtained by AAEM method (input parameters were defined by Model B3 and Model B3 improved).

ψ_2 factors obtained by the analytical analysis for series of beams D and E made of ordinary concrete C30/37 are given in table 1.

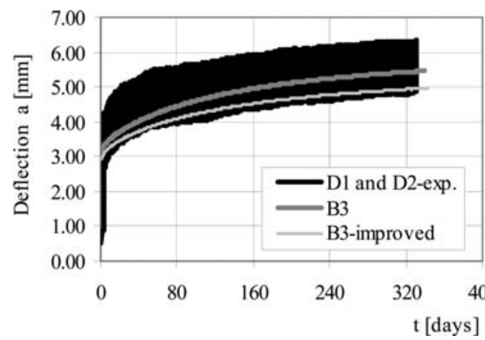

Figure 1. Diagram deflection-time for series D beams concrete class C30/37.

Table 1. ψ_2 factors for series of beams D and E made of ordinary concrete C30/37.

Series	Permanent action G (kN)	Variable action Q (kN)	Total load G + Q (kN)	ψ_2 factor	Quasi-permanent load G + ψ_2Q (kN)
D	2×4	2×7.6	2×11.6	0.49	2×7.7
E	2×4	2×7.6	2×11.6	0.66	2×9.0

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Modeling of concrete creep based on microprestress-solidification theory

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ABSTRACT

Creep of concrete is strongly affected by the evolution of pore humidity and temperature, which in turn depend on the environmental conditions and on the size and shape of the concrete member. Current codes of practice take that into account only approximately, in a very simplified way. More realistic description can be achieved by advanced models, such as model B3 (Bažant & Baweja 2000) and its improved version that uses the concept of microprestress (Bažant, Hauggaard, & Ulm 1997). The value of microprestress is influenced by the evolution of pore humidity and temperature.

The complete constitutive model for creep and shrinkage of concrete can be represented by the rheological scheme shown in Figure 1. It consists of (i) a non-aging elastic spring, representing instantaneous elastic deformation, (ii) a solidifying Kelvin chain, representing short-term creep, (iii) an aging dashpot with viscosity dependent on the microprestress representing long-term creep, (iv) a shrinkage unit, representing volume changes due to drying, and (v) a unit representing thermal expansion. All these units are connected in series, and thus the total strain is the sum of individual contributions while the stress transmitted by all units is the same. This model has been implemented into the finite element package OOFEM (Patzák & Bittnar 2001), which was used for running numerical simulations.

In this paper, values of parameters used by the Microprestress-Solidification theory (MPS) are recommended and their influence on the creep compliance function is evaluated and checked against experimental data from the literature. Certain deficiencies of MPS are pointed out and its modified version is proposed and verified. One example showing measured experimental data (Fahmi, Polivka, & Bresler 1972) and the results of numerical simulations with the original and improved material model is shown in Figure 2.

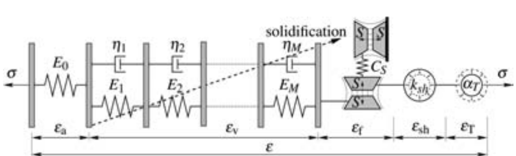

Figure 1. Rheological scheme of the complete hygro-thermo-mechanical model.

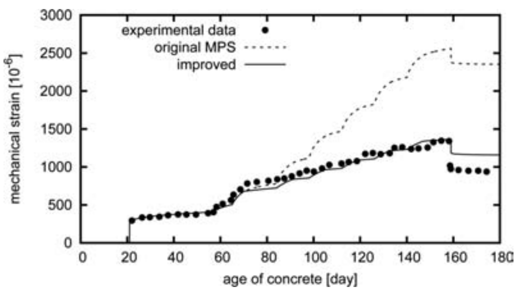

Figure 2. Evolution of mechanical strain of a sealed concrete sample subjected to several thermal cycles.

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Updating B3 model for long-term basic creep

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ABSTRACT

The B3 model for concrete creep and shrinkage was first published in 1995 (Baweja & Bažant 1995). Today, it is clear that long-term creep data were not presented sufficiently during the calibration stage, raising questions about long-term creep predictions of the B3 model. From the current, slightly appended Northwestern creep database (Bažant & Li 2008), containing 675 datasets (and 12258 points), only 20 datasets contain measurements after 10 years of loading. Brooks (Brooks 2005) measured 30-year creep using 18 concrete types from normal and light-weight aggregates. He concluded that the creep compliances were underestimated by all prediction models (CEB, GL, B3, ACI, BS). These data were appended to the creep database later on.

It is evident that excessive creep has consequences on concrete bridges. Up today, 66 large-span, segmentally erected bridges were gathered, exhibiting excessive deflections (Bažant, Hubler, & Yu 2011). A partial explanation, beside other factors, lies in micromechanically-nonlinear creep, which needs to be added to the B3 model.

Basic creep experiments lasting for 30 years demonstrated that after approximately 1000 days of loading the final slope of basic creep increases (Brooks 2005). These experiments covered ordinary concretes with w/c ratii between 0.5 and 0.8. This phenomenon can be captured in B3 model by multiplying the flow term (parameter q_4) with

$$\left[1 + l_l \cdot 13(w/c)^3 \cdot \left(1 - \frac{1}{1 + (0.0005(t-t'))^3}\right)\right] \quad (1)$$

where l_l is the load level at the time of loading. For a specimen loaded at the load/strength ratio of 0.3, the $l_l = 1$.

Figure 1 demonstrates the differences between original version of the B3 model and the updated version with a modification according to Equation (1). The figure plots the basic creep data from two concretes, where North Notts is of good quality. Interfacial transition zone around aggregates seems to be unchanged in both concretes since the slope change occurs roughly at the same time.

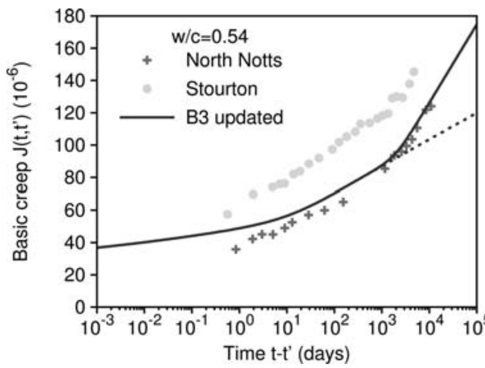

Figure 1. Prediction of the original B3 creep model and updated B3 model.

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Application of B3 prediction model to analyze prestress loss in prestressed concrete members

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ABSTRACT

Reliable design of prestressed concrete elements and structures must be based on the accurate determination of prestressing force (including losses and their time development) to satisfy stress as well as deformation limits. Commonly used obsolete computational routines and methods neglect or incorrectly describe real effects and the results obtained by analysis based on these methods are thus necessarily wrong – great differences from results obtained by measurements on real structures have been found. The paper is directed to long term prestressing losses due to creep, shrinkage and cross section warping.

The long-term deflection behavior of long-span prestressed concrete box girder bridges has often deceived engineers monitoring the deflections. A survey of many bridges monitored in various countries showed that all of them have experienced similar deflection histories. It has frequently been experienced that the box girders of many prestressed concrete bridges deflected far more than predicted in design. The deflection evolution has often been counterintuitive, with slowly growing deflections in the early years, followed later by a rapid and excessive deflection growth.

Reliable design of prestressed concrete elements and structures must be based on the accurate determination of prestressing force (including losses and their time development) to satisfy stress as well as deformation limits. Commonly used obsolete computational routines and methods neglect or incorrectly describe real effects and the results obtained by analysis based on these methods are thus necessarily wrong – great differences from results obtained by measurements on real structures have been found.

Results of experimental studies performed on real structures and their analyses confirm that present methods for evaluation of prestressing losses due to concrete creep and shrinkage are not realistic, prestressing losses are extremely underestimated. These methods ignore lots of important factors:

- The true behavior of concrete prestressed structures is three-dimensional – the simplified presumption that cross sections remain plane after deformations is quite far from reality. It is inevitable (when applying advanced computational analyses) to take into account (without any problems for computational process) 3D effects as shear lag and shear deformations of webs resulting in the cross section warping.
- The real development of concrete creep and shrinkage – for its prediction have to be used apposite and calibrated mathematical models; due to this fact, application of the B3 model, which has a strong theoretical basis, is proved as the best choice.

The rheological non-homogeneity of cross sections – creep and shrinkage development is significantly affected by cross section slabs and walls thicknesses. Application of creep and shrinkage models that realistically describe the moisture diffusion process, which causes that the shrinkage and drying creep is significantly affected by cross section slabs and walls thicknesses, is essential.

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Model B3.1 for multi-decade concrete creep and shrinkage: Calibration by combined laboratory and bridge data

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ABSTRACT

The prediction formulas for long-term creep and shrinkage have gotten new attention in response to a connection drawn between excessive bridge deflection and an underestimation of these concrete mechanisms (Bažant 2010). Subsequently a collection of 69 bridge deflection data sets from all around the world was analyzed to determine if the phenomena is widespread. The most prominent trend the data displays is a terminal linear slope in log time scale that is present in all datasets. However, this linear trend is not reflected in the ACI, CEB-fib, GL and JSCE creep prediction formulas which assume a final horizontal asymptote. Model B3 on the other side already has the correct long-term shape and needs only to be recalibrated to better agree with the new data. On overview of the models is given in (ACI 2008).

Concrete prediction formulas are typically matched empirically to laboratory test data which are performed on a relatively short time range, dominated by tests of duration <6 years. The incorporation of multi-decade bridge deflection data, related to the creep and shrinkage behavior of the concrete, helps to overcome this restriction. In a first approach the parameters of model B3 that determine the final slope were scaled to meet the long-term slope of the bridge data. However, modern structures are built from newer concretes with admixtures, fly-ash, slag, and fillers. Consequently, experience gained from multi-decade deflections of existing structures alone is not sufficient to develop an accurate prediction model. An expanded database of laboratory test data, covering both traditional and new concretes, provides the necessary information for increasing the range of applicability of model B3. A multi-objective optimization strategy based on the comprehensive laboratory database and the asymptotic properties of multi-decade creep, gained by analyzing the bridge deflection data, is applied to achieve an improved model B3.1 that can be used both for the design of new and the performance assessment of existing structures.

Previous efforts have been made to collect creep and shrinkage data for statistical analysis and fitting. The RILEM 1992 database of 426 curves was augmented with 20,000 data points collected at Northwestern University and made available online in 2008 (Bažant 2008). Using this existing set of creep and shrinkage data as a starting point, the database entries were reevaluated and combined with another database provided by JSCE, Japan (Sakata 2010). An additional collection of creep and shrinkage tests performed on concretes containing admixtures has recently been assembled at Northwestern University. This new addition provides information on the chemical effects causing autogeneous shrinkage and self-desiccation. The admixtures considered are as follows: silica fume, fly ash, fillers, viscosity agents, accelerators, retarders, air entraining agents, and superplasticizer. The 2008 database contains 934 creep and 1180 shrinkage curves. Compared to the 1998 Northwestern database, the number of available curves was augmented by 29% for creep and 51 % for shrinkage, respectively.

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Application of special non-destructive testing methods to different kind of structures

Organizer: M. Reiterer

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Low-cost sensor for integrated durability monitoring and life-cycle assessment of reinforced concrete structures

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ABSTRACT

This paper gives an overview how embedded sensor can be used to monitor the development of corrosion on new or existing reinforced concrete. It will be shown how the service life prediction of corroded reinforced concrete structures can be improved by means of monitoring data.

The main focus of this contribution lays on the development, practical testing, verification and optimization of novel mini-sensors for the estimation of rebar corrosion in concrete members. These calibration-free sensors are made with a single or several parallel arranged, ca. 0.05 to 1 mm thin steel wires, serving as 'watch-dog-sensors' for the determination of rebar corrosion initiation and damage progress. By means of such robust corrosion sensors the penetration of the depassivation front into concrete can be monitored depth-dependently by means of transient resistance measurements.

Main advantages of these small-sized filament-sensors are the simple design and measuring principle. Also a subsequent sensor installation into boreholes with a shrinkage compensated, filling grout and a two dimensional sensor arrangement are practicable too, Holst et al. (2007). Figure 1 shows exemplarily a cylindrical multi-wire sensor for the posterior installation into boreholes.

A modified filament-sensor types with several initially cutted silver wires enable also the measurement of the conductivity of the surrounding concrete, i.e. detecting the ingress of water and conducting salt ions, e.g. chloride into concrete.

The functionality and the application of the smart multiprobe-sensors were successfully tested at many specimens and during a long-term study at an experimental concrete bridge, Holst & Budelmann (2010). Based upon these investigations and the application experience of more than seven years the advantages of the sensor are discussed.

Moreover in the paper will be reported about achieved improvements and several further developments of the filament sensor, cp. also Figure 2. Here a distributed sensor, 2D-sensor arrays, an injection packer sensor and a corrosion thershold sensor for the corrosion rate will be presented as well as future trends like a wireless type of the filament sensor.

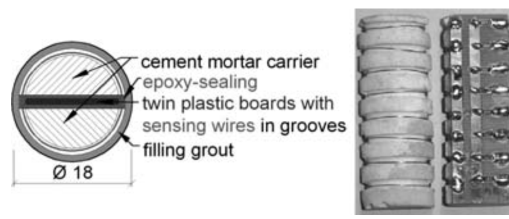

Figure 1. Filament-corrosion sensor with half-rounded PCC-mortar pieces for drill-hole installation, *left*: Sectional sketch of a double sensor, *right*: Photo with sensor's front and rear side.

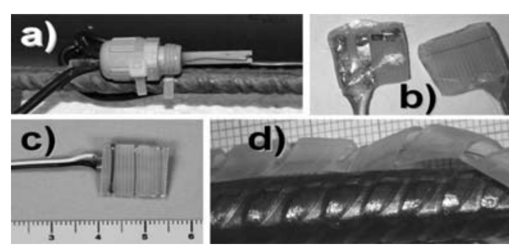

Figure 2. Further developments of the filament-sensor: *a*) and *b*): 1-wire sensors as elements of a 2D-sensor array, *c*) 3-wire sensor with varying wire diameter as a corrosion threshold value sensor, *d*) distributed, long gage sensor in a spiral hose.

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Development of a virtual reality-based support system for bridge inspectors

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ABSTRACT

This study describes a novel education system for training engineers to become bridge inspection specialists. The proposed education system uses virtual reality and three-dimensional computer graphic (3D-CG) technologies. These technologies can reproduce the slow time-dependent deterioration of concrete bridges on computer screen in any chosen time frame. Thus, the system provides the trainees with illustrative and educative insight into deterioration problems. The design tool of the proposed system can be used to create a three-dimensional model of a concrete bridge, including surfaces, viewpoints and walkthrough paths. The viewer of the system shows the designed bridge model in the virtual space on computer screen, and allows us to walk through the virtual bridge according to the designed path or walk freely by joystick manipulation. With the help of the virtual bridge model, an experienced bridge inspection specialist can teach various deterioration phenomena of concrete bridges to trainees. Thus, the proposed system can be used effectively to improve the quality of inspection data provided by bridge inspectors.

The main conclusions obtained in this study can be summarized as follows:

1) A Virtual Reality (VR)-based bridge inspection support system has successfully developed to assist in bridge inspection education and inspection result evaluation in this study.
2) The purpose of the system was explained, and a method for making effective use of the system was proposed.
3) Next, the procedure for developing the system was explained. Then, the effect of the system was verified with respect to the variability of visual inspection results by comparing the results of the inspection in 2009 carried out following a pre-inspection hearing conducted by using the proposed system, and the results of the inspection in 2008 carried out without conducting such a hearing. As a result, the fact that the variability of inspection results was reduced by conducting a pre-inspection hearing using the proposed system has shown that the system is a useful tool that contributes to reduction of the variability of inspection results.

A next challenge is to evaluate the usefulness of the proposed system in the education of visual inspection personnel, which is another area of application of the proposed system. Another challenge is to add concrete core test data and hammer sounding data and functions such as the function of displaying deterioration and damage processes in cross-sectional view.

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Modern acoustic NDT methods for the off- and online detection of damages in composite aeronautic structures

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ABSTRACT

The concept of "Damage Tolerant Design" has been successfully implemented in the aeronautic industry. This concept is based on the idea, that a certain amount of damage can be tolerated in a structural component, before it has to be repaired or exchanged. In order to guarantee the structural integrity of such a component continuous non-destructive inspections have to be done. In addition the percentage of composite structural parts in commercial aircraft is continuously increasing. Still their potential has not been reached, mainly due to uncertainties in the manufacturing process, the presence of barely visible impact damages and the prediction of the long term behavior in use, leading to much higher safety factors compared to metals to date. If a non-destructive inspection system could be permanently applied to the structure of interest, especially when made of composite materials and operated online (Structural Health Monitoring), a strong reduction in the down time and subsequent costs of maintenance and also a reduction in weight with a further reduction of fuel consumption could be expected. Worldwide activities in the field of "Structural Health Monitoring" are continuously growing since more than two decades. Analyses show that mainly the acoustic methods either passive as acoustic emission or active as lamb waves are able to cover larger areas with a sufficient size of detectable damage.

Impact Damage detection, localization and quantification has been done by a special algorithm that uses only the energy of the located Acoustic Emission events acquired for a de-fined number of load cycles The algorithm has been verified on Glass Fiber Reinforced Plastic (GFRP) plates that have been damaged by impacts of different incident energies. A typical result is shown in figure 1.

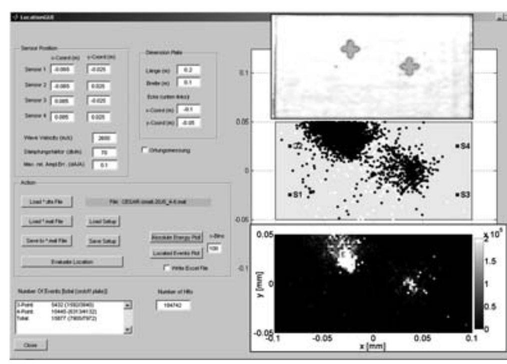

Figure 1. GUI of a Matlab code (overal), color coded AE energy distribution for a 20 J impacted panel loaded for 100 cycles (bottom right) and US C-scan for a 20 J impacted panels (top right).

In addition the ability for damage detection, localization and quantification using an active ultrasonic Phased Array SHM System based on Piezo-Actuators (PA) and Piezo Sensors (PS) has been verified on different flat plates made of Al and CFRP.

With both concepts the introduced impact damages could be detect, located and quantified. The acoustic emission on demand concept show a better accuracy in the assessment of the impact damage location and neglect able influence of the environmental conditions compared to the active method, but need a minimum strain for the activation of the AE sources. The advantage of the active US phased array technique is that no external loads are needed but the strong sensitivity to environmental effects such as the temperature requires very good compensation techniques. For future application a combination of both methods could help to overcome the drawback of the individual methods.

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Experimental modeling of fatigue processes to detect the real degree of deterioration

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ABSTRACT

Dynamic Loads can result in fatigue phenomena within the material concrete which are not totally explored even in their beginnings. Especially in the fields of foundations for wind energy plants on- and offshore fatigue is a big problem. The Fatigue associated load combinations are mostly the decisive ones for design and dimensioning of the structure. The targets of the research are the development of a monitoring system for detecting the initiation or the early stage of a fatigue process in concrete and for the identification of the degree of deterioration in the concrete structure. The full scale model of a new type of gravity base foundation for offshore wind turbines in Cuxhaven (Figure 1) projected by the Ed. Züblin AG is an optimal possibility to test the monitoring system within a concrete structure of real dimensions.

The objective of this contribution is to provide a short review of concrete fatigue properties, to discuss, demonstrate and portray preliminary analyses results which are decisive for the final fatigue test layout at the STRABAG-gravity base foundation in Cuxhaven. The conduction of the fatigue tests at the gravity base foundation are planned in the beginning of the year 2013.

With the presented fatigue test it will be possible to experience a process of fatigue close to reality. Concerning the cyclic loading of the gravity base the directed exciter is a fundamental part of the whole test-setup. The higher the stresses within the concrete cross section are the sensible behaves the whole system. All the more important is the stepwise control of the excitation frequency.

In order to validate and check the local effects a nonlinear calculation was carried out. The result is a confirmation of the expected stress values. But it became evident too, how important it is that the carrying out of the weakening is done very precisely. As soon as the remainder of the cross section is bigger than calculated the stresses will decrease. That could be a reason to carry out the weakening in a different form than the 2 horizontal continuous cuts.

With the results of these fatigue tests at the gravity base it will be possible to calibrate the developed monitoring system and to make an inverse analysis between numeric model and the real structure.

An aim of the research at the University of Natural Resources and Life Sciences in the fields of concrete fatigue is the discovery of a monitoring system in order to comprehend and assess the real behavior of a concrete structure under cyclic loading. As a result a possibility will be found with which concrete structures can be designed, dimensioned and maintained more effectively and economically with the ulterior motive of saving resources.

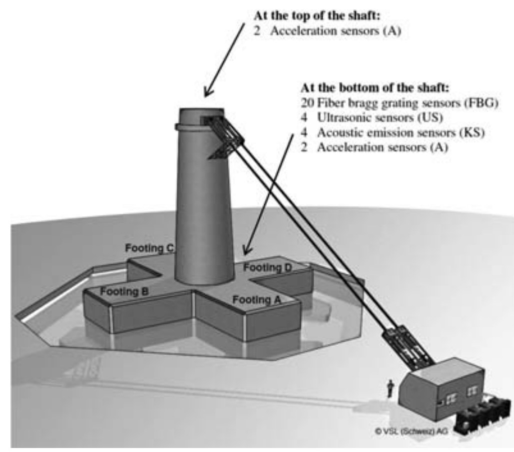

Figure 1. Sensors at the gravity base in Cuxhaven.

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Damage process monitoring on the hot spot of a real steel component by means of ultrasonic guided waves

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ABSTRACT

Structural health monitoring is the common standard for structure monitoring in terms of integrating a sensor technology into a structure and record the time depending signals (Brownjohn 2007). The benefit of the collected data is to use them for a "condition assessment" by a following diagnosis. Civil infrastructure owners are interested in monitoring their structures when they are exposed to extraordinary loads or structural changes – e.g. retrofitting – are applied. Actually, introducing monitoring equipment to civil structures during the manufacturing process to collect data over their life time, is increasing. Preferred structures of applications are dams, bridges, offshore installations, towers, nuclear installations and tunnels. The characteristic of monitoring infrastructure is that no prototype is present and the testing goes along with the use of the structure. The monitoring system has to be adapted to the physical requirements, the number of sensors, the availability and the environment.

In this paper a hot spot monitoring system is introduced by Ultrasonic Guided Waves (UGW) (Su and Ye 2009). At an arbitrary point in time the UGW system starts to record the wave propagation signals within an distributed sensor array. The first or a defined sum of signals is the initial system state $s_{i0}(t)$, whereas faults may already exist. Every signal recorded later in time $s_{ik}(t)$ will represent a system state to be compared with. The subscript i denotes the i-th actuator sensor path within the USW array and k the k-th recording. After filtering and normalisation of the signals they are subtracted (Croxford et al. 2007).

$$r_{ik}(t) = s_{i0}(t) - s_{ik}(t) \tag{1}$$

Mathematically an deviation of $r_{ik}(t)$ from zero is a structural change and might be a fault or crack. At this point the sensitivity of the residual signal $r_{ik}(t)$ becomes apparent because every time shift has to be evaluated. This time shifts appear due to operational effects. The affection and compensation of mechanical stress to the wave guide is demonstrated and the detection of a crack and crack growth, while a fatigue test is running, is demonstrated (Figure 1).

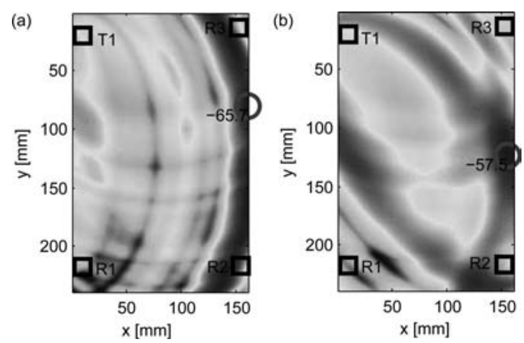

Figure 1. Results of crack detection with the UGW sensor array and the maximum echo. (a) image after 1.101.078 and (b) after 1.399.629 load cycles.

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Concrete fatigue monitoring on large scale structures using acoustic emission and an ultrasonic actuation and sensing system

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ABSTRACT

A better knowledge of the fatigue state of large concrete structures, is of great interest for their designer, operator and maintainer. Therefore a system for reliable monitoring of the fatigue process of concrete under dynamic load and a realistic estimation of remaining life would be of great use. It could help to define new design rules for engineering structures using high strength and ultra high strength concrete. We present an on-line fatigue monitoring concept together with first measurement results for a large scale concrete structure, a Gravity Base Foundation (GBF) for a wind energy plant. The monitoring system consists of an acoustic emission and an ultrasonic sensor part. Monitored parameters have been the acoustic emission activity and the ultrasonic signals time-of-flight respectively travel velocity. Experiments have been realized on the large scale test structure with a specially constructed towing device, that has been used to induce cyclic dynamic loads. Acoustic emission data shows a correlation to loads and acoustic activity of the concrete structure. Results for ultrasonic measurements show good reproducibility and that no relevant integral fatigue levels have been reached yet. To verify the measurement results on the large scale structure as well as to improve the monitoring system a test campaign on small scale specimen was carried out and similar AE and US-measurements as on the GBF were performed. In course of this campaign the specimens were (under laboratory conditions) exposed to varying cyclic, loading levels. The range covered was from the onset of fatigue processes to the complete damage of the specimens. The results of this campaign were in accordance to the large scale tests and showed the capability of the proposed methods to trace the fatigue process of concrete.

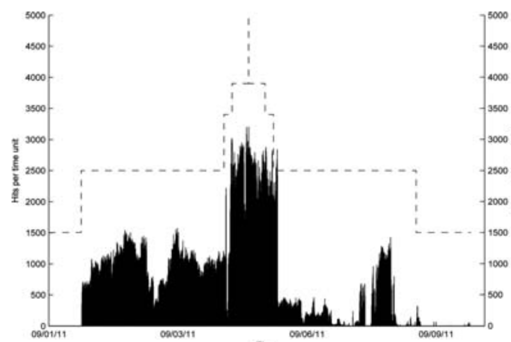

Figure 1. Hitrate from an AE Sensor during a load scenario.

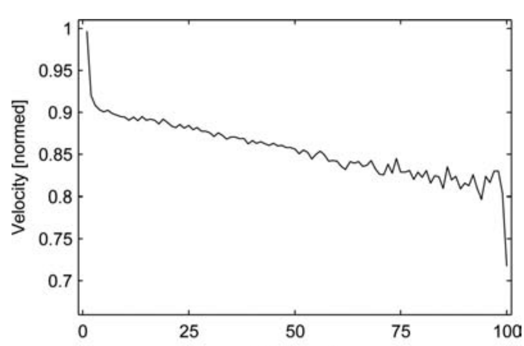

Figure 2. US-velocity over a small scale specimen's life time.

Seismic system identification for life-time-prediction (SEISMID)

Organizers: H. Wenzel & P. Furtner

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Assessment of the global dynamic behavior of a historic residential brick-masonry building in Vienna

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ABSTRACT

This paper reports about recent experimental tests on a historic residential brick-masonry building, to investigate the influence of structural and non-structural elements on the global dynamic structural behavior. The building located in the city centre of Vienna, Austria, was constructed about 120 years ago and remained almost unchanged since then. Figure 1 shows a photograph of the test object.

The assessment of building dynamics was based on the measurement of vibrations induced by an impulse-like load applied to a central load-bearing wall at the last floor level. Several three-dimensional accelerometers distributed inside the building recorded the dynamic response, see Figure 2. Subsequently natural frequencies, mode shapes and the damping ratio were identified from the data.

Initially, the dynamic properties of the original building were determined, see Table 1. Subsequently, partitioning walls and wooden floors were removed step by step, and the experimental tests were repeated according to the test set-up of the initial measurements on the undisturbed building. From the results of measurements the stiffness distribution of the building was determined by application of a finite element update procedure. These analyses were based on the identified mode shapes and natural frequencies. The influence of non-structural elements on the global stiffness could be determined, because the dynamic parameters at different stages were known from the experimental investigations. The calculated stiffness of the load-bearing walls of the lower story was verified from the results of the study on the partially demolished structure.

Figure 1. Photograph of the test object.

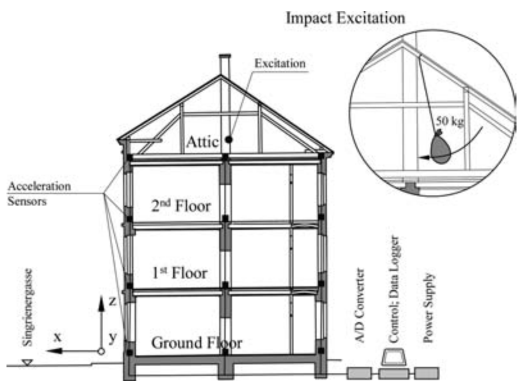

Figure 2. Vertical distribution of the acceleration sensors within test object, and experimental test set-up.

Table 1. Natural frequencies of the original undisturbed building.

	Natural frequency [Hz]	
Mode number	Measurement	Numerical analysis
1	3.8	3.78
2	4.2	4.29
3	4.6	4.67

These investigations were performed within the Austrian research project SEISMID, which aims among others to develop a methodology to assess the actual seismic bearing capacity of historic buildings. SEISMID is funded by the ZIT Center for Innovation and Technology, which is a subsidiary of the Vienna Business Agency (WWFF). This support is gratefully acknowledged.

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Seismic capacity of old masonry buildings in Vienna: Laboratory testing on bricks, mortar, and small-scale brick masonry

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ABSTRACT

In the City of Vienna historical brick masonry buildings, mainly constructed in the period between 1850 and 1918 (the so-called "Gründerzeit"), represent a substantial amount of the existing stock of residential structures. Retrofit of these buildings requires the assessment of the load-bearing capacity of the entire structure. However, for many buildings the earthquake resistance cannot be assessed successfully, because no detailed knowledge of the physical behavior of their structural elements, i.e. the load-bearing walls, was available until recently.

The goal of a subproject of the Austrian national research project SEISMID (Achs et al. 2011) was the identification of mechanical properties of brick masonry in old buildings of this period located in Vienna. This masonry consists in general of solid bricks of the so-called "Old Austrian Format" (29 cm × 14 cm × 6.5 cm) bonded with mortar in the head and bed joints. A series of experiments on small-scale test specimens was conducted at the Technical Laboratory for Research and Testing (TVFA) of the University of Innsbruck.

Complete masonry specimens could not be extracted from buildings without destroying their integrity. Thus, at first the material behavior of both masonry components, i.e. bricks and mortar, was investigated separately. The constituents of original mortar were identified, and subsequently, test specimens of small-scale masonry elements consisting of old bricks and reproduced mortar were tested.

In this paper the test set-up of all experiments is described, the experimental results including the statistical evaluation are presented, and the mechanical behavior of the considered brick masonry is evaluated and discussed. In particular, stress-strain relationships from displacement controlled compression, tension, and shear tests were determined and mechanical properties such as compressive and tensile strength, Young's modulus, Poisson's ratio, and specific fracture energy were identified.

As an example Figure 1 shows stress-strain relations from compression tests on column-type masonry specimens of three and five brick-layers, respectively. In Figure 2 a five-layer specimen is depicted at crack initiation (left photo) and its condition at failure (right photo).

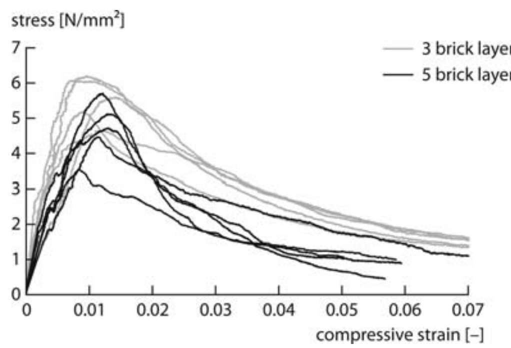

Figure 1. Stress-displacement relations from compression tests on column-type test specimens.

Figure 2. Compression tests on column-type test specimens. Left photo: Crack initiation. Right photo: Failure pattern.

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Seismic capacity of old masonry buildings in Vienna: Numerical modeling of load-bearing brick masonry walls

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ABSTRACT

In the City of Vienna and in many other European cities historical brick masonry buildings, mainly constructed in the period between 1850 and 1918 (the so-called "Gründerzeit"), represent a substantial amount of the existing stock of residential structures. Retrofit of these buildings requires the assessment of the load bearing capacity of the entire structure. However, for many buildings the earthquake resistance cannot be assessed successfully, because no appropriate constitutive model and set of material parameters for their structural elements, i.e. the load-bearing walls, was available until recently.

In this paper numerical modeling of old brick masonry walls subjected to in-plane loads is addressed in order to predict more reliably the seismic capacity of historical brick masonry buildings in Vienna. Material parameters for the masonry constituents brick and mortar are experimentally determined and subsequently numerically simulated in order to validate parameter sets for old brick masonry on the meso-scale. Exemplarily, Figure 1 shows the stress-displacement relations from experiments and numerical analyses of compression tests on small-scale masonry test specimens. For computations of the considered masonry on a larger scale, a macro-model in the framework of multi-surface plasticity theory is implemented in a finite element program. The composite yield surface of the macro-model is shown in Figure 2. Based on the constitutive quantities on the meso-scale a numerical homogenization procedure results in the material parameters for the macro-model. In an application the representative load-bearing wall of a historical brick masonry building is identified. Homogenized material properties are assigned to this wall, and a pushover analysis is performed to derive a global force-displacement relation of this wall. The capacity spectrum methodology is applied to prove the earthquake resistance of this building.

Note that the results of this paper are an outcome of the investigations conducted within the Austrian national research project SEISMID (Achs et al. 2011).

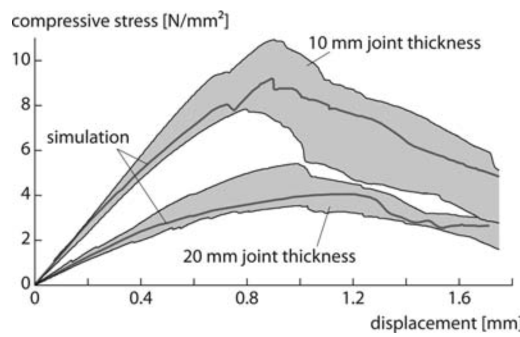

Figure 1. Stress-displacement curves of masonry compressive tests with one mortar joint. Experimental scatter and results from numerical simulations.

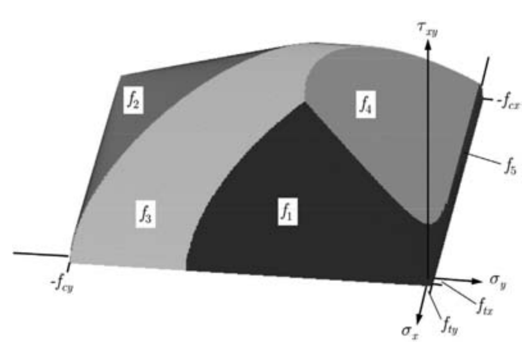

Figure 2. Composite yield surface of the macro-model.

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The assessment of soil-structure-interaction by measurements

F. Kopf, A. Bekö, D. Schäfer, M. Pietsch & L. Rossbacher
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ABSTRACT

In common buildings the Soil-Structure-Interaction (SSI) is governed by the elastic behavior of the building. The best conditions to see the soil effect predominate would be a dynamic rigid body movement of a massive building. For this reason measurements were carried out on an old anti-aircraft tower from World War II on typical Viennese soil conditions. The dynamic properties of the soil could be determined in two different ways. The first one is the common way using the dispersion of the Rayleigh waves on the soil surface and the back calculation to shear wave profiles. The second approach is the back calculation of the soil stiffness based on the soil-structure-interaction.

With the simultaneous measurement of the dynamic movement of the building the kinematic behavior of the SSI could be determined. The tilting oscillations of the tower in both directions with different frequencies transmitted to the soil could be measured in the surroundings. The decay of these vibrations and their influence on the H/V-method results was studied.

The results of the dynamic measurements were compared with different methods of numerical simulation. The benefits and the disadvantages of the methods can be compared. The calculation values of the numerical models could be calibrated with the real dynamic properties of the measured SSI-system.

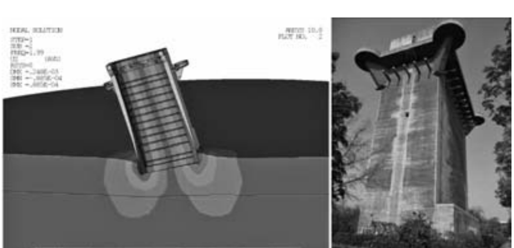

Figure 1. FE calculation result: Deformation in z-direction, 1st harmonic. Picture of Leitturm, Augarten, Vienna.

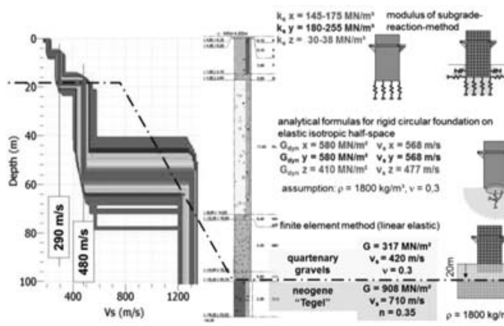

Figure 2. Comparison of the obtained soil-parameters: Results from measuring the Rayleigh-waves and from the soil-structure-interaction.

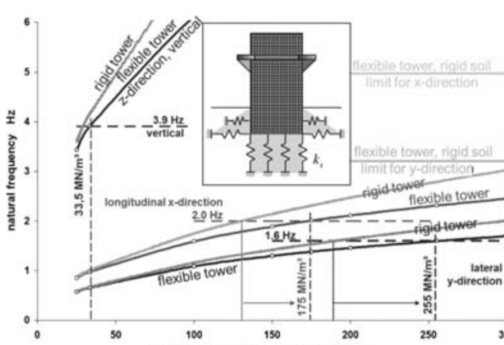

Figure 3. Coherency between modulus of subgrade reaction and Eigenfrequency for different modes of vibration for rigid body movement and elastic building characteristic, comparison of FE-computations with measured results.

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Measurement principles for masonry buildings

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ABSTRACT

Intensive scientific works in the field of earthquake research caused the classification of new earthquake zones in wide areas of Europe, also in Austria. The introduction of the compulsory Eurocode 8 changed the estimation of earthquake risk for buildings in Austria's cities significantly. The endangerment by earthquakes for buildings in the Vienna Basin became a deciding factor, not only in planning, but also financially. In order to eliminate this problem for the real estate economy, the research project SEISMID was launched and thanks to financial contribution of the "Technologieagentur der Stadt Wien (ZIT)" it could be finished successfully on 31.12.2010. This project was the first step to compensate the lack of knowledge and experience in this sector in Austria. Historical masonry buildings are very complex structures. Their global dynamic behavior is affected by many components (foundings, walls…), that consist of many subcomponents themselves (bricks, joints, etc.). There are many constraints for the dynamic system identification of the micro- and macro-behavior while taking measurements. This article describes the technical aspects of the measurements, which was acquired within SEISMID and could be tested successfully.

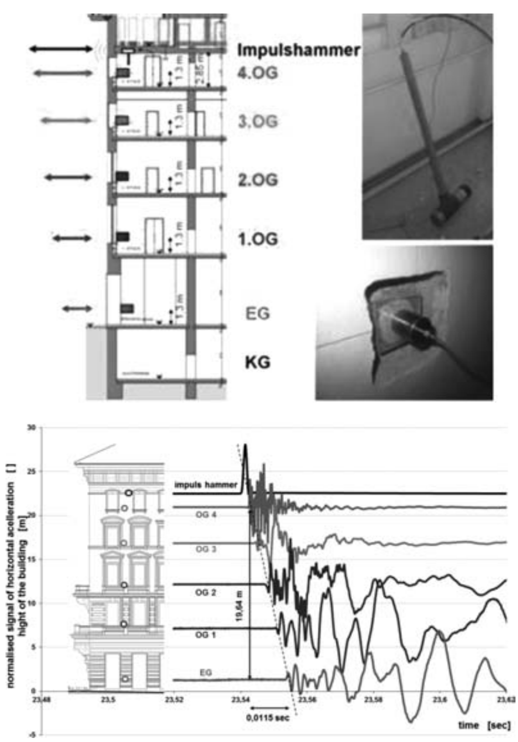

Figure 1. Experiment for wave dispersion in masonry wall.

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Remote sensing and GIS contribution to earthquake disaster preparedness in the Vienna area

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ABSTRACT

When catastrophic earthquakes happen and affect cities and settlements, immediate and efficient actions are required which ensure the minimization of the damage and loss of human lives. The inhomogeneous spatial distribution of damages caused by earthquakes is not only controlled by the earthquake depth, the orientation of the fault plane solution and magnitude, it is influenced as well by local geological site conditions. Information regarding the geological structure, the physical and dynamic characteristics of the rocks in the subsurface, the possible geological dynamical processes, and the response of the soil in case of a stronger earthquake should be taken into account. When knowing areas with aggregated occurrence of causal ("negative") factors influencing ground motion or earthquake related secondary effects, this knowledge can be integrated into disaster preparedness. The main purpose of this study is to investigate local site conditions during stronger earthquakes in the Vienna area by using remote sensing and GIS technologies. Society, infrastructure and economy periodically and extensively suffer from the impact of natural disasters. Especially, in the case of an earthquake hazards that affects cities and settlements, immediate and efficient actions are required, which ensure the minimization of the damage and loss of human lives. The Vienna Basin is one of the main seismic active areas in Austria (Hausmann et al., 2010). Most of the epicentres line up along the Vienna Basin Transfer Fault System (VBTF). Responding local and national authorities should be provided in advance with information and maps where the highest damages due to unfavorable, local site conditions and earthquake related secondary effects such as landslides, liquefaction, soil amplifications or compaction can be assumed. Technical interdependencies between infrastructures have a potential in triggering widespread cascading effects of failure or loss of service. The Vienna area has more than 2 million inhabitants and sensitive infrastructure. Therefore seismic hazard assessment and mitigation is an important task. Seismic and secondary technological hazards identification and analysis, as well as risk assessment and mapping, are the important steps in prevention strategies. The use of Remote Sensing (RS) and GeoInformationSystems (GIS) methods combined with the related geo-databases can assist local and national authorities to take action in order to meet the above objective. These tools support efficiently the organization of public protection activities (Theilen-Willige et al., 2011). As a prerequisite for earthquake preparedness a detailed inventory of sites in the Vienna area, that are more susceptible to earthquake damage and to earthquake related secondary effects due to local site conditions, has to be carried out.

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Evaluation of the safety index of old masonry buildings in Vienna: Non-linear analysis based on seismic capacity

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ABSTRACT

The design and structure of masonry buildings are based on experiences of many centuries. Although these experiences are used worldwide, knowledge about the material behavior of masonry is still afflicted with uncertainties. In particular, the reliability assessment of masonry buildings with respect to earthquake loadings is a complex challenge. The assessment is highly influenced by uncertainties in the material characteristics, geometrical quantities, structural details and seismic conditions. The primary objective is to minimize the risk of earthquakes and the vulnerability of buildings; Bachmann (2002).

The EC 8 demands according the earthquake safety a stability of the building for a design earthquake with a return period of 475 years and an occurrence probability of 10% in 50 years. There should be no damage or use restriction, where the cost of repair would be disproportionate to the construction costs – "Requirements for damage control".

For detailed and realistic consideration of seismic effects on buildings also local effects must be incorporated. The design according to EC 8 allows a simplified analysis for regular building shapes in plan and elevation. A sufficiently accurate calculation method is the response spectra method; Chopra (2002). Thereby loads from higher modes of vibration and torsional effects are neglected. For this simplification each vibration direction is considered separately. Only the horizontal component of ground acceleration is taken into account. The entire system is represented by a single degree of freedom. Hence the stress for the linear elastic design is addicted. In linear calculation the plastic behavior is taken into account by reduction of the design spectrum ordinate with the so-called behavior factor q. This factor represents an approximate value of the ratio of those seismic forces that would act on the structure if the answer at 5% viscous damping would be ideally elastic, to those forces that may be used for the design with a linear model to just receive a satisfactory response of the structure. In addition to the 5% viscous damping ζ an effective damping ζ_{eff} can be used in the calculation if information is available; Chopra & Goel (1999). The behavior factor q and the parameters for the energy dissipation e.g. the hysteretic damping ζ_{hyst} are of crucial importance for the design of new structures and the assessment of existing buildings regarding the seismic performance and capacity respectively.

To incorporate the damping and the non-linear material behavior in a more appropriate way the capacity spectrum method was used to assess the performance of a structure due to earthquake loading. This paper presents the application of a deformation-based verification procedure adapted for masonry structure in earthquakes zones to a typical old masonry building in Vienna. Material characterization is based on experimental results of cyclic shear wall tests. The shear wall capacity curves are determined in a stochastic way by non-linear simulation approach. With the stochastic approach uncertainties for material characterization as well as for the impact loads are considered. This is a novel approach for the evaluation of the safety index for both existing buildings without and with a loft conversation but also for new buildings. Moreover it serves as a basis for life-cycle assessment on masonry structures.

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Life-cycle assessment for sustainability evaluation of buildings

Organizers: R. Smutny, C. Neururer & M. Treberspurg

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Life-cycle assessment of steel constructions

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ABSTRACT

The Life-Cycle Assessment (LCA) has become an integral part of the sustainability evaluation by reason of the German sustainable building certificate (Deutsches Gütesiegel Nachhaltiges Bauen, DGNB). With the introduction of the system variation "industrial construction" in 2009 the LCAs issued for these building types have increased. Thereby, steel structures take on a key role in the development of sustainable buildings since steel constructions as supporting structures and light steel constructions for building envelopes are prevalent for industrial and commercial buildings.

In order to gain knowledge about the environmental impacts of different construction techniques for the supporting framework as well as for the building envelope, several construction methods have been examined within a study (Kuhnhenne et al. 2010). Since a comparison of the environmental performance of different types of construction is only useful and meaningful within the building context, different variations of a simplified hall have been selected as reference object. This hall is intended to act as a model and to show principles of life-cycle assessment of hall-like buildings. Another part of the study is the comparison and examination of various databases (Ökobau.dat 2010, EPD data).

The reference hall (Kocker, R. & Möller, R. 2009) provides the opportunity to realize small and medium-sized steel frame buildings with quality-checked statics and plans. The structural design is based on an optimization with respect to use and cost of steel as well as production and assembly. Basis for the structural design are a two-hinged frame and fixed columns with hinged trusses. The reference hall is suitable for different uses and can be realized in a heat-insulated (e.g. production facility) or uninsulated (e.g. unheated warehouse) way. Here, the primary energy demand for heating, cooling, ventilation and lighting as well as the energetic quality of the building envelope have to meet the national energy requirements (EnEV 2009).

From the studies it can be observed that the differences between Ökobau.dat and EPD are noticeable in absolute values, however, for the comparison of different designs the two data sources lead to the same result.

For the future, it is important to continue expanding the data base for environmental performance of construction products. On the one hand data must be captured for more building products (e.g. through the wider dissemination of EPDs), on the other hand for the existing production processes it must be provided the appropriate information for assembly into the building, maintenance and cleaning as well as end-of-life.

From the studies it can also be concluded that the total primary energy demand for the operation phase, even with increased energy requirements and an observation period of 20 years, is the dominant size. Here, it should be noted that determining the environmental impact of production and disposal of the construction, only the mass-relevant components (building envelope and load-bearing structure) are included in the analysis.

This simplified approach and its impact on life-cycle assessment of hall-type buildings should be reviewed as part of future investigations.

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Ecological and economic impact of various materials and constructions for buildings over the whole life-cycle

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ABSTRACT

Calculations of life-cycle assessment and life-cycle cost are becoming increasingly important in the building process, mainly due to the rise of sustainability certifications in the building sector. This paper looks at the environmental and economic effects of different materials and constructions in the complete life-cycle of buildings.

The evaluation was carried out on a reference building – a two story detached family house (Sohm 2009). For the building an array of construction methods was considered and calculated. The construction methods were as follows: timber framing (EL), massive timber construction (MH), external wall with external thermal insulation composite system (DF) and masonry construction (ES).

The calculations are carried out with the computer tool LEGEP. The program LEGEP is a tool for integral life-cycle analysis in the building sector.

With this tool it is possible to calculate the costs and the environmental impacts over the whole life-cycle. All calculations can be conducted for the whole life-cycle or separated as individual phases like erection of building, operation, cleaning, maintenance und end of life. All data complies with German codes. The *ökobau.dat* is used as environmental data. The results in life-cycle analysis includes Global Warming Potential (GWP), Acidification Potential (AP), Primary Energy renewable (PE e) and non renewable (PE n.e.) In addition to this, the weight of the construction is always calculated. The economic calculations are worked out in Euros.

The following phases are calculated: erection of building, utilization phase, end-of-life. The phase of utilization includes the operation of the building and the services and maintenance of the construction over a period of 80 years.

Results outlined, that the differences in these categories over the whole life-cycle are only a few percent (1–3%). These calculations do not lead to favor certain constructions. The comparison in primary energy shows an increase in primary energy renewable for timber constructions and especially for the massive timber construction. The categories primary energy non renewable and primary energy altogether are very similar in output. This means, that the impact of the operation period is far bigger (70 to 90%) today than that of the construction itself. Differences in the materials are small and will only have an effect, if energy efficiency is increased to a very high level.

The small differences in the environmental calculations meant, that the construction principals were also compared without operation period, maintenance and end-of-life to locate the differences in material itself. Results are: Massive construction stands out in terms of global warming potential because of its negative impact. This is due to the capacity to store carbon in the wood. The part of renewable energy also is significantly higher than in other constructions.

In addition differences in insulation material and cladding material in terms of environmental and economic issues were examined. Results were, that construction with soft boards comes forward in the categories GWP and renewable primary energy. In terms of façade surfacing there are no major differences in the overall output, as the façade surfacing has only a minor mass of the whole building.

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Wooden products – positive material in life-cycle analysis

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ABSTRACT

The construction industry accounts for a large part of material and energy consumption. Up to date the influence of the operation of buildings has the maximum impact on environmental pollution. Various calculations on life-cycle analysis show, that the operation phase accounts for around 65% in buildings by realizing the energetic standard up to date. When energy efficiency for the operation is increased, the impact and effect of the construction process on the environment will come into the foreground. Because of the capacity of storing carbon and being a renewable material wood is examined in detail. In the building sector the classification of different materials is roughly divided up into mineral and organic materials according to divisions in chemistry. The classification of the origin of carbon in the material in renewable or non-renewable resources is not considered in that separation (König 2011). This paper only looks at the organic materials and their influence on life-cycle analysis in buildings. In that context the term renewable material means that all material made of plants have the capacity to store carbon during growth and can be used as a carbon sink until the material is burned at the end of life releasing the CO_2 again. This is the characteristic of renewable materials. Data for wooden products in life-cycle analysis and calculations of timber buildings in life-cycle analysis are discussed. Large part of the primary energy in the wooden product can be allocated to primary energy renewable. The life-cycle analysis of various wooden materials shows the ecological impact of the different wooden materials. It is dependent on length of the production process from raw material to final product. Discussion in various research and standardisation projects in the past showed that simplification and transparency in LCA data for wood products are necessary. The issues of bound solar energy, imbedded carbon and renewable energy consumption have to be addressed. For example includes the category of 'primary energy renewable the solar energy incorporated for the growth of trees. The bound solar energy hereby appears as primary energy consumption of wooden products. This results in high values for primary energy renewable and also in higher values in primary energy in total. 'Primary energy renewable' consumption and bound solar energy therefore is accumulated in life-cycle analysis calculations for buildings. But this mixes up the energy consumption and the potential of naturally imbedded solar energy, see Figure 1. Further research also needs to be conducted in end of life scenarios for wood and possibilities of a "regrowing potential" for wooden products.

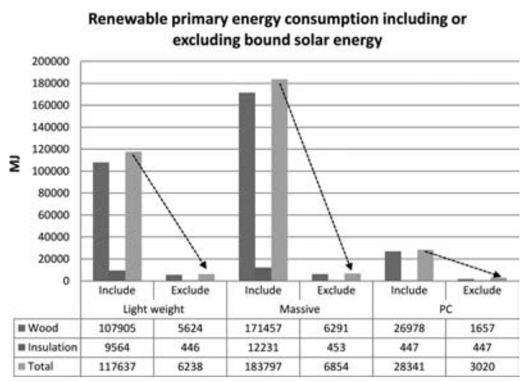

Figure 1. Change of renewable primary energy consumption when the bound solar energy of wooden product is excluded, in the three box buildings (Linkosalmi 2011).

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MINERGIE-ECO® 2011 – definition of thresholds in an LCA-based building label

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ABSTRACT

The Minergie base standard was introduced in 1998, with the more stringent Minergie-P® and Minergie-P/-A-ECO® standards appearing later. Together they set performance criteria for materials, energy efficiency and comfort. Compared to a conventional Swiss building, a Minergie building consumes around 60 percent less energy.

"ECO" is an extension of the MINERGIE® label for new buildings, covering the aspects of health (e.g. high indoor air quality) as well as environmentally friendly building concepts and products.

In order to simplify this application process, methods and software libraries to integrate a full Life-Cycle Assessment (LCA) were developed, along with a short questionnaire into existing energy performance calculation tools for buildings.

In order to decide whether a project fulfils the requirements for embodied energy (non-renewable primary energy) and Global Warming Potential (GWP) for new or retrofitted buildings, specific requirements are defined. For new constructions, the thresholds are based on the 2000-Watt goals that Switzerland has committed itself to. These top-down derived thresholds for different building categories (living, office and schools) have been broken down into values for geothermal probes, construction per m^2 of heated area and for photovoltaic and solar collectors per m^2 of these appliances. If a project foresees the use of one or more specialty energy systems, these will be integrated in the total threshold to avoid penalty.

In the case of retrofit projects, basis thresholds for different building envelope components (roof, outer wall, base plate and windows/doors) were defined in a complex process per m^2 component. The final threshold is the sum of all (manipulated) component-related thresholds and the specialty energy systems similar to the threshold of new constructions (geothermal probes, photovoltaic, etc.). The chosen approach has been tested with more than 10 case studies and proven practical.

For the first time it is now possible to optimise a building with a holistic energy approach.

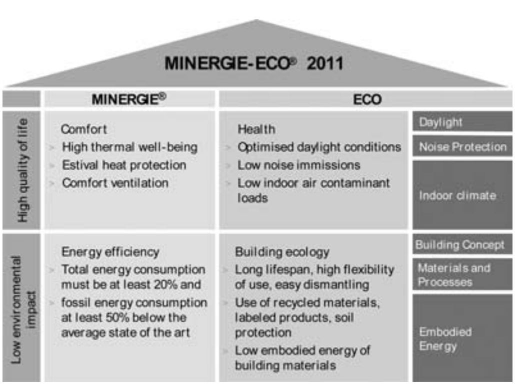

Figure 1. Newly structured criteria of MINERGIE-ECO® 2011.

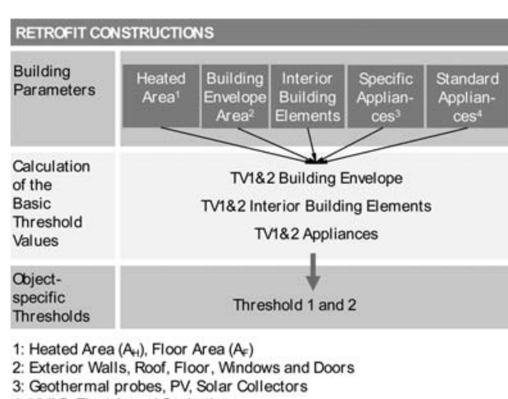

Figure 2. Methodology for the calculation of the object-specific Thresholds for retrofit constructions.

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Sensitivity of life-cycle analysis results to the required service life of buildings

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ABSTRACT

This preliminary study shows the influence of the Required Service Life (RSL) on the LCA of buildings with mineral primary structure.

With the aim of evaluating the sustainable quality level of new built buildings, in 2008 Germany introduced a certification system for buildings called BNB/DGNB (Bewertungssystem Nachhaltiges Bauen/Deutsches Gütesiegel für Nachhaltiges Bauen). In this system the assumed Reference Service Life (RSL) for buildings is 50 years (y). For this reason the potential of construction materials with long persistence like mineral materials, will not be taken into account.

A preliminary study, which was established by the association of mineral construction products, Berlin (BBS), should find answers to the following questions: Which is the influence of different reference service life assumptions on the result of the Life-Cycle Assessment (LCA) of a building?

Which part of resource consumption and environmental impact are underestimated in the different life-cycle phases when choosing short reference service life? Which is the influence of the structural material on the overall result?

Comparable calculations for LCA need some basic requirements:

– Use Type: e.g. Residential Building
– Reference building: e.g. Apartment-house, 3 storey and roof, 7 units.
– Energetic demand: e.g. German Energy Standard (EnEV 2009)
– Calculation rules: e.g. German BNB-certification system addressing LCA.
– RSL: e.g. 50 years.

Based on these choices the representative building type (modelled in the project database of the used software, in this case the German software LEGEP), value corridors for the LCA of different RSL were elaborated. The different RSL vary between 30-50-80-100-and 150 years. The elaborated analysis tried to describe the in- and output in several steps from the overall results of the building including production, use and end of life until the primary construction as the smallest unit.

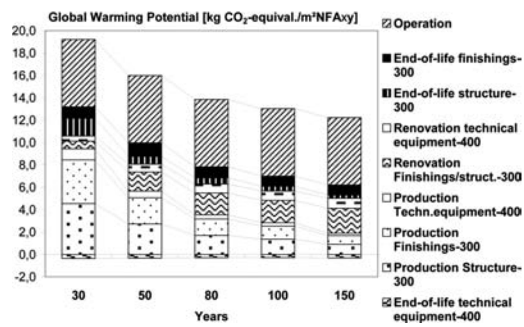

Figure 1. GWP. Phases: production, operation, replacement/renovation and end of life (for technical equipment, load bearing structure and finishes).

The results of the building type and its variations have been statistically analysed and values for different indicators concerning the influence of the primary construction has been determined. The selection of the indicators follows the core model of EN 15978. As an example the indicator Global Warming Potential (GWP) is shown.

A longer RSL tends to higher values between 50–63% of GWP, Ozone depletion and non-renewable primary energy. The rate of the primary construction is reduced continuously from 32% to 5% (GWP).

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"Mountain wood vs. lowland wood", an ecological process assessment – a case study

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ABSTRACT

How ecological is the harvesting of Tyrolean mountain wood compared to wood harvested in Bavarian lowlands? In this study the felling area is described for both regions, from forest site to sawmill. Thereby we create mean values related to timber harvesting and transportation, which are applicable to the whole of Tyrol and Lower Bavaria. Moreover, the eco-balance of the mountain and lowland wood is established. It is based on the following impact categories: global warming potential, acidification potential, eutrophication potential, ozone depletion potential and non-renewable primary energy.

This research project pursued several objectives

Determining impacts of transport routes and assessing to which extent the use of domestic products adds value to the regional economy.

In our eco-balance the functional unit per m^3 wood is specified, which is the foundation for the mass allocation distinguishing between structural timber and fuel wood. The data are based on the current timber industry figures. Growth periods vary in wood production and therefore cannot be specifically defined.

The results revealed that the management of both timber and fuel wood in mountainous regions have the same ecological impacts.

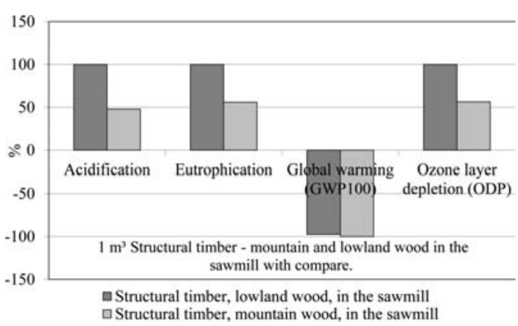

Figure 1. Comparison of mountain wood – lowland wood.

The figures of the impact assessment for lowland wood differ, because the transportation part for fuel wood is smaller. Figure 1 compares the impact assessments of mountain wood and lowland wood. It becomes clear that mountain wood shows, on the whole, a 50% lower impact than lowland wood in the impact categories acidification, eutrophication and ozone layer depletion.

Evaluation mountain wood – lowland wood: Table 1 lists the results of the impact assessments for timber and fuel wood for both mountain and lowland wood. The negative global warming potential value indicates that in the process from felling area to sawmill more CO_2 is bound than is used up by machines and transport.

The evaluation of the process chains of the individual impact categories for lowland and mountain wood suggests that the decisive factors for the global warming potential – acidification and eutrophication – are in the areas of transport and fuel gas (diesel for engines). On the other hand, mineral oil production processes are mainly responsible for the ozone depletion potential.

Table 1. Results of impact assessment.

GWP kg CO_2-eq	AP kg SO_2-eq	EP kg PO_4^3-eq	ODP kg CFC-11-eq	PEI_{ne} kg MJ-eq
Mountain wood – fuelwood				
−996	0.0515	0.0129	1.3 E-6	147
Mountain wood – structural timber				
−996	0.0515	0.0129	1.3 E-6	147
Lowland wood – fuelwood				
−974	0.0889	0.0191	1.75 E-6	198
Lowland wood – structural timber				
−970	0.107	0.0231	2.30 E-6	252

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Life-cycle assessment as a planning tool for sustainable buildings

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ABSTRACT

The building sector is a "key sector" to achieve the goals of climate protection and for a sustainable resource management. Therefore buildings with an optimum of environmental effects and optimized cost during the whole life-cycle can positively contribute to a sustainable development.

During the planning of buildings and constructions, the task to reduce the production costs often dominates. In parallel, numerous studies with a holistic life-cycle approach show that the follow-up (running) costs of a thermally conditioned building dominate. Production costs can be up to 20% of the total life-cycle costs of a building. The follow-up or running costs (use, operation, repair and demolition) can be up to 80%. The construction and the use of buildings cause many environmental effects and impacts (polluting emissions).

In terms of a sustainable built environment these effects must be recognized and reduced. The materials for the primary construction, the facade and the energy concept play an important and decisive role.

Parallel to the above conditions, buildings around the world are increasingly evaluated according their ecological and economic qualities. Not only international certification systems, but many activities of governments support these developments.

The paper addresses the planning and optimization of the ecological quality of a building. Using an example of an office building, the principles and the method of the life-cycle assessment based of existing European standards is shown. Results and possible potentials to reduce the pre-and follow-up environmental effects through recycled materials and a consistent energy concept will be discussed. The paper shows the connection between the life-cycle based ecological accounting (Life-Cycle Assessment) and the economic accounting (Life-Cycle Costs Analysis) and discusses the implications for planning. The paper demonstrates finally the importance of such an analysis for the documentation of building qualities, for example for the certification of sustainability with an international focus.

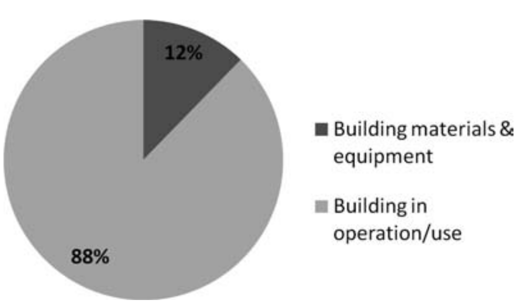

Figure 1. Ecological assessment at example of carbon emissions equivalents, share of construction and operation.

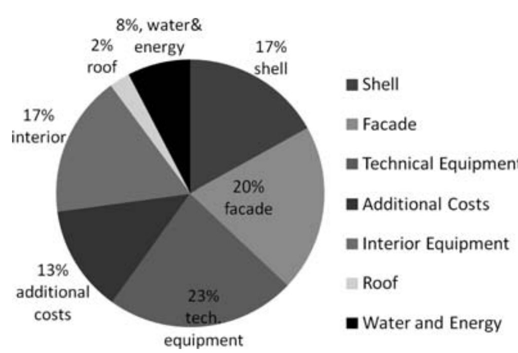

Figure 2. Optimized life-cycle costs of an office building: Costs of energy and water in Operation less than 10%.

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Environmental life-cycle analysis of housing estates in Austria

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ABSTRACT

The overall objective was to optimize the environmental life-cycle performance of housing estates and to derive recommendations for planners and other stake holders involved in design and building. 31 housing estates in Vienna (Treberspurg et al. 2009) and Linz (Treberspurg et al. 2010) have been analyzed. The buildings have been completed recently (2002–2007) and 10 building fulfill the Passive House (PH) Standard.

The research focused on the following questions: Do the buildings meet the planned targets? How much improvement of the life-cycle performance can be achieved by the PH Standard? Which other energy efficiency concepts have a significant improvement of the LC performance? How much do building materials influence the LC performance of housings?

The methodology for the assessment of the life-cycle performance is based on ISO 14040, ISO 14044, EN 15978, EN 15603, OIB Richtlinie 6 (conversion factors) and the German certification system for sustainable buildings (DGNB, BNB).

The reference building is a LEH in concrete construction supplied by fossil gas with a boiler efficiency of 85% and 7 kWh/($m^{2\cdot}$a) electricity consumption outside of the apartments. The environmental effect of building materials was estimated by the reference values of the DGNB system. The effects of several environmental optimization concepts have been compared:

- "Ventilation": Mechanical fresh air ventilation system with heat recovery
- "PH": Passive Houses with "Ventilation"
- "Solar thermal": Solar gains cover 70% of final energy demand for domestic hot water
- "Timber construction": Massive timber walls and floors (Merl, 2005)
- "Switch DH": Switch of energy supply from fossil gas to district heating
- "ALL": All concepts together

The energy switch to district heating is most effective to optimize the life-cycle performance.

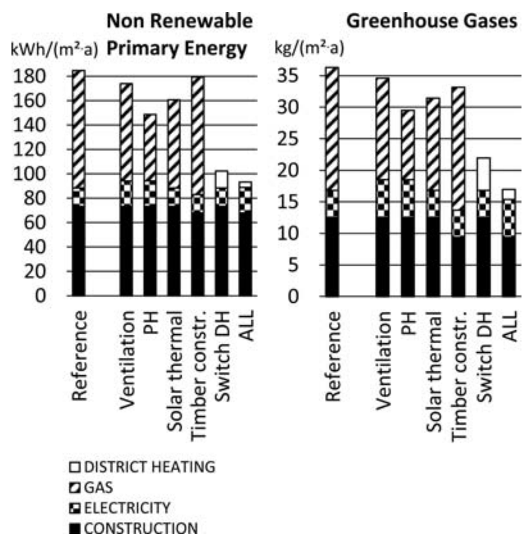

Figure 1. Improvement potentials of the environmental life-cycle performance by energy efficiency measures and environmental friendly building materials.

Furthermore the Passive House standard and solar thermal plants have significant positive effects.

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LCM – Life-Cycle Management, integrated management philosophy for building projects

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ABSTRACT

In construction project management the main goal was to realize the project in the shortest time, at minimum costs and in the best quality.

Nowadays construction project managers are constantly facing new challenges. Investors and users require sustainable buildings, low operating costs and flexibility. These requirements are a result of current problems such as extremely high operating costs due to high-tech buildings or difficulties in the maintenance due to inflexible structures.

The management philosophy Life-Cycle Management in construction is the sustainable way to cope with these requirements. It combines the core skills of professional construction project management with expert knowledge of life-cycle-oriented design and build under the consideration of the four dimensions market-, environment-, customer- and employee-orientation.

Core skills of a Life-Cycle Manager include expertise in project management, construction management, soft skills and professional knowledge of life-cycle oriented design and build.

For life-cycle orientated design und build it is essential to consider the experience of future project phases (operating phase, rebuilding, . . .) during project development and design.

This requires

– the integration of experience from future project phases (e.g. maintenance phases and the demolition or subsequent use of buildings) in the early stages of design.
– optimising projects in regard to operation,
– considering the requirements of redevelopment and rebuilding already in the design phase and
– user-orientated design.

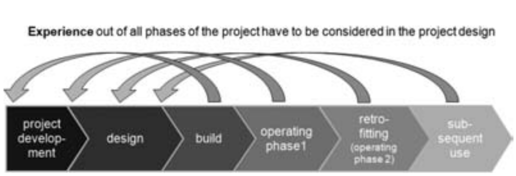

Figure 1. Experience out of all phases of the project have to be considered in the project design.

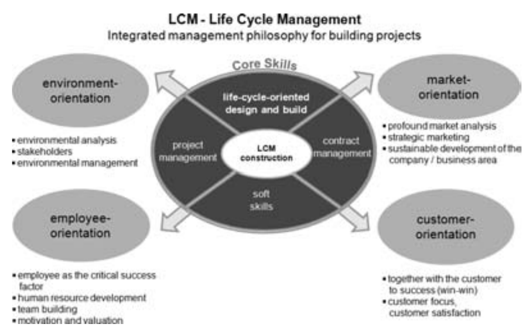

Figure 2. Four dimensions of integrated life-cycle management approach.

In order to meet these requirements the life-cycle manager has to be an expert concerning the mentioned core skills. Furthermore the integration of environmental-, employee-, market- and customer-orientation in the management philosophy is essential to achieve sustainable building projects (as shown in den following chart).

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Lehrgang LCM-Bau Nachhaltiges Bauprojektmanagement www.donau-uni.ac.at

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Has sustainability become the norm in the planning and execution of building projects?

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ABSTRACT

After many years of indecisiveness the building industry is now finally moving towards a holistic view of its product, the "building project". The reason why this is so is that existing patterns of consumption and production have proved to be unsustainable and cannot survive, because we are currently already using over 150% of the resources the planet can renew in one year (Ewing et al. 2010). Therefore the entire Life-Cycle Costs (LCC) of a project and more general aspects of sustainability will have to be considered in the future. Existing systems of project execution and contracts have to be adapted because they are frequently subject to litigation in our part of the world even now, and because the inclusion of criteria of sustainability, which are sometimes difficult to quantify, into the tendering procedure and the execution of projects will increase the number of potential conflicts even further.

Such a holistic approach is difficult to put into practice in a market with such a variety of players like the building industry. For constructions above ground level several different certifying systems (LEED, BREEAM, DGNB, ÖGNI etc.) already represent a first step towards sustainability, whereas constructions below ground and infrastructure projects are lagging behind, although the environmental impact assessment prescribed for large infrastructure projects guarantees that at least quality aspects of sustainability have to be considered in their execution.

As a result, the tendering procedure for construction work is slowly being adapted to include sustainability in accordance with the Austrian law on tendering procedures (BVergG), which makes "*considering the environmental justice of a project*" a criterion for awarding the contract (art. 19, par. 5 BVergG); the Austrian "*Action plan for sustainable public procurement*" (NAP) is another step in this direction. But the processes used in the planning and execution of projects are lagging far behind when it comes to including aspects of sustainability systematically; indeed, sustainability is not yet the norm.

Table 1. Example – assessing the quality of a facade according to ÖGNI sustainability criteria.

B*	Average U-value (W/m^2K)	R**	LEK-value (W/K)	R**	Costs (€/m^2)	R**	Weighted points
A	0.15	3	7.06	3	120	1	2.50
B	0.28	1	14.56	1	80	3	1.50
C	0.26	2	11.94	2	82	2	2.00

*Bidder. **Ranking.

The criteria used to assess tenders need further development since no models exist for the calculation of LCC or the evaluation of sustainability criteria. Technical and ecological quality criteria can be assessed on the basis of ÖGNI descriptions. During a tendering or a competitive dialogue procedure, say, for a facade, the heat transmission coefficient, the LEK-value and the price offered can be evaluated. Based on the rankings of each individual criterion ($1 =$ last position), the assessment points are calculated by weighting them and then the best bidder is found.

If the criteria for awarding a contract cannot be quantified mathematically, a commission will have to evaluate the tenders. This has advantages and disadvantages, in particular as far as the transparency of the decision is concerned. The prerequisites are suitable models for contracts and project execution and their adaptation to the requirements of sustainability as discussed here by the authors of this study.

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Passive House – best practice examples of cost effective building solutions with high-quality living

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ABSTRACT

Rising energy costs, growing dependence on energy imports from instable regions and CO_2 pollution need to be addressed with sustainable and cost effective solutions. Buildings are one of the greatest opportunities for conserving CO_2. The Passive House Standard (PH) developed by Dr. Wolfgang Feist in the early 1990's offers a concept that is adaptable to different local climates and is a cost effective solution to these crucial global problems.

The objective of this paper is to give an overview of practical examples from modern sustainable city development to newest building development in Vienna. Implementation of beyond state of the art technology with economic design and high life quality is of great importance in each project. Life-cycle costs, energy monitoring and social aspects are standard design practice at Treberspurg & Partner Architects ZT GmbH. A close cooperation with national and international research facilities to develop and monitor innovative solutions is of high importance. The following best practice examples will be explained in detail:

– "SolarCity Linz Pichling": a model for sustainable city development including the first multi-family PH in Upper Austria.
– "Roschégasse Vienna": first biggest Multi-family PH in Europe (from 12/2006–01/2007) with carefully planned construction details, an economical design by a committed project team and determination by the building developer to implement sustainable solutions.
– "Austria House for the Olympic Games 2010 in Canada": first PH in Canada and the most energy efficient building ever constructed for the Olympic Games.
– "Young Corner Vienna": Design for special requirements of young residents with low-cost but high-quality living. Furthermore economic feasibility of different façade constructions have been analysed to gain additional usable area.

Scientific research based on Life-Cycle Assessment could document the advantages of PH-Standard compared to low energy houses. All given examples demonstrate that cost effective buildings with high-quality living for now and future generations are feasible if principles of solar architecture to achieve PH-Standard are applied already in an early design stage with a committed interdisciplinary project team. Future research and development need to focus on refurbishment of the existing building stock to ensure social security and equality.

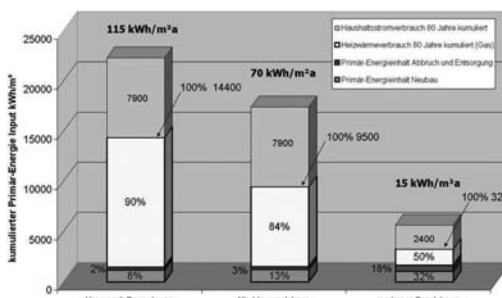

Figure 1. Life-Cycle Assessment comparing standard building stock (according to building code in 2000), low energy standard building and passive house standard building.

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LCA of multi-storey timber building and comparative estimation with alternative building materials

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ABSTRACT

The purposes of this paper are to estimate the sustainability of multi-storey timber buildings and compare their ecological implication to steel and Reinforcement Concrete (RC) models.

The inspected 7-storey house is realized in 2007 and considered as a benchmark that multi-storey wooden building is adaptive in the urban context (Deutsche Bauzeitung, 2009, Wiederkehr & Makiol, 2008). As various researches have proved, wooden products and constructions exhibit excellent environmental performance in terms of global warming potential and energy depletion for manufacturing (Buchanan & Honey, 1994, Gustavsson et al., 2010, Petersen & Solberg, 2005). For constructions, however, the quantitative testimony remains insufficient and comparative studies are intensively concerned.

Firstly, in order to evaluate the eco-efficiency of a timber building, the material inventory of the targeted wooden house is established. This inventory states the volume and sort of the primary building materials and is intended for subsequent sustainability evaluation. Second, the appraisal is carried out based on the concept of life-cycle assessment and by means of SimaPro, which consists of enormous databases and is widely applied for Life-Cycle Assessment (LCA) (ISO14040, 2006, Verbeeck & Hens, 2010). The third step is to compare the environmental impact caused by timber, steel and RC constructions. Steel and RC models are established depending on the original wooden building. These alternative models comprise not only similar appearance and profile but also equal structural performance as the timber one. Here, horizontal resistance against earthquake is set as the criteria about structural performance, i.e. the same lateral deformation due to seismic attack. Setting these criteria provides an equilibrium base for comparing diverse structures composed of different building materials. In this research, RSTAB helps figure out the steel and RC models. Finally, individual recycling scenario associated with specific material is taken into account for LCA.

The consequence demonstrates that, with the same seismic performance, the wooden construction consists of better ecological efficiency. Timber generates significantly less burden as far as greenhouse-gas emission and fossil fuel consumption are concerned.

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Life-cycle cost analysis and risk analysis for buildings

Organizers: T. Lützkendorf & S. Geissler, H. Kreiner

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OPIK-International: An international comparison of single life-cycle processes in hospitals

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ABSTRACT

Since the year 2001 the research project OPIK (Optimization and Analysis of the Processes in Hospitals) has been successfully running with cooperation partners from science, hospitals, professional associations and industry at the Karlsruhe Institute of Technology (KIT), analyzing different life-cycle processes in hospitals. As one of the results of this research project standardized facility management processes with single process steps, definition of the customers, responsible departments, characteristic variables (cost and quality factors) and the interfaces to the core business were defined.

This project becoming international, the question arose: "How do the same processes look like in other countries?", "Can one process be transferred from one country to another one by one?" "What are the similarities, semblances and differences and what are the factors that influence them?"

Therefore three – very technique oriented processes – that were already analyzed in the OPIK-Germany project were chosen and analyzed in a new research project, "OPIK-Iran". The processes were: maintenance of medical equipment; maintenance and repair of technical facilities and laundry management.

Results of this comparison and a developed "System transferability method" are presented in this paper. One outcome of this project was the implementation of Facility Management at Tehran University, the establishment of a German Iranian Facility Management Competence Center and the start of pilot projects were FM is realized in health objects.

The success of this project spread to other countries, like Namibia and South Africa were a Bachelor course is realized since 2009. These projects are accompanied by transfer of knowledge, information and know-how which sustains discussions and effects an examination and optimization of the own system, structure and used methods.

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Impact of semi-transparent building-integrated photovoltaics on building life-cycle cost

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ABSTRACT

The Energy Performance of Buildings Directive requires all new buildings to be nearly zero-energy buildings by 2020 (European Parliament; Council of the European Union 2010). In order to achieve this goal, energy efficiency measures have to be combined with on-site renewable energy production. Especially for high-rise buildings with large façade areas, the installation of a Building-Integrated Photovoltaic (BIPV) system is an attractive technical solution to reach a high share of renewable energy generation. Evaluation of the effect on building life-cycle cost is relatively straightforward for opaque BIPV systems. However, semi-transparent BIPV modules have multiple effects on building life-cycle cost: They generate electricity and influence the building's energy demand for heating, cooling and lighting. The associated costs account for a major share of building life-cycle cost. Mende et al. (2011) have analysed the impact of the change in lighting, heating and cooling demand induced by semi-transparent photovoltaic modules that are integrated into the façade of an office located in Freiburg, Germany (Clarke 2001), (Reinhart & Walkenhorst 2001). This paper analyses the economic impact of a BIPV system equipped with mono-crystalline, multi-crystalline, amorphous silicon or CIGS modules on building life-cycle cost. The cost of electricity generated is contrasted with the cost change in lighting, shading, heating and cooling demand caused by the shading effect of the semi-transparent photovoltaic modules over a 20 years time period. Different system configurations are evaluated by calculating the net present value. Also the change of carbon dioxide emissions (CO_2) from final energy demand is taken into account as an ecological aspect. All calculation results are compared to the life-cycle cost when the system is equipped with thermally insulating windows without photovoltaically active components. The result of the life-cycle analysis leads to the conclusion that the installation of a BIPV system is still associated with extra cost compared to a conventional glazing system. However, for mono- and multi-crystalline silicon with a medium transparency ratio these extra cost are below 100 €/m^2 in total for 20 years which might be considered a worthwhile investment for an investor taking also non-quantifiable factors such as the increase in image and building value from the BIPV system into account. With CO_2 abatement costs close to zero or even negative for mono-crystalline and multi-crystalline silicon the BIPV system has the potential to reduce the impact on global climate. Therefore building-integrated photovoltaic systems are an economic option for the provision of electricity from a renewable source and thereby saving CO_2 emissions.

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Specific life-cycle cost indicators and design recommendations for life-cycle cost optimized buildings

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ABSTRACT

Economic sustainability and life-cycle optimization

Sustainability is a paradigm, the new paradigm of today: The term appeared the first time 1987 used as "sustainable development" in the so called Brundtland report; this is a development that meets the needs of the present without compromising the ability of future generations to meet their own needs. The concept of the three pillars of sustainability, leads to "buildings and real estate are economically sustainable, if they can be utilized permanently and affordably fulfilling the needs of the present users without endangering the abilities of an economical and affordable utilization for future users."

Standardized life-cycle costs

To overcome the vagueness and complexity of life-cycle costs of buildings in total, experts of the Austrian standards committee 240 set up a consistent clear and detailed described structure of the follow-up costs. Life-cycle costs are defined as the sum of building costs and follow-up costs of a building. All costs are considered as present values, these are the costs discounted to the present day for comparing them directly to the building costs. The content and scope of all cost-corresponding actions of all cost groups of the follow-up costs are described in detail in two levels.

All described cost denotations and cost groups refer to the accumulated follow-up costs generated by the whole building over a considered calculation period. To compare different buildings it is necessary to compose key cost indicators. The key cost indicators for the building costs (*ERK*) result in $ERK/\mathrm{m}^2_{\mathrm{NFA}}$ or $ERK/\mathrm{m}^2_{\mathrm{PA}}$. This cost indicator can be denoted as specific building costs. For the different main cost groups of the costs of utilization, *ONK*, specific utilization costs can be derived.

These indicators are dependent on the considered calculation period. It makes a difference, if you consider the specific costs of cleaning and cultivation (ONK_4) over the first decade or over the whole assumed life span of e.g. 36 years.

Examples

In the following the life-cycle calculation results of the generally refurbished building of the retirement home and of the new headquarter of Greiner are presented. The calculations have been made by help of LEKOS, the life-cycle cost analysis tool, developed by the Department of Building and Environment at the Danube University Krems.

Criteria for life-cycle optimized buildings

Out of the examples it can be shown that building technology is a follow-up cost driver, caused by the maintenance of the equipment, the permanent consumption of electrical energy and the short life span of its components. All surfaces which need periodic cleaning, like e.g. glass and high polish floors are the second big follow-up cost driver.

A third follow-up cost driver is the not used or not easy usable space as a consequence of a vague space function programme.

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Life-cycle cost analysis of building components and materials used in hospitals

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ABSTRACT

Sustainability and increasing requirements on the economic, environmental and social performance of buildings are causing growing demands on building design. Regarding the current situation in hospitals, the increasing demand for technical equipment is leading to a shortened service life of the installed building products as the refurbishment cycles decrease rapidly. Furthermore the development is reinforced by the special functional and technical requirements of hospitals. These circumstances reflect an urgent need for life-cycle design in hospitals.

In Styria the biggest hospital association, which goes by the name KAGes, implemented a new sustainable strategy on utilization costs by improving the construction guidelines and introducing ecological revisions.

A new model for estimating life-cycle costs of different construction covering types like walls, floors and ceilings have been developed. The structure of the new LCC model is based on national and international guidelines, normative regulations e.g.ÖNROM B 1801 (ASI 2011), ÖNORM EN 15643-4 (CEN 2010b) as well as specific requirements of hospitals e.g. TR-PBB 004 (KAGes 1994) and is described by the following modules:

Module 1 – Database (input material/price data)
Module 2 – Build-up building part (modeling of building parts)
Module 3 – Algorithm (set up options/dynamic calculation)
Module 4 – Room concept book (results sheet/evaluation)

The developed calculation method records all processes and materials with their respective costs, to determine life-cycle costs for a set period of time. The prices ofthe positions and materials are taken from the „BKI BAUKOSTEN Positionen 2009“ (Fetzer 2009) and have been implemented in the new developed material database of the model.

The approach of the utilization costs which are incurred directly in the use of the building are based on the framework ÖNORM B 1801 and consist of the following costs:

- Maintenance costs – maintenance and repair costs
  – maintenance and restoration costs
- Operating costs – building cleaning
- Removal costs

The technical Service Life (SL) and the function conditions provided by KAGes, are clearly defined for each room and building parts. The provided SL is shorter than the Lifespan (LS) of the used materials to make sure that the maintenace intervals are not determined by the LS of the used materials.

To gain practice-related results the dynamic method of an investment calculation has been used in LCCA.Therefore a practical applicability is shown by the evaluation of a hospital in Graz (AT) and the results are presented with a comparison of different building components and examined life-cycle cost performances in this paper.

This new developed calculation model can be used to analyse and forecast LCC of selected building parts in hospitals and shows, that the LCC calculation implies important benefits in the early planning phase and helps to reduce building operation and maintenance costs.

Furthermore with this method it is possible to compare different construction components and to optimize used materials related to their life-cycle costs.

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Life-cycle cost method for the early design phase of non-residential buildings

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ABSTRACT

Consideration of Life-Cycle Costs (LCC) during the early planning phases of buildings is insufficient at the moment. The reasons for this are that on the one hand the focus of clients for whom a building is being built most often remains on the initial investment costs. On the other hand available software tools and methods are complex and the data needed to use them properly is vague during the early design phase – the phase where cost minimising can be most efficient.

The Figure below describes the essential problems of existing LCC-tool methods. Today it is common that the projected investment and operating costs of buildings are based on benchmarks of existing buildings. Top-down approaches do not exist in sufficient detail to be used in the early design phases of a building, when different types of building systems with altering costs have to be compared.

Existing software tools for calculation of LCC are based on the bottom-up approach which makes it necessary to enter itemized data (i.e. lime cement plaster, or type of paint coating/finish of paint).

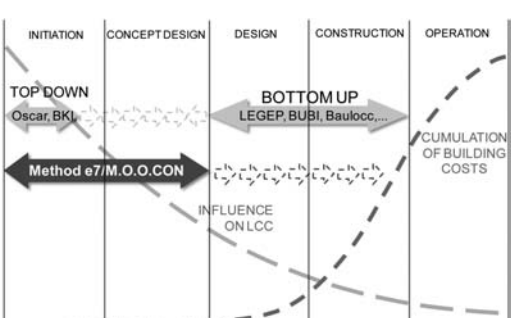

Figure 1. Missing link in models for calculating life-cycle costs during the design phase (Source: original illustration).

On the one hand this requires a great deal of data entry while on the other hand the data is simply not available at the required level of detail in the initiation and early design phases. A quick simulation of different variations is only possible through a great expenditure of time and effort.

The objective of the newly developed approach was to model the building in such a way that LCC can already be calculated in the early design phases; even at a point of time when no design for the building is yet available at the definition of requirements. The main concept is to have an 80/20 Pareto principle that is applied in the design process: by using approximately 20 percent of input efforts 80 percent of the indicators should be calculated. Furthermore, the first LCC calculation should be carried out before the first architectural concept is drawn.

In order to make use of this approach models for generating the space allocation program and volume program for the building as well as data for construction costs and operating costs are necessary on an aggregated level. This eventually enables entries to be made before the beginning of design. In addition, an energy calculation model that is directly incorporated in the LCC model should illustrate the interdependency between the building design, the façade, and the building equipment system. If this is executed in this way, no additional energy calculation tool is needed. The model design should enable LCC analysis during the design phase for optimisation of the building concept and, to a lesser extent, during the preparation for construction for the optimization of the building components.

By integrating the necessary input data for this model into a software tool, it is possible – with an acceptable expenditure of time and effort – to make reliable statements on prospective investment and operating costs of the building and thereby accelerate the realization of sustainable and energy efficient construction concepts.

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The use of life-cycle analysis for planning and assessment of construction works: Topics and trends

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ABSTRACT

Investors, architects and engineers traditionally are concerned with the effects of planning decisions on the building during its life-cycle. The definition of the designed service life is already an important task. In early design stages takes place among other tasks the adherence to standards and laws, the formulation of user requirements and their fulfillment, as well as the cash flow and profitability analysis.

In the past long-term considerations were partly neglected. Through the dealing with issues related to the implementation of sustainable development principles in the construction industry also the long-term perspective has started gaining in importance. The valuation object of a sustainability assessment is always the construction work including its associated plot of land and its behavior over its entire life-cycle. At the same time, apart from technical, functional and economical issues are also increasingly considered social and ecological aspects. In the meantime, both the international (e.g. ISO TC 59 SC 17) and European standardization attempts set high requirements with regard to the complexity of a sustainability assessment. Here a transition was accomplished from a predominantly qualitative to a predominantly quantitative assessment leading to an increase in demand for suitable methods, tools and metrics of calculation and valuation.

A basic approach for improving the planning process and supporting the sustainability assessment is the life-cycle analysis. So far by this term normally it was understood a combination of life-cycle assessment and life-cycle cost calculation. Both of them are based as a general rule on an identical building model and assumptions on the building life-cycle. Through the combination of life-cycle assessment and life-cycle cost estimation can the time and effort spent for describing the building be minimized, as well as a plausibility test can be carried out.

For these the range of e.g. construction costs can be consulted. In the meantime an ecological assessment is based partially on the results of an ecological balance as well as an economic evaluation is based partially on the results of a life-cycle cost calculation. There is a need of suitable metrics of valuation (benchmarks).

This form of life-cycle analysis needs to be redeveloped. On the one hand, the impacts on environment and health (risks), as well as the impacts of planning decisions on users' comfort and satisfaction must be to an even greater extend considered. On the other hand, the different assumptions taken into account during the life-cycle assessment and life-cycle cost estimation (utilization, service life, maintenance cycle etc.) can influence the final result significantly. These parameters are associated with uncertainties.

In days to come for the life-cycle analysis the handling of uncertainties becomes a substantial task. Uncertainties stem from the development of the market and thus the demand for the building (risk of changes in market), from the behavior of the users, from the change of boundary conditions and uses (e.g. climate change) as well as from the reliability and the long-term behavior of construction and building products.

A need for action in the area of benchmarks, uncertainty and user satisfaction is pointed out.

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Risk management and robustness as part of sustainability assessment

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ABSTRACT

A trend to make the performance of buildings comparable and to assess them in terms of sustainability is highly visible. This development is encouraged by the European Commission, as manifested by various strategies to change competition and framework conditions as well as by binding directives and regulations e.g. the new Construction Products Regulation. However, this development does not take into account potential risks buildings are exposed to and the probability of their failure in case of unplanned impact, particularly if not load-bearing structures are affected, and if these impacts are other than those causing frequently injury to persons, such as fire or earthquakes. Especially the EN 1990 defines reliability as the ability of a structure to meet the specified requirements within the intended life span: load-bearing capacity, serviceability and durability.

No comparable information on useful life, failure probability, etc. is contained in the regulations for non-load bearing structures and structural parts that are of little or no relevance in terms of safety.

At present, potential risks to which construction works may be exposed due to the impact of hail, torrential rain and floods are not considered sufficiently. In Austria, an Online-Platform called HORA (http://www.hora.gv.at/) has been developed as a database for natural hazards, which can be used from insurance companies for risk assessments. A link to the respective construction products standards regarding the resistance of exposed construction products is missing. For example, the construction products standards of non-load bearing structures have to consider methods of testing and categories regarding the resistance against the impact of hail. This applies to all parts that are relevant for finishing and completion including, e.g., components of the building envelope and façade or roof coverings, which are known to be particularly vulnerable to extreme weather events.

Generally, there are three basic rules which – depending on constraints and installation environment – are apt to achieve adequate durability if either applied individually or in combination with each other:

- appropriate choice of materials,
- constructive detailing,
- additional protective measures.

Today, there is a growing interest in more robust construction products/building components/constructions and in improving the planning of constructive detailing and, above all, in enhancing the quality of workmanship. This also includes the inspection and disclosure of building properties after completion. In view of the growing demand for sustainability certificates, particularly for larger buildings, it would be desirable to integrate the risk aspects mentioned above into certification systems as well as to develop them as stand-alone tools rather than to resort to separate risk-oriented building passports that are exclusively tailored to the interest of the insurance industry. The numerous overlaps with building certification systems which are currently in use, e.g. as regards the assessment of a structure for recyclability (ability to be dismantled, separated and reused), demonstrate that robustness and ease of exchange and repair can be included in the assessment with comparatively little additional effort. For buildings of smaller size all the way down to single-family dwellings, simpler and less costly evaluation concepts have to be developed accordingly, an issue which still needs to be addressed in building certification. However, true success will only be achieved if this kind of proof is demanded by the market, and if these building certificates find their way into the assessment of real estate.

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Life-cycle assessment and construction costs of a low energy residential building

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ABSTRACT

The identification of the construction sector as one of the most energy consumer dates back several years and led to enact some rules that are pushing to build near zero energy buildings, by the reduction of energy consumption during use. Instead, the low energy buildings involve a significant increase in energy and raw materials consumption and in pollutants production, during their first life stage, from components production to building construction. For this reason the Technical Committee CEN/TC 350 of the European Committee for Standardization (CEN) has been working for several years on new standards which drive the sustainability level assessment of buildings, in a life-cycle perspective.

The life-cycle assessment of a newly built residential complex allowed us to highlight the environmental impacts due to materials, energy consumption during construction and during the use phase. The object is constituted by four buildings of two floors, placed on a wide underground basement. The construction techniques are traditional: reinforced concrete for structures; thermal bricks with external insulation in rock wool for perimeter walls; wood structure, rock wool insulation and concrete tiled for roofs. Winter heating, summer cooling and domestic hot water production are assured by a central heat pump that takes advantages from an horizontal borehole heat exchanger. It is fed with electric power produced through the photovoltaic panels installed onsite. So this complex can be judged like *near zero energy building*.

The LCA evaluation related to the building materials shows that the structural elements are responsible for about 70% of the total impacts, instead materials for masonry, insulation and plaster account for 10% each. The windows are responsible for very low impacts, around 1%, even if they are made by PVC. The high prevalence of structural elements is linked to the construction of the basement, which absorbs 75% of the materials delivered to the site. Among those, 90% is reinforced concrete. During the construction phase the impacts are mainly generated by the use of electricity, on some indicators; while in other cases the waste category of recyclable packaging materials is prevalent. On the contrary the construction debris have very low impacts. The weight of the building site impacts is contained between 1% and 10% of the outputs related to the construction materials, depending on the indicator. Thanks to the high efficiency of the buildings, the impacts generated during the use phase overcome those related to construction materials after a life span included between 25 and 50 years.

Concerning the construction costs, the impact of structural materials is about 55% of the total and the weight of windows is equal to 23%. It is important to underline that the less impacting category for the environmental point of view is the second regarding the economic aspects. Instead, the cost of the construction phase is equivalent to only 4.5% of the cost of components. The small demand for electric power from the national network allows the inhabitants to limit the annual expenditure for energy supply so much so that it takes about 40 years to equal the exercise cost and the cost of building materials.

To conclude, we can state that in this case study the high energetic performances have a lower influence than the structural elements. Therefore it is important to support the energy researches with the appropriate architectural choices, without deny the environmental advantage assured by envelope and plants efficiency, because of impacts of structures.

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Combined database for LCC, LCA and life-cycle quality

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ABSTRACT

The focus of the research project "life-cycle improvement of the building quality" is to develop, analyze and evaluate methods to assess planning decisions about their consequences on the life time of a building. The project has two main outputs. One is the creation of a common database on building life-cycle data; the other is a comprehensive guideline on sustainable building.

The comprehensive guideline is programmed as web oriented database and is structured as a dynamic matrix. The vertical trace follows the five chronologic phases of the life-cycle of a building: concept, planning, construction, utilization, demolition/recycling. The horizontal succession of the columns is called "layers" and helps to detail on the topics (LQG 2010). The second layer allows the main phase to be more precisely described. With the third layer topics like "project management, energy, building biology, building environmental impact, economy may be selected, while the fourth layer provides additional information that serves as an overview for better understanding of the challenges inherent in building construction. The descriptions should be for people with lower levels of experience in planning sustainable buildings. The fifth layer provides the detailed information on specific topics. The sixth layer provides the space for references, important links or documents such as reports, that are available without copyright. The focus of the common database will lie on LCC- and LCA- Data. As the approaches to LCC are various, the calculated LCC are also very different and the market does not trust the calculated figures yet. Therefore two main improvements are suggested: The usage of all given information and the establishment of continuous and constantly narrowing calculations similar to the investment calculation, which is getting more and more accurate during the planning process. The first improvement is achieved via consequently using regression analysis instead of mean values to evaluate benchmarking data (Schrag 2011). The latter is achieved through a framework, where different benchmarking pools can be used and the calculated results can not only be compared to each other but also be combined with more detailed calculations for specific building elements. Life-Cycle Assessment (LCA) is a useful tool to describe life-cycle characteristics of products, services, or entire systems from an environmental point of view. Environmental assessments of buildings are often focused on the production of building components. In order to optimize common building types with from an environmental perspective, a Life-Cycle Assessment of the whole building is necessary. All relevant life-cycle stages are accounted for, starting with resource extraction, the manufacturing of materials and the building itself, continuing with the use and maintenance stage, and ending with the consideration of the End-of-Life stage. Furthermore the consequences of different allocation methods for recycling have been analyzed (Piringer 2010).

As LCC and LCA prediction is especially important for new building elements, the development of different new energy saving facades is part of the project and the application of the methods on these new facades will be shown. As both of these Analyses need a calculation of the energy demand, the data are to be accomplished with the data from the energy performance certification. The comprehensive guideline and the data base will be placed finally in an internet portal, which shall give pubic and quick access to the scientific data. A first prototype within excel has been used to show how to evaluate two different facades in an example. As the LCC and LCA calculations are based on an energy certificate, the tool is called enhanced energy certificate.

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Life-cycle assessment of a passive house and a traditional house – comparative study based on practical experiences

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ABSTRACT

The paper presented a comparative study of two residential buildings, one built as a traditional house and the other as a passive house. Life-cycle analyses have been performed considering the cost evaluation, but also the environmental impacts related to the construction, use and end-of-life phases.

The main conclusions that result from the LCC are the following:

– The initial investments in the passive house were about 27% higher, mainly due to the higher quality and quantity of thermal insulation but also due to the special mechanical equipment's;
– During the use phase of 50 years, mainly due to the high energy consumption for building utilities, the costs for the traditional house became 53% higher, leading to an overall advantage of 46% for the passive house (Fig. 1).

Performing an LCA analysis some important conclusions could be underlined:

– As a general trend, the greatest impacts of the two houses were related to fossil fuels, respiratory inorganic substances and climate change, due to the manufacturing of building materials and the energy demand during the use phase. Other smaller impacts were on land use, due to the damages on land caused by wood exploitation and ballast pits;
– In a single score analysis, taking into account the initial boundary conditions, the impact of the passive house in the construction and end-of-life phase (7535 points) is about 24% higher the impact of the traditional house in the same stages (6058 points). This difference is given by the better thermal insulation of the envelope;
– Integrating the use phase in the LCA analysis, the effect of the better thermal insulation is very remarkable. The impacts in use phase are about 10 times higher for traditional house and about 3 times higher for the passive than in the other stages, which reduce significantly their effects;
– The higher initial impact of the passive house is transformed in much more efficient operation phase, due to the reduction of the energy consumption. Expressed in eco-points, the life-cycle impact of the passive house (30402 points) is about 54% smaller than the impact of the traditional house (65707 points) (Fig. 2).

Considering a life-cycle approach the design and execution principles and techniques of a passive house offer beside economic benefits also lower environmental impacts, which will make it in the near future to a target for the new residential dwellings.

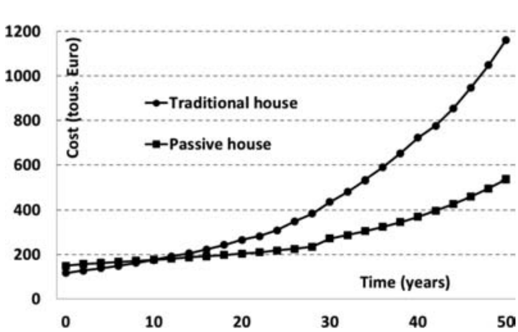

Figure 1. Life-cycle cost evaluation over 50 years.

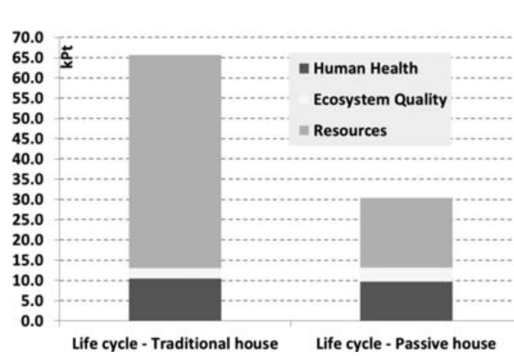

Figure 2. Final comparison on environmental impact – entire life-cycle (single score).

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Thinking future with risk management – a substantial tool of life-cycle management

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ABSTRACT

Buildings influence the environment und build the future. Nowadays Life-Cycle Management means not only to define the best technical lasting. It goes further and tries to find the optimized building in the fields of efficiency, technical quality, maintenance, impact on the environment, comfort for the user and of course economic performance. To achieve all these aims it's essential to have the right management tools to integrate the different demands and to create future scenarios for the decision making.

Risk Management is the tool which uses scenario-based designs to create different situations considering influences and impacts on the future building and its environment in the short term as well as in the long term. Therefore it is a powerful and indispensable tool for Life-Cycle Management which treats the consequences of aims and decisions at the phase of design to the built and operated object till the reuse of the object.

Risk Management integrates practical experience, scientific research and trend analysis for the forecast to build scenarios out of different perspectives. It is not only about finding a right figure – for example the total life-cycle costs, it identifies the main issues, it shows the main aspects which are to manage, it specifies measures that has to be taken, it creates a system for dependences and sensitivity, it detects time risks, it implements chances and it evaluates the knowledge out of other projects.

The main achievement of Risk Management is the increased reliability in design, costs and schedules. For Life-Cycle Management it means an improvement of the whole design process and the output. Risk Management is the main tool to animate Life-Cycle Management.

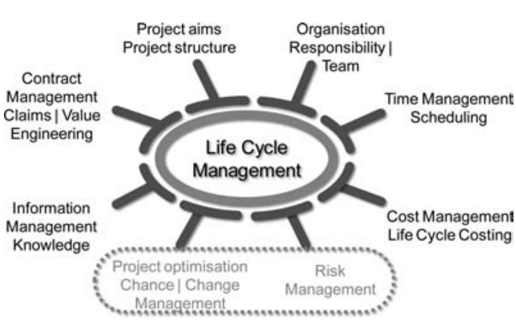

Figure 1. Tools for Life-Cycle Management.

Figure 2. Perspectives of Risk Management.

Figure 3. Elements of Risk Management.

Life-cycle design and engineering of facades and building envelopes

Organizers: O. Englhardt & A. Merl

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Decision support method for flat roofs with focus on life-cycle costs using a probabilistic method

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ABSTRACT

A probabilistic method based on a decision tree (Nusser 2012) is summarized in this paper. The tool calculates the impact of a specific roof construction on the life-cycle cost and energy efficiency of single family houses. Using this method the decision for a certain roof can be made incorporating durability aspects, energy requirements and costs. By varying environmental and constructional parameters the solution with the lowest life-cycle costs can be determined. In the first part of this paper an approach for a probabilistic tool to predict the interior climate is used. User-factors like moisture production, ventilation, etc. are brought into a relation to forecast the indoor relative humidity. In the second part of this paper the fault tolerance of the construction is used to calculate the life-cycle costs. By varying factors like the interest rate the most sensitive measure on the life-cycle costs is evaluated.

In the first part of this paper method is used, which estimates the indoor relative humidity with a certain probability. The method takes user dates like persons per household, moisture production or ventilation (window opening, mechanical ventilation system, airtightness of the building) into account. The output is a cumulative frequency distribution for the mean relative indoor humidity in January.

The second part of the paper calculates the lifetime and further on the life-cycle costs of flat roofs depending on environmental and constructional parameters. Factors are temperature, humidity, radiation, pressure difference between the interior and the exterior, shading, solar absorption, air tightness, etc. Depending on the amount of moisture which is accumulating in the flat roof from year to year, the lifetime of the construction can be determined.

Depending on the money which is necessary to construct a new roof at the end of the lifetime, it is possible to name the present cash value, which has to be put aside now to pay this new roof at the end of the lifetime.

This tool is helpful in the decision making process, which investment in retrofitting (airtightness, ventilation system, etc.) is the most sensible. With this method it is possible to optimize the life-cycle costs by choosing the right flat roof for each situation.

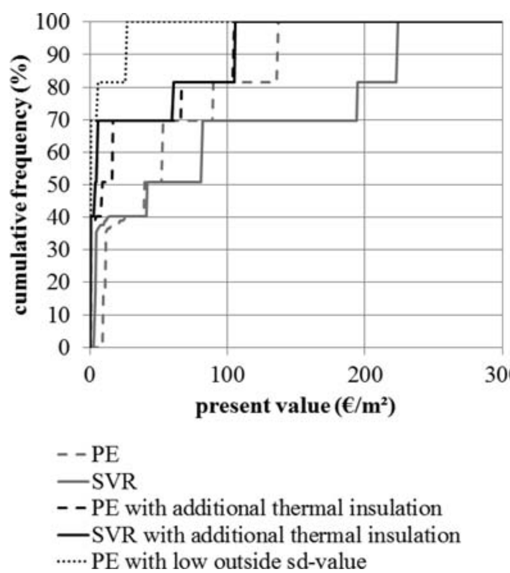

Figure 1. Cumulative frequency of the present value with 10% mvs and a low n_{50}-value for different flat roof constructions with a building price index of 3% and an interest rate of 4%.

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Green homes through LCC(A)-based planning of multi-functional building skin

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ABSTRACT

The paper presents a "pioneer-project" from Frankfurt in many aspects: For the first time in Germany, multi-family homes are created with the highest standards of energy efficiency and sustainability in a new monolithic construction with a multi-functional building skin. The buildings are built by the ABG Frankfurt Holding. The five apartment buildings with 50 apartments consist of different types of houses and are designed as passive houses in a new monolithic construction, which is now used for the first time in combination with such a high energy standard. The challenge for the planners was the consideration of different aspects and their integral involvement in the planning using holistic valuation methods. On the basis of predetermined parameters of urban development plans, planning and requirements specifications for example number of housing units required, aspects of family-friendliness, accessibility and infra-structure – for a long term social sustainability- should be implemented a design with the highest standards of resource efficiency in the broadest sense.

Aspects of energy efficiency, Life-Cycle Costs analysis (LCC) and Life-Cycle Assessment (LCA) of the new construction were investigated, to ensure the long-term ecological and economic sustainability for users and owners. The solution was achieved by an integral approach. Structure and envelope; function and performance of the buildings were connected. After a comparative analysis of materials, a monolithic new brick was used. It takes both loads and high insulation properties. Thus it was possible to achieve a high energy standards and at the same time to realize a sustainable economic building skin. The Paper reports on the role of LCC and LCA as planning tools creating ecological and economic benefits for the users and owners of the shown project green homes. First experiences with the new multi-functional building skin in connection with high energy standards will be discussed.

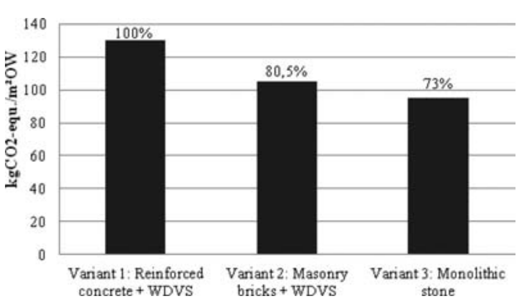

Figure 1. CO_2-equivalents as an exemplary ecological effect of production, maintenance, and demolition of the investigated three variants of a wall construction.

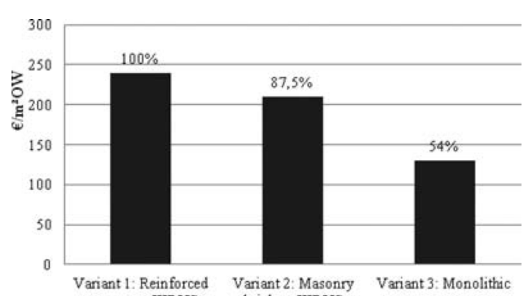

Figure 2. Life-cycle cost analysis of the investigated three variants of wall construction.

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Parameter study of a prefabricated retrofit façade system

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ABSTRACT

In the field of energy efficient retrofit of buildings the improvement of the building envelope is central, to gain a low energy demand and user comfort in modernized buildings on a sustainable basis. An improvement of the building envelope towards the passive house level is a necessity and a cause of construction material consumption.

The change in perspective towards the building stock gradually gives insight to the economic and social resources lying there, sleeping. Refurbishment that is largely driven by ecologic reasons can activate them when the material input and resource consumption are bargained at the same time.

Because of the reduction of the building performance of existing buildings compared to new ones, they are going to be less attractive and lose worth on distinct quality criteria. A lower quality leads to devaluation and the obsolescence of the object, (Thomsen et al. 2011a). If an object is no longer used literally it is gone lost and can be erased or replaced by a substitute. A sustainable development has to take care of that the life-cycle of buildings is prolonged, (Thomsen et al. 2011b).

The conservation of approximately fifty up to more than ninety percent of bound Primary Energy (PEI), reused from a building itself is a big achievement, compare Figure 1. But in the medium term diminishes the success of savings from seventy-five to eighty-five percent of the operating energy, when it is compared-with the material input side. This is reflected in the long payback period for expenses of input bound primary energy.

The duration of payback time for energy savings is related here to an alternative retrofit method which is primarily based on renewables. The case of the application of a conventional method of refurbishment means a greater input of PEI_{nr} and thus a poorer amortization of the chosen solution. For the different scenarios should each optimum calculated, what expenses in the context of the scenario are sustainable.

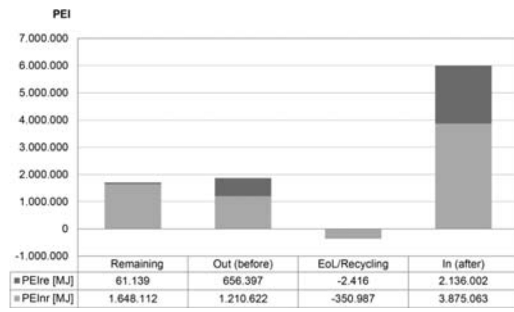

Figure 1. Comparison of PEI over various product stages of the multistorey dwelling case study.

Interesting within this context is the question whether an optimization of energy levels and the resulting expenditure for the refurbishment makes an earlier time of obsolescence acceptable. For example in buildings, which have functional, social, cultural and urban low quality would require a disproportionate economic effort for quality improvements.

Both case studies show buildings with alternative retrofit façade systems/curtain wall systems. In the case of the total renovation of the apartment, the input to PEI_{nr} is so high, that the return on invest of resources is according quite long. Therefore it comes, despite renewable resources in the façade, not to a considerable reduction of PEI expense.

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Implementation of energy efficient measures in apartments in Macedonia

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ABSTRACT

In order to decrease the consumption of heating energy, nowadays the mankind strives towards design and construction of new energy efficient buildings and towards undertaking repair measures of the existing ones. Repair of existing buildings incorporating thermal insulation materials in the building envelope, as well as construction of new energy efficient buildings requires special consideration on the type of insulation used, the thickness of the insulation layer, the location and the manner of installing the insulation.

There is an imperative demand for low-cost, energy-efficient housing in Republic of Macedonia. According to the census from 2002, the multi-storey buildings represent 38–40% of the total housing. 65% out of them are three to four storey buildings, 30% are buildings with seven to ten floors, while only 5% are 10 to 16 floors.

The buildings that had been built during the past century (50-ties and 60-ties) in Macedonia do not have thermal insulation incorporated and, therefore, they manifest huge consumption of electricity for heating. The analysis was carried out on a typical four-storey apartment building with prefabricated RC panel systems, named “Karposh”. Nearly 15000 apartments were built in this style, with a total area of 760000 m^2.

The measures that have been undertaken for energy conservation of the analyzed buildings referred to the phases of: installation of thermal insulation (placing of thermal insulation on the external walls, roof and the floor, i.e. on the whole building envelope) and replacement of the old windows with new ones. The influence of insulation and window-retrofits interventions in increasing the life-cycle performance of buildings has been investigated.

For the analyzed buildings, and for each structural element where thermal insulation has been incorporated, the type of the thermal insulation material is different, and the thickness (5, 10, 15 and 30 cm) of the insulation layer has been varied. The thermal insulation materials have been included with their lowest coefficient of thermal conductance, as follows:

- Rock wool, $\lambda = 0.035$ W/mK
- Glass wool, $\lambda = 0.037$ W/mK
- Expanded polystyrene, EPS, $\lambda = 0.027$ W/mK
- Extruded polystyrene, XPS, $\lambda = 0.025$ W/mK

The ENSI Key Number Software is a very efficient tool, used both for identification and evaluation of measures. The existing condition of the building for heating energy consumption before the energy conservation measures is calculated. After the implemented measures, the total energy consumption is calculated, as well as the energy consumption for heating. The total energy savings, the budget power for heating, the thermal losses of the separate elements in the building envelope, as well as the total thermall losses were analyzed and commented.

The performed analyses lead towards conclusions about which measure is the most profitable to be applied for the analyzed buildings, as well as which type of thermal insulation material and which depth of the insulating layer is the most appropriate. The manners and methods of in-built of the thermal insulation materials in the structural members of the building are represented as well. The proposed optimum insulation thicknesses have a significant impact on the life-cycle energy savings.

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Life-cycle of building's facades: Service life prediction of natural stone wall claddings using the factor method

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ABSTRACT

In this study the factorial method proposed in the ISO 15686:2000 standard is applied to predict the service life of natural stone wall claddings. This method consists on determining the estimated service life through the product of a reference service life by a set of modifying factors resulting from different characteristics of the claddings.

The method proposed is based on a survey of the degradation state of 130 stone cladding systems in the region of Lisbon, Portugal. The anomalies and degradation extent of the claddings are analysed and associated to various maximum levels of degradation in order to define, as a function of the case studies and the authors' perception, the maximum admissible level of degradation that leads to the estimation of the reference service life and the definition of sub-factors for the factor method.

The modifying sub-factors are analysed based on various scenarios. In scenario 1 the sub-factors are quantified by comparing the estimated service life of each subgroup of the sample with the reference service life. Independent degradation curves are drawn for the various characteristics that affect the durability of the claddings (Silva et al. 2011), which allow obtaining estimated service lives for each of them. The sub-factors are obtained through the ratio between the estimated service life corresponding to each characteristic and the reference service life (for the whole sample). In scenario 2 the reference values established in the ISO 15686 standard (0.8 for the unfavourable situation, 1.0 for the current situation and 1.2 for the favourable situation) are used and the values are adjusted to the data collected on site relative to the degradation of stone claddings in real use conditions.

The model is validated by comparing the service life of each case study estimated using the factorial method with the corresponding one obtained using a graphical method (where the degradation evolution is evaluated based on adjusting degradation curves to the scatter of points corresponding to the cases analysed on site). This comparison allows adjusting the sub-factors in order to optimize their values by minimizing the difference between the two methods in terms of estimated service life.

Two more scenarios are thus created. In scenario A the sub-factors are fine-tuned in order to minimize the differences between the models without imposing any numerical restrictions on their values. This scenario ends up being purely statistical, since the errors are minimized without taking into account the physical sense of the sub-factors analysed. On the other hand, in scenario B some restrictions are imposed in order to fine-tune the sub-factors bearing in mind the actual influence of the various characteristics on the degradation of stone claddings and also acquired experience/knowledge.

The determination of the sub-factors is adjusted to onsite reality. By comparing the service lives estimated using the graphical method with those estimated through the various scenarios of the factor method it is found that scenarios 1 and 2 lead globally to the worse results and the greatest percentage of failed estimates. As expected, scenarios A and B show the best results.

This study is a first approach to applying the factor method to the service life prediction of stone claddings in buildings' walls.

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Analysis of influence of water vapor condensation in building construction envelopes

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ABSTRACT

Moisture that accumulates in the constructions of buildings as a result of water vapor condensation is carried into its surrounding. This movement of liquid moisture depends on the moisture conductivity coefficient, as well as the moisture gradient.

In considering the density of the mass flow of liquid water with a moisture gradient, the following applies:

$$\frac{\partial w}{\partial \tau} = \kappa_m \cdot \frac{\partial^2 w}{\partial x^2} \qquad kg.m^{-3}.s^{-1}] \tag{1}$$

In order to solve partial differential equations, it is necessary to state initial conditions and boundary conditions. In this particular case, depending on the orders of the partial derivates, there is one initial condition (first derivation according to time τ) and two boundary conditions (second derivation according to x). The most suitable numerical solution to this partial differential equation of the moisture field seems to lie in using the finite difference method.

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:

$$\lim_{\tau \to \infty} w(x, \tau) = konst. \tag{2}$$

It is possible to express the constant in (2) – due to integral correction – as follows:

$$konst. = \frac{G}{|x_{\max} - x_{\min}|} \tag{3}$$

where G is the total amount of liquid water at a given moment in time.

Four different flat roof constructions were studied in both standard and extreme climatic conditions. In each case, the effect of capillary conductivity of moisture was included in the calculations.

Current technical standards specify the calculations that are to be carried out to determine the amount of water vapor condensed and evaporated over the period of one year. However, these standards do not take into account the process of capillary conductivity of moisture, which nearly always takes place when water vapor condenses in a build-in construction.

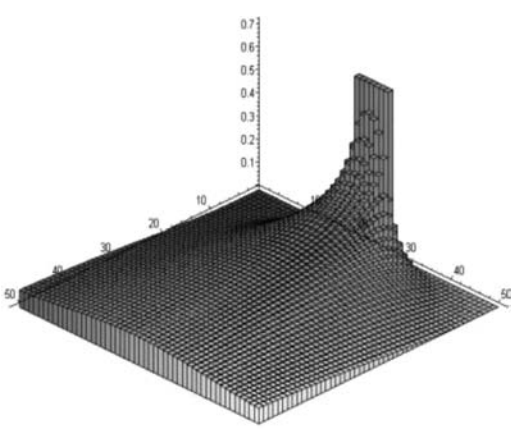

Figure 1. Graph of the function $w(x, \tau)$, verifying convergence for $\tau \to \infty$.

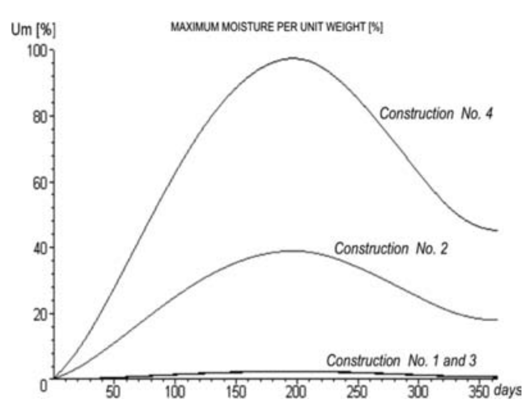

Figure 2. Changes in the values of moisture per unit weight in the most critical spot within the building construction over time.

Structural and thermal retrofitting of buildings

Organizers: O. Englhardt

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Sustainable retrofitting solutions for precast concrete residential buildings

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ABSTRACT

A large part of the Romanian urban population lives in collective multi-storey residential buildings made out of reinforced concrete structure on large prefabricated panels (Radoslav et al. 2010). Most of these buildings are over 30 years old, and the materials used for thermal insulation are already out-dated.

One of the main issues of the existing building stock in precast reinforced concrete large panels is related to the small living area of the apartments (see Fig. 1). The repartitioning of the internal spacing through the horizontal and/or vertical unification of two or more apartments is a good solution for the improvement of interior comfort. This solution will also lead to reducing urban density. However, the repartitioning could only be performed by operating openings into the diaphragm walls and by strengthening the affected zones.

The structural response is investigated by FE analyses under gravitational and seismic loadings. The new structures must fulfil the requirements of current design codes.

The responses of the numerical analyses on the structure with new walls openings were compared to the responses of the initial structure and to code limitations. The values of the normal stresses resulting from seismic combination show a very small increase in the diaphragm walls that were not intervened upon. Moreover, the stress redistribution due to the interventions changes the stress diaphragm field. Substantial changes are generally seen in the door lintels. The results show that shear stresses are five times higher in the case of repartitioned structure, due to the redistribution of stresses. The most sensitive areas are at the lower levels, generally in the connecting zones of the large prefabricated panels.

In consequence, solutions are proposed for the strengthening the affected diaphragms. They are steel-based solutions that create composite reversible structures thought for an easy erection. Steel frames are connected with reinforced concrete diaphragms by means of steel connectors, i.e. chemical anchors.

In the first case, frames are one storey high, fixed above the slabs onto the reinforced structure with chemical anchors. The beam-to-column connection is welded, and columns are pinned onto the slab and the lower frame girder by bolts, at the column basis. In the second case, the structural system is made as a continuous frame with rigid joints.

The use of such solutions leads to technical advantages such as reversibility, easy erection, on-shop partial manufacturing, easy interventions in case of impairment, easy checking of the execution quality.

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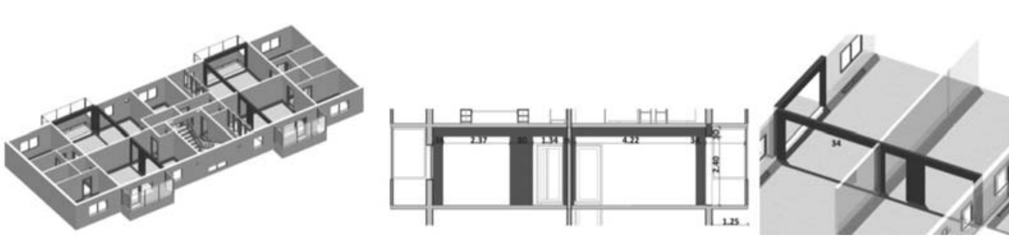

Figure 1. Example of space reconfiguration.

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A comparison of three schools renovated to the Passive House standard

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ABSTRACT

The Zirbitzkogel Grebenzen nature park school in Neumarkt in Steiermark is a typical case of a local school complex that was built and gradually changed over decades and has now been renovated after intensive project development that took into account the existing structure and materials. Construction units A and B were improved, construction unit C (classroom wing) was renovated to the Passive House Standard using timber elements and central comfort ventilation, and construction unit D (gymnasium and event hall for 800 people) was renovated to the Passive House Standard with a comprehensive insulating façade and its own comfort ventilation system.

After project development, the St. Leonhard elementary school in Arnoldstein was designed to be a place where children feel at home. The elementary school had been rebuilt in the late 1960s, and it was now time for a renovation. Renovation using prefab timber elements was ideal for the building's structure, and a central comfort ventilation system could be used because of the ceiling heights. Thanks to the project development process, at least three buildings could be combined in one (school, kindergarten, after-school care, and public library), thereby conserving energy and contributing to the village center.

An important factor in each of these projects was detailed analysis followed by an open project development process to develop goals in cooperation with future users (village residents, politicians, educators, custodians, and local government agencies). Our task as planners and architects is to use this information to make the correct decisions for each individual case. Besides detailed planning, an important factor for construction is on-site quality assurance, without which renovation quality will decrease despite pre-made materials.

Once the building is used, adjustments and fine-tuning based on measurement results can help ensure the project's success.

Energy efficiency and air quality is frequently controlled to ensure the quality and efficiency of the buildings over decades.

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- Energy Efficiency Award 2011

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New envelopes for old buildings – the potential of using membrane systems for the thermal retrofitting of existing buildings

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ABSTRACT

The project "membrane structures for the thermal retrofitting of buildings (MESG)" is aimed at the development of new concepts for the use of membranes for retrofitting existing buildings. The goal is to achieve significant energy savings in the area of heating, cooling and lighting energy consumption. The findings of this research project are suited to be applied to new buildings also. As part of this research project, various options and applications were examined, such as the use of membranes as sun-, heat- and glare- protection elements, and/or as elements for generating heat, cold, electricity or for ventilating buildings.

In addition to improving the thermal properties of membranes, this project also looks into the options of using additional technologies and systems to reduce thermal losses and to maximize passive solar gains through the use of suitable materials, coatings and components. Examples are special coatings to reduce heat radiation losses (low-e coatings), Phase-Change Materials (PCM) to store thermal energy, or translucent silica aerogels, foams or Vacuum Insulation Panels (VIP) to reduce thermal transmission. These applications offer a high potential for achieving considerable energy savings.

Membrane structures are not only suitable for use in new buildings, but can also provide an important contribution to increase the energy efficiency in existing buildings. Textile structures open up new ways in the energy-related renovation/refurbishment.

However, the roofing of courtyards and building areas only represent one of several approaches, since the field of facade structures offers great potentials also. By optimizing the functions of membranes (low-e coatings, etc.) or in combination with other components, the thermal properties can be greatly improved.

Through the passive use of solar radiation through transparent and translucent membrane structures or the integration of photovoltaic elements into the building envelope, weather protection can be combined with the production of thermal or electrical energy. Thus the materials and components that will be presented provide system planners with new possibilities in order to develop tailor-made and efficient solutions for the energetic improvement of buildings.

Project partners in this collaborative project are:

– Bavarian Center for Applied Energy Research, Würzburg, Germany
– University of Applied Sciences – FH München, Munich, Germany
– College of Technology (HFT) Stuttgart, Stuttgart, Germany
– Hightex GmbH, Bernau, Germany
– Lang Hugger Rampp GmbH Architects (LHR), Munich, Germany
– TAG Composites & Carpets GmbH, Krefeld, Germany
– Dörken GmbH & Co. KG, Herdecke, Germany
– Roto Frank Bauelemente GmbH, Bad Mergentheim, Germany

The project is funded by the Federal Ministry for Economy and Technology, Germany, based on a decision of the German Bundestag in the framework of the funding programme EnOB (Research for energy-optimized building).

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Indicators for sustainability assessment of renewables

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ABSTRACT

Environmental and operational loads are the design drivers of steel support structures for Offshore Wind Turbines (OWT). Besides design and installation a holistic design also includes sustainability aspects which dominate the decision process and the cost effectiveness of future renewable constructions. Within a large research project with 3 research institutions consulted by over 30 industrial partners sustainability issues for renewable energies are investigated. Hence, this extended abstract deals with special indicators developed to evaluate the sustainability of the steel support structures.

Regarding the structural design of buildings sustainability aspects are already taken into account. Established rating systems e.g. such as the German Assessment System for Sustainable Building (BNB 2010) or the German Sustainable Building Council (DGNB 2011) facilitate the evaluation and certification of buildings. Due to missing methods for other constructions, the paper makes a contribution to establish a rating system for steel constructions for renewable energy systems. In a first step, proven indicators originating from the building industry and characteristics reported in literature were used to determine new indicators. Beside environmental, economic and social indicators effects of the process quality and location were taken into account. In a second step, composed characteristics were transferred to the specification of steel support structures for OWT.

Concerning social characteristics, the complexity of installation and implementation of offshore steel structures was analysed. Therefore effects on employment were considered. Further investigations focussed on environmental effects resulting from a life-cycle assessment. Ecological consequences were quantified by the global warming potential and the cumulated energy demand. Recycling aspects of the constructions as well as deconstruction costs were also included.

Due to the basic understanding of sustainability reflecting the elements: environment, economy, society, process, and technique, the rating system for steel constructions of renewables is based on these sustainability elements. For all indicators and categories the assessment has to take into account the decisive life-cycle stages as discussed in Schaumann et al. (Schaumann et al. 2011). Figure 1 shows the main life-cycle stages referring to Hauke & Siebert (Hauke 2011). Especially the Life-cycle stage D plays an important role to the evaluation of steel structures due to the high recyclability of steel.

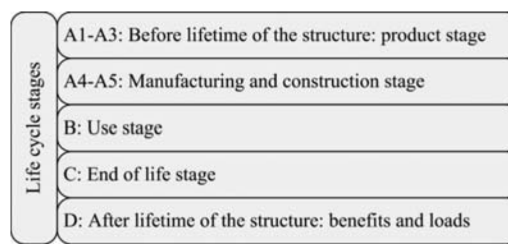

Figure 1. Life-cycle stages referring to (Hauke 2011).

Focussing on the applicability of developed indicators selective reference structures were chosen and analysed. Results are shown in the paper and first ideas about a full sustainability assessment method for renewable constructions will be given.

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Sustainable thermal retrofitting solutions for multi-storey residential buildings

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ABSTRACT

The official statistical data on the Romanian building stock shows the fact that more than one third of the Romanian population, lives in collective building types, such as apartment houses with the bearing systems on concrete diaphragms, concrete frames or masonry, the major part of them being built between 1960 and 1990 (INSSE, 2003).

The preliminary analysis performed on such structures reveal the fact that the largest majority satisfy the actual requirements in terms of resistance performance, including the seismic behaviour. The main problem of these buildings represents the low thermal efficiency of envelopes. Analysing several stratifications of walls it could be concluded that depending on the year of construction, the thermal resistance of initial envelopes is between (0.54–1.84) m^2K/W. However, in case of the majority of buildings the thermal resistances of envelopes do not fulfil the current requirements for thermal resistance.

In a modern design, based on the integrated strategy, the retrofitting process is based on a multi-criterial analysis, assessing all the issues that may interfere. In the case of thermal rehabilitation at least the technical, structural and economical aspects should be considered. In consequence, a global methodology could be conceived following the basic steps in thermal retrofitting of buildings: evaluation – design – construction. The sustainability should be considered as an additional parameter in the design and constructional phase. The choice of retrofitting solution should be based on an initial evaluation through an integrated design. The social, economical and environmental parameters could be considered by different approaches.

As a case-study on thermal retrofitting of a concrete building, four solutions are proposed for thermal rehabilitation of the envelope of a concrete structure through over-cladding solution:

- Solution 1: additional polystyrene layer insulation;
- Solution 2: rigid mineral wool and fibreboard;
- Solution 3: mineral wool insulation layer installed on steel stud framing and protected by fibreboard;

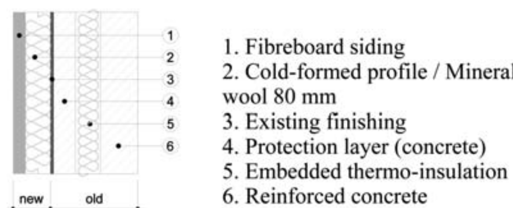

Figure 1. Thermal retrofitting solution – example (Tuca et al. 2011).

- Solution 4: mineral wool insulation layer installed on steel stud framing and protected by PVC board.

In the decisional process three parameters are considered: thermal resistance, environmental impact and economic aspect. A realistic estimation of parameters was performed. The retrofitting choice is discussed according to three solutions:

- *single indicator solution*: solution oriented towards a certain indicator.
- *multi-axial representation*: for each analysis parameter corresponds one axis and each solution is represented inside the so-created space. The final decision is taken by the point closest to the target.
- *characterisation factor method*: each parameter is multiplied with a factor expressing the weight of this in the final decision. The decision is taken by the aggregation of results.

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Sustainability certification of new and of existing buildings

Organizers: A. Passer & H. Wallbaum

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Rating tools for the evaluation of building sustainability

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ABSTRACT

Sustainability is defined at the confluence of the environmental, economic and social dimensions. The construction industry plays an important role regarding resource consumption, energy supply and greenhouse gas emissions, but also contributes to the economic growth, with social responsibilities. Therefor at the sustainability evaluation of construction works, all three dimensions must be considered in equal way.

The actuality and importance of sustainability is underlined by the ongoing interest and preoccupation of international organizations and committees to elaborate standards related to different construction activities. The standardizations can make the sustainability assessment of construction works more objective and transparent, creating a common platform for the comparison of the results. The first part of the paper is a brief presentation of international standardizations of ISO/ TC and CEN TC350 related to sustainability of construction works. The objectives of already published standards and of standards under development are shortly described.

In many countries rating and certification tools have already been developed in order to assess environmental/sustainability performances of constructions. Although there may be a different perspective in the approach of sustainability, most of the rating tools cover the same categories of performances like energy, water, resources, indoor environment, *etc*. They are applicable mostly on new or existing buildings of any typology. The characteristics of BREEAM, LEED and DGNB are presented. The tools are very comprehensive, but present also some disadvantages and conceptual gaps: do not cover all dimensions of sustainability, are difficult to apply or cannot be used for all types of construction works.

The authors developed two evaluation models, which intend to solve some of the above mentioned problems. The global model is an evaluation model developed for the assessment of residential buildings. The physical boundary includes the building itself with its entire components, the construction site, occupants but some categories may have an influence also on the local or global environment. The model should be applicable in design and operational stage. 16 major sustainability issues, with 46 criteria and over 50 qualitative or quantitative indicators are covered, combining all the three dimensions. The model uses a semi-objective weighting system, but keeping the proportion of 40%-30%-30% for environmental, economic and social dimensions. The quantification is based on a scoring system between 0 and 5 points, representing insufficient and best practices. The final result is a Building Sustainability Index BSI.

The specific model is flexible and target oriented evaluation tool which was developed mainly for the comparison of different solutions, but can be used also for self-assessment. It can be applied on partial building works, production of building materials, re-habilitation works, transport of prefabricated elements, construction technologies *etc*. This tool considers only quantitative parameters, from each sustainability dimension. Depending on the type of the construction works, parameters are carefully selected and evaluated, in order to permit a correct and objective assessment of each proposed solution. The parameters are combined using specific equations. The components of the equations are ratios between the calculated and reference values of the parameters. In case of a comparison between different solutions, the reference value represents the best value from each solution; while in case of a self-assessment the best practices available are taken as reference.The result is a sustainability index, with a dimensionless value between 0 and 1, where 1 is the best and 0 the worst value. The proportion of the weightings keeps the same values of 40%-30%-30% as in case of the global model.

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Case studies with the DGNB certificate and the OPEN HOUSE methodology – practical experiences and results

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ABSTRACT

Buildings have a large impact on the environment in a social, economic and ecologic dimension. To assess all these impacts a lot of certificates and methodologies have been developed. Starting with systems of the first generation, like the "British Research Establishments Environmental Assessment Method" (BREEAM) (Beck et al. 2011) in 1990 or the American label "Leadership in Energy and Environmental Design" (LEED) in 1996, as well as methods of the second generation, like the DGNB Certificate 2009 (Eberl et.al. 2010). With projects like OPEN HOUSE, SuPerBuildings or the standards from ISO TC 59/SC 17 as well as CEN/TC 350 a process has been initialized to harmonise all these approaches.

The DGNB Certificate started its pilotphase for certification in late 2008. Since then many objects have been certified and a lot of datas and informations are accessible.

The EU project OPEN HOUSE started in 2010 in order to harmonise current approaches which are assessing the sustainability of buildings and to create one common methodology for Europe. In 2011 a first version of the OPEN HOUSE methodology is available and will be tested with case studies in 23 European countries.

In this paper the experience gained from real case studies with the DGNB Certificate and the OPEN HOUSE methodology will be exemplified and further analysed.

Also a comparison of the outcomes from the different assessment methods will be shown as a real case example at the building Zentrum für Umweltbewusstes Bauen (ZUB), which acts as a demonstration and research building in the german city Kassel.

Figure 1. South-West façade of the ZUB. (Photo: Constantin Meyer, Cologne).

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Environmental evaluation of steel plates and steel sections sold on the French market

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ABSTRACT

The construction materials sector is the third-largest CO_2 emitting industrial sector world-wide, as well as in the European Union (UNSTATS 2010). Furthermore, over the past decades, the demand for natural resources has increased so much that it is now widely considered as a serious threat to our economical and social equilibrium (Millennium Ecosystem Assessment 2005). One of the key sustainability challenge for the next decades is thus to improve the management of natural resources in order to reduce current levels of anthropogenic environmental pressures.

Life-Cycle Assessments (LCA) on civil engineering structures point that materials production phase is responsible for significant environmental burdens on air and water emissions and resources consumption. To do so, engineers and researchers needs specific and relevant environmental data for materials in order to accurately design and promote structures that significantly reduce the human environmental pressure (Bouhaya et al. 2009, Kawai et al. 2005).

However, available data for these studies are often too generic to be well-adapted to one specific context due to the variety and specificity of civil engineering materials and to the local industrial practice that can differ from one country to the other. The present paper focuses on steel plates and steel sections. Concerning this material, the current available life-cycle data have been developed by the Institute Construction and Environment (Institut Bauen und Umwelt e.V. 2010). These data are an Environmental Product Declaration (EPD) according to ISO 14025 for steel section and steel plates sold on the Germany market. The main objective of this study is then to obtain environmental data corresponding to steel sections and steel plates sold in the French market in 2011 and to compare the results with the available data. The present evaluation makes a distinction between section and plates which is not the case for the German EPD; a single value for each environmental impact is assigned to these two products. Furthermore, this present study focused on steel sections and steel plates is part of a wider project leaded by the French Civil Engineering Society (AFGC) that aims to propose a public LCA database for construction materials, called DIOGEN (Habert et al. 2011, Tardivel & Tessier 2012).

As a conclusion, the comparison between the EPD values and the results obtained thanks to the model highlight the fact that EPD "average" the impacts of plates and sections which represent very different impacts on production life-cycle phase.

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EU-project OPEN HOUSE: Benchmarking and mainstreaming building sustainability in the EU based on transparency and openness (open source and availability) from model to implementation

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ABSTRACT

Numerous sustainability assessment methods exist in Europe and at the international level, but most of them are proprietary. There are also significant differences in the approaches of these methods, even within Europe. To fill the gaps between the different assessment methods and to create a common assessment method in Europe, the development of the new European building assessment methodology OPEN HOUSE was initiated. Central in the OPEN HOUSE concept are transparency and collective development in an open way across the EU. The process to develop the OPEN HOUSE assessment methodology for the planning and construction of sustainable buildings covers the following steps:

1. Assessment of methodologies, norms, standards and guidelines for the sustainability of buildings at national, European and international level
2. First set of recommendations for standardisation of the baseline
3. Definition of indicators, sustainability performance levels and procedures to evaluate them
4. Development of the OPEN HOUSE baseline

Nineteen partners from eleven EU-countries participated in the development of the OPEN HOUSE methodology. The basis for the new methodology is the already existing standards (CEN/TC 350 and ISO TC59/SC17) as well as other European and international assessment methods (BREEAM, LEED, DGNB etc.). The OPEN HOUSE methodology has been developed after the analysis of these existing methodologies with a main focus on the identification of indicators.

The newly developed OPEN HOUSE methodology covers 56 qualitative and quantitative indicators (full system indicators) from existing international and European assessment methodologies. For a basic assessment, a core system with a set of 30 indicators based on the indicators from the OPEN HOUSE full system is available as well (see Figure 1).

Category	Nr.	Indicator
Economic Quality	3.1	Building-related Life Economic Cycle Costs
	3.2	Value Stability
Technical Quality	4.1	Fire Protection
	4.2	Durability of the structure and Robustness
	4.6	Quality of the building shell

Core Indicators

Figure 1. Excerpt full/core system indicators.

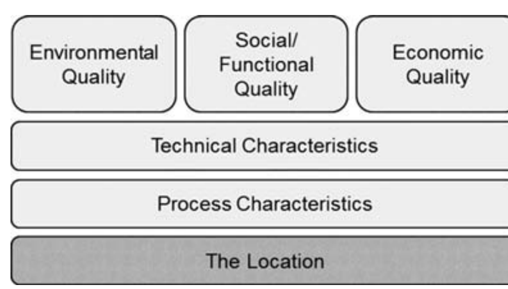

Figure 2. OPEN HOUSE assessment categories.

All indicators are listed in six categories related to sustainability and connected to all life-cycle stages of a building as shown in Figure 2.

The OPEN HOUSE methodology consists of a “basic and quick sustainability assessment” achievable within several days and based mainly on estimations. No stringent documentation is needed. The “complete assessment” is applicable when the building is finished and based on calculations. Complete documentation of the “OPEN HOUSE – core indicators” as well as the “basic and quick sustainability assessment” for the rest of the indicators from the “OPEN HOUSE – full system” is required. After evaluating all indicators, a label can be awarded.

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Statistical cluster analysis as a means to complement LCA of buildings

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ABSTRACT

Life-Cycle Assessment (LCA) models generated for evaluating the environmental impact of buildings usually allow for an abstracted view of one or more individual buildings. However, in order to optimise the LCA procedure in the future, it would be a prerequisite to model "typical" buildings instead of individual ones. As each building is a unique and complex system, the existing approach to LCA modelling needs adaptation when trying to establish a generic alternative model for "typical" buildings. This paper introduces a new approach to complement LCA modelling, suggesting the application of statistical cluster analysis to enable classification of buildings according to the inherent similarity of their individual characteristics. The concept will be illustrated for this paper by applying it to a small sample of eight newly built Swiss apartment buildings. The building data of the sample contains general descriptive information along with facts on the amount of materials used for building construction, construction costs, details on building installations and the buildings' annual energy demand for heating and mechanical ventilation. Through statistical cluster analysis, this sample of individual buildings is then sorted by certain conceptually highly ranked building characteristics into groups (clusters), thus disintegrating the concept of the buildings' uniqueness through the revelation of latent substantial similarities. Each of the clusters preferably contains buildings that share a strong similarity in their attributes (variables) while, at the same time, each cluster exhibits a maximum diversity from the others. Variations in the combination of the cluster variables and the number of clusters typically lead to slight changes in the composition of the clusters so that the main challenge here is the identification of the optimum variable combination and number of clusters. Subsequently, "typical" building examples are distinguished for each of the clusters. These typical buildings will be analysed at a later point in a comparative LCA, with the goal of identifying the most relevant building components in terms of the environmental impact of "typical" new Swiss apartment buildings. The following consecutive steps are applied, using the numerical computing software MATLAB:

– Data transformation
– Cross-correlation
– Choice of suitable cluster variables
– Cluster analysis
– Identification of "typical" buildings

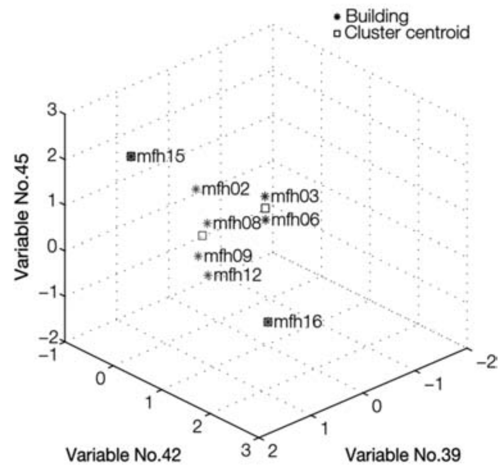

Figure 1. 3D cluster visualization.

Figure 1 shows the favourable clustering result as presented in this paper, classifying the exemplary building sample by three variables into four clusters.

In a final sensitivity analysis, the validity of the clustering result needs to be verified, with regard to the stated objective of allocating LCA results to the identified typical buildings.

This approach is generally applicable and offers detailed insight on the latent correlations and interrelations of a building's composition and its resulting environmental impact, while simultaneously enabling a simplification of the LCA procedure.

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Interdependency of LCCA and LCA in the assessment of buildings

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ABSTRACT

Sustainability aspects in the assessment of buildings using the life-cycle approach have become more and more common. This includes the assessment of the environmental, economic, the social and functional as well as the technical performance. Currently decisions for certain construction techniques/quality levels are mainly based on initial cost, and rarely on life-cycle cost. Due to the increasing focus on systems thinking, LCCA and LCA are becoming more and more important, as published in (Hunkeler D., Lichtenvort. K., Rebitzer G. 2008), (Vester F. 2008), (Cole R. J. 2011), (Passer A.; Kreiner, H. and Kainz F. 2009), (Passer A.; Kreiner, H. and Maydl P. 2009), (Passer, A.; Kreiner, H. and Maydl P. 2012), (Wallbaum H. and Hardziewski, R. 2011).

This paper gives an overview of the role of environmental and economic performance in current building certification systems. Furthermore it focuses on the interdependency of Life-Cycle Cost Analysis (LCCA) and Life-Cycle Assessment (LCA) in the assessment of buildings in the case of ÖGNI/DGNB building certification system (ÖGNI). The main part of the paper draws the attention to a new method to improve building performance behind a systematic approach.

Based on a case study in Graz all aspects that are currently covered by the ÖGNI/DGNB building certification system (e.g. energy performance, construction materials and building maintenance) are evaluated with respect to their LCCA and LCA performance, and this allows identification of those design options that are most influential. (Kreiner H. 2012).

In summary the results show, that the improvement of building performance by a linear approach is only suitable for criteria that do not interact with each other. If there is an interaction between criteria a systematic approach seems more appropriate to improve buildings performance. Therefore, those design options which have the lowest economic and environmental impact as well as the highest qualitative target achievement are recommended for realization in practice.

Only by ensuring that already design options are based on a holistic life-cycle performance assessment, the demanding challenge of a holistic improvement of building performance can be reached in future.

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Building sustainability assessment system corresponding to needs of users

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ABSTRACT

There is no universal assessment system fitting to all users for all possible purposes. The choice of suitable form and extent of an assessment system depends on many factors such as purpose and objectives of the system, intended type of users and involved stakeholders, expected form of presentation of results and available human and financial resources dedicated to assessment task. Research project of the 7th Framework Programme Sustainability and Performance assessment and Benchmarking of Buildings (SuPerBuildings) has approached the stakeholders with surveys and further analyzed their needs.

For real impact of an assessment system there is needed not only quality (represented by right selection of assessment indicators, setting benchmarks and weights among criteria), which ensures that the requirements of the system are challenging, but there is also quantity (market share of assessed buildings). The quantity depends on motivation to use sustainability assessments in general and in motivation to use particular assessment system.

Basically there are four main motivations to assess in general: research; non-financial benefit; financial benefit; and obligation. These benefitsarefurther described in the full paper.

Financial benefits are significant driving force to assess buildings when constructing a new building or purchasing property. They can be motivating in when financial incentives are granted, when refurbishing a building, when renting for longer period or in some cases of property tax discount. When assessment is being executed while none of these scenarios is place, there are probably another non-financial motivations.

The main motivation to use or do not use particular assessment system is suitability to the purpose of the target stakeholders' group. Purposes of assessment systems can be divided into the four main groups: design support; performance proof; performance rating; and reporting. Each of these groups requires different approach to assessment and different level of aggregation.

Decision to assess a building has economical consequences. The main cases when assessment can make a difference are when some improvements of building are being planned or financial transactions are being made (simplified). These cases are further described in the full paper.

Important factor that can discourage users of particular assessment system from the financial point of view is scale of the building compared to work amount (and budget) needed to provide an assessment. For projects smaller than apartment building there is a need for assessment schemes less expensive than the actually most common certifications on the market. The price can be lowered by simplification of the assessment system, by utilization of smarter calculation tools or by partial self-assessment carried out by the owner. Justification for the conclusions above is provided in the full paper.

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Environmental assessment of building refurbishments in SBToolCZ – criteria setup

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ABSTRACT

It is generally known that the biggest deal of energetic savings in built environment is possible through reducing energy demands in existing building stock. New buildings represent just a minor portion (the annual growth rate of new buildings added to the housing stock is 1,5%) of the total number of housing stock.

As there already exist various multicriterion tools for environmental assessment of new buildings, methodology for refurbishments are still in development in many countries. It has to consider various aspects and values of each particular building and seems to be much more delicate task to set up a general methodology for refurbishments than for newly built buildings.

At the CTU in Prague, Department of Building Structures a method is being developed for assessment of refurbishments that belongs to the SBToolCZ family. First version of criteria setup was finished in December 2011.

The main aspect that, in contrast to new buildings, affects renovations of existing buildings is their cultural-historical value. It has fundamental influence on potential renovation and so on the scale of improvement of overall building profile. It is obvious that it is necessary to approach differently to a building with cultural-historical value and building with minor cultural-historical significance.

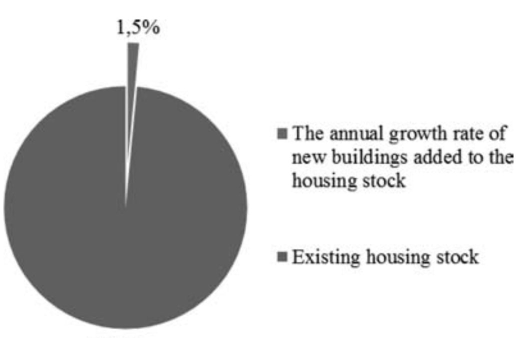

Figure 1. Proportion of existing and newly constructed buildings in EU (Ad-hoc Industrial Advisory Group 2009).

Assessment methods that nowadays exist for evaluation of existing buildings do not consider their cultural-historical value.

From point of view of building structure, it is obvious that cultural-historical value respectively its preservation during renovation or reconstruction significantly limit reaching the best environmental performance quality.

With respect to above mentioned facts, **two main goals** were set up for the proposal of multicriterion assessment method for existing buildings within SBToolCZ:

– Assessment of complex quality of new buildings and renovated/reconstructed buildings in a way that the resulting quality of both will be comparable
– Systematically take into account cultural-historical qualities of buildings

From the point of multicriterion assessment of existing buildings, reSBToolCZ is a completely new method and differs from all existing methodologies in the world.

The assessment method reSBToolCZ and its benchmark settings are nowadays being tested on several case studies and their results will be presented in October 2012.

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Utilizing GIS as a geospatial tool to inventory LEED certified buildings and Construction and Demolition (C&D) waste flows in the United States

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ABSTRACT

Markets have developed around green building products which may provide incentives for developers to track specific building materials and identify strategies for both environmental and economic improvement. We use geospatial analysis to inventory Leadership in Energy and Environmental Design (LEED) Buildings and subcomponents in two urban centers in Pennsylvania and track construction material stock.

Cidell and Beata used GIS to track the performance of a select LEED credits and represented performance per EPA regions throughout the US (2009). The authors demonstrated that spatial variation in the implementation of LEED standards does exist across the United States and that green building construction is uneven across the United States. Our work focuses on the Material and Resources category pertaining to Construction Waste Management credit in Philadelphia, PA, to expand previous studies that have not addressed this specific credit.

Using case study methods we investigate the LEED building stock for Philadelphia, PA and identify waste material flow patterns that result from architecture/designer choices to meet LEED certification, and integrate these with regional Construction and Demolition (C&D) patterns to understand recipient secondary C&D markets. We evaluate the existing LEED building stock in the US and separate the projects that diverted a portion of their C&D waste from landfills. Data mining techniques were applied to the USBGC completed projects database (USGBC 2011).

We use material flow analysis of region-specific C&D waste stocks and flows to understand material exchange and opportunities for recycling C&D waste. This analysis was supplemented with expert elicitation to gain an insight to the performance of a local C&D material reclamation facility that receives construction and demolition waste flows and channels them to the various secondary materials markets with multi-year annual volume of materials processed in the Philadelphia region. We find there are ample resources available in C&D waste in Philadelphia, with potential to be diverted from the solid waste landfills, thus improving material recycle efficiency. As indicated in the overview map, Figure 1, a cluster of LEED buildings and landfill infrastructure is around the two urban centers studied – Philadelphia and Pittsburgh.

The map shows the spatial proximity of LEED sites to city centers in 25 (40 km) mile steps. The map generated in this study identifies the close proximity of projects to C&D outlets. This indicates high rates of buildings that obtain LEED certification will generally pursue the CWM credit.

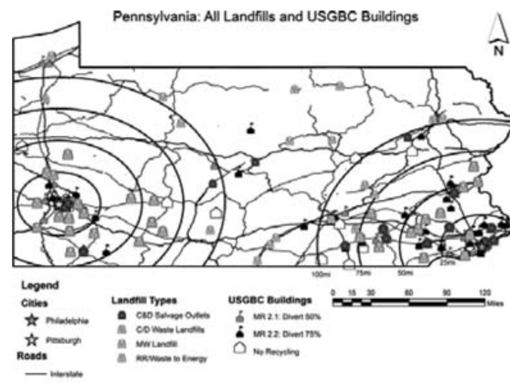

Figure 1. Pennsylvania with USGBC certified buildings and Waste Outlets indicated and major cities in GIS.

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Tools and processes for life-cycle engineering: Experience from the European project OPEN HOUSE

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ABSTRACT

There are currently many methodologies used for the assessment of building sustainability. Almost every European country has developed its own tool based on national building standards, regulations and benchmarks. Thus, buildings can be certified in France with the methodology developed by the Association HQE (High Environmental Quality), in Germany with the DGNB Certificate (German Certificate for Sustainable Buildings) or in the United Kingdom with BREEAM (Building Research Establishment Environmental Assessment Method). Moreover, the growing market of Sustainable Buildings' labels is consequently fragmented and allows competitive poles like the American certification system LEED (Leadership in Energy and Environmental Design) to gain popularity in Europe.

The European project OPEN HOUSE was established under the framework of a FP7 R&D program by a European consortium of 19 stakeholders, coming from research institutions, the building industry and the political sector (from February 2010 to July 2013). Its objective is to merge existing methodologies towards a common view, widely adopted because collectively and transparently developed, until it becomes the mainstream and reach the label level.

The development of the set of indicators is based on existing assessment methods as well as international (ISO TC59/SC17) and European (CEN/TC 350) standards promoting the concept of life-cycle thinking for environmental, economic and social aspects.

Implementing a new assessment methodology for buildings' sustainability in Europe requires the development of new concepts regarding assessment process and documentation. The analysis of current working documents, data requirements and evaluation process in different methodologies allows the development of a new procedure promoting simplicity and user-friendliness as well as accuracy and comprehensiveness.

As a result, the following assessment process has been developed, supported by an online platform.

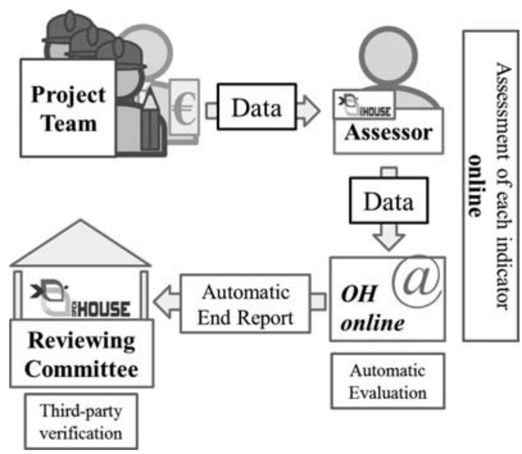

Figure 1. OPEN HOUSE assessment process.

The OPEN HOUSE assessment methodology and its related services are tested in 68 case studies all around Europe, from December 2011 to July 2012. This will allow a refinement of the technical specifications as well as an improvement of the assessment process thanks to the feedbacks of numerous building sustainability experts.

The OPEN HOUSE project will give birth to an open European online platform for the assessment of sustainability of buildings, providing free tools and guidelines as well as an open discussion platform. Therefore, it will set the basis for a better communication and comparison of building performance in European countries, paving the way for more sustainable construction practices.

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Longlife – energy efficiency and sustainability

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ABSTRACT

Longlife develops practices, innovative technologies, unified procedures and guidelines for and subsequently the design of a pilot project of a sustainable, energy efficient and resource saving residential building in the Baltic Sea Region. Longlife guidelines and unified procedures for energy efficiency, sustainability, resource saving buildings and low life-cycle costs will lead to a reduction of fossil energy consumption and CO_2 emission during a building's life-cycle. Longlife represents the focus of planning aspects of environmental, social and economic sustainability. These factors are in terms of environmental issues of great importance. To build sustainable requires greater planning and investment costs, which amortize in the life-cycle of the building, but it is difficult to motivate an investor to invest in these costs. The European Commission has also recognized the, and in its energy strategy to 2020, which was adopted in November 2010, formulated, that they are looking for proposals by mid-2011, to Solve in the rental market the investor-user dilemma. Longlife aims to optimize methods and construction, adapts and implements new technologies for buildings and harmonizes building procedures between the countries. These will lead to a reduction of energy consumption during a building's life-cycle. The main objectives of the project are to define benchmarks and to complete planning, administrative and tendering documents for a new energy efficient residential building. The project collects data and practices in the participating countries and evaluated the results in order to harmonize procedures and technologies for energy efficient construction. The outputs are the Longlife Report 1 (ISBN 978-3-7983-2213-4): Analysis and comparison (Rückert 2010), Longlife Report 2 (ISBN 978-3-7983- 2247-9).: Development of standards, criteria and specifications (Rückert 2010) and Longlife Report 3.1–3.3: Sustainable, energy efficient and resource saving residential design, Longlife Prototype Catalogue, Pilot Projects (Rückert 2011). An additional output is the Longlife Glossary. This is a collection of terms and definitions concerning energy, ecology and building aspects with respective translations in 18 languages.

The Longlife benchmarks and the prototype catalogue of elements provide data on energy consumption, CO_2 emissions and life-cycle costs for building elements e.g. walls, roofs and technologies which use renewable energies and reduce the CO_2 emissions, the primary energy consumption in the life-cycle of the building and reduce operational costs. In consequence, the technologies and engineering equipment should be evaluated and monitored to examine the minimization of the operational costs. Longlife develops the design of pilot projects in all participating countries under the use of the Longlife benchmarks and the Prototype Catalogue. The performance of the pilot projects will be evaluated and audited in reference to the economical and ecological benchmarks. These benchmarks are evaluated and measured in the "Longlife Performance Pass" through three main indicators: operational costs, energy consumption and CO_2 emissions. All criteria are measurable. There are no subjective criteria. The LPP enables a comparison with other buildings and will be a basis for decision of owners and investors.

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Ecological life-cycle assessment of structures made of UHPC – systematic and practical relevance

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ABSTRACT

Ultra High-Performance Concrete (UHPC) is a very densely structured and corrosion-resistant concrete with a compressive strength of between approximately 150 and 200 MPa. It enables the construction of material-saving, filigree and yet extremely strong and very durable structures. One example of this is the Gärtnerplatz bridge in Kassel. The approximately 133 m long hybrid construction was built in 2007. It consists of a three-dimensional truss made of steel pipes. The longitudinal girders and the slabs of the bridge deck consist of very slim prefabricated prestressed fibre reinforced UHPC. For the first time ever in the world, the deck slabs were glued to the girders by means of an epoxy resin adhesive (Fehling et al., 2008).

In a comprehensive study, the above-mentioned construction was compared with regard to its environmental effects to an otherwise identical construction in which the three-dimensional framework also consists of prestressed UHPC struts. In addition a conventional bridge made of prestressed ordinary concrete – which was originally planned instead of an UHPC construction – was evaluated. The environmental effects related to the manufacture, to the foreseeable maintenance and repair as well as to demolishing the construction at the end of its lifetime and to recycle the building rubble were covered (Stengel & Schießl 2008). For this holistic examination of the life-cycle, a service life of 80 years was assumed.

The contribution of the emissions to acidification (Acidification Potential AP; reference variable is the SO_2 equivalent), the greenhouse potential (GWP; CO_2 equivalent), the over-fertilisation potential (NP), the potential for the formation of ground-level ozone (summer smog POCP, ethene, C_2H_4 equivalent) and the Ozone Decomposition Potential in the stratosphere (ODP) were calculated (Jerebic 2005).

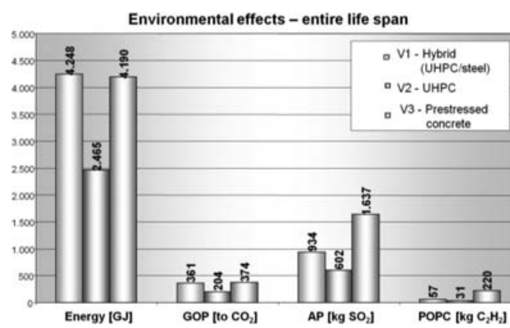

Figure 1. Environmental effects of three examined designs for the Gärtnerplatz bridge over the entire life span.

As can be seen from Figure 1, the pure UHPC variant evaluated exhibits by far the lowest impact on the environment. The total energy expenditure is about 41% smaller than for a standard concrete bridge, the contribution to the greenhouse effect about 46%. The UHPC construction contributes around 63% less to the acidification and even around 86% less to the formation of ground-level ozone.

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Life-cycle assessment of buildings for sustainable development

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ABSTRACT

During the life of the building leads to numerous changes and it is difficult to estimate its entire life-cycle. In addition, many environmental impacts to occur during its use. A certain simplification of life-cycle assessment is the certification of buildings. Unlike LCA methodology allows SBTool assessment of the impact of construction on the environment, including social and economic aspects. SBTool methodology evaluates some impact categories are the same as most of the characterization models for methodology LCA. There is no global or continental uniform method for the certification of buildings. In June 2010 at the international conference CESB 10 was officially introduced methodology certification of buildings for the Czech Republic.

SBToolCZ certification is a tool at the national level. It expresses the quality of the building in accordance with the principles of sustainable construction. The methodology is based on the international SBTool scheme, which is developed by the International Initiative for a Sustainable Built Environment (iiSBE).

The methodology is based on multi-criteria approach, which enters in the evaluation set of criteria that take into account the principles of sustainable buildings. Range of criteria vary by type of building (the methodology is currently developed for residential and administrative buildings). Buildings are evaluated in one phase of the life-cycle, namely at the design stage. The methodology shows for possible improvements in the monitored parameters of the building. Evaluation of the certificate of buildings is given only by one number or graphic symbol.

The resulting value of 10 points corresponds to the best available technologies, 5 points corresponds to the high quality construction, and 0 points reflects the usual state or to meet current legislative requirements.

Content of this work is evaluation two types of residential buildings according to the methodology SBToolCZ. The first evaluation concerns a circular single family detached house with a wooden supporting structure and envelope created from straw bales. The subject of the second assessment is a set of four residential buildings. Evaluated residential buildings are traditional brick construction.

Table 1 shows the points selected environmental criteria and utility areas evaluated buildings. A complex of four residential buildings is rated as a whole. For some criteria can not be evaluated each building individually.

The resulting value for the family house is 5.5 and for the group of four residential buildings is 3.5. Methodology for assessment of residential buildings is still relatively time-consuming. Currently is under preparation an Internet database of materials and structures for environmental assessment of buildings for SBToolCZ.

Table 1. Weighted points obtained in the selected criteria.

Criteria	Block of flats	Detached house
Usable area of building (m^2)	1050.05	118.8
E.01 Global Warming Potential (GWP, $CO_{2,ekv.}$)	0.29	0.91
E.02 Acidification Potential (AP, $SO_{2,ekv.}$)	0.35	0.00
E.03 Eutrophication Potential (EP, NO_x)	0.11	0.14
E.04 Ozone Layer Depletion Potential (ODP, R-$11_{ekv.}$)	0.03	0.00
E.09 Primary energy consumption from nonrenewable sources ($MJ/m^2.a$)	0.75	1.28

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Towards sustainable dams and embankments

Organizers: M. Wieland & W. Wu

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Reliability analysis for rainfall stability of municipal solid waste landfills on slope

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ABSTRACT

In this research, a method to assess the reliability for the stability of Municipal Solid Waste (MSW) landfills on slope due to rainfall infiltration is proposed. Parameter studies are first done to explore the influence of factors on the stability of MSW. These factors include rainfall intensity, duration, pattern, and the engineering properties of MSW. Then 100 different combinations of parameters are generated and associated stability analyses of MSW on slope are performed assumed that each parameter is uniform distributed around its reason ranges. Following, the performance of the stability of MSW is interpreted by the Artificial Neural Network (ANN) trained and verified based on the above-mentioned 100 analysis results. The reliability for the stability of MSW landfills on slope are then evaluated and explored for different rainfall parameters by the ANN model with First-Order Reliability Method (FORM) and Monte-Carlo Simulation (MCS). It is found out that the evaluation model of ANN-based FORM or ANN-based MCS is superior to traditional reliability method in view of many aspects, such as system modeling, computational efficiency, and analysis precision. Based on these methods, the Performance-Based Design (PBD) of MSW landfills on slope can be implemented easily.

According to the analysis results of a hypothetic site subject to rainfall infiltration as shown in Figures 1–3, it can be concluded that all the rainfall characteristics, including intensity, duration, and pattern, have obvious influence on the reliability for stability of MSW landfills on slope. Thus, the variation of rainfall condition should be investigated and considered in the analysis. By the quantitative reliability method proposed in this study, it will be beneficial to MSW landfills design and provide a guideline to achieve the target reliability considering rainfall scenarios.

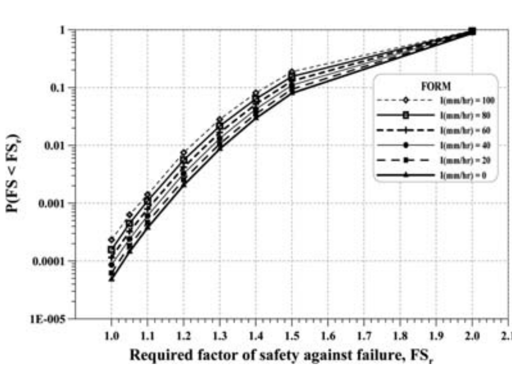

Figure 1. The relationship between required factor of safety and failure probability for different rainfall intensity (T = 36 hr).

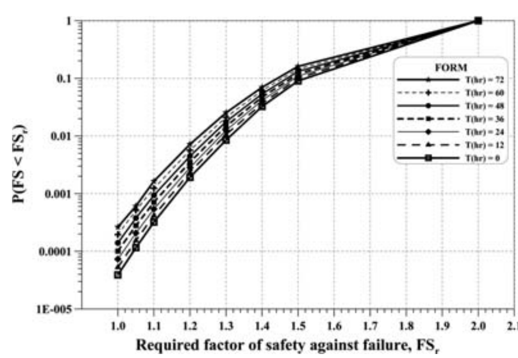

Figure 2. The relationship between required factor of safety and failure probability for different rainfall duration (I = 50 mm/hr).

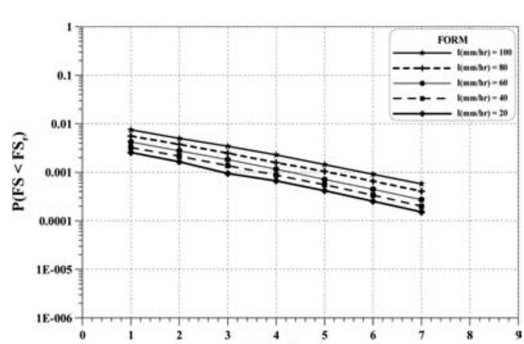

Figure 3. The relationship between rainfall pattern and failure probability for different rainfall intensity with $FS_r = 1.2$.

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Life-cycle assessment of Tuttle Creek Dam seismic retrofit

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ABSTRACT

We consider life-cycle performance of cement bentonite slurry walls installed to increase the seismic stability of Tuttle Creek Dam in Manhattan, Kansas, USA. While there is a desire to move towards sustainability in dam construction and operation, an equally important consideration is dam safety, particularly with respect to seismic stability. Owing to updated earthquake data and new analysis techniques, many existing dams have been found seismically deficient and require retrofitting to improve seismic stability. Ground improvement techniques are frequently employed for in-situ strengthening of potentially liquefiable foundations soils. Studies of seismic retrofit techniques of embankment dams within North America have been completed by Hynes (1993) and Carter et al. (2003). Selection of the appropriate retrofit technique for a given project involves balancing numerous costs and benefits as well as site specific geologic constraints. However, decision makers often neglect the environmental impacts over the project life-cycle. In an ideal world, a retrofit alternative would both meet the multiple criteria necessary to increase the seismic stability to acceptable levels and have a minimal environmental footprint.

We evaluate the environmental life-cycle performance of the seismic retrofit used at Tuttle Creek Dam using Life-Cycle Assessment (LCA). LCA has become a widely used tool for quantifying environmental sustainability aspects of engineering infrastructure and it is applied in this study to the ground improvement technique utilized for seismic rehabilitation of the Tuttle Creek Dam. The environmental performance was assessed using global warming potential (GWP100), cumulative energy demand, and nitrogen oxides (NOx). After determining the life-cycle materials and processes for the grout walls used for seismic retrofit, the life-cycle impacts were calculated (Table 1).

Table 1. Results of Life-Cycle Assessment.

Material/Process	GWP100 (kg C02 eq)	Cumulative energy demand (MJ eq)	nitrogen oxides (kg)
Portland cement	6.75E+03	2.59E+04	17.3
Blast furnace slag	2.18E+03	1.39E+04	3.39
Bentonite	257	6.49E+03	0.547
Water	3.29	6.39E+01	0.00641
Excavation	6.65E+04	1.01E+06	760
Grout mixing	2.03E+02	1.60E+04	0.293
Transport of excavated materials	6.43E+02	1.05E+04	2.01
Total	7.65E+04	1.08E+06	7.84E+02

Portland cement has the largest impact in GWP, cumulative energy demand, and NOx. It comprises roughly 50% of the grout mix, but produces about 70%, 40%, and 80% of the GWP, cumulative energy demand, and NOx, respectively. The blast furnace slag comprises approximately 50% of the grout mix, however, it contributes significantly less environmental impact across all three categories than Portland cement.

We used LCA to evaluate a broader set of sustainability metrics for decision-making for mitigating liquefaction risk for dam embankments. The work will subsequently be extended to include the other remedial alternatives considered in the design stage in order to compare the environmental performance of different techniques.

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Life-cycle performance of a concrete shear-wall structure

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ABSTRACT

Concrete building structures of a nuclear facility, other than a containment structure housing a nuclear reactor, are shear-wall structures, i.e., the lateral forces from earthquake or wind are transferred to the foundation by concrete walls deforming either as flexural-shear bending members or, when the ratio of height/length is small, primarily through truss action in shear. These structures are initially designed/constructed to perform their intended functions during the anticipated man-made and natural environmental (e.g., earthquake, flood) demands.The deterministic process generally used for the design of these structures has served the nuclear industry well, as evidenced by the satisfactory performance of nuclear facilities in last 60 years. However, the process fails to recognize uncertainties in material behavior and the likelihood of occurrence of natural and man-made events. To address the uncertainties in performance of these structures and the risk of a facility to public health and safety, the nuclear regulatory agency has taken a lead in this area, and has changed the regulatory requirements to make them risk-informed performance-based (USNRC 2007, ASCE 2005).

To address the uncertainties in the performance of structures, stochastic modeling is generally performed (Lu et al. 2008) using either a Monte Carlo simulation technique or approximate methods, based on assumed probability distributions (e.g. first-order or higher-order reliability methods, moment methods). However, sensitivity of the performance of a structure, e.g. probability of failure, to uncertainties in various parameters, such as probability of pitting corrosion and modeling, is not evaluated.

Recent study (Shah 2010) reported on how the performance of a typical concrete shear-wall in a nuclear facility would change with time during an earthquake event, for specified mean values pitting corrosion of reinforcing bars, along with increased concrete strength. The study described in this paper uses a stochastic method using a Monte Carlo simulation technique to examine the effects of uncertainties in pitting corrosion of reinforcing bars and modeling along with the assumed concrete strength gain with time, to determine performance of a concrete shear-wall during an earthquake event. Results of the stochastic analysis are then evaluated with those based on the mean values of these parameters reported by Shah (2010).

Based on the results of the evaluation described above, it is concluded that the performance of a typical concrete shear-wall during an earthquake event, based on the mean values of the pitting corrosion and modeling uncertainties, compares well with those based on considering these parameters as random variables in a stochastic method using the Monte Carlo simulation technique. Further research similar to the one reported in this paper needs to be performed for concrete shear walls where flexural failure mode may govern, and also to address uncertainties in material properties and various environmental demands.

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Effect of geogrid in rockfill dams during strong earthquakes

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ABSTRACT

Rockfill dams are usually preferred to concrete arch dams in seismically active regions due to their ductility to accommodate large permanent deformation during earthquakes and due to their ability to cope with movements at discontinuities and faults in the footprint of the dam. Such earthquakes can lead to failure of the soil structure as a consequence of large settlements.

This numerical study focuses on the serviceability aspect, in particular the effectiveness of properly placed geogrids to reduce settlements.

The performance of a large rockfill dam was investigated numerically under high seismic load using the finite difference code FLAC3D. The dam has a cohesive core, a high frictional rockfill, filters and transitional layers. The valley and the bedrock were also modeled.

Mohr Coulomb model was chosen to simulate the behavior of dam materials and the linear elastic model was used, instead, for the valley and the bedrock.

The simulated dam showed a large settlement. Also, the seismic load propagated to the dam crest with just a little reduction of the acceleration.

In order to reduce settlements, geogrids were placed at the top of the dam through the whole length of the dam with a variable spacing.

Geogrids in FLAC3D are plane stress elements resisting tensile stress providing a shear frictional interaction with the soil. Geogrids were simulated using a linear elastic constitutive law showing no failure.

After placing the geogrids, the crest settlement dropped to a more acceptable value. Moreover, the vertical acceleration in the top of the dam was reduced by a factor of 10, proving the effectiveness of geogrids.

Both minimum and maximum soil stresses dropped in the top of the dam, mainly because the soil stress was partially taken by the geogrids.

In this numerical study geogrids placed in the upper part of a rockfill dam proved to be an efficient mean to improve the performance of the structure against a high seismic loading, reducing the settlement and improving the dynamic stability of slopes.

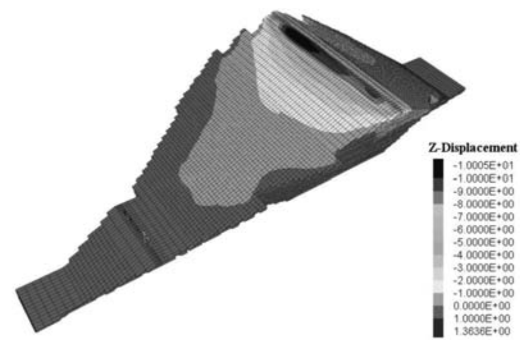

Figure 1. Settlement contour without geogrids.

Table 1. Geogrid parameters.

Parameter	Symbol	Value	Unit
Elastic modulus	E	20	GPa
Poisson's ratio	ν	0.2	
Thickness	t	0.1	m
Stiffness per unit area	k	2.3	MN/m^2
Cohesive strength	c	4	kPa
Friction angle	ϕ	30	°
Density	ρ	950	kg/m^3

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Safety aspects of sustainable storage dams

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ABSTRACT

The basic element in any structure or infrastructure project is safety. Therefore for sustainable storage dams the emphasis must be in the long-term safety of the dam and the safety-relevant elements such as spillway, bottom outlet and all related control equipment and power supply. Today, for dams with a large damage potential an integral safety requires concept is used, which includes the following element:

(i) Structural safety (main elements: geologic, hydraulic and seismic design criteria; design criteria and methods of analysis may have to be updated when new data are available or new guidelines, regulations or codes are introduced);
(ii) Dam safety monitoring (main elements: dam instrumentation, periodic safety assessments by dam experts, etc.);
(iii) Operational safety (main elements: reliable rule curves for reservoir operation under normal and extraordinary (hydrological) conditions, training of personnel, dam maintenance, sediment flushing, engineering back-up, etc. and the most important activity is regular maintenance of all structures and components); and
(iv) Emergency planning (main elements: emergency action plans, inundation maps, water alarm systems, evacuation plans, etc.).

The importance of these four safety elements is discussed.

The long-term safety includes, first, the analysis of all hazards affecting the project, which are hazards from the natural environment, hazards from the man-made environment and project-related hazards, which include the vulnerability of the dam to selected hazards, and secondly the compliance of the structure with current design criteria, design and safety guidelines, methods of analysis, etc. All these elements may change with time; therefore, a mechanism is needed, which ensures that a dam complies with new regulations. The periodic update of the dam safety taking into account new information and developments is one of the tasks of the detailed safety inspection of storage dams, which generally takes place every five years. By following such a procedure, it is, for example, also possible to cope with the effect of climatic change on storage dams as its impact on dam safety is mainly related to the flood hazard.

It is also important to know the safety reserves of a project. This is, for example, almost never the case for seismic actions. The earthquake hazard and seismic design and performance criteria are changing must faster than any actions from the natural environment including climatic change effects. The special features of the seismic safety of dams are discussed, as today the safety of large storage dams is often governed by its earthquake safety whereas in the design of most of the existing dams this hazard played a minor role.

Ageing of structures and maintenance are the main factors, which affect their condition and thus their safety. A dam which was safe at the time of completion does not automatically remain safe forever.

A sustainable dam is one which is safe and generates benefits to all stakeholders for a long period of time. The life-span of a dam and other infrastructure projects is basically as long as regular maintenance can be guaranteed. Thus the life-span of a well-designed and well-maintained concrete dam can be much more than 100 years and that of rockfill dams with impervious core can last even longer.

Finally, the paper also serves as an introduction to the subject of the mini-symposium ‘Towards Sustainable Dams and Embankments’, to be held at the conference.

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GENERAL SESSIONS

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Estimation of steel weight loss due to corrosion in RC members based on digital image processing of X-ray photogram

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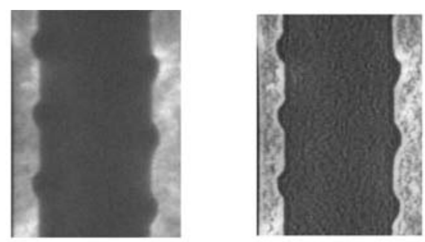
ABSTRACT

Since structural capacity of RC members depends strongly on localized condition of reinforcements, it is important to model the spatial variability of steel corrosion. However, steel corrosion in RC members can only be observed after severely damaging the concrete specimen. In order to understand the growth process of steel corrosion and how the spatial variability of steel corrosion increases with time a continuous monitoring is necessary. Recently, X-ray technology has been applied to concrete. Otsuka & Date (2000) developed an inspection technique based on X-rays photography using contrast media which can directly inspect internal cracks. Using this technique, they investigated the behavior of fracture process zone in concrete. Beck et al. (2010) examined the steel surface within the mortar specimens by X-ray tomography. They reported that X-ray tomography was a suitable tool to visualize the propagation of localized corrosion attack on reinforcement in mortar with a cover of approximately 35 mm. However, the total weight loss of corrosion steel, determined by weight measurement was about 40–60% higher than that determined by X-ray tomography. Therefore, the estimation of the accuracy of steel weight loss using X-ray needs to be improved.

In this study, X-ray photography and digital image processing are applied to observation of steel corrosion in RC members. Figure 1 shows examples of the images of sound rebar in a RC member before and after applying the edge detection to the X-ray photogram. Using X-ray photograms with different viewing angles taken by rotating the specimen, the volume of non-corroded rebar is calculated and steel weight loss is obtained. Figure 2 shows the comparison of steel weight loss due to corrosion in a RC members by weight measurement with that estimated by the method of edge detection. The mean and coefficient of variation of the ratio of steel weight loss by weight measurement to that estimated by the digital image processing are approximately 1% and 10%, respectively. Estimation accuracy by using the digital image processing is independent of experimental parameters such as water to cement ratio and concrete cover. Encouraging results for the applicability of X-ray photogram to the observation of corrosion process in RC members are reported.



(a) Before edge detection (b) After edge detection

Figure 1. Detection of edge between rebar and concrete.

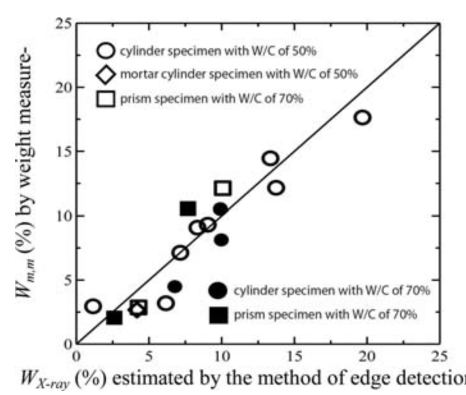


Figure 2. Comparison of steel weight loss by weight measurement ($W_{m,m}$) with that estimated by the digital image processing ($W_{X\text{-}ray}$).

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Numerical simulations of occupants evacuation within the context of life-cycle engineering in building construction

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ABSTRACT

The safety of occupants of office and public buildings is one of the major concerns of fire protection engineering. The intended use and purpose as well as the load-bearing capacity of the structural elements of a building may change frequently over time. These changes impose new and extra restrictions that should be taken into account in the design and assessment process of egress systems during the life-cycle of a building. In this paper a probabilistic framework for the evaluation of egress systems over the life-cycle of a building is presented. A performance-based approach of egress design is adopted within this framework. An essential part of this approach is the usage of numerical simulations of pedestrian evacuation in verifying the life-cycle-related modification of an egress system.

The proposed methodology is based on reliability analysis as a common quantifier since all the models and design rules for life safety engineering are usually based on a so-called Limit or Failure State (LSF), which can not be over- or underrun as a violation would lead to an unsafe egress situation with possible casualties. Additionally, nearly all parameters are subject to uncertainties which can be expressed by using stochastic models based on defined parameters of the various distribution functions. This has the advantage that the probability (p_f) of the violation a limit-state can be quantified and thus yields a quantitative parameter for comparison–regardless of complexity or scale of the underlying simulation methods.

In order to provide for a holistic framework for probabilistic life-cycle management, an adaptive methodology based on event tree system analysis is introduced. Looking at life safety analysis in a systematic approach, it can be decomposed into basic scenarios which each fail with a certain probability (Albrecht and Hosser 2011). The approach shown herein used event tree system analysis to encompass the entire system holistically. The life safety design is regarded as a system and will be decomposed into various components whose failure probabilities can be described by either predefined values, simple analytic equations, or complex numerical models (Klinzmann 2008).

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Assessment of structural damage due to ground settlements by using the DInSAR technique

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ABSTRACT

Ground movements have the potential to damage existing buildings. A powerful remote sensing technique that allows the monitoring of these settlements is the Differential Synthetic Aperture Radar (SAR) Interferometry (DInSAR); it is able to produce ground deformation maps and time series with centimeter to millimeter accuracy. In urban areas, this monitoring is very important for both large scale evaluations and detailed analyses of single buildings. At a large scale it allows the identification of critical areas subject to significant movements, where the structural damage is more likely to occur. At a smaller scale, it gives the time-history of the settlements under specific buildings.

These measurements can then be used for the evaluation of the structural conditions of the constructions.Given the ground deformation, different approaches exist for the assessment of the structural damage; they range from empirical estimates to detailed finite element calculations.

In the first part of the paper a brief explanation of the considered DInSAR technique, referred as Small BAseline Subset (SBAS) is given.

In the second part various procedures for evaluating the structural response to a specific pattern of differential settlements and identify the structural damages are briefly presented. Their potential and limitations are discussed.

In the present work, a semi-empirical model is applied to some buildings in a urban area in the southward part of Rome (Italy) (Figure 1). The model, originally proposed by Finno et al. (2005), considers each building as an equivalent laminated beam, where the layers represent the floors and the core material reproduces the infill walls. This model is a compromise between the purely empirical methods that ignore many of the properties of the building, and the detailed finite element analyses that are time and computationally very expensive.

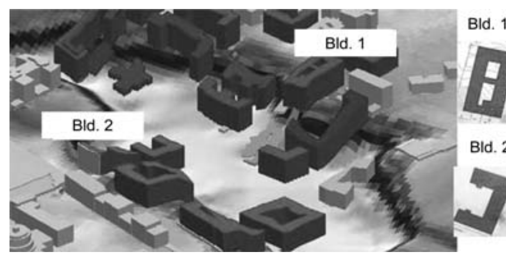

Figure 1. Three dimensional GIS imagine of the area (the scale of the settlements is emphasized).

The damage assessment procedure that integrates the DinSAR data and the laminated beam model consists of four steps:

- The DinSAR data are processed and the displacements under the buildings are obtained. In the considered case, the available dataset included registrations of the movements every 35 days from 1992 to 2010 (Bonano et al., 2012).
- The equivalent laminated beam models of the investigated buildings are created (considering material, geometrical characteristics, etc.).
- The displacements are applied to the models.
- The damage is classified according to its severity.

The results obtained with the chosen model have been compared with the observable damages and the results are in substantial accordance.

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Performance-based retrofit of a prestressed concrete road bridge in seismic area

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ABSTRACT

The bridge structures can be usually defined such as "essential" constructions due to the reduced level of hyperstaticity. Thus, the possible severe seismic damage of a bridge structural member can lead to collapse of either the whole structural system or large parts of it. Therefore the seismic design process of these structures requires extreme care in both the dimensioning of the structural details as well as in the use of seismic protection levels higher than other structural typologies. In Italy a significant number of existing bridges, currently in exercise on the national road network, were designed and realized without any form of earthquake resistant criteria. It is therefore to be expected to many of these bridge structures are particularly vulnerable to the effects of an earthquake or are able to withstand the seismic actions of lower intensity than the seismic design levels provided by the current seismic code provisions (EC8, Italian Code NTC2008).

In this context, the paper deals with the proposal of a procedure for the seismic retrofit of an existing prestressed concrete bridge built on the "Domitiana" state road in Campania region. Starting from the identification of the bridge structural geometry as well as the material mechanical properties based on the original project designs, the seismic vulnerability assessment of the existing bridge was carried out. With this aim a Nonlinear Static Procedure (NSP) based on the Capacity Spectrum Method (CSM) as well as the Inelastic Demand Response Spectra (IDRS) already used for other structural typologies (Fajfar 1999, Vidic et al. 1994), was applied. According to the Performance-Based Earthquake Engineering (PBEE) criteria, these procedure make it possible to explicitly correlate the different performance levels to varying intensities of seismic action.

Then an innovative seismic protection strategy based on the use of isolating system located between pier top and continuous deck was appliedby using Friction Pendulum System (FPS) devices. In particular a design process consisting of the appropriate application of capacity-design principles as well as Direct Displacement Based Design (DDBD) approach was applied (Priestley et al. 2007).

The seismic response of the bridge, modelled with an "exact" damping matrix, was subsequently carried out by means of a linear time-history analysis with the direct integration of motion equations using a set of recorded accelerograms whose spectra are consistent with the design spectrum proposed by the Italian Code.

Finally, the results obtained highlight the effectiveness of the seismic retrofit strategy, which avoids the use of additional energy dissipation devices and involves the significant reduction of internal forces in the piers, that are able to withstand the new seismic action effects remaining in the elastic range.

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Efficient design and construction of the APM major bridge project in Saudi Arabia

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ABSTRACT

In the present paper, the design and construction of the major 12 Km APM elevated bridge in PNU in Saudi Arabia, which is based on precast concrete technique chosen to efficiently speed up the project execution, are presented. The project is structurally divided into simply supported straight and bifurcation parts and continuous curved and straight parts.

Both straight and bifurcation parts are constructed using simply supported twin precast unsymmetrical pre-tensioned C beams connected together, after the pre-tensioning in the precast yard and the placing of the precast beams on bearings at top of pier, through stitching concrete to form the box girder section. The 15 cm topping slab and the end diaphragms are then cast in place and connected to the precast slabs by shear connectors.

For bifurcations, the deck width is variable and consequently, the twin precast beam spacing is variable as shown in Figure 1. The arrangement of the pre-tensioning strands in the precast C beams is chosen in such a way that, at transfer, the distortion (side sway) which may occur due to the unsymmetrical characteristic of the precast beam is minimized (Ayoub et al. 2011).

The curved parts of the APM elevated bridge project are constructed using the precast segmental concrete technique. Curved segments of 2.9 m width and consisting of the typical box section used in the project are connected in the yard through shear keys, glue and top and bottom post-tensioning to form the curved bridge single span. The assembled curved span is lifted and placed on pier bearings using gantry cranes. After the placing of the precast slabs and the hardening of the cast in situ topping slab and diaphragms, the continuity top and bottom post-tensioning cables are installed and stressed to ensure the continuity of the curved bridge part. In final phase, the curved bridge is monolithic at piers; while, it is supported on bearings at the expansion joint locations. The same procedure used to achieve the curved bridge continuity is adopted for the continuous span straight parts consisting of the precast unsymmetrical twin C beams.

It is concluded from the present paper that the precast concrete technique used extensively in the APM major bridge project in PNU had a big impact on the ease of construction and consequently on the time saving achieved in such big bridge project. Moreover, the above technique ensures the concrete quality of the project structural elements. Of course, the cast in situ technique as an alternative construction solution using props will dramatically increase the time frame of project with such big scale. Also, the aesthetic feature of the bridge is realized by using box section composed of two precast unsymmetrical sections for the straight and bifurcation parts part and segmental precast box sections for the curved parts. The tendency of the unsymmetrical precast beam to distort during transfer is minimized using a proper arrangement of the pre-stressing strands and as a result, only camber deformation is expected and recorded in site at load transfer.

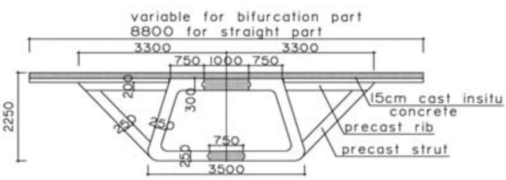

Figure 1. Box girder section used in the APM project in PNU.

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Probabilistic description of foundation capacity for design of electrical transmission lines

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ABSTRACT

The current demands for sustainability have posed an increasing pressure on costs of competing alternative renewable energy sources. Regarding these alternatives, Brazil stands in a privileged position due to its high hydropower potential. However, most of the hydroelectric power plants that are being built, or will be built in the future, are located far from the consuming centers. In this way, there is a strong emphasis on costs associated to the construction and maintenance of electrical Transmission Lines (TL's). Foundation costs account for a great part of the total construction costs related to TL's (roughly 50%). As such, there is an increasing pressure for the minimization of these costs. While the design of these structures has to be undertaken under a life-cycle approach, it is clear that due to the many uncertainties that affect the problem (physical, decision, measurement, modeling, and statistical uncertainties), it has to be dealt with in a probabilistic framework.

The probabilistic framework, in its turn, requires the probabilistic description of both loads and capacities. While the probabilistic description of loads in TL's has been fairly investigated, little has been done regarding the probabilistic description of foundation capacities, in special those subjected to tension. Due to the complexity of the stochastic process depicting soil variability (see Fig. 1), the practical treatment of the foundation capacity estimation has largely resorted to experimental results, usually limited in number and thus resulting in considerable statistical errors. Furthermore, current foundation design codes leave up to the designer's discretion the selection of the model for capacity estimation. In this way, both the incorporation of capacity prediction errors and risk consistency among different designs are not attained.

In this paper, the probabilistic description of foundation capacity of TL's subjected to tension is sought. To this end: (i) a review of current deterministic approaches for capacity estimation is made; (ii) a large database on soil properties and test results of foundation capacity of actual TL structures is assembled;

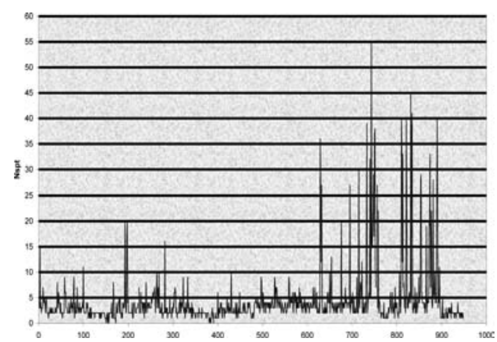

Figure 1. Soil profile at 3 m depth as a function of N_{SPT}.

(iii) model errors associated to capacity prediction are estimated; and (iv) the probabilistic description of foundation capacity of TL's is made.

The main goal of the research reported herein is to obtain the probabilistic description of the strength of foundations in TL's subjected to tension, $f_R(r)$. This probabilistic description, i.e. type of distribution and corresponding parameters, is a basic requirement for the development of either full- or semi-probabilistic design codes. This goal has been accomplished for foundations in sand soil subjected to tension.

The results obtained have shown that the strength estimates obtained via Grenoble method (Biarrez & Barraud 1968) are reasonably conservative and associated to a small coefficient of variation (COV = 0.06). The minimum, mean, and maximum values associated to the model error, ε_R, are 1.37, 1.46, and 1.66, respectively, which means that the analytical estimates are in the range 37 to 66% higher than the values obtained in full-scale tests.

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The SBAS-DInSAR technique: A tool for deformation monitoring in the urban damage assessment

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ABSTRACT

The structural damage is a critical problem of soil-structure interaction and it strongly depends on the uniformity and the rate at which ground settlements occur. Accordingly, monitoring of displacements affecting single structures is of key importance for damage assessment of urban areas.

Until now, detection and analysis of such deformation events have been difficult and expensive, thus limiting surveys to very restricted areas. On the contrary, remote sensing radar techniques allow non-invasive analyses over wide areas and on long time intervals by exploiting the availability of large archives of space-borne data, as those acquired over the last two decades by the Synthetic Aperture Radar (SAR) sensors on-board the ERS-1/2 and ENVISAT satellites of the European Space Agency (ESA).

Within this framework, Differential SAR Interferometry (DInSAR), based on the exploitation of the phase difference (i.e. interferogram) between pairs of SAR data acquired over the same area at different times, has emerged as a valuable microwave methodology to detect and map surface displacements with centimeter to millimeter accuracy.

More recently, advanced multi-pass DInSAR techniques have been developed, aimed at investigating not only single deformation events, but also the temporal evolution of the detected phenomena. As results, spatially dense deformation maps and associated time-series showing the displacements occurring in the analyzed area during the observed period can be produced.

Among these advanced DInSAR approaches, we focus on the Small BAseline Subset (SBAS) algorithm (Berardino et al. 2002) that relies on the combination of DInSAR data pairs, characterized by a small separation between the acquisition orbits (baseline), in order to produce mean deformation velocity maps and corresponding time series, maximizing the coherent pixel density of the investigated area. Moreover, the SBAS technique allows detecting and monitoring surface deformation at two spatial resolution scales (Lanari et al. 2004), namely regional and local scales, and acquired by different sensors (Bonano et al. 2012).

In this work, we investigate the effectiveness of the full resolution SBAS approach to analyze local deformation phenomena occurring in urban areas, In this framework, we focus on deformation phenomena affecting the city of Roma (Italy) by exploiting full resolution ERS-1/2 and ENVISAT SAR datasets covering the 1992-2010 time interval in order to generate long-term deformation time-series (Bonano et al. 2012).

The presented results demonstrate the capability of the proposed methodology to investigate the spatial and temporal patterns of deformation affecting single buildings and human-made structures, pointing out as such approach may play a key role within infrastructure diagnostics and urban damage assessment, as well as within conservation strategies of the historical heritage, monuments and artistic artifacts.

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Rehabilitation of concrete bridges using Ultra-High Performance Fibre Reinforced Concrete (UHPFRC)

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ABSTRACT

Rehabilitation of deteriorated concrete bridges is a heavy burden from the socio-economic viewpoint since it also leads to significant user costs due to traffic disruptions. As a consequence, novel concepts for the rehabilitation of concrete structures must be developed that really achieve the required durability over the design service life. Ultra-High Performance Fibre Reinforced Concretes (UHPFRC) provide a unique combination of (1) extremely low permeability which largely prevents the ingress of detrimental substances such as water and chlorides and (2) very high mechanical strength.

The basic conceptual idea is to use UHPFRC only in those zones of the structure where the outstanding UHPFRC properties in terms of durability and strength are fully exploited; i.e. UHPFRC is used to "harden" the zones where the structure is exposed to severe environmental conditions (f.ex., deicing salts, marine environment) and high mechanical loading (f.ex. impact, concentrated loads, fatigue).

Since 2004, UHPFRC is applied in Switzerland on existing reinforced concrete bridge deck slabs as watertight overlays as well as reinforcement layer in R-UHPFRC providing both protection and load bearing functions for bridge elements (Fig. 1).

These applications revealed that UHPFRC is suitable to establish the required durability and mechanical performance of rehabilitated concrete bridges.

Figure 2 shows the evolution of the performance of a rehabilitated bridge over its service life, using two different rehabilitation technologies.

For concrete bridges, Strategy A may be achieved using the novel advanced UHPFRC rehabilitation technology. Conventional Strategy B with multiple interventions is still considered by many researchers as being given or inherent to structures. This approach is however outdated.

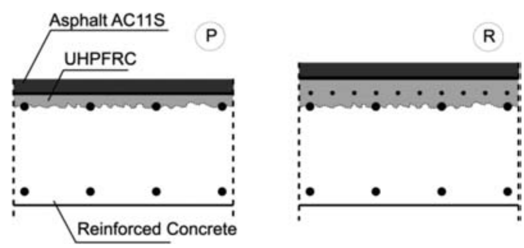

Figure 1. Applications of UHPFRC for protection (P) or as R-UHPFRC for reinforcement (R).

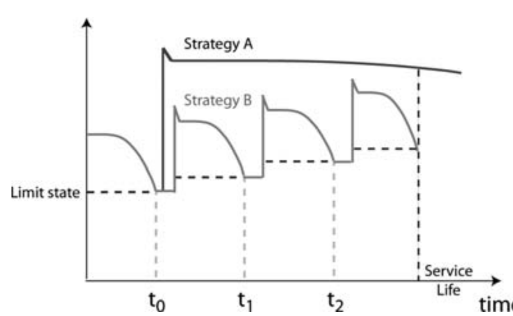

Figure 2. Evolution with time of the demand and supply for two different maintenance strategies.

Obviously, Strategy A is highly desirable, and it ought to be the objective of any rehabilitation intervention. A fundamental change in paradigm, i.e., a change from Strategy B to Strategy A, is thus needed to seriously tackle the question of infrastructure management. Research efforts need to be made in the domain of material and structural engineering rather than in the domain of life-cycle costing and management.

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Fatigue safety examination of a 150-year old riveted railway bridge

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ABSTRACT

This paper deals with the examination of the oldest existing wrought-iron railway bridge still in service in continental Europe. This bridge is part of a railway line belonging to the Zurich suburban railway system. The main objective of the present examination was to verify the structural and fatigue safety for a long term future utilisation of the bridge and to show the consequences in terms of rehabilitation interventions while preserving the cultural values of the bridge.

Both owners, the Deutsche Bahn AG and the Swiss Federal Railroads SBB, want to exploit the railway line in a long term perspective. For the bridge, the question arises regarding the necessary interventions to reach this objective in view of an increase in Line class to C3 as a minimum requirement, i.e., maximum allowable axle and line loads of 200 kN and 72 kN/m' respectively, for a maximum train speed of 60 km/h and an additional service life of at least 80 years. Also, the study should reveal the necessary interventions for the scenario of higher line classes (e.g., D4 or E5).

This paper has the objective to explain the methodology applied in the examination of this more than 150 years old riveted railway bridge. Emphasis is given on the verification of the structural and fatigue safety for a long term future use of the bridge. The concept for rehabilitation interventions is outlined and economic as well as life-cycle aspects are presented.

The investigated bridge (Fig. 1) crosses the Rhine river in northern Switzerland to carry a one lane railway line between Koblenz (Switzerland) and Waldshut (Germany). It was built in 1859 and comprises riveted wrought iron members. The straight lattice-truss structure is one of the last examples of a construction type that was typical for the railroad construction boom in Europe during the third quarter of the 19th Century.

The examination of the performance of the railway bridge over the Rhine river between Koblenz

Figure 1. Railway bridge over the Rhine between Koblenz (Switzerland) and Waldshut (Germany).

(Switzerland) and Waldshut (Germany), revealed that no extraordinary interventions need to be performed to keep the bridge in service for the next 80 years.

The study showed that preservation and further use of the more than 150 year old bridge is much more economical both in terms of costs for maintenance interventions, compared to bridge replacement (an option that was often and still is commonly chosen in similar situations).

This important monument of structural engineering will remain in service for modern railway traffic despite its relatively high age. There are no "old" bridges; there are only bridges that provide adequate performance (or not). This approach is clearly in agreement with the principles of sustainable development. Extending the service life finally means giving value to bridges as well as appreciating the art of structural engineering and the identity of structural engineers.

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Rehabilitation of two bridges in construction in the motorway M-410 in Madrid, Spain

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ABSTRACT

The new highway M-410 in Madrid has been constructed during the year 2007. This motorway near to Parla city crosses the road A-42 from Madrid to Toledo. To solve this crossing it has been needed to construct three bridges, the central with two spans over the existing motorway and the other two with one span at each side of the previous one. All the bridges has deep foundations with piles of 1.00 m of diameter and separations of 1.25 m each one.

Just after the construction, it has been made sonic transparency essays, which have showed some anomalies on the tip of the piles of the two lateral bridges called PS-18 and PS-19, which implies the necessity of a more deep investigation.

It has been decided to make two boreholes one at each bridge in order to obtain samples of the soil near to the tip of the piles. This investigation has showed that the earth at the level of the tip of the piles, was sandy and with a great water flow, so the conclusion reached is, that it was impossible to retire the earth under the concrete slab on the top, because the piles are not correctly founded

This paper shows the project and the construction of the rehabilitation of bridge, consisting of two sheets of micropiles, deepest at each abutment of the two bridges, to resist all the loads and after this, it would possible to remove the earth under the concrete slab above.

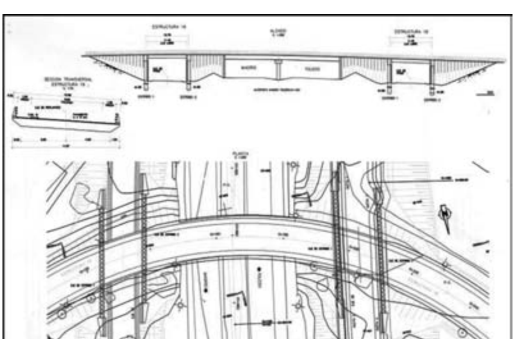

Figure 1. Bridges over motorway Madrid-Toledo M-410.

Figure 2. Central and lateral bridges prior the construction.

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Testing of pre-stressed masonry corner for tri-axial stress-strain analysis

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ABSTRACT

During the revitalization of masonry structures the post tensioning of masonry structure is used frequently for improving the structural mechanical properties. Masonry is heterogeneous material with varied material properties, in older buildings affected with different type of damage and that is why in engineering practice the intensity of pre-stressing is often designed according to engineering judgment of the designer.

From experience of practical design it can be concluded that during the post-tensioning the failure occurs most often in the zone of pre-stressing force anchoring (Bazant & Klusacek, 2004). The authors' aim is to contribute to a better understanding of post-tensioned masonry, and therefore it was decided to perform the testing of tri-axial stress/strain conditions in post-tensioned masonry corner.

At VSB – Technical University of Ostrava unique equipment was designed for experimental testing of the tri-axial state of stress and strain of a pre-stressed masonry corner, Figure 1. Plan dimensions of the tested corner are 900 × 900 mm, the thickness of the wall is 450 mm and the height is 900 mm.

Experiments started in 2011 with masonry corners made of clay bricks and general purpose mortar. Bricks were obtained from a demolished building and lime cement mortar was prepared from a designed dry mixture.

The masonry corner was exposed to an arbitrary vertical load, and then the pre-stressing was installed in the lower bar (direction I) and released. In the next step pre-stressing was installed in the upper bar (direction II) and released. In the end the pre-stressing was installed in both directions together. Short-term deformations are measured in a network of measuring points.

Pre-stressed masonry corner was analysed also using FEM analysis in ANsys computer program as micromodel. Bricks and mortar joints are assumed as two different materials with their real dimensions and real geometrical arrangement in the structure, (Cajka & Kalocova, 2007), (Materna & Brožovský, 2007).

Figure 1. Testing equipment for pre-stressed masonry corner.

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Health monitoring of civil engineering structures – what we can learn from experience

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ABSTRACT

In a first, general part, the meaning of "health monitoring" when applied to civil engineering structures is discussed. How should we define "health"? "monitoring"?

The discussion is then concentrated on "health monitoring of civil engineering structures using dynamic methods".

Dynamic methods means: Possible parameters to be monitored are:

- natural frequency,
- mode shape,
- damping.

Damping can be neglected as a reliable "health" parameter. Damping can not be determined with the necessary reliability.

Natural frequency and mode shape are dependent on mass and stiffness. Since the the mass of a structure is usually very constant, this is not relevant for health monitoring purposes.

Therefore, message No.1 of this paper is: Stiffness is the only "health" parameter being subject of health monitoring using dynamic methods.

Trying to detect changes in "health" through monitoring stiffness assumes that no other effects are also influencing "health". This is usually not the case.

Message No. 2 of this paper is: Successful application of dynamic methods to monitor a civil engineering structure is possible if the influence of other parameters is identified and eliminated only.

Practically speaking: In all cases where temperature influences the structure's stiffness, at least a one-year pre-monitoring period is needed where no changes in health are to be expected.

These messages are discussed in the paper main part through presenting some projects having been performed in the last years.

Tests performed on the Ganterbridge in Switzerland illustrate the uncertainty in experimentally determining e.g. a structure's natural frequency due to the non-linear behavior of the structure.

Tests performed on the Westend Bridge, Berlin, and the Z24 bridge in Switzerland illustrate the possible influence of temperature on a bridge's natural frequencies.

Z24 tests and tests on a concrete wall show that it is possible to identify a "health" problem (stiffness reduction) using dynamic methods under certain conditions:

- the stiffness reduction must be important enough to significantly influence the structure's natural frequencies,
- temperature effects are eliminated.

Z24 tests also show that it is even possible to locate the "health" problem if:

- the measurement point density is high enough to allow determination of a mode shapes' curvature.

Placing one 3D-accelerometer on a bridge is definitely not the approach leading to success in this case.

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The research on the time-dependence of recurrence times of great earthquakes based on the historical records in North area of China

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ABSTRACT

As it is well known, the prediction of the recurrence times of great earthquakes is important for the life-cycle reliability analysis and risk assessment of structural engineering. The Poisson process with exponential interval time distribution in seismic risk analysis is common used because its constant risk ratio is convenient to utilize. Although this hypothesis is more or less confirmed by the small and medium size of earthquakes in a long interval time and large region of interesting, it is failed by the observations of the recurrence of large earthquakes. In fact, many deep inspections on the historical records of earthquakes have revealed that the magnitude and recurrence times of earthquakes, especial for the great earthquakes, are temporal and spatial variant. In the efforts to describe the time-dependence of great earthquakes, several statistical models, such as renewal process model, Markov process model, semi-Markov process model, cluster model and the recent self-organized model are proposed. In the present paper, ignoring the spatial variation, we analyze the dependence of the recurrence times of great earthquakes based on the long term historical records in North area, China, from 231AD to 1996 AD. It turns out that, ingeneral, the occurrence of earthquakes in this area is quasi-periodic and could be subdivided into two stages, which are inertial stage and active stage. The recurrence of great earthquakes in the active stage retains a complex of temporal and size variance, while the small and medium size event has a robust constant hazard ratio. The recurrence time of great earthquake depends on the occurrence time and magnitude of the previous event. That means, in the active stage of seismic cycle, the longer since the last earthquake,and the larger the magnitude of the event, the bigger probability of next earthquake would be. No matter what the length of the waiting time is, all the probabilities of great earthquakes are getting close with time in the active stage, which implies the clusters of earthquakes.

However, the data statistical analysis in the inertial stage shows that the risk of earthquake is time-independent in this stage. Therefore, the Poisson process with exponential interval time distribution is adaptable.

We also suggest the conditional seismic risk assessment for structural engineering which has two implications. One is that the stage of seismicity should be considered because the risk ratio for different earthquake varies violently in inertial stage and active stage of seismic cycle. The other is that the risk assessment should be made based on the magnitude and occurrence time of previous earthquake.

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Development and verification of a typhoon wind hazard model for Taipei

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ABSTRACT

The first step in evaluating the risk due to wind is to identify and quantify the possible strong wind events in the considered timeframe in a probabilistic manner. It is a challenging task, especially for regions with strong winds caused by rare events such as typhoons. Direct statistical analyses of historical wind speeds are not accurate enough; therefore, a new typhoon wind hazard model is developed in this study and is applied to assess the wind hazard for Taipei, Taiwan. The proposed model simulates artificial typhoons and then predicts Taipei's wind speeds conditional on the simulated typhoon parameters. First, sub-models for simulating the occurrence and transition of typhoons are established by analyzing the historical typhoons in the Northwest Pacific Ocean; the considered typhoon parameters for a typhoon at a time consist of the central location, the moving speeds, the central pressure, the near-center maximum wind speed, the radius of whole-gale winds as well as the radius of near-gale winds. Next, Gaussian process based regression sub-models, correlating Taipei's wind speeds with typhoon parameters, are also developed based on historical data and are adopted to predict Taipei's wind speeds corresponding to the simulated typhoon parameters. The prior model for Taipei's wind speed is assumed to be Gaussian with a covariance function containing uncertain hyper-parameters. The posterior distributions for the hyper-parameters are obtained by Bayesian analyses and the posterior distribution for Taipei's wind speed is subsequently derived and employed for predictions. Numerical studies show that the wind speed of Taipei is, in general, most sensitive to the relative position between the typhoon center and Taipei as well as the typhoon central pressure regardless of the invading path. In addition, the comparison between the predictions made by the proposed models and the observed wind speeds reflects that the predicted wind speeds exhibit the similar trend revealed by the observed wind speeds and the 95% confidence intervals for predictions generally envelop the respective observations. The obtained results provide the basis for strong wind risk analyses of structures at Taipei.

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Seismic reliability and LCCs of a RC building considering earthquake events within a specified service period

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ABSTRACT

Calculating the cumulative damage due to various earthquake events generally requires knowledge of the number of events of magnitudes that are likely to occur at each fault in a given time-window. Therefore, several analytical models have been developed for earthquake occurrences. To simulate events of large magnitudes, some recent models have assumed a lognormal or Weibull distribution of return periods, whose hazard functions are time-dependent. The number of occurrences in a given period depends on the knowledge of the time of occurrence of the last event. These models accurately represent the occurrences of large earthquakes in many regions. However, in addition to providing statistical evidence to support seismic gap and a typical earthquake hypothesis, engineers often have difficulty in using simulation models that correspond to lognormal or Weibull distributions of return periods in order to estimate the number of earthquake occurrences. Therefore, the Peak Ground Acceleration (PGA) of each earthquake for a building within a specified service life is determined by using the Cumulative Distribution Function (CDF) of PGAs for a selected site. Additionally, for convenience, earthquake occurrences for a specified period are simulated using the Poisson process (Kumar et al. 2009). Notably, owing to its lack of memory nature, the Poisson process cannot effectively deal with large magnitudes; however, because the Poisson process is not physically unreasonable for events of small to medium magnitudes, it can still be used to study the effect of the cumulative damage in this work.

Estimating LCCs caused by earthquakes involves an effective method for assessing seismic structural damage. Based on the model of Park and Ang (1985), this work assesses seismic structural damage to an RC building. By simulating life-cycle earthquake events within a specified period and using nonlinear dynamic analysis, including earthquake occurrences and their PGAs, this work also derives the damage states of an RC building considering the effect of the cumulative damage. Additionally, besides life-cycle earthquake events, a simplified model is developed to modify the structural properties of a structure without seismic repair after earthquakes. Given the uncertainty of the occurrence time and PGAs of earthquake events, the seismic reliability and expected current values of LCCs are calculated using Monte Carlo Simulation (MCS). Finally, a case study involving the estimation of the optimal design base shear force demonstrates the applicability of the proposed procedure.

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Threshold-based network-level transportation infrastructure management

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ABSTRACT

Transportation infrastructure life-cycle management deals with maintenance decision-making of facilities such as pavements and bridges. Various methodologies have been adopted to determine the optimal allocation of limited funds over multiple-periods. Among these methodologies, optimal control is the state-of-the-art for maintenance decision-making. However, most of these models generate highly detailed plans describing maintenance policy for every facility at every time period. Such maintenance plans are not well-accepted by maintenance agencies because these plans are incompatible with their workflows for maintenance and rehabilitation. In specific, these solutions are difficult to understand and explain to their supervisors and funding authorities. Moreover, they often change dramatically when available funds change slightly (Task Force on Pavements and the AASHTO 2001).

In practice, these agencies use threshold-based rules for maintenance decision-making. However, maintenance thresholds are generally determined by engineering judgment or past experience in practice. Valuable maintenance resources thus cannot be used effectively, which motives the research need for the optimization of maintenance thresholds. Hybrid Dynamic Models (HDM) (Bemporad & Moran 1999, Torrisi & Bemporad 2004) is adopted for realistic threshold-based maintenance process modeling. The threshold optimization problem is categorized as a nonlinear mixed-integer bi-level programming problem (NP-hard) and a solution method is proposed to solve this problem. Finally, the methodology is tested with a road network for its capability of generating thresholds intuitive to highway maintenance agencies.

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The economic impact of photocatalytic concrete in an urban industrial setting

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ABSTRACT

Photocatalytic concrete is an innovative advancement in material science which enables a photocatalytic process that reduces airborne pollutants such as SO_x, PM_{10}, VOCs, and NO_x. It uses the power of the UV-A portion of sunlight to accelerate the natural oxidation process to decompose pollutants. The utilization of photocatalytic concrete in the construction industry has been assessed through pilot studies in France, Italy, the Netherlands and Japan.

The thrust of this research is to analyze the true cost of photocatalytic concrete infrastructure located in a highly industrialized urban city utilizing Life-Cycle Cost Analysis (LCCA). Hamilton is an industrial city with a population of 500,000 located in Ontario, Canada. The city is known for its steel mills and is linked through an extensive highway system to Toronto, located approximately 100 km to the east. The NO_x and SO_x concentrations in the city are created from transportation sources and industrial point sources. Pollutant concentration data has been reported in the literature using a mobile sampling unit which highlights the effect of wind speed and direction on NO_x levels.

This study examines the economic feasibility of constructing photocatalytic concrete median dividers in comparison to conventionally designed concrete 'Type F' median barriers. The LCCA captures the valuation of the pollutants that are broken down by the photocatalytic processes and also the pollution emissions during manufacture. In this study, the estimated service life of concrete infrastructure is critical to the LCCA since it directly impacts the net amount of pollution degradation.

Key findings from this study revealed that:

- Application of photocatalytic concrete does not present a strong economic incentive to implement on a large scale.
- Optimal conditions (i.e. environmental factors) are necessary for the present value life-cycle cost of PCAT barriers to be greater than GU barriers.
- Comparison of the present value life-cycle cost of PCAT barriers to GU range from 2.9% to −4.4%
- Outcomes from this study and many others reported in the literature have are limited to the examination of NO_x removal by photocatalytic processes.
- Greater accuracy in LCCA investigations requires further understanding on the degradation rates of particulate matter and other pollutants (SO_x, VOCs, and ground level ozone). Even though the pollution avoidance cost for particulate matter is quite high compared to other pollutants, the amount of particulate matter that can be degraded is still small due to the low, pollutant concentrations on primary road networks in Canada.

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Life-cycle optimization based on progressive collapse monitoring of irregular RC structures located in seismic active areas in Europe

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ABSTRACT

This paper presents a strategy for life-cycle optimization of an irregular multi-story structure, based on experimental available data and a set of Finite Element Analysis (FEA) done for the simulation of its progressive collapse. In introductory part, some relevant issues of design methodology of European Code EC8 compared to Romanian National regulations for this irregular RC class of constructions, located in seismic areas are presented. The main focus of our paper is modelling & simulations studies for life-cycle optimization of these irregular structures using Dynamic Modal and Nonlinear Pushover Analysis Methods, respectively.

As study case, a RC model of an irregular structure has been considered, for which the experimental dynamic data have been available from SPEAR project, [ELSA Laboratory, JRC European Commission], as represented in Figure 1.

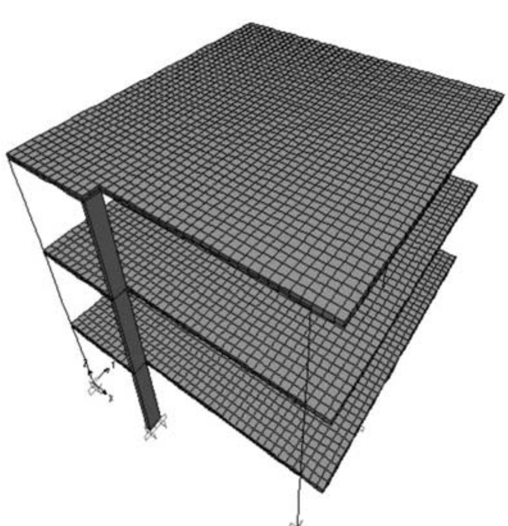

Figure 1. FE Analysis model of the SPEAR structure.

This model is analyzed in different stages of behaviour, first in its initial non–retrofitted stage, in order to validate the dynamic results obtained in FEA towards the benchmarking with available experimental data. Next, the retrofitted model was considered in accordance with Romanian regulations for rehabilitation of existing buildings.

For the two different stages of lifetime behaviour considering the non-retrofitted and retrofitted RC models respectively, the Push-over and Progressive Collapse procedures are applied, revealing the collapse sequences along with relevant changes in structural seismic safety.

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The use of advanced remote sensing techniques for monitoring of slopes affected by slow movements

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ABSTRACT

This case-study concerns a complex gravitational movement characterized by displacements of few mm/year, and made up of a deep sliding surface and another more superficial. Such slope has been affected by the construction of a highway tunnel, provoking the temporary acceleration of the slope movements and the reactivation of some portions of the slope that were apparently steady.

Our goal is to illustrate the applicability of a new technique to the monitoring of slopes affected by slow movements, in order to identify the most critical areas. The latter should be then monitored with a careful conventional system of monitoring such as inclinometers, strain gauges and crack meters.

During the realization of the tunnel we analyze traditional monitoring results, comparing and crossing them with ground-based SAR interferometry. Results allowed us to focus on the landslides and structures falling within the settlements trough analysis. We want to draw attention particularly on a pre-existing structure, the only that was already located on a landslide area. Numerical analysis have been disposed for the most damaged buildings, then, different three-dimensional finite element models are developed. The goal of such analysis is to define maximum differential settlements of each building, depending for both the state of real consistency and the structural typology of the building. The displacements that have been taken into account in the analysis are on the line to those detected by the monitoring system.

The use of advanced remote sensing techniques has many advantages over conventional monitoring techniques, but at the moment the SAR interferometry informations are to be considered complementary to conventional techniques rather than an alternative tool of slope monitoring.

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Performing of non-destructive measurement methods on existing arch bridge structures

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ABSTRACT

"An arch bridge can stand anything but no static calculation" (Proske & van Gelder (2009)). For refusing of this quotation the following contribution points out various measurement methods which have been performed on an existing arch bridge. The principal item of the measurement campaign was to analyse the condition and bearing capacity of an historical arch bridge structure.Concentrating only on non-destructive measurement methods, ground penetration radar, laservibrometer and linear variable differential transformer measurements were conducted. The Case Study object is located in Burgenland/Austria, close to the Hungarian border in Rohrbach bei Mattersburg and is an arch bridge which was built between 1845 and 1847 as a part of the Mattersburg railway. Based on the obtained measurement data and some additional calculations a model for experimental test series in the laboratory shall be set up. These experiments shall be made for the determination of the structural response of arch bridges due to various impacts. The mentioned non-destructive testing methods can also be applied on other structures than arch bridges and other materials like wood, steel or glass. It still needs to be checked whether it makes sense to use one of the applications or a combination with other systems. By combining at least two of the three mentioned methods errors in performing the measurements and misinterpretations can be avoided or identified in an early state of the research.

Non-destructive testing methods are methods for the analysis which don't have any impact on the material characteristics and the bearing capacity. Such methods can give information about size, build-up, density, form and homogeneity of individual parts of the structure.Most methods base on the characteristics and/or the change of the characteristics of an electromagnetic wave under examination. Chances can be detected in the wavelength, the wave frequency or delay.

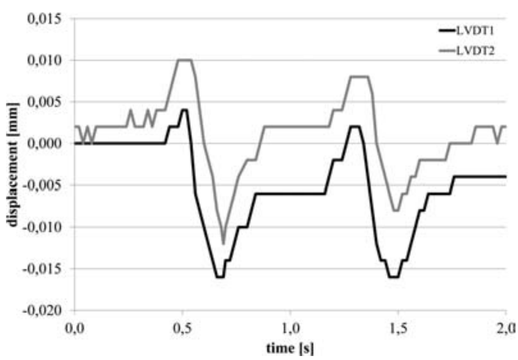

Figure 1. Displacements measured in the direction of the corresponding pairwise transducers caused by a crossing single railcar type 5047.

An existing arch structure can be analysed on the base of various measurements e.g. ground penetrating radar, the laservibrometer and LVDT.

Figure 1 shows as an example the measured displacements at one of the pairwise applied measurement crosses. The most important data is the maximum peak of the measured displacements, which occurs if one of the axes of the railcar passes the arch bridge. In the visualization of the measurement data the number of the axes which crossed the bridge can be identified easily by counting the number of peaks (see Figure 1). By the algebraic sign of the data it can be made an estimation whether a contraction (negative data) or extension (positive data) was recorded.

Further research should deepen the investigations into this field as preliminary studies for laboratory tests and nonlinear modelling concepts.

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Post-earthquake geomatic survey of a monumental building in L'Aquila, Italy

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ABSTRACT

The results of a survey carried out on Palazzo Camponeschi, a monumental building situated in the heart of the old city center of L'Aquila (Italy) and seriously damaged by the April 6, 2009 earthquake are presented. Structural monitoring was carried out performing automatic topographical measurements repeated at different time for several months. Kinematic collapse mechanisms recognized on the building (overturning of the facades and other forms of either local or global structural collapse) were monitored observing a number of strategically selected points of the structure. In particular, a grid of 27 points was decided as monitoring network along the buildings fronts facing the internal courtyard (Figure 1).

Survey methodology was based on the use of mini-reflectors, installed on the above mentioned selected positions, to be observed by an advanced motorized total station TS30. Horizontal and vertical angles measurement were performed by the total station with LED technology; electro-optical distance measurements were performed by the TS30 with a visible laser beam coaxial to the optical axis and transmitted by the EDM system. A 6 layer reading procedure was applied to angles and distances measurement: this made possible the statistical treatment of the data, the elimination of the systematic errors and the achievement of the final desired precision. Two independent geomatic triangulations and GPS measurements were also performed to ensure the steady position of the total station over time, allowing a precise recalculations of its absolute coordinate before any new set of measurements functional to the structural control of the monitored building (Dominici 1989). After clearing a three-dimensional angle and distance compensation, the assessment of the structural deformations was then carried out in real time with the accuracy of the mm. Results confirm the interesting possibilities offered by modern techniques of geomatic surveys in the monitoring of important buildings, both during the immediate post-earthquake period and during their reconstruction and recovery. The just started monitoring of Palazzo Camponeschi represents the precursor of modern survey techniques possible to be extended to a number of other monumental buildings in the town (Dominici et al. 2011). On the other hand, structural control on old historical and monumental buildings remains a complex operation to be set time by time on purpose, mainly because of the great number of irregularities and peculiarities characterizing this type of buildings.

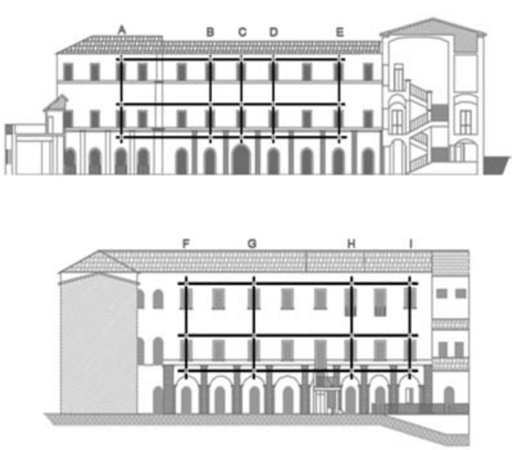

Figure 1. Grid of monitored points set on the building' SW (above) and NW internal fronts (below).

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Transverse load distribution in masonry arch bridges

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ABSTRACT

Masonry arch bridges comprise a significant proportion of the bridge stock in many countries (See Figure 1). Due to increased traffic loads and possible deterioration of the bridge structure, it is essential that the load carrying capacity of such bridges be regularly assessed. Environmental benefits can be gained if the assessment provides an accurate evaluation of the capacity of the bridge, thereby avoiding unnecessary remedial work or bridge replacement. Several methods are currently used to assess masonry arch bridges including the popular modified MEXE method and mechanism analysis. An issue of particular interest is how the loads are distributed transversely in the arch. The current load model for masonry arch bridges calls for the utilisation of an effective strip when dealing with the transverse distribution of an applied load (The Highways Agency, 2001).

In this paper LUSAS Finite Element Analysis (FEA) software is used to assess transverse load patterns on masonry arch bridges. An arch is modelled using thick shell elements to represent voussoirs and a reduced stiffness is assigned to the mortar joints between each voussoir. The overall aim is to develop a load distribution model that improves upon current mechanism methods for the design and assessment of masonry arch bridges.

In this study, three masonry arches have been modelled using LUSAS FEA software. The arch geometries are outlined in Table 1. Arch 1 is representative of a typical arch bridge and Arch 3 is akin to a culvert or tunnel. Arch 2 is not a typical arch geometry and was modelled solely for comparison purposes. Arch 1 can be seen in Figure 2.

Preliminary results indicate that using a uniform effective bridge width along the length of the span is overly simplistic. It is shown that the load effect fans outward with increasing distance from the application of the point load; this is at variance with the assumption of using a uniform effective bridge width. It is also found that where the load is located away from the crown of the arch, the thrust is spread more widely across the arch width than when the load is applied to the arch crown.

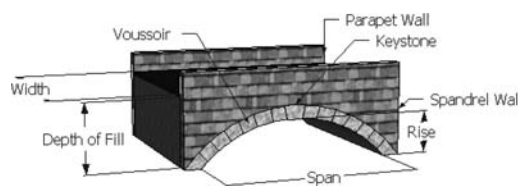

Figure 1. Typical masonry arch.

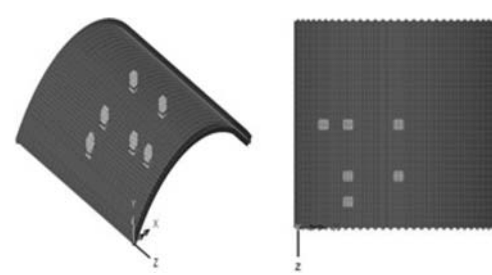

Figure 2. Arch with point loads.

Table 1. Arch geometries.

	Arch 1	Arch 2	Arch 3
Span	10 m	10 m	10 m
Width	10 m	1 m	50 m
Shape	Segmental	Segmental	Segmental
Height Midpoint	2.5 m	2.5 m	2.5 m
Height Quarterpoint	1.84 m	1.84 m	1.84 m
Ring Thichness	0.5 m	0.5 m	0.5 m
End Conditions	Pinned	Pinned	Pinned

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Time dependent reliability for existing structure based on the moment method

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ABSTRACT

The moment method is one of the most efficient methods for structural reliability analysis. However, this approach is mainly used for time independent reliability, and has not been extended to time dependent reliability. In the present paper, the extension of the moment method to time dependent reliability is investigated. Firstly, a time parameter t is introduced into the performance function, and an approach for probability moment evaluation of time dependent univariate and multivariate function is proposed. In order to simplify the moment evaluation of multivariate function, the weighted sum of univariate functions is used to approximate the original multivariate function. Rosenblatt transformation or Nataf transformation is introduced into the evaluation, so that the calculate points and corresponding probability weights can be used in unification. Secondly, the relationship between probability moments and reliability in existing moment methods is introduced and the moment method for time dependent reliability is put forward. Finally, two numerical cases, one of which is a simply supported reinforced concrete beam with degraded material and the other is a plane steel truss structure with time dependent loads applied on, are analyzed to elaborate the detailed implement of the proposed method and to verify the proposed approach.

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Preservation of structural and functional integrity in the interaction of new and existing structures: The case of London Underground's Green Park Station

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ABSTRACT

The London Underground system is not only the oldest of its kind worldwide, but also one of the busiest, serving approximately a billion passengers annually. Both of these characteristics induce an accelerating necessity for maintenance and rehabilitation of the present infrastructure, which of course includes the upgrading and extension of existing underground stations. Next to that, especially with London being one of the most condensed regions in the world, influence of tunneling works on the built environment can hardly be overmatched. This poses a huge challenge for the designer, who comes to develop an appropriate balancing solution that guarantees, apart from the new structure's safety, the functionality and structural integrity of the existing neighboring structures and surrounding assets and utilities. In that sense, the design is strongly de-pendent on the different life-cycle stages of the individual elements within the same system due to the varying construction types, building standards and philosophies, varying ages and maintenance levels, and socioeconomic priorities. For this challenge to be confronted in practice, prediction of deformations during construction is a key-agent, whereat the successful combination of three main features can be signified: (a) advanced finite element modeling, (b) expertise-based assumptions, and (c) response monitoring of the existing assets throughout the entire project execution phase. These critical aspects of urban tunneling are discussed on the basis of the accomplishment of the London Underground's Green Park station extension and upgrading project, while the experience gained from the completion of this project may provide a solid reference for future cases.

Based on the London Underground Green Park Station Upgrade case-study, certain small to large scale aspects of infrastructure life-cycle engineering are highlighted. At first, detailed aspects of an intervention design with particular focus on the impact of existing adjacent structures is presented here on the basis of this London Underground Station Upgrade project in Central London. The main feature of this project is the expansion of the existing Tube station and the simultaneous operation maintenance and structural protection of the existing century-old neighboring assets (rest of the station, buildings on the surface).

In this context, 3D modeling combined with an extensive monitoring program proves to be vital for the deformation control. This allowed for the works to be driven efficiently in a heavily burdened urban environment without interruption to the operation of the adjacent infrastructure and businesses. Furthermore, the 3D finite element prognosis assisted that implementation of highly consuming protection measures was limited or even abandoned. Simultaneously, the particular project provides key information for the stress-strain, loading and deformation behavior in the underground space of London. This serves as a reference project for several similar construction attempts in London (e.g. Bond Street Station, Tottenham Court Road Station) thus comprehensively supporting the extensive efforts for the London Underground network maintenance.

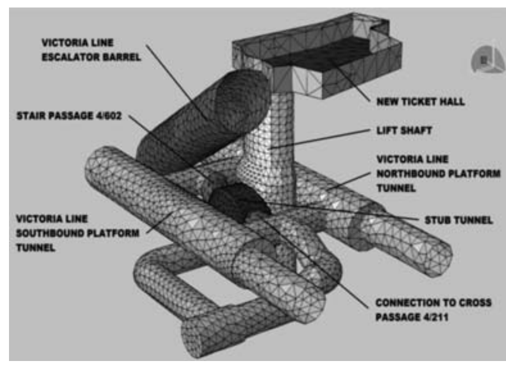

Figure 1. Model of the Green Park Station in London; new and existing structures.

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Progressive collapse of seismic resistant multistory frame buildings

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ABSTRACT

Civil structures are designed to withstand loads from their occupants and the natural environment. Such structures may have little reserve capacity to accommodate abnormal loading conditions that have very low frequency of occurrence, but extraordinary consequences resulting from sudden changes to the building's geometry and load-path. Furthermore, social and political factors also have led to an increase in these hazardous events that may produce a risk to buildings. Finally, changes in design and construction practices over the past several decades have lessened inherent robustness in certain modern structural systems, making them vulnerable to abnormal loading conditions. In particular, the local failure of a primary structural component produced by abnormal loads may lead to the additional collapse of adjoining members. In such cases the sudden loss of a critical load-bearing element initiates a chain reaction of structural element failures, eventually resulting in partial or full collapse of the structure. The process will continue until the structure can find equilibrium either by shedding load as by-products of elements failing or by finding stable alternative load paths. As a consequence, in recent years there has been a growing interest in the study of collapse scenario associated to abnormal loads (faulty construction practice, foundation failure, accidental impacts, gas explosion, machine malfunction, bombs, volcanic eruptions, landslides).

In this study the progressive collapse-resisting capacities of steel moment-resisting frame buildings designed according to the Italian Code are evaluated. At first, a stepwise increment of amplified vertical loads (pushdown analysis) for a variety of possible scenarios regarding the location of the missing column is employed. Then, as the nonlinear dynamic analysis for progressive collapse analysis does not require modeling of complicated hysteretic behavior and may be used as a practical tool, it is applied for evaluation of progressive collapse potential with the same first story column suddenly removed. Finally, the dependence of structural response on the variables such as applied load, location of column removal, or number of building story is investigated.

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Sustainability of tunneling based on life-cycle analysis

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ABSTRACT

Sustainability is becoming increasingly important in our consciousness and specifically in the construction industry. Although sometimes dismissed as a marketing strategy or a current trend, the core values of sustainability in construction are essential for the survival of our society.

In the structural engineering sustainability issues are already duly considered whereas in the infrastructure industry they do not play a significant role yet. The values of sustainability should definitely be applied also to the construction of civil engineering.

The introduction of a common database and common assessment criteria is necessary to make the sustainable features of buildings measurable and comparable. Since the various infrastructure projects have different characteristics, a general list of criteria will not make much sense. The following approach could be more appropriate: Separate sets of criteria or common sets of criteria with sub-systems for the different infrastructure projects such as tunnels, bridges, roads, hydropower plants, etc.

Some invariable criteria should be defined as well as several variable criteria. The criteria established by DGNB (German Sustainable Building Council) could serve as a starting point. The first step would be to filter out those that can be used also for the evaluation of infrastructure. Then the infrastructure-related criteria are added. The five main criteria groups defined by DGNB are Environmental quality, Economic quality, Sociocultural and functional quality, Technical quality and Process quality.

Since the proportion between new buildings and renovations has shifted strongly in recent years and will shift even further, it is essential to think already of renovation and maintenance measures while building.

A sustainability certification should not lead to additional bureaucracy. When thinking of the countless documents which are necessary during the process of an infrastructure project, among other things in order to obtain a positive assessment, then it becomes clear that the introduction of a sustainability certification could also be an opportunity. An opportunity to call into question the usefulness of some constraints and their impact as well as to streamline the approval procedures and processes.

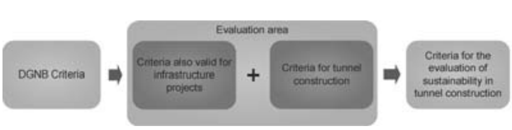

Figure 1. Identification process of the criteria for the evaluation of sustainability in tunnel construction.

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Safety evaluation of masonry arch bridges by nonlinear finite element analyses

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ABSTRACT

With the focus on railway bridges, a concept for sufficiently accurate and computationally efficient nonlinear finite element analyses of masonry arch bridges is proposed. Increasing traffic loads and changing structural conditions may require the re-analysis of such structures.

The influence of the assumed material parameters on the model predictions has been investigated in parametric studies. Numerical results obtained for real and imaginary bridges are presented.

Cracking of masonry under tension as well as nonlinear deformation under compression is taken into account. For the simulation of cracking, the concept of nonlinear fracture mechanics, which is widely used for describing the behavior of concrete, has been adopted. Figure 1 shows the crack pattern at the bottom side of an arch as obtained by numerical simulation.

Two failure criteria are used. The first one is met when the maximum principal stress in the uncracked arch reaches a critical value, i.e., when the first cracks are being formed. The other one is based on a critical crack length in relation to the arch thickness.

The masonry tensile strength assumed for the numerical simulations has a strong influence on the model predictions, whereas the influence of the fracture energy is less significant.

The soil-structure interaction is modeled by incorporating the soil into the finite element model and assigning a yield criterion to this material. The Drucker-Prager plasticity model has proved to be suitable for describing the behavior of the soil supporting masonry arch bridges. By considering the nonlinear material behavior, unrealistic tensile stresses in the ground may be eliminated. The simplifying assumption of a linear elastic behavior of the soil results in a noticeable underestimation of the internal forces in the masonry arches.

For certain bridges, it appears to be inevitable to use 3D models. This may be the case for curved bridges or if the effects of load eccentricities and lateral forces may not be neglected. Figure 2 shows the finite element model of a curved single-track masonry arch bridge the load-carrying capacity of which would have been overestimated by using a 2D model and disregarding the out-of-plane loading.

Experimental data obtained under regular traffic conditions may support the formulation of appropriate analysis models for masonry arch bridges. It has to be considered, however, that these data do not allow for a complete validation of nonlinear analysis results. The latter may only be validated by load tests above the service load level. Nevertheless, measurements under regular traffic conditions are a comparatively cost-efficient method to improve the validity of the simulation results.

Figure 1. Cracks at the bottom side of a masonry arch bridge.

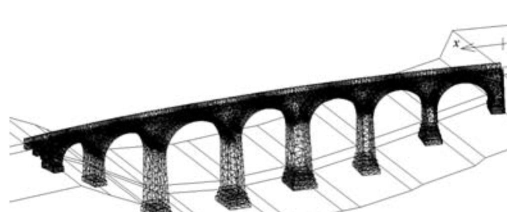

Figure 2. Finite element mesh for a curved bridge (mesh for the soil not shown).

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Model-free identification of uncertain time-dependent material behaviour for long term structural analysis

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ABSTRACT

The application of new materials in engineering as well as the evaluation of existing structures require knowledge about their long term properties. Time-dependent structural behaviour can be investigated by structural monitoring. In situ monitoring and material tests can be performed. Often, only a short time period – with respect to the aspired lifetime – is regarded within the structural monitoring due to restrictions in time and costs. But the obtained data sequences can be used to identify time-dependent material behaviour, which is required to predict structural responses, e.g. lifetime and reliability.

If experimental investigations are carried out, it is difficult to assign precise quantities to the observed events. Data uncertainty occurs which may result from scale-dependent effects, varying boundary conditions which are not considered, inaccuracies in measurements and incomplete sets of observations. Measured results are more or less characterized by data uncertainty which originates in imprecision. Imprecise parameters can be described as fuzzy data or intervals (Möller and Beer 2008). Here, fuzzy processes are utilized for measurement values representing time-dependent material behaviour. They are required to identify deterministic or fuzzy material formulations for structural analysis.

Commonly, material models are utilized to describe time-dependent phenomena (stress-strain-time dependencies). However, often limited and only imprecise information is available for the selection or the development of adequate material models and the identification of their parameters. An alternative are model-free material formulations based on soft computing methods. Artificial neural networks can be applied to identify dependencies between time-varying stresses and strains from uncertain measured data. Recurrent neural networks for fuzzy data (Graf et al. 2010, Freitag et al. 2011) have been developed to identify deterministic or uncertain dependencies in fuzzy processes. The recurrent neural networks can be trained and validated directly with data series representing components of the stress and strain tensors or indirectly with uncertain data of inhomogeneous stress and strain fields. The indirect approach requires a numerical simulation of the real experiment.

Identified uncertain stress-strain-time dependencies can be considered for long term structural analyses. The neural network based fuzzy material formulation is applied within a fuzzy stochastic finite element analysis (Graf et al. 2011). In an example, the reliability of a pavement structure is assessed.

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A multi-dimensional approach on maintenance of distributed infrastructure: ASFINAG's maintenance strategy

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1 INTRODUCTION

ASFINAG conducted an evaluation of its maintenance strategy during the second half of 2011.

The objective was to create a maintenance strategy that includes all types of assets (roads, bridges, tunnels and electrical and mechanical equipment). During the process it became clear, that ASFINAG's maintenance strategy so far consisted of different partial strategies which operated on unequal levels due to organisational reasons in the past. During the evaluation the team tried to identify and consider all relevant stakeholders and surroundings. First of all the condition of assets (age, condition of surface and structural conditions of buildings and equipment) has the biggest impact on maintenance efforts.

2 INTEGRATED MAINTENANCE STRATEGY

The ASFINAG Maintenance Strategy is included in the general ASFINAG strategy.

One customer related objective is to minimize disturbance due to construction sites along the road network and provide high availability. Another objective is to increase traffic safety.

The focus of the financial objectives is on both sustainable investing and optimizing the asset life-cycle.

Another objective is to guarantee sustainability of refurbishment measures. The main aim is to reach the best performance at the lowest possible cost while obtaining the highest possible sustainability.

3 PROCESS AND ORGANISATION-RELATED MEASURES IMPLEMENTED BY ASFINAG

The Maintenance Group – which is part of the Asset Management Department – surveys ASFINAG's assets and defines required measureson the basis of inspections carried out. During the annual planning process the Asset Management Department prepares a plan which includes measures to be carried out within a time period of six years taking into account ASFINAG's budget. In addition, the Asset Management Department is responsible for both the implementation of the ASFINAG Maintenance Strategy and current updates.

ASFINAG uses a net-wide maintenance planning process which includes all existing assets.

3.1 *ASFINAG Asset Management System*

3.1.1 *Pavement Management System (PMS)*

The PMS includes all steps which are necessary for maintenance planning. A net-wide-survey which is conducted using the Roadstar System allows for optimal maintenance planning on the basis of a damage-related measure catalogue which is part of the system.

3.1.2 *Engineering structure management and electromechanical assets*

The condition of engineering structures and electromechanical equipment is surveyed by engineers of the Maintenance Group or by civil engineers. The results of these inspections serve as a basis for the planning of required measures. ASFINAG looks into the possibility of implementing a data-based planning tool similar to the PMS used for engineering structures and electromechanical equipment, in order to improve the planning system.

4 OUTLOOK AND UPCOMING CHALLENGES

In 2011 the process of defining an integrated maintenance strategy succeeded. In the organisation a clear division of duties is implemented by now.

In the next years ASFINAG has to apply the strategy to the highway network in Austria and to develop it further.

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Availability orientated operation of automated people mover

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ABSTRACT

The guarantee of a highly reliable system can be assured within a Life-Cycle Management System (maintenance management information system), that is able to use all available life-cycle data covering all product phases efficiently including complete information on the state of compliance with the customer's expectations and requirements with regard to RAMS (Reliability, Availability, Maintainability and Safety).

The functional reliability, availability, maintenance and safety of these technical systems have a significant influence on customer satisfaction and therefore on the success of the company.

In order to ensure the reliability of these complex technical systems, a reliability and safety management system is required, such as the RAM(S) Management System with the support of an MMIS (Maintenance Management Information System). The aim of this system is to make the factors relating to functional reliability, availability, maintainability and safety manageable and controllable.

At DCC GmbH, this RAM(S) management is taken care of by the Maintenance Management Information System (MMIS).

In order to establish availability (RAM) and Safety (S) for a project, a variety of information is required. This information is obtained by analysis of the system reliability and/or availability as well as risk analysis which are required within the context of RAMS management.

The aim of the RAM(S) management process is to ensure that RAM(S) requirements for all phases of the entire product life-cycle are logically specified and also fulfilled by the system being operated.

The management process supports:

– The definition of the RAM(S) requirements and the measures to fulfill the RAMS requirements through,
– identifying and avoiding negative influences on the RAMS properties
– the planning and implementation of RAMS activities
– providing fulfillment of the RAMS requirements
– ongoing monitoring of fulfillment during the life-cycle and
– therefore the Life-Cycle Cost Controlling.

The main objectives for the structured return flow of data within a Life-Cycle Management system:

– To obtain data and information on the product behavior (reliability, life span and costs).
– To determine weak points and potentials for technical optimization based on data taken from the field.
– The transparent itemization of costs over the entire life-cycle.
– The comparability of costs at a component level and across installations (maintenance and repair costs).

Further use of the results within our organization:

– Strategic product/service optimization
– Condition-oriented maintenance
– Life-Cycle Cost Controlling
– Development of service models

The successful implementation of the Life-Cycle Management System, e.g.:

1. Reorganization of the maintenance team to increase productivity and reduction of maintenance costs and labor of more than 40%.
2. Product Improvement due to return of information from operation.
3. Redesign and fabrication a Maintenance Vehicle that is better for maintenance use; reducing of mobilization time and increasing of safety.

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Time-dependent compressive strength of concrete in existing buildings

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ABSTRACT

Concrete strength in existing buildings is changing with time during their life-cycle. For retrofit and dynamic reliability analysis of existing buildings the concrete strength is an essential influencing factor. So it is necessary to know the time-dependent concrete strength at service age, based on the existing research the most significant influencing factors that affect the concrete strength are freeze-thaw cycle, sulfate attack, chloride corrosion, carbonation and temperature and so on. A lot of research on the change rule of the concrete strength at service age has been carried out to model time-dependent concrete compressive strength in existing buildings. To understand the time-dependent concrete strength a quantity of experiments has been done to investigate the action of influencing factors. However, most of the experiments focus on study single or double factors and usually the acceleration simulation experiment are used under laboratory environment. As known, in the atmosphere environment the influencing factors exist simultaneously and sometimes they interact on each other. Thus the laboratory environment cannot simulate the real influencing environment accurately. Therefore the applied range of the most of the existing models is limited and they are not suitable to estimate the concrete strength in existing buildings. So the best way to understand the time-dependent concrete strength is to test the concrete strength in the real structures at different service ages. In this paper about 400 buildings serving ages from 1 to 59.5 years are investigated in fields. Based on the compare of these models, the test data by rebound method cannot be expressed by existing models correctly and all related coefficients are very small. Due to this, the residual error is chosen as the criterion to determine the time-dependent model of concrete strength. Finally the Plowman model and Lew model are suggested to predict the concrete strength by rebound method in existing buildings in Shanghai area. The quadratic polynomial expression is suggested to predict the concrete strength by core drilling method and unfortunately the sample data is a little small.

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Application of satellite radar interferometry for structural damage assessment and monitoring

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ABSTRACT

Radar satellites allow the measurement of ground displacement to millimeter accuracy, thanks to a particular technique known as the 'multi interferogram' approach . The latest development of this technique is SqueeSAR™, which allows the identification of discrete ground points and their displacement in time (Ferretti et al. 2010). Analysis of ground displacement is possible due to the exploitation of data obtained from all radar systems: the distance between the sensor and an 'illuminated' target.

The ground measurement points on the Earth's surface: typically responding to houses, buildings, metallic objects, pylons, antennae, exposed rock surfaces, pipelines, outcrops, detritus, etc. The technique does not require the installation of any ground instrumentation. One of the main advantages of SqueeSAR™ technique is the possibility to process data archives of Space Agency since 1992, enabling an historical review of movements.

The case studies illustrated in the paper, regarding both infrastructure monitoring and structural damage assessment. Several applications to railway sector have been presented showing how satellite technique monitoring supports the infrastructure entire life-cycle (design, construction and operation phases).

In the framework of High-Speed (HS) Milan to Naples railway construction, a tunnelling work under the city of Bologna (Italy) has been monitored combining *in situ* monitoring system with radar satellite data. The analysis demonstrates how satellite radar data can provide useful displacement data covering an area much wider than that covered by traditional monitoring systems. Moreover the processing of archive satellite data allowed additional assessment of deformational behaviour *ante operam* (i.e. *before* the start of tunnelling activities), and therefore independent of any construction work.

The HS railway case study was an opportunity to carry out an interesting comparison between satellite radar data and other measurements collected from conventional techniques. The optimal correlation between the two data-sets confirmed the precision of the SqueeSAR™ technique for the detection and estimation of surface displacement phenomena (Pigorini et al. 2010).

Regarding structural damage assessment applications three different case studies have been reported relevant in particular to urban excavation and correlated building damages (Jurina & Ferretti 2007).

Displacements monitoring in the city of Rovigo (Italy) is one of the first applications of the radar satellite technique. In the year 2002, during a forensic case, many historical buildings of the centre of Rovigo were declared to suffer from a significant and concerning structural crack patterns. All the cracks occurred simultaneously in 1994 within the period of a few months, eight years before any forensic analysis had been conducted. Research showed that during that period an underground car park was built not far from the damaged buildings. The radar satellite technique allowed to measure the settlement that had occurred eight years before, and to demonstrate that the structural damage was the consequence of the parking excavation.

The case studies described in this paper show that this kind of data may become extremely useful in the entire life-cycle of a structure or infrastructure.

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Energy harvesting for the life-cycle of structures and infrastructures: State of art, recent trends and future developments

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ABSTRACT

In the upward trend of renewable energy growth, several proposals have been made concerning energy harvesting devices in structures and infrastructures. The objective, concerning higher power extraction, is to supply power to auxiliary systems (e.g. road lights or information panels), thus, satisfying the requirement for sustainable transportation infrastructures. This paper deals with the state of art, the recent trends and the future developments in energy harvesting for structures and infrastructures. A literature review is provided (both for low- and high-energy applications), and after a taxonomy, focus is given to the definition of a broader framework of energy extraction for such systems.In this sense, it is possible to evaluate the system efficiency for different meso-scale energy harvesting applications in the civil engineering field (Figure 1). Asurvey for the possible research issues and synergies among different research sectors and some points for discussion are provided for the life-cycle analysis.

RESOURCE Defined in terms of E_H, E_C^{SS} +		From natural resources $E_H = \infty; E_C^{SS} = 0$	From artificial resources $E_H < \infty; E_C^{SS} > 0$	From natural resources, modified by artificial systems $E_H = ?; {}^*E_C^{SS} > 0$ * Greater than if a system of extraction was implemented
EXTRACTION SYSTEM Defined in terms of E_C^{ES} =		Magnetic induction converter Electrostatic converter Piezoelectric converter	Thermal energy converter Photovoltaic converter Radian energy converter	RF converter ...
COUPLED SYSTEM Defined in terms of E_C, $\hat{E}_H$, ΔE		Example: wind turbines for a wind farm.	Example: piezoelectric converters placed on deformable joints of a flexible structure.	Example: wind turbine on a skyscraper (the source is natural but it is placed in high attitude due to the skyscraper).
ΔE	ACTIVE ($\Delta E>0$)	SUCCESSFUL[1]	SUCCESSFUL[3]	SUCCESSFUL[3]
ΔE	PASSIVE ($\Delta E<0$)	UNSUCCESSFUL	SUCCESSFUL[1] UNSUCCESSFUL $\hat{E}_H - E_C^{ES} > 0$ $\hat{E}_H - E_C^{ES} < 0$	SUCCESSFUL[1] UNSUCCESSFUL $\hat{E}_H - E_C^{ES} > 0$ $\hat{E}_H - E_C^{ES} < 0$
ΔE	BALANCED ($\Delta E=0$)	UNSUCCESSFUL	SUCCESSFUL[2]	SUCCESSFUL[2]

SUCCESSFUL[1]: Production of energy; SUCCESSFUL[2]: Production of an amount of energy equal to the one consumed;
SUCCESSFUL[3]: Production of an amount of energy higher than the one consumed
Definitions: E_H ~ Maximum extractable energy; E_C^{SS} ~ Energy cost of the structural system; E_C^{ES} ~ energy cost of the extraction system; E_C ~ Total energy cost of the coupled system; $\hat{E}_H$ ~ Effective extracted energy of the coupled system; ΔE ~ Energy balance of the coupled system; $\Delta E'$ ~ Energy balance of the extraction system

Figure 1. Classification of the energy production schemes.

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Integral railway bridges for high speed trains – from conceptual design to construction by the example of the Gaensebachtal Bridge

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ABSTRACT

The standard design so far used for long railway bridges according to German Rail (DB Netze AG), has a typical structural system, which consists of simple beams resting on bearings on massive piers. The superstructure spans typically 44 m and has a hollow box girder with a construction height of 3,40 m and more. This kind of bridge was extensively used for new high speed train links in the last decades without respect of the surrounding and without any relation to the landscape.

On the new high speed railway link between Berlin and Nuremberg, part of the German Unity Transportation Project VDE8, several new integral railway bridges are currently under construction or have been recently finished. One of these is the Gaensebachtal Bridge with a length of 1001m (Schenkel et al. 2010). It is an alternative design in accordance with the guideline "Design of Railway Bridges" ("Leitfaden: Gestalten von Eisenbahnbrücken") of the DB Netze AG, see Schlaich et al. 2008.

A large variety of different designs are offered by the integral bridges described here, where the piers are connected directly or even monolithically with the superstructure without bearings resulting in a frame-like structure. Furthermore, integral bridges offer several advantages against the standard design such as robustness, lower erection and maintenance costs and sustainability due to less material.

The challenges in designing integral or semi-integral bridges result from a conflict of goals. On the one hand high longitudinal stiffness for braking forces is necessary, on the other hand flexibility in case of reaction forces is required.

According to German Rail Authority (EBA), this new type of integral railway bridges is not "proven technology" yet. Consequently, these integral railway bridges for high speed railway traffic required a couple of additional approval procedures. Further technical challenges such as the proof of rail-stresses, the proof

Figure 1. Gaensebachtal bridge during construction.

of dynamic behavior under design speed of 360 km/h, check of resonance and check of the fatigue behavior of monolithic connections had to be resolved.

The finished bridge demonstrates, that the additional work was worth the effort: The slender Gaensebachtal Bridge fits more harmoniously in the shallow valley than the original commissioned standard design. With a minimum of joints and bearings it promises low maintenance and inspection cost.

The example of the Gaensebachtal Bridge demonstrates also that integral railway bridges are at least equivalent with regard to load bearing capacity, serviceability and traffic safety as well as cost, even for high speed traffic.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)
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Maintenance costs calculation over the life-cycle: A method for the usage of elementary influence factors on the technical durability of technical components and constructed assets

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ABSTRACT

Life-cycle costing of buildings has gained importance during the last decade. This development is based on different facts, e.g. commercial reasons as well as societal trends. A common indicator is the prominence of sustainability of buildings and constructed assets, where life-cycle costs are of significant influence.

The essential mechanism to determine the economic success of a life-cycle project lies within the accurateness of the life-cycle costs. But despite the fact of an impressive number of life-cycle projects, a method does not exist to anticipate the prospective costs in an appropriate way.

The basis of this lack is the deficit of reliable data concerning the durability, which is needed for the calculation. It can be shown, that all existing reference-data are of minor validity concerning actuality, structure and precision and are therefore of no value for a resilient calculation. Especially the usage of static figures is inadequately because of the complex interdependency in the field of technical components and constructed assets. The given range of life-cycle data points this fact out.

From this need of specific data, the idea was created to replenish the existing methods (especially ISO 15686), by developing an approach that includes all relevant influence factors on the durability of the components. The method takes into account the existing standards, which do already use a factor approach. The relation between cause and effect is used to advance and optimize the methodology by adding a consecutive correction to the reference data.

Due to the lack of data sets a separate survey was required to gain the necessary information. An elevation among different experts was accomplished to gain more basic data and on the other hand determine the influence factors on the durability.

As a result, a method to use elementary influence factors to specify the calculatory durability could be obtained.

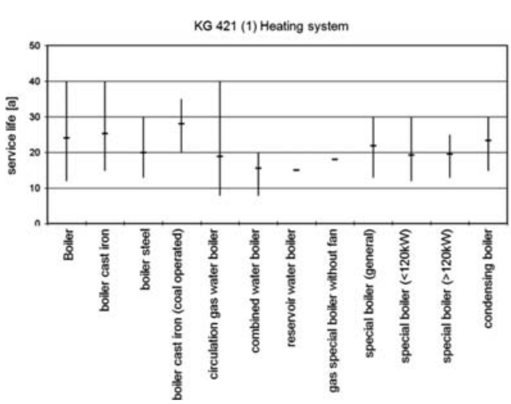

Figure 1. Example of literature research results for the service life of technical building equipment.

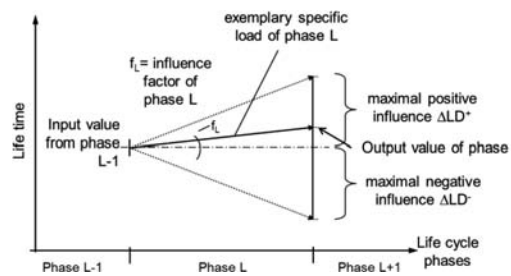

Figure 2. System of influence-factors during life-cycle.

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Load induced damage in masonry bridges, some observations on stiffness and force flow

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ABSTRACT

Masonry bridges suffer damage which is rarely readily explained by analysis. Indeed, most of the available analyses tell us very little about real behaviour. There are two reasons for this. The first is that the detailed geometry and material properties of the bridge and its supporting soil are rarely known. The other is that long established rules of thumb are used in analysis and they are often wrong.

The author has been involved in the study of masonry bridges for many years and has only recently found defensible explanations for some forms of aberrant behaviour. These relate to the complex interconnected stiffnesses in a bridge and the difficulty of understanding them.

In railway viaducts it is common for the deep section over the pier to be brought up to level with internal longitudinal walls capped with either stone slabs or arches. Some bridges of this construction suffer cracks in the arch under the inside edge of the outer spandrel wall.

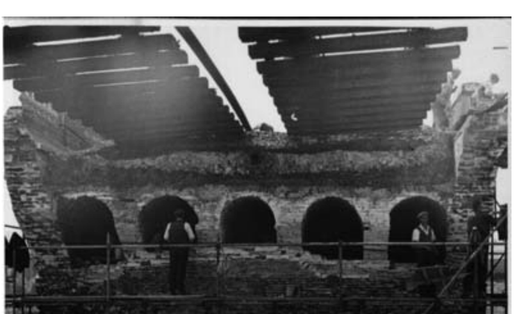

Figure 1. Cross section of a major bridge.

Figure 2. Internal spandrel walls in a small bridge.

Although the inner walls carrying the loads are extremely stiff, they, and the arch, provide little resistance to rotation of the block over the pier as live loads pass. The outer spandrel wall is, though, very much stiffer in this sense so that considerable strength is required in the thin sections between walls if the body of the bridge and the outer walls are to rotate together. If any part of the ring of material involved gives way, the crack will propagate into the arch and the bridge will begin to articulate.

It has recently become clear that even small railway bridges often have internal spandrel walls without caps but with fill between them. The complex stiffness of this system defies simple analysis.

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Flexible floor slab systems for long service life

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ABSTRACT

Contemporary buildings and building elements have almost mono-functional characteristics and provide only poor resource efficiency and slight aptitude for changes in utilization. Frequently, such buildings do not allow for the implementation of technical innovations or they disregard architectural issues. Therefore, a significant number of buildings in urban areas is demolished or elaborately remodelled before reaching their economic life time. In order to achieve higher efficiency and exploit the buildings' full range of life time, adaptive structural systems with a high degree of flexibility have to be developed.

Multifunctional and flexible floor slab systems have great potential to benefit the planning, construction, and operation of sustainable buildings. The present paper summarizes fundamental requirements for multifunctional slabs and presents different state-of-the-art solutions for slab systems in composite construction. Subsequently, general principles of construction for integrated slab systems are deduced. According to these principles, the paper describes the development of an innovative slab system that incorporates building services and technical installations into the structural element by means of an integrated installation floor.

The innovative slab element features a composite cross section, composed of halved HEA800 steel profiles and a prestressed concrete layer of 10 cm thickness at the bottom side (Figure 1). The concrete chord is attached to the steel beams by innovative puzzle shaped shear connectors that are immediately burned into the web of the steel profiles.

Due to the concrete chord at the bottom side, the system features improved physical and fire protection characteristics. The storage capacity and thermal performance of the concrete chord can optionally be upgraded by use of PCM-packs and an integrated hose

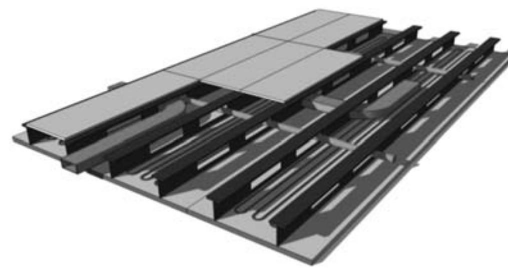

Figure 1. Innovative floor slab system.

assembly allows for thermal activation of the slab. The prestressing of the concrete layer limits deformations and concrete cracking and provides wide spans for increased flexibility at the same time. Large web openings in the steel beams enable flexible arrangements of all installations and detachable hatches allow for maintenance of installations during the buildings utilization stage. At the end of life time the flexible floor slab system is compatible to promising innovative ways of recycling, like the recycling in terms of reuse of entire buildings.

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A standardized life-cycle costing framework for flexible and rigid pavements in Austria

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ABSTRACT

Flexible and rigid pavements show a number of different failure types with considerable impact on road safety and rehabilitation needs. Standardized Pavement Management Systems (PMS) commonly address these issues based on deterministic weighted condition indices and utility value approaches. Due to the stochastic nature of most input parameters deterministic approaches allow only a rough approximation. Additional setbacks in common approaches are based on uncertainties towards weighting as well as an overestimation of expected service life or time between necessary maintenance and rehabilitation measures. Furthermore, addressing the specific dam-ages in the optimization process cannot be achieved due to averaging effects in the calculation of the condition indices and the lack of a full LCC – approach (Hoffmann et al. 2010).

With the presented deterministic framework and LCC – approach it is possible to overcome most of the limits of such conventional PMS - approaches. Based on a true optimized life-cycle approach an optimal treatment selection for any combination and extent of failures on road section, project and net-work level is feasible. Based on an iterative process even timing and work-zone length can be optimized allowing an optimal use of available resources (Hoffmann 2012).

The necessary amount of data for accurate predictions remains a challenge. While the standardized failure type and measure catalogues for flexible and rigid pavements are already finished; the available pavement performance functions and impact functions of measures are somewhat outdated. With average costs and service positions together with a de-tailed description for all measures already available further research into cost development and distribution remains necessary. However, based on all avail-able data and some estimates it is already possible to achieve reasonable deterministic results.

If the stochastic nature of most input parameters is considered it is necessary to extend the presented approach accordingly. With such a probabilistic approach the stochastic condition distribution over time and the remaining service life of all surviving road sections can be calculated. Furthermore it is possible to predict the occurrence probability of necessary preconditions for a successful measure application together with the survival probability against multiple failure types. A probabilistic model based on the presented deterministic approach allows an optimization of the entire life-cycle from dimensioning and construction until deconstruction. Such a fully probabilistic approach for Pavement Management was recently developed.

For the implementation of a full probabilistic approach further research and the results of the implementation from the deterministic approach are needed. With a thorough investigation of all available data of already existing PMS – Systems in Austria and data from pilot applications it seems likely to obtain all relevant input parameters for a full probabilistic Pavement Management System based on life-cycle costs with reasonable efforts.

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Risk-based monitoring, inspection and maintenance framework for coastal structures

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ABSTRACT

In the framework of a joint research project between Leibniz Universität Hannover (LUH) and Technische Universität Braunschweig (TU BS), Germany, a risk-based strategy for Monitoring, Inspection and Maintenance (MIM strategy) for coastal structures is being developed, which will be a key component of an overall framework for life-cycle engineering and management (see Figure 1).

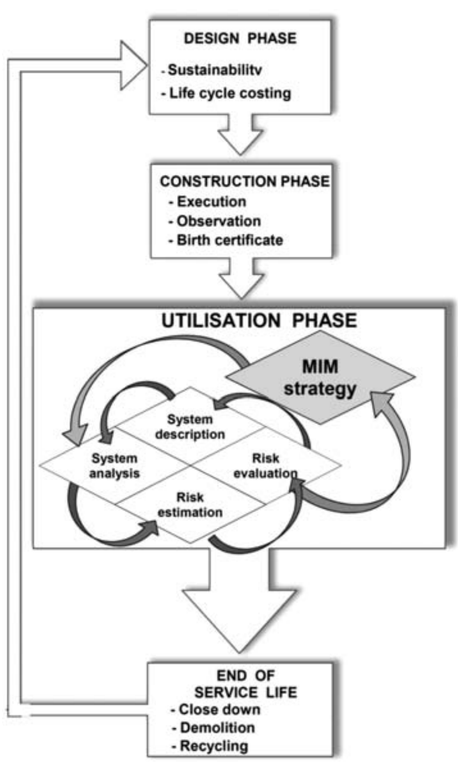

Figure 1. Life-cycle phases including Monitoring, Inspection and Maintenance (MIM) strategy (Horstmann et al., 2012).

The new MIM strategy is outlined and illustrated using typical examples of coastal structures: sea and estuary dikes as well as quay walls. First, a scientific basis for an improved understanding of the failure mechanisms and their effects on failure probability and serviceability of the aforementioned structures had to be established. As the MIM strategy is risk-based, the prospective methods and models explicitly account for the associated uncertainties (probabilistic approaches) and failure consequences (risk analysis), including Bayesian updating techniques.

This paper will particularly focus on the application of the MIM strategy to dikes for which extensive research on the failure mechanisms has already been carried out in the past years (e.g. Voortman (2002), Kortenhaus (2003), Steenbergen & Vrouwenvelder (2003), Vorogushyn (2009)).

However, in those reliability analyses the time dependency of the processes and the degradation mechanisms leading to dike breaching were either not considered at all or not properly taken into account. A methodology to consider the time dependency of processes in terms of duration, sequence, and simultaneity are proposed for different time scales.

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Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

On the sustainability of deconstruction and recycling: A discussion of possibilities of end-of-lifetime measures

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ABSTRACT

The total production of Construction and Demolition Waste (C&DW) in the European Union is about 450 million tons per year and represents the most part of waste stream (WMU, 2000).

The lifetime of buildings depends on their flexibility to be reused and, more and more, on economic questions as the annual costs for energy. Beside, most of the towns in dense populated environment do not grow anymore to their outskirts, but are redeveloped on their existing area. Consequently buildings of the last decades are replaced by modern buildings with sustainable and architectural appealing design.

Hereby the deconstruction and the management of construction and demolition waste of the existing buildings is a key issue in the development of sustainable construction. Prevention, re-use and recycling are the basic approaches to waste management.

Recycling is not only a question of the total costs of the deconstruction, but also depends on the fractions of the materials, which do not all permit recycling. Typical buildings at their end-of-lifetime today were not constructed in the past with any thoughts on possibilities of later reuse of materials or structures. But even today, the deconstruction as one possible strategy at the end-of-lifetime is not deeply investigated.

Thus, this paper focuses as well on the question, whether we can reach zero-waste deconstruction techniques in the near future or not. But even if we could, what additional possibilities exist today for concrete-gravel or other recycling products? Especially in high populated areas of Europe the amount of recycled concrete or masonry coming from deconstruction projects is about twice as much as the need for recycled material e.g. as an alternative to natural gravel (Figure 1). While complete reuse of buildings will be even in the next decades a suitable possibility in only a few cases, todays focus lies on pushing recycling rates higher.

Putting the focus on regular housing, today recycling rates of 75% are realistic, if 3rd order recycling is not taken into account (thermal use).

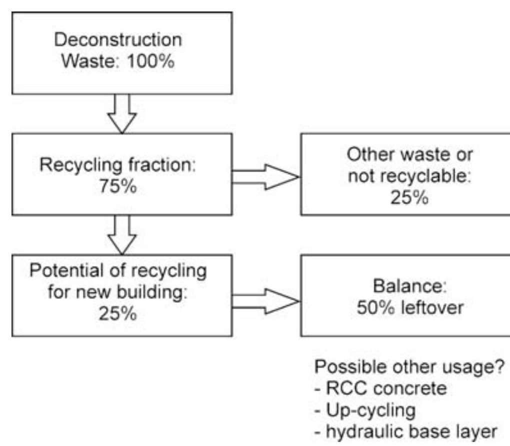

Figure 1. Material stream of deconstruction waste. About 75% of the total waste are recyclable. In the end, about 50% are leftover. The balance could be enhanced with the help of other products which make use of recycled crushed mineral waste.

Because deconstruction today means dealing with contaminations from the past, zero-waste deconstruction is not reachable for the next years (Kamrath & Hechler, 2011).

Technologically all requirements to minimize the impacts due to deconstruction are given: Separation is the key to prepare recycling on the local level.

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Long-term maintenance of deteriorating infrastructure: Inspection strategies for incipient failures

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ABSTRACT

Large structures and infrastructure components, such as bridges, tunnels, etc., are complex engineered systems representing significant public and private investments. These systems deteriorate as a result of their use and the external environmental conditions. Maintenance activities, which are directed to correct degradation effects, are a major component of life-cycle cost analysis (Frangopol et al. 2012). While expensive, they are also crucial in mitigating the risk of catastrophic failures, which have much more significant financial and social consequences.

This paper develops a stochastic framework for modeling deterioration of infrastructure systems. The model assumes that a structure has an initial nominal life Y (e.g, initial structural capacity), and it deteriorates over time according to a random process described by the instantaneous rate of degradation. The main interest is the time required for the remaining capacity of the system to reach a predefined performance threshold k^* (incipient failure state); i.e., $L = \inf\{t \geq 0 : V(t) \leq k^*\}$.

We assume that incipient failures are not self-announcing and can be identified only through scheduled inspections, which occur at predetermined times $\tau_1, \tau_2, \ldots$. When a scheduled inspection finds the system failed, maintenance is performed. Then, the objective is to determine a sequence of inspection times $\tau_1, \tau_2, \ldots$ that efficiently balance inspection capacity (measured by the rate of inspections per unit time) with maintaining the structure in an acceptable operating state. Two performance measures are considered in this paper: the *limiting average availability* A_{av} and the *long-run inspection rate* β.

Furthermore, two distinct inspection strategies are studied. First, we will assume that successive system lifetimes are *iid* random variables with known distribution function. In this first case, we rely on the regenerative property of the cycle times to determine limiting availability. In this case it is shown that knowledge of the lifetime distribution can lead to inspection strategies that outperform periodic inspections (Yang and Klutke 2000)). In the second strategy, we consider explicit deterioration models that may include both graceful (continuous) and shock degradation (Sánchez-Silva et al. 2011). We show that in certain cases, the regenerative structure of the cycles is maintained, but in other cases, successive lifetimes are no longer independent and identically distributed (Klutke and Yang 2002). In these cases, inspection scheduling is considerably more complex, and we identify several new areas for research.

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Determination of mechanical and temperature induced deflection based on monitoring data for assessing the load factor

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ABSTRACT

Jointless bridges are characterized by integral abutments and their lack of bearings and expansion joints. Recently this construction type has gained much popularity among bridge owners, since reduced costs in maintenance and rehabilitation are to be expected. This is caused by the absence of significantly endangered details like joints and bearings. There are uncertainties concerning material strength development and traffic load models as well as time-dependent effects (temperature, creep/shrinkage etc.). Currently those topics are being addressed by a number of research projects which focus on monitoring the actual structural response. Details on these different research projects as well as results and numerical simulations can be found in Strauss et al. (2011), Wendner et al. (2011) or Krawtschuk et al. (2011).

In this paper a suitable monitoring concept for all relevant structural parameters of a chosen structure is being presented and in particular the determination of mechanical and temperature induced deflection based on monitoring data is discussed. Further results of the application of an advanced finite element model are being discussed with regard to a comparison between monitored data and numerical modeling as well as an estimation of the actual influence of the different loading situations. The focus is laid on the separation of the overall deflection data into the individual parts due to temperature, dead load of the structure and traffic. Finally an estimation of the load factor α with respect to normative demands is carried out by using an advanced finite element model.

The separation procedure of the totally monitored deflection measured by high-frequency weigh in motion is divided into different steps which are shown in figure 1.

By comparing temperature-related deflection to total deflection (a) a separation of the high-frequency variable processes from the long-term variable processes and (b) a determination of time-delayed deflection effects of temperature can be done.

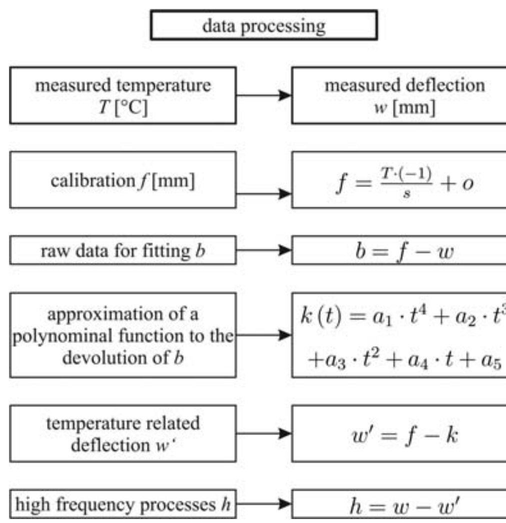

Figure 1. Separation process of measured data into individual portions.

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The development of the combined LCC analysis system for bundling public facility maintenance

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ABSTRACT

Public facilities are national and social properties whose safety and functionality are maintained during their service lives through preventive maintenance plans and facility management budgets based on cooperation between government and local autonomous entities. Korean public facility maintenance is characterized by delayed repairs and comparatively long-term replacement works because of facility management's limited budgets.

An LCC analysis is legally mandatory only for construction projects costing over 5 billion won; thus, the calculation bases of facility management budgets for public facilities costing less than that are inadequate. Moreover, public facilities managers believe the utilization of LCC analyses to be pointless because of their poor professional understanding of the LCC analysis results; they cannot see why the analyses are necessary but simply consider their costs a serious burden.

The bundling method utilized in the Private Finance Initiative (PFI) project is introduced in this study through a discussion of the development of the Web-based combined LCC analysis system, which carries out combined LCC analyses focusing on regions, projects, and repair and replacement items and manages the related records to enhance public facility maintenance planning and performance.

A Web-based combined LCC analysis system was developed in this study for the efficient application of bundling maintenance to public facilities. It has been verified through domestic public facilities maintenance activities, setup classification schemes for system-based DB, and a system prototype design and pilot test.

Through a preliminary study, a classification scheme for domestic public facilities and a repair and replacement item classification scheme were established based on a pilot test at K-corporation. The Web-based program was developed with user convenience and system extendibility in mind.

The combined LCC analysis system for public buildings will prevent duplicate investments and enhance economic feasibility by combining activities on identical projects in the pursuit of preventive maintenance and repair and replacement in accordance with long-term repair programs. Moreover, prioritizing maintenance activities would be made easier, as future costs could be estimated during public facility management budgeting.

Additional functional developments, such as importing Excel LCC calculation sheets and facility status evaluations, would be required for the Web-based combined LCC analysis system proposed in this study.

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Early warning system of roofs overloaded by snow based on measurements and inverse analysis

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ABSTRACT

Gaining information on the loads acting on a structure based on the observation of the response, e.g. deformations, stresses, belongs to common inverse problems in engineering practice (Maincon, 2004). One of such inverse task is an identification of snow load distribution on roofs. It is a part of development of early warning system of roofs overloaded by snow which is a critical load case for light flat roofs. System gives online information about load distribution in every part of the roof during snow-fall, snow-drift, snow removal, etc. Such information is important for detection of roof overloading and its possible collapse. It also helps with the best strategy of snow removal for particular roof. Development of the system goes hand in hand with several light roof collapses in 2006 in Czech Republic due to extreme snow fall.

Early warning system uses inverse analysis for identification of snow distribution on the roof from monitored response of the structure (stresses, deformations). Here, methodology of inverse analysis based on artificial neural network and stochastic analysis using small sample simulation technique Latin Hypercube Sampling originally developed and used for material parameters identification (Novák & Lehký, 2006) has been implemented.

For automatic identification of actual load distribution on the roof within early warning system software tools Strecha_Mosnov and SVV has been developed. The first one is a command line application and serves as fast solver of structural response for various combinations of snow load in different parts of the roof. It is used for preparation of neural network training set. The second software tool is SVV program where artificial neural network as a key stone of inverse analysis is implemented (Fig. 1). Its aim is the real-time identification of snow load intensity in individual loading zones for given response of the structure obtained from sensors.

Verification of methodology and software tools has been carried out using testing hypothetic situations, i.e. snow removal and situation during snow drift and

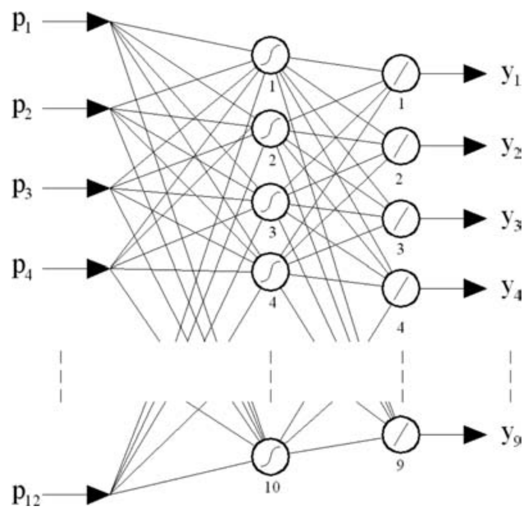

Figure 1. Scheme of artificial neural network implemented in early warning system.

ice-up of snow on the roof. From comparison of the real/original load and load identified from response of the roof can be concluded that proposed early warning system is able to successfully identify snow load distribution on the roof and to give information if critical load intensity in some part of the roof is exceeded. Proposed methodology can also help with design of optimal positions of measuring sensors within the framework of early warning system of newly built roofs.

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Total life-cycle assessment of steel constructions

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ABSTRACT

The assessment of the total service life-cycle of steel constructions will become much more important by the increasing meaning of building preservation and building modernization. The main cause for failure of existing structural steel constructions under cyclic loading is material fatigue. However the service life time of steel constructions is not limited by the appearance of the first crack initiation. Most steel constructions are failure tolerant. This behaviour can be considered for economic reasons by including the crack propagation phase into the assessment. The total life of steel constructions under the criteria of material fatigue results as sum of the life span up to the crack initiation plus the additional residual life time of crack propagation.

It is possible to compute the first part of the total life time using classical fatigue assessment concepts. For the following residual service life time, the principles of macro crack growth are valid. The determination of the crack propagation rate and the crack growth direction represents central questions. The task to answer these questions can be solved with methods of fracture mechanics.

For the determination of the residual service life time the phase of stable crack growth is considered. Termination criteria can be defined by reaching the required maximum number of load cycles, the ultimate limit load, exceeding of material fracture toughness, excessive deformation or the exceeding of permissible plastic strain rates.

Goal of the executed simulations is to give answers to some practical questions like optimization of inspection intervals or definition of the necessary maintenance and repair measures. Additionally these simulations provide data for the optimal scope of inspection of welded structures and give references to the appropriate selection of execution classes according EN 1090.

The contribution will present the developed FEM software tools and results of executed simulation of total lifetime. The presentation also includes results of accomplished computations on existing steel bridges and machinery parts of cable way structures.

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Study of the identification of aggregates of construction and demolition waste by using object recognition methods

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ABSTRACT

Construction and Demolition Waste (CDW) are the biggest waste flow in Germany. There was an amount of 84.5 million tons of CDW in the year 2008 (KWB 2011). The recycling rate amounts to about 80% (67 million tons). Regarding to application of C&D aggregates, most of them are used in road pavements and earth works, not really substituting the natural aggregate applications. Only a very small part of around 1% flows back in the production of recycling concrete.

The target is to overcome the down cycling scenario and the realization of real closed cycles and a high standard of quality in recycling, because the use of land will be critical in the future and land filling should be avoided.

The recycling industry of building materials is dominated by simple technologies. For instance the single-stage crushing is used with advance sieving and separation of reinforcement steel by over belt magnetic separator. For the processing of building materials sorting processes are only used for the separation of light components until now. These technologies are not able to separate the incidental mixed aggregates. They are suitable in no way for "new building materials" including connected building materials, which will use more and more in building industry.

In fact, the composition and physical properties of C&D aggregates are variable in a wide range. The heterogeneity prevents the profitable reuse. It is indispensable to separate the CDW mixtures to establish a reliable and demanding reuse. This is the basis for the development of specific products which based on the characteristic properties of the materials. And it is also the basis for the return of pure material as secondary material in the production of primary material.

This paper discusses the possibility of optical separation in the wavelength range of visible light for secondary aggregates of CDW. The aim is to develop an optical identification method for secondary aggregates to improve its quality and built the basis for an innovative sorting method on the field of processing of CDW.

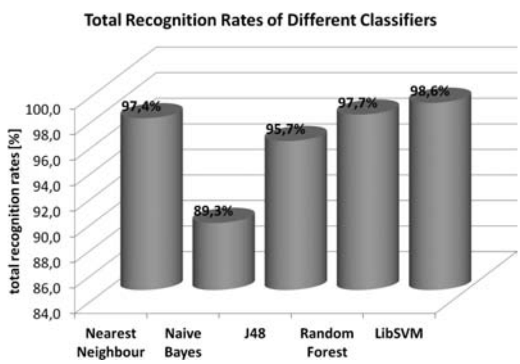

Figure 1. Classification performance of different classifiers by using the 56 of the best ranked features.

For the investigations images of the five aggregate classes (concrete, aerated concrete, lightweight concrete, dense brick, porous brick) were captured and analysed by algorithms of image processing and machine learning. A Principal Component Analysis (PCA) and a visualisation of object clusters in feature space was realised for obtaining a better understanding of the given recognition problem and the separability of the five classes by using 188 feature values. Different classification algorithms were tested on the given dataset and their results were shown.

The approaches demonstrated the SVM and Random Forest as the best classification algorithms for this recognition task. The parameter optimized LibSVM achieved a total recognition rate of 98.6% and the Random Forest classifier of 97.7% for the given dataset (Figure 1).

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LCC in Norway: State of the art 2012

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ABSTRACT

Since 1978, when LCC was first set on the agenda in Norway, Multiconsult has been an active part of the development of methodology, models and tools. Much has happened since 1978. The Norwegian Standard NS 3454 "Life-Cycle Costs for Building and Civil Work, Principles and Classification" came in 1988(revised in 2000), with both taxonomy and methodology for calculations(The Norwegian Standards Association, 2000.). After the publication of NS 3454 Multiconsult has published a number of guidelines regarding different aspects of life-cycle costs. The first, "Annual Costs – Calculation Guides" (Bjørberg, S. et al. 1993), gives information and guidance on the use of LCC-calculations during the process of planning, design, construction and use. Calculations are performed at three different levels depending on the purpose.

Later, in 2003, a series of new publications within the field of "Facility Management and Life-Cycle Planning" where published. In the LCC-field we got "LCC-Calculation Guide" (Bjørberg, S et al. 2003) and "Decomposition and Depreciation of Building Components" (Bjørberg, S et al. 2005). A revised law public procurement (2001) requires LCC considerations in the planning phase. Further in 2011 Multiconsult published a guideline for service life of building components, which is an important aspect in calculation of life-cycle costs(Bjørberg, S. et al. 2009).

In recent years several computer aided calculation tools for LCC has been developed. One of the most used in Norway now a day is a web based tool called LCCWeb, which is developed by the some of the largest public property owners in Norway; Statsbygg (governmental property manager and advisor) and The Norwegian Defence Estates Agency. This tool is based on the earlier excel based tool "LCProfit" (developed by Statsbygg).

Today Multiconsult is developing a new model for LCC calculations(Listerud, C. et al. 2012). The new model aims at calculating costs that corresponds with the building rent, development and depreciation. This is done by calculating LCC over a shorter period of time, which is more in line with the actual rate of reconstruction and renewal of technical installations, than the more traditional 40–60 years calculation period.

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Life-cycle cost management mode based on information management platform

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ABSTRACT

This paper combines the cost management with information management technology. Its purpose is to establish the index and parameters, to establish a life-cycle cost management mode of construction projects based on information management platform, and to realize the life-cycle cost information of project communication and cooperation between each party that joined in, through the analysis of the project cost information flow. What the innovation of the essay is that an information management platform of engineering is created, through which we can solve the sharing problem of project cost information. Due to the dynamic management of project cost information with full staff, whole process and all directions, we can realize the whole system goal of construction projects, which is superior quality, saving cost and reasonable period.

Based on existing domestic research, we learn advanced experiences and management techniques from foreign countries and connect them with China's actual conditions, then establish a management model for the whole life-cycle on project cost which is based on the platform of information management. This provides some theoretical basis and technical support for China's information construction on project cost.

With the life-cycle cost information platform for communication and cooperation, we can guarantee the shortcut, promptness and unobstructed of project cost information's transfer, and each project participant is able to exchange information and work collaboratively, to ensure the coordination and communication of project cost information in time, to promote the transparency of the construction project cost management, and is able to know the whole spending of the engineering project. What's more, with the platform, problems in investment and cost control can be found easily. The general objective and sub-objectives of project cost can be implemented and controlled, so we can guarantee that the life-cycle cost of construction projects is superior, that the waste of investment is reduced, and that the benefit of investment and society is enhanced. Therefore, there is a wide application prospect for the life-cycle cost management mode based on information management platform.

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Probabilistic aspects of Offshore Wind Turbines: Influences of in situ assembly of grouted joints

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ABSTRACT

In recent years, the wind energy industry has been engaged in expanding the commercial running of offshore wind farms. There are plans to build more than 8,000 Offshore Wind Turbines (OWTs) in the North Sea and the Baltic Sea. Out of these, 2,000 OWTs have already been approved for erection, while the others are going through the approval process. Because of this large number of wind farms, application of probabilistic assessments and life-cycle concepts are worth discussing. The existing limited experience in life-cycle engineering, especially with regard to this new form of power plant, is in intense contrast to the planned lifetime of 20 years under the harsh offshore conditions.

OWTs are structures which are segmentally prefabricated onshore and later assembled offshore. Contrary to the onshore manufacturing, the offshore assembly of OWTs is much more complex. The so-called grouted joint is normally used to connect the foundation and the supporting structure. Grouted joints are pile-sleeve connections where the annular gap is filled with high-performance grout or concrete. Different types of grouted joints are used in different positions exposed to the seawater depending on the supporting structure (Figure 1).

Furthermore, no reliable information is available concerning the in situ material properties of grout in grouted joints which is worthwhile for probabilistic analysis and the life-cycle concepts of wind farms.

Taking these facts into consideration, the Institute of Building Materials Science has developed a special transparent formwork to simulate the material behavior in the annular gap during the application process of grouted joints. This formwork is 1.5 m high, 0.6 m wide and has a flexible gap. Moreover, the transparent front panel offers the possibility of an optical evaluation of the filling process. Further objects, such as shear keys, can be added to the back and front panel.

Grouted joint test-specimens will be produced with this formwork to predict the influence of the in situ assembly on the grout material properties. After the hardening of the grout material, samples will be taken from these hardened, grouted joint test-specimens to determine the density of the hardened grout, compressive strength, tensile strength, and stability against segregation. These material properties will be compared with standard test samples and reference grouted joint test-specimens.

The influence of the grout application method will also be considered, as well as the influence of the geometry of shear keys and the width of the gap of the in situ material properties of the grout.

Furthermore, a large-scale test facility for grouted joints is under construction for full-scale grouting tests.

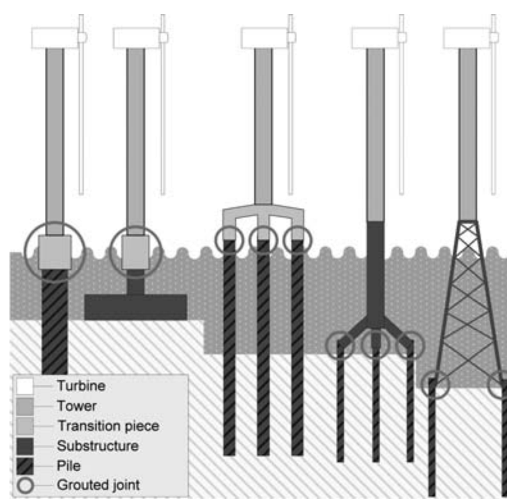

Figure 1. Supporting structures, from left to right: monopile, gravity based foundation, tripile, tripod, jacket.

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An optimum design approach for the wind and seismic design of wind turbine supports in Mexico

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1 INTRODUCTION

This paper presents results of a recent study concerning the optimum design for wind and seismic hazards in Mexico of wind turbines supports. A well establish optimum design procedure for estimating optimum base bending moment due to earthquake and wind hazards is outlined. This procedure is based on the minimization of the total cost due to the initial cost and the cost of losses when structural failure occurs during its life. It is required a realistic description of the construction cost in terms of the intensity, as well as a complete and accurate description of the hazard given by excedance rates curves. Utilizing this optimum design model and characterization of wind and seismic hazards in Mexico through their excedance rates of maximum gust wind speeds and peak ground accelerations, optimum base bending moments have been quantified and presented in contour maps for Mexico.

2 OPTIMIZATION OF TOTAL COST FOR WIND AND EARTHQUAKE HAZARDS

Considering the wind and seismic hazards, the total cost, normalized with respect to C_0, is given by the sum of the initial cost, *CI(M)*, and the present value of the expectation of losses. The following expression represents the total cost as the objective function to be optimized:

$$\frac{C_T(M)}{C_0} = \frac{CI(M)}{C_0}\left(1 + (1 + S_L)\frac{\lambda_w(M) + \lambda_s(M)}{\gamma}\right) \quad (1)$$

where C_0 = the cost of the structure corresponding to the gravitational load design, $\lambda_w(M)$ and $\lambda_s(M)$ = the excedance rate of wind and seismic hazards, respectively, and γ = the discount annual's net rate of the value of money which is normally taken equal to 0.05. S_L = a factor that measures the importance of loss structures.

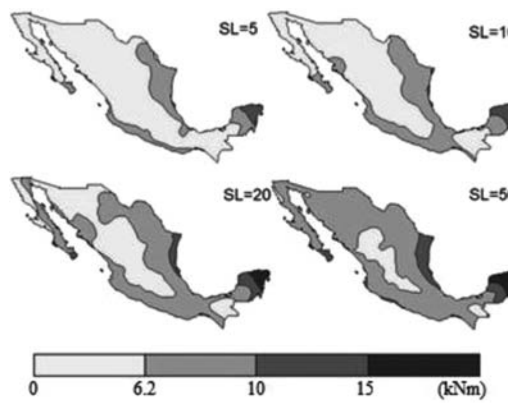

Figure 1. Optimal base bending moment considering aeolian and seismic actions on wind turbines for different structural importance values, S_L.

From a multivariate analysis, an optimum base bending moment map has been elaborated. In Figure 1, it is shown a map corresponding to both hazards, for the case of Mexico and for different values of the important factor S_L.

3 CONCLUSIONS

For this study, the selected structures were wind turbines supported by steel towers. Since the design of these structures is controlled by flexocompression, the intensities are described in terms of the base bending moment. The first result is a collection of maps of optimal base bending moment, which takes into account that the parameter that controls the safety level are the consequences related to the cost of losses, or in other words, the importance of the structure, called S_L. Four values of structural importance were analyzed which explain that the importance of the structure depends on the cost of loss of structures.

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A comparative analysis of environmental impacts of ordinary concrete and structural lightweight concrete

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ABSTRACT

A comparative life-cycle impact assessment of lightweight concrete and normal concrete beams is presented in this paper.

First part of this paper presents the results of experimental research of the basic properties of lightweight aggregate concrete. Two pairs of concretes were made for comparison. In each pair, both normal and lightweight concretes were designed with equal compressive strength and same workability (Table 1).

Second part is dedicated to comparative analysis of environmental impacts. In order to compare environmental impacts of observed types of concrete, Life-Cycle Impacts Assessment (LCIA) analysis was performed. Analyzed environmental impact categories are: Global Warming Potential (GWP), Eutrophication Potential (EP), Acidification Potential (AP) and Photochemical Ozone Creation Potential (POCP).

A precast concrete beam with defined span and loads was chosen as a functional unit.

After comparing overall impacts with analyzed impact categories (Figure 1.), it is clear that lightweight concrete beams have higher impacts.

Based on the presented results, it can be concluded that the use of structural lightweight concrete in structural members loaded primarily in bending, in terms of categories of the observed effects, is not justified.

Table 1. Concrete mix composition.

Concrete mix	LC1	LC2	NC1	NC2
CEM I 42.5R	450	400	350	300
Water	180	180	180	180
Additional water	15.3	15,6	–	–
River aggregate 0/4 mm	940	955	685	694
River aggregate 4–16 mm	–	–	1271	1288
Leca–Laterlite 4–15 mm	333	339	–	–
Chemical admixture SIKA VSC 4000BP	3.15	2.4	5.2	4.6

*All values are in kg per m^3 of concrete.

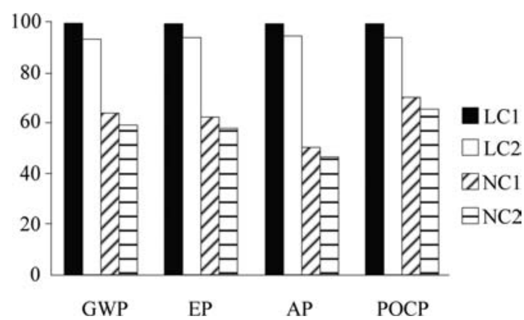

Figure 1. Comparison of beams for all impact categories in percents.

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Artificial neural networks in calibration of nonlinear models

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ABSTRACT

Development in numerical modelling provides a possibility to describe a lot of complex phenomena in material or structural behaviour. The resulting models are, however, often highly nonlinear and defined by many parameters, which have to be estimated so as to properly describe the investigated system and its behaviour. The aim of the model calibration is thus to rediscover unknown parameters knowing the experimentally obtained response of a system to the given excitations. The principal difficulty of model calibration is related to the fact that while the numerical model of an experiment represents a well-defined mapping from input (model, material, structural, or other) parameters to output (structural response), there is no guarantee that the inverse relation even exists.

The most broadly used approach to parameter identification is usually done by means of an error minimisation technique, where the distance between parametrised model predictions and observed data is minimised (Stavroulakis, Bolzon, Waszczyszyn, & Ziemianski 2003). Since the inverse relation (mapping of model outputs to its inputs) is often ill-posed, the error minimisation technique leads to a difficult optimization problem, which is highly nonlinear and multi-modal. Therefore, the choice of an appropriate identification strategy is not trivial.

Within the several last decades, a lot of attention was paid to the so-called intelligent methods of information processing and among them especially to soft computing methods such as Artificial Neural Networks (ANNs) and evolutionary algorithms. A review of soft computing methods for parameter identification can be found e.g. in (Kučerová 2007).

The ANNs can be employed for model calibration in two different scenarios. In a forward mode/direction, the ANN is applied to approximate the model response. The error minimization technique then becomes a minimisation of a distance between the ANN's predictions and experimental data. The advantage of this strategy is that the ANN is used to approximate a known mapping which certainly exists and is well-posed. On the other hand, the important shortcoming of this method concerns an inevitable minimisation process, which remains often ill-posed and needs to be solved repeatedly for any new experimental measurement. This way of the ANN application to parameter identification was presented e.g. in (Aguir, BelHadjSalah, & Hambli 2011).

The second philosophy, an inverse mode, assumes the existence of an inverse relationship between outputs and inputs. If such a relationship exists at least on a specified domain of parameters' value, it can be approximated by an ANN. Then the retrieval of desired inputs is a matter of seconds and could be easily executed repeatedly for any new experiment and no other optimization process is necessary. Here the ANN training represents the whole computational costs and a solution of the ill-posed problem. See e.g. (Novák & Lehký 2006) as the inverse mode example.

In this contribution, the advantages and disadvantages of the two modes of ANNs application are demonstrated in details on calibration of the affinity hydration model (Šmilauer 2012).

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Probabilistic assessment of working life for bridges

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ABSTRACT

An extended use of existing structures is of a great importance for many countries. It has significant economic, social and cultural impacts. Verification of the serviceability limit states of an existing bridge is commonly based on estimation of its remaining working life. Current European and international standards give general recommendations only. Recently developed Czech national standard CSN 73 6222 (2008) provides supplementary, currently in Eurocodes not given guidance for determination of the load-bearing capacity and for estimation of the remaining working life of existing concrete bridges. Several bridge categories of prestressed and reinforced concrete bridges are distinguished and limiting values of crack width are there recommended.

The probabilistic methods are applied for verification of the reliability level and remaining working life of the bridge with respect to the serviceability limit states of crack width. The limit state function $g(.)$ is expressed in terms of the limit value of the crack width w_{lim} and the random crack width $w(.)$ calculated under the quasi-permanent combination of actions and theoretical models given as

$$g(\boldsymbol{X},t) = \xi_{\lim}\, w_{\lim} - \xi_w\, w(\boldsymbol{X},t) \tag{1}$$

where X = the vector of basic variables, $\xi_{\lim}$ and ξ_w = the coefficients of model uncertainties for the requirements on the crack width limit and the crack width model, respectively and t = considered time.

The probability P_F of a random crack width exceeding the crack width limit $w_{\lim}$ for the time dependent problem may be assessed as

$$P_{\mathrm{F}}(\boldsymbol{X}, t) = P\{\xi_{\lim}\, w_{\lim} - \xi_w\, w(\boldsymbol{X}, t) < 0\} \tag{2}$$

The bridge may be considered as reliable if the condition $P_F \le P_{Ft}$ is satisfied where the probability of failure P_{Ft} is the specified (target) value that should not be exceeded during the design working life. The crack width models recommended in the Eurocode EN 1990 (2002) and Model Code (2010) are taken into account. The reliability analysis of a reinforced concrete bridge with respect to the serviceability limit states of crack width indicates that the uniform corrosion leads to a smaller reduction of the reinforcement area and higher reliability indices than the pitting corrosion.

The results of probabilistic analysis of a selected deteriorating reinforced concrete bridge indicate that its reliability after the first half of bridge working life may be rather low ($\beta < 1{,}3$). Thus, to achieve the recommended target reliability level during the whole working life of the bridge, additional provisions need to be accepted in the design (e.g. increase of reinforcement cover, acceptance of protective measures).

The application of crack width model of Model Code (2010) for the assessment of bridge remaining working life leads to more strict requirements than the theoretical model provided in Eurocodes.

The serviceability constrains recommended for the assessment of the remaining working life of a bridge in current prescriptive documents should be further analyzed and calibrated. The type of corrosion (uniform, pitting) and potential consequences of failure should be taken into account.

It appears that the probabilistic assessment of existing bridges may facilitate the optimum decision regarding their safety and serviceability, and indirectly contribute to a sustainable development.

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Probabilistic working life assessment of power-producing components

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ABSTRACT

The paper is focused on probabilistic durability assessment of existing structural components of power producing facilities in hydroelectric power plant and estimation of their remaining working life.

Presently hydrotechnical construction works are not given in the scope of current generation of Eurocodes and supplementary national provisions are needed including their reliability classification.

Application of probabilistic methods for specification of working life of an existing structure is illustrated in Figure 1. It is assumed that the assessment (inspection) of an existing component of a power producing facility is performed in time t_{pr} from the beginning of the structure completion. In case that the time-dependent resistance $R(t)$ of a component and load effects $E(t)$ are known, the remaining working life of the component may be specified.

For estimation of the residual working life t_{res} of the component, the following expression is given as

$$P_f(t_{res}) = \mathrm{P}\{R(t_{res}) - E(t_{res}) < 0\} \approx P_{f,t} \qquad (1)$$

facilitating decision about its repair or replacing.

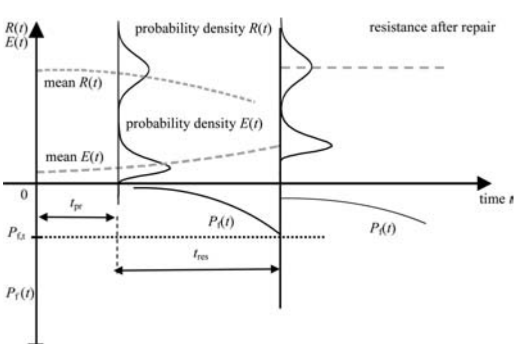

Figure 1. Probabilistic assessment of remaining working life.

General requirements for the probabilistic assessment of structural serviceability are applied for the reliability analysis of quick-closing valves. The steel components having age of about 50 years are gradually deteriorating due to non-uniform corrosion.

Initial reliability of the cover plate non-affected by corrosion (reliability index $\beta = 5.2$) satisfies the target reliability level for considered reference period of 80 years.

It appears that the working life of the component may be estimated to approximately 70 years when the reliability index $\beta(t)$ decreases to the target reliability level. Thus, the remaining working life of a structural component may be estimated to be about further 20 years. When the reliability index approaches the target reliability level, a new reliability assessment of the component should be made on the basis of updated material characteristics and corrosion models to decide about repair or replacement.

The probabilistic assessment of existing structures makes it possible to effectively estimate remaining working life of structures and to plan their maintenance and required economic resources.

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The influence of bridge maintenance on their durability and bearing capacity

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ABSTRACT

Maintenance of bridges is key factor in terms of their durability and capacity. This can be drawn from previous projects for repairing and strengthening bridges in Macedonia. Many years of inadequate or absence of maintenance have questioned their usefulness. In combination with the modern heavier vehicles, their bearing capacity was also seriously threatened. Having in mind that during the actual design of the bridges seismic force is neglected, need for rehabilitation is necessary. Serious damages were established on all of the bridges.

A long term cooperation between the Faculty of Civil Engineering in Skopje and NATO-NAMSA project for "Strengthening and repair of bridges and roads in the Republic of Macedonia" last for many years. In order to determine the present condition and the bearing capacity of the bridges, an expert group has performed a detailed inspection of nearly 100 bridges on few national road sections. Serious damages due to the inappropriate and not efficient maintenance were established on all the bridges.

Over the past years, in collaboration with NATO, a project for repair and strengthening of 50 bridges was performed. A large portion of them have had inadequate and poorly maintained drainage system, which later caused serious damage to the structure from the influence of atmospheric water. At some bridges the damages were so big that even the bearing capacity of the bridges and the safety of the traffic were suspicious.

The analyses were made and it was concluded that the bridges have not capacity for carrying the loads defined in the present national standards, as well as European and NATO standards. Therefore the country could be excluded from the international routes. After the priority list was made, the project was divided in three phases:

1. LOT1: Strengthening and repair of 30 bridges on section Blace-Skopje-Veles
2. LOT2: Strengthening and repair of 20 bridges on section Negotino-Udovo-Bogorodica
3. LOT3: Strengthening and repair of 15 bridges on section Veles-Skopje and reconstruction of carriageways.

After determining the necessity of strengthening the characteristic cross sections of all structural members separately, the strengthening methods have been chosen. It should be pointed out that different members are strengthened with different measures, as a combination of some of the following measures:

- increasing the concrete cross sections;
- increasing the cross section of the reinforcement;
- strengthening with carbon strips

According to the results of the design analysis, performed testing, as well as the experience of the construction works, following conclusions can be drawn:

- Adopted system ensures effective, relatively simple strengthening of the bridge elements and it is not time consuming.
- The strengthened bridge can bear the new increased design loads with sufficient reliability.
- The undertaken measures for bridge repairing should ensure efficient protection of the inbuilt materials from the aggressive atmospheric influences.

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An advanced probabilistic updating algorithm for life-cycle analysis of civil structures

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ABSTRACT

An advanced methodology for reliability analysis of structures, which considers the updating of the structural resistance curve, is presented in this paper. The objective is to introduce such methodology into a probabilistic Life-Cycle Analysis (LCA) framework. Accordingly, this methodology is validated with a set of reinforced concrete beams tested up to failure in laboratory (Matos et al. 2010).

This methodology is developed in MatLab®, according to Figure 1, being used one random model generation module, based in Iman and Conover algorithm (Iman & Conover 1982), one nonlinear finite element analysis module for processing these models, in ATENA® (Cervenka et al. 2009), and a module for statistical analysis of obtained results. Additionally, one probabilistic updating module, based in a Bayesian inference algorithm, is introduced through WinBugs® (Lunn et al. 2009).

The main conclusions of this paper are: (1) the developed methodology is validated and it can be easily applied to a real structure; (2) the inference procedure results in one improvement on the reliability of the probabilistic model (Table 1); (3) the probabilistic updating permit to identify a capacity reserve of tested beam, due to an increase on reinforcing steel quality, which was not considered in design (Table 1); (4) resistance curves were respectively updated with acquired data, minimizing the existent gap between the moment data is collected and the one in which it is included in the model; (5) the methodology can be easily introduced into a probabilistic LCA framework.

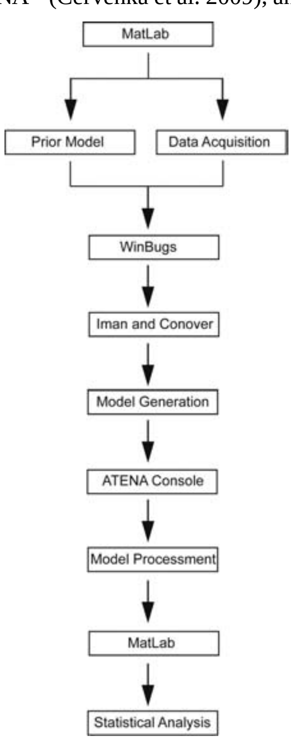

Figure 1. Organization chart of developed methodology.

Table 1. Failure load.

Numerical model	μ [kN]	σ [kN]	Index-p [%]
Without Bayesian inference	28.49	3.79	94.66
With Bayesian inference	31.69	4.19	96.96

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Life-cycle cost analysis for real estate

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ABSTRACT

Sustainable real estate management encompasses investment decisions based on the understanding of Life-Cycle Costing. The life-cycle cost analysis of a property therefore replaces the sole optimization of investment costs.

The International Facility Management Association Switzerland (IFMA) and the German Facility Management Association (GEFMA) have developed a life-cycle cost model for real estate. The documentation consists of a base model and a calculation aid based on Excel. Professor Dr. Andrea Pelzeter from the Berlin School of Economics and Law (HWR) gave the scientific support for this cost model.

Life-cycle costs encompass all costs from project development to deconstruction. The model includes the cost for: construction, management, insurance, utilities and disposal, cleaning, grounds, inspection, maintenance, reconditioning and replacement. The calculation of the life-cycle costs is based on the net present value method and considers calculation parameters like the calculatory interest rate and specific inflation rates.

The earlier in the stage of a design of a building the greatest the influence on life-cycle costs (Figure 1). The calculation tool gives the opportunity to define and optimize life-cycle costs of different project types during three different design phases. A systematic approach of phase-appropriate input data allows comparative results based on the different depth of information in each phase. The calculation tool enables the user to identify levers which have an influence on life-cycle costs of buildings. Essential levers are investment costs, flexibility for different space and user needs, logistical and structural requirements for cleaning, basic and reasonable mechanical systems in combination with easy accessibility and maintainability, long service lives of building components and a high energy standard.

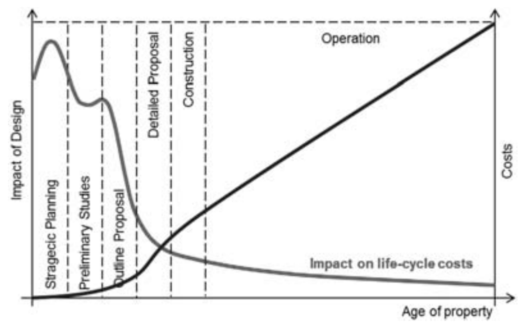

Figure 1. Impact on life-cycle costs.

Life-Cycle and Sustainability of Civil Infrastructure Systems – Strauss, Frangopol & Bergmeister (Eds)

Simulation of random behavior of engineering structures: From parameters identification to reliability assessment

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ABSTRACT

The paper presents a complex methodology for statistical and reliability analyses of engineering structures. It describes the virtual simulation concept and tools used on the way from assessment of experimental results to reliability analysis. The methodology is represented by consequent steps starting by material parameters identification based on artificial neural networks and finite element modelling. The aim is finally to perform advanced reliability assessment.The attention is given to those techniques that are developed for analyses of computationally intensive problems like nonlinear FEM. Sensitivity analysis is based on nonparametric rank-order correlation. Statistical correlation is imposed by the simulated annealing.

As FReET software development is performed in a complex project and system for reliability assessment of concrete structures SIMSOFT, the full role of software FReET will be also shortly described – including degradation module FReET-D and methodology for inverse analysis and identification.

The paper presents briefly a complex methodology for statistical, reliability and risk analyses of concrete structures. It starts by material parameters identification based on soft computing – artificial neural network. The methodology is valid generally, not only for concrete structures. It describes the virtual simulation concept and tool based on soft computing approaches. The whole approach is based on small-sample randomization of nonlinear fracture mechanics finite element analysis of reinforced concrete structures. Efficient techniques of both nonlinear numerical analysis of concrete structures and stochastic simulation methods have been combined in order to offer an advanced tool for assessment of realistic behaviour of concrete structures from reliability and risk points of view.

The stochastic response requires repeated analyses of the structure with stochastic input parameters, which reflects randomness and uncertainties in the input values. The system uses the nonlinear computer simulation for realistic prediction of structural response and its resistance. As the nonlinear structural analysis is computationally very demanding, a suitable technique of statistical sampling should be utilized, which allows relatively small number of simulation. Final results are: statistical characteristics of response (stresses, deflections, crack width etc.), information on dominating and non-dominating variables (sensitivity analysis) and estimation of reliability using reliability index and theoretical failure probability.

Software tools developed are: SARA – a software shell which controls the communication between following individual programs: ATENA (Cervenka et al. 2007) – FEM nonlinear analysis of concrete structures; FReET (Novák et al. 2011) – the probabilistic engine based on LHS simulation; DLNNET – artificial neural network software; FReET-D (Teplý et al. 2010) – degradation module.

Finally, the paper documents two examples of successful application of this methodology and software tools for life-cycle assessment of concrete structures: bridge and fiber-reinforced concrete facade panels.

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Fatigue behavior of stay cables

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ABSTRACT

Fatigue tests of stay cable systems are one of the basic prerequisites for the construction of new cable stayed bridges. In a research project, currently carried out at Vienna University of Technology, it is investigated if different testing parameters influence the fatigue strength. Various tensioning systems are being tested with frequencies ranging from 6 to 26 Hz. The two-year research project which started 2010, consists of numerous fatigue tests with different sizes of anchorages (1, 7, 12, 19, 31, 43 strands). To obtain more information about the influence of the testing frequencies on the fatigue behaviour of stay cables, the test program was designed to test the same anchorages and prestressing steels with frequencies under 8 Hz and later with frequencies up to 26 Hz. The principle purpose is to get data of the impact of stress range, load cycles and upper load on the fatigue strength of tendons.

Parallel to the experimental tests numerical simulations of tension members will be carried out. The ambitious aim of the numerical simulation is the composition of models offering the possibility to optimize stay cable and prestressed systems. Another great advantage is the opportunity to simulate tests on large specimens under conditions close to the real loading condition of a structure during its lifetime: different mean stress levels, variable stress amplitudes and various number of load cycles. In addition to this, supplementary material screenings with a scanning electron microscope are planned to explore processes at the fatigue fracture and the occurring failure mechanism.

The first interpretations of the experimental tests showed that there are no influences of the test frequency and the load cycles (up to 10 million) detectable for frequencies up to 26 Hz. Because of the limited number of tests carried out so far, all results only show evidence of the influence of different parameters. Therefore, upcoming tests have to confirm the first results of these tests. At least 32 experiments with mono-anchorages, six tests with anchorages for 7 strands and two tests with 19 strands are planned for the year 2012. To obtain reliable results, a high number of tests with equal parameters are necessary. Based on the results of the experimental and numerical tests, further development of the tendons should be possible. Such a development could be an improvement of the fatigue resistance and therefore lead to a more economic design of the components. For the infrastructure operator the research projects provides the opportunity to evaluate existing bridges and prestressing structures and thereby save maintenance costs in regard to new and existing structures.

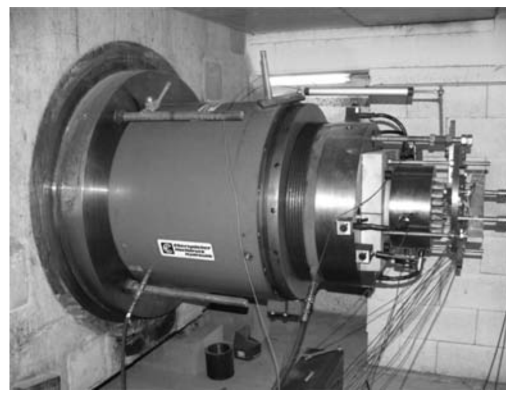

Figure 1. Hydraulic jack with a stay cable anchorage for 43 strands.

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Reliability index for wind turbines subjected to wind and seismic actions

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ABSTRACT

Mexico is frequently hit by moderate and strong hurricanes and earthquakes. Specific maps for seismic and aeolian design intensities are based on different criteria. In the last years, wind turbines have been widely accepted in Mexico and it is expected to reach major diffusion in the next years. Several criteria have been explored to choose the best design intensity for both wind and seismic actions.

Although wind and seismic hazard distributions in Mexico has been well studied, structural failures are still occurring mainly due to hurricane winds and earthquakes.

The procedures to compute wind and seismic hazard are described. The relations between acting moment with regional wind velocity and acting moment with spectral seismic acceleration were obtained for a typical wind turbine at an aeolian farm administrated by Mexican Electrical Utility (CFE, as per Spanish abbreviation).

Afterwards, exceedance rates corresponding to the acting bending moment due to earthquake, wind and the combined effect of both actions are determined. Total excedance rate of the acting bending moment is calculated for wind and earthquake acting separately (Figure 1).

Two criteria to select the design intensity have been studied: constant resistance in all the country and optimum design criterion. Several contour maps are displayed (e.g. Figure 2) in order to show the variation of these indexes in all the country. Finally, design intensities related to different constant failure rates are shown. A discussion of the pros and cons of each design criterion is presented.

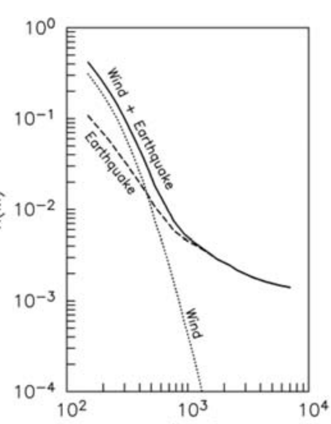

Figure 1. Exceedance rates of the bending moment acting in a wind turbine steel supporting structure, produced by wind, earthquake and both actions.

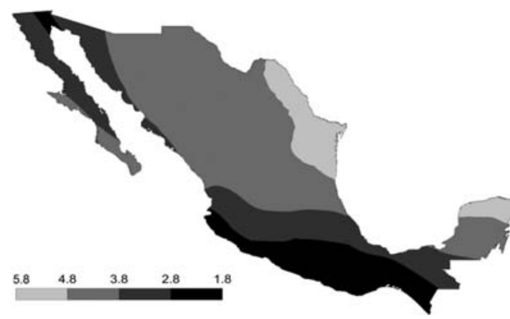

Figure 2. Map of reliability indexes considering seismic action on wind turbine studied for $S_L = 10$.

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An energy harvesting application in a long span suspension bridge

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ABSTRACT

In the last years, Energy Harvesting (EH), i.e. the process of extracting energy from the environment or from a surrounding system and converting it to useable electrical energy, is a key research topic in applications in structures and infrastructures, and focuses on different aspects. A first classification can be made with regard to the principal object of study and the associated scale, as it would be classified following a multilevel point of view. Under this philosophy, three main research groups can be individuated: i) research focusing on structural and non-structural (device-constitutive) materials for EH. This research belongs to a micro-scale level; ii) re-search focusing on devices for EH, in a meso-scale level and, iii) research focusing on structural systems, in a macro-scale level. The last group is further subdivided in: iii-a) research finalized to the optimal coupling of conventional and well-known structural systems with devices for EH and, iii-b) research finalized to the conception of non-conventional or innovative structural systems for EH. Focusing on the research field (iii-a), efforts can be devoted in the coupling of proper devices with flexible structural systems in order to extract energy from vibrations induced by external (environmental and traffic) loads. In this view, promising structural typologies are auxiliary systems (e.g. road lights or information panels).

Considering the above, a novel device for the vibration energy harvesting, based on piezoelectric material, is modeled in a commercial FEM (Finite Element Method) code, in order to optimally extract energy from vibrations induced by external loads. A long span suspension bridge serves as case study, presenting some exceptional characteristics (main span of 3300 m, and the length of the 60 m wide deck from side to side, including the side spans, of 3666 m).

The efficiency of the devise is tested by placing it in the attachment between a hanger rope close to the bridge mid-span (Figure 1) and the bridge deck (Figure 2).

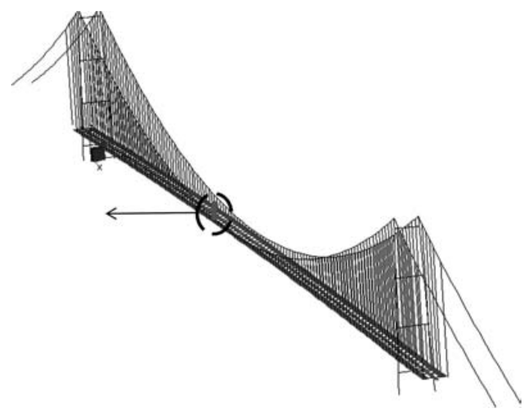

Figure 1. FE model of the bridge and positioning of the EH device.

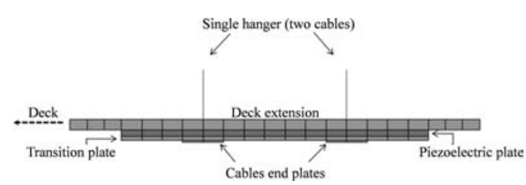

Figure 2. Position of the EH device on the bridge.

A 3D Finite Element model of the entire bridge is implemented, in which the total number of elements (beams, no compression cable elements and gaps) is 1614, and the number of nodes is 1140. A large displacement finite element formulation has been adopted in order to take into account the geometric non-linearity and a Newmark time integration scheme has been adopted for the developed transient step by step analysis. The energy production is inquired for different loading conditions (wind and train loading), and numerical results are presented in terms of input displacements, generated voltage, obtained power and stress on the piezoelectric material.

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Coupled damage in assessing the lifetime of bridge and viaducts

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ABSTRACT

Exceptional and cyclic external actions on structures affect the structural performance of bridges. As the prediction of the remaining life for all deteriorated structures is a key issue to maintain a safe network, the development of assessment models to evaluate structural changes in these structures are needed. Fatigue, seismic, and other concurring minor damages as corrosion need to be investigated in coupled manner. In this paper, damage causing cracking or even failure is analyzed together with different scenarios of propagation. Prediction of the remaining life for a real structure is illustrated.

A peculiar characteristic of seismic loading is the application of a few cycles of high cyclic strain that could exceed the material yield strength. These strain cycles may cause small fatigue cracks at a notched location, which reduces the fatigue strength (Kondo 2005). However the continuing use of these structures, their fatigue strength may be reduced below the pre-earthquake level. The objective of this study is to produce a preliminary and simplified method in order to evaluate the coupled damage of welded steel girders that could experience the seismic loading. A second detrimental action has been considered, corrosion, by modeling different propagation rate scenario. This paper deals with the coupled damage assessment of bridge and viaducts, considering seismic, fatigue and corrosion issues, by using the LEFM (Linear Elastic Fracture Mechanics) approach, implementing a finite strip FEM analysis. The numerical example have shown the capability of the proposed method. The results reveal that the fatigue reliability of a steel girder bridge needs to be integrated to other concurring damages that could affect bridge and viaducts during their lifetime, such as the seismic and the corrosion damage. Conversely, the realistic damage scenario is not assumed, and the assessment could be considered to be partial. The case study is represented by a composite steel-concrete on steel-girder bridge with cover plates. The LEFM approach is used to evaluate the fatigue reliability of an end weld of the girder cover plate. The principle girders are shown in Figure 1, while loading condition have been modeled according to Pipinato et al. (2011).

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The influence of design life in life-cycle civil engineering

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ABSTRACT

Some structural engineering design standards include requirements that vary according to the nominal design life of a structure. The intention is that structures should be designed so that they are suitable for their intended use during their design working life. Accordingly, the International Standard ISO 2394:1998, General principles on reliability for structures, states that "in particular, they shall fulfil, with appropriate degrees of reliability, the following objectives: (a) they shall perform adequately under all expected actions; (b) they shall withstand both extreme actions and frequently repeated actions occurring during their construction and use".

The stated objectives relate to both structural serviceability and structural safety, and they refer to actions occurring during the design life of the structure. Accordingly, some design standards specify design loads corresponding to return periods that are roughly proportional to the design life.

The specification of design loads that vary with the design life might appear to be consistent with the principles of life-cycle engineering, but the performance requirements for structural serviceability are often related to the frequency of serviceability 'outages' (independent of the design life), and requirements for personal life-safety should not be dependent on the design life of the structure. Some design requirements relevant to the minimization of life-cycle costs (accounting for maintenance, damage, lifetime reliability and failure costs) might justify the use of design loads that vary with the design life, but a rational basis for varying the relevant design loads needs to be established.

Structural design standards should account for the design life with regard to possible strength degradation, and for lifetime reliability the design loads should be based on the relevant distributions of anticipated loads (for extreme load events and fatigue).

The usual target values of the structural reliability index β_t derived from code calibration studies for typical structures (for a design life of 50 years) may be taken to satisfy the typical requirements for both structural reliability and life-safety. The calibrated values of β_t may be taken to be appropriate in relation to the lifetime reliability for any design life, but for a structure with a design life of less than 50 years the target value of the lifetime reliability might be further constrained by life-safety requirements which should be evaluated with regard to the potential consequences of a structural failure.

The Australian/New Zealand Standard AS/NZS 1170 has been discussed as an example of current design practice based on the design life of a structure. In this Standard the ultimate limit state design loads for natural hazards (wind, snow and earthquake) are defined in relation to the return period of the design load, dependent on the importance and design life of the structure. The specified return period of the design load is roughly proportional to the design life, and the specified return period of the design load for a 'major' structure is typically twice the period for a 'normal' structure. The influence of these design loads has been evaluated with regard to the resultant values of the structural reliability index for structures subjected to wind loading in accordance with the Standard.

It has been shown that these design loads do not give consistent levels of structural reliability, and the reliability levels for 'normal' and 'major' structures are not significantly different. Furthermore, it was found that the structural reliability levels achieved in practice are determined primarily by a load-effect factor which acts as a de-facto load (safety) factor, and variations of the design load return periods have only a minor effect.

Therefore it is concluded that it may be appropriate to define characteristic values of design loads based on specified return periods that are proportional to the design life, but appropriate load (safety) factors must be used in order to achieve appropriate levels of structural reliability.

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Extension of bridge lifetime by use of structural health monitoring

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ABSTRACT

The history of Structural Health Monitoring applications to engineering structures is very long but the real breakthrough of this technology is still missing. Today there is a big potential for application of Monitoring systems to bridge structures and the bridge owners will have big advantages by use of this technology. The important point is to convince our national authorities and to ensure a confidence by the use of Structural Health Monitoring.

Very good examples for the successful application of Structural Health Monitoring Systems are the projects conducted by RED Bernard for the Provincial Government of Styria where monitoring systems were applied to extremely slender bridges, see picture of such a bridge in Figure 1.

In 2005 RED Bernard was introduced to available problems with three pre-stressed concrete bridges in Styria. The actual problems of these bridges are given by a large vertical deflection of the midspan in static equilibrium position and by tie bars which are clamped to the foundation and covered with soil. Hence, a visual inspection of these tie bars is not possible and in case of a failure of the tie bars the bridge is endangered to collapse, see Figure 2.

Due to the fact that it was observed that the vertical deflection increases with bridge lifetime and that the soil covered tie bars are not visual visitable the Government of Styria thought of two options: (1) Demolish the bridge and built it new which is of course very cost effective and (2) Look for other possibilities to extend the bridge lifetime for additional 25–30 years.

RED Bernard was contracted by the State of Styria to look for the most economical method for bridge maintenance. After analyzing the possible collapse scenarios by numerical simulations RED Bernard suggested the application of a permanent Structural Health Monitoring where different types of sensors measure the ongoing deformation process and the possible failure of the soil covered tie bars. The Monitoring System was attached in the end of 2006 and since January 2007 the permanent measurements were recorded and displayed on an Internet Website. A developed alarm plan ensures that in case where critical limit states are exceeded the bridge will be closed for traffic.

Figure 1. Bridge structure in Styria attached with a Structural Health Monitoring conducted by RED Bernard.

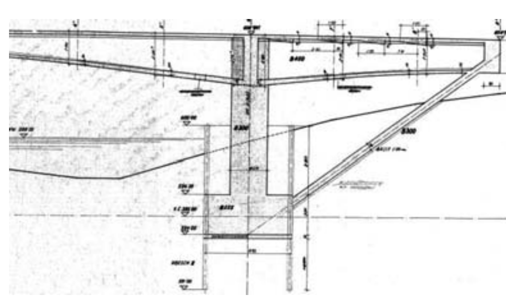

Figure 2. Tie bars clamped to foundation and covered with soil.

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Estimation of asphalt pavement performance by using risk analysis method

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ABSTRACT

Asphalt pavement performance during use depends notably on the quality of the used material, on the composition of the asphalt itself, on the circumstances while laying and compacting the asphalt layer, and, of course, on the traffic load and weather conditions while using.

In order to enable estimation of probabilistic asphalt pavement performance and thus the pavement's serviceable life with consideration of all relevant influencing parameters as well as their variation and uncertainty respectively, a new approach was developed (Rosauer 2010). This approach bases on Darmstadt Risk Analysis Method (DRAM), which permits a relatively easy handling and – supported by a special tool (Darmstadt Risk Analysis Tool, DRAT) – calculation of multidimensional probability distributions. (DRAM is well described in Bald & Heimbecher (2010).)

Basis for the comprehensive approach is a model, consisting of several part models, which range from the production of the asphalt onwards to laying and compacting asphalt pavements to using and loading it. According to DRAM, these part models consist of active and passive elements, which model the cause-and-effects chains, with passive elements representing a property, a characteristic, or a situation, and active elements resembling the process or interaction occurring between passive elements. The part models are linked by passive elements that are output parameter of the previous part model and input parameter of the following one. Thus, model of pavement performance, which is oriented to common surface characteristics, contains the inherent performance ability predetermined by the previous processes.

By transforming the model parameters, their variation, and their interaction into mathematical definitions and formulations, the initially qualitative model is compiled to a quantitative model. The overall quantitative model can be applied at different times in the pavement's life-cycle as well as with different issues, and thus can be used e.g. to calculate pavement performance and serviceable life respectively.

That transformation was done exemplarily in terms of the 'development of rutting'. The resulting model was applied to two test sections. Thereby, the definition of general functions and factors based mainly upon correlations known from literature review. Specific input parameters of the sections derive predominantly from measured data in the framework of acceptance tests. The test sections are comparable to a great extend, the big difference consists in the structure: One section was conventionally built, the other one was built with compactasphalt.

First application of the model with estimating rut depth for a five year period of use showed satisfying results, especially in comparison to the rut depth that was measured within condition survey at that time. The different behavior of the two test sections is consistently reproduced, and estimation of development of rut depth for a 20-year-period corresponds mostly to the behavior known in practice, too.

The paper comments technique of DRAM, depicts a first application of the underlying model in terms of estimating the development of rutting of asphalt pavements, and touches possibilities of further application.

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Optimizing a structure's life with genetic algorithms and emergence theory

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ABSTRACT

Optimization advancements have led to increased efficiency in structural design. These developments are founded on principles of growth and behavior; mathematically described through genetics and theories of emergence. Evolutionary algorithms based on non-determinate approaches to design are used in multi-variable and multi-objective searches in large through often poorly defined, search spaces. The method, influenced by the seminal work of Darwin, uses advanced trial and error techniques of mass populations as a basis for optimization.

The designs of the Al Sharq Tower and the Wuxi SPG Tower both utilize advanced concepts of genetic algorithms to develop optimal solutions for the structural systems. With initial target design parameters set including minimal material for target displacement limits, multiple solutions are considered. These solutions are considered in developing a fitness score that considers normalized deflection and weight.

Emergent organisms respond to constraints using basic components related by intrinsic rules to create and reach a state of equilibrium. Emergence or self-organization is the interaction between simple common elements having singular and common characteristics, each functioning according to its own simple rules, resulting in complex behavior, without an *obvious* central controlling force. Most consider emergence as a random act of organization, but perhaps a collective intelligence is at work assembling a natural and involuntary response given particular environmental and boundary limit conditions. The properties of an emergent system cannot be deduced from its components and are more than simply the sum of its parts: one element repeated and collected into a complex interaction results in emergence.

Emergent Theory is used in developing the designs for the Shanda Interactive Center and proposed ultra-tall tower for the Gemdale Shenzhen project. In both projects the structure was considered as a continuum, a membrane, rather than a collection of beam and columns. Strain Energy Theory was used to consider placement of structural material for applied lateral and gravity loads.

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The tunnel inspection database of the cold region tunnel for maintenance

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ABSTRACT

In Hokkaido, 350 tunnels have been constructed over the past 30–40 years, of which many tunnels were constructed during the so-called economical developing period in Japan. Since the time of maintenance for these tunnels is approaching, many tunnel linings will repair in the near future. This project therefore aims to establish a management system for tunnel structures based on life-cycle cost in cold region (Sato et al. 2010).

In this paper, an asset management methodology is presented to investigate the optimal repair po1icies of the public facilities under uncertainty of economic lives. And, it also establishes the tunnel management system from inspection data in Hokkaido.

The model of tunnel management system presented in the paper is applied to the asset management of the road tunnels. Also, this research project will be developed to the efficient tunnel maintenance system and a quantitative criterion from pictures of tunnel lining concrete using the life-cycle cost theory (Figure 1).

In this maintenance system is consist of the following parts. Firstly, the database of the foundations of the basic specifications of the national highway tunnel in Hokkaido, a repair history, check data, etc. is created. Secondly, demand performance of tunnel linings in cold region are identified by the Analytic Hierarchy Process (AHP) model based on the weights and Consistency Indices (CI) using the tunnel user, tunnel inspection engineer's and tunnel management engineer's interview (Sutoh et al. 2008). Finally, numerical examples are worked out to select of the suitable repair time and the optimal repair construction method for tunnel lining (Figure 2).

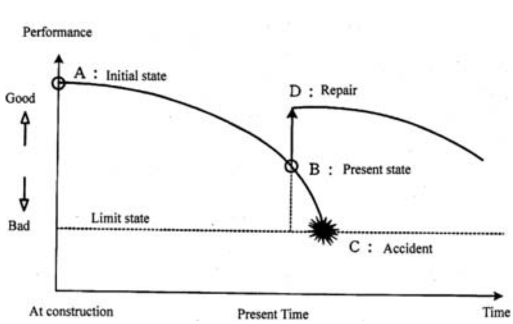

Figure 1. Life-cycle cost for tunnel performance.

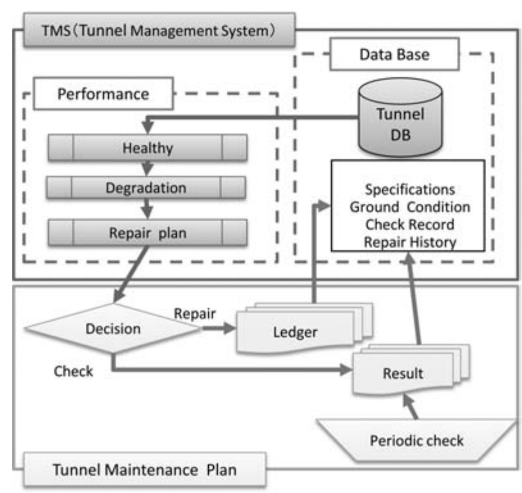

Figure 2. Tunnel Management System.

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"Whitetopping" of asphalt and concrete pavements with thin layers of Ultra-High-Performance Concrete: Construction and economic efficiency

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ABSTRACT

An advanced pavement construction method, named Whitetopping was developed to improve the load-bearing capacity of underdesigned or to reconstruct damaged road pavements. Even for very heavily trafficked roads a comparatively thin layer of only about 120 to 150 mm made of High- or Ultra-High Performance Concrete with a compressive strength of about 150 MPa developed at the University of Kassel reinforced with fibers and/or with steel bars is placed on top of the existing structure. Thus the existing structure must not be removed. It furtheron acts as a high grade base course.

In a comprehensive study the technical and the economical benefits of this kind of high performance Whitetopping were compared with conventional methods to strengthen or to renew pavements by means of layers consisting of unreinforced ordinary concrete. After definition of the technical conditions an overall "life-cycle" economic evaluation was executed. The criteria commonly essential for a financial decision of the road construction authorities and enterprises – primarily the initial investment, the costs of maintenance, capital- and administration cost in relation to service-life – were embedded into a total system in order to receive a valuation method for strategic decisions. Thus further targets for the preservation of the road infra-structure were considered becoming increasingly important, like safety, trafficability, substance preservation und environmental suitability. Furthermore the road user costs like vehicle operating cost, travel expenses and accident costs, have been estimated and implemented for each respective construction method.

It could be demonstrated that the initial costs of the high-performance Whitetopping structures are only marginally higher than for the conventional reference methods based on 260 mm thick unreinforced slabs. Due to the higher quality and the longer service–life of the high-performance structure (estimated to 54 instead of 26 years for the conventional system) the capitalized costs for maintenance und renewal are only half of the reference method. Significantly longer renewal intervals lead to a further reduction of construction-, traffic congestion- and accident cost.

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Study of the quantity of gypsum in Construction & Demolition Waste (CDW)

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ABSTRACT

The enormous increasing use of gypsum materials in the building industry in the last decades will cause a huge mass of this material in the future waste stream. As an extraneous material at the reuse of Construction and Demolition Waste (CDW) it will be a growing problem. Gypsum can cause damage in concrete as well as in road subbases by volume increase induced by the formation of ettringite and thaumasite. Therefore the content of gypsum in CDW has to be reduced.

The threshold for the reuse of CDW is already exceeded by very small gypsum contents. According to DIN 4226-100 for the use of recycled concrete aggregates in building constructions at most 0.2% by mass of impurities are allowed. Such components are gypsum, glass, ceramics, light-weight concrete, aerated concrete, metal, wood and plastics.

On the one hand gypsum has to be separated directly on the demolition site, in order to be able to produce high-quality recycling concrete. On the other hand more effective processing methods have to be established. Also more detailed knowledge is required on the gypsum material streams and the gypsum content in the rubble, which can be expected in future.

This study shows different approaches to calculate the present amount of gypsum in CDW in Germany. Each of them bases on different sources. The possible future development of gypsum waste amount was calculated by a dynamic model. One of the objectives was to show the main stream of the material gypsum in the building industry.

The results of the calculations show that the amount of gypsum in the waste stream will increase strongly in the next years. An important result of this study is that in the next 20 years the output of gypsum waste in CDW almost will be doubled.

To get an estimation of the amount of gypsum in real buildings different samples were taken from demolition sites and analysed regarding the sulphate content. So the proportion of gypsum could be calculated for those buildings.

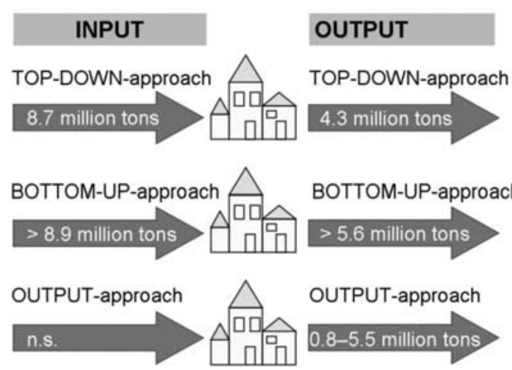

Figure 1. Results of the different approaches by comparison (Müller et al. 2011).

Additionally to analyse the influence of the demolition and processing technology on the output of sulphate a recycling company in Thuringia was contacted. The production of the recycling material in a stationary recycling company was documented. The quality of his products regarding the sulphate content was tracked. For different steps of processing the material is analysed.

At present in Germany there is no solution for an environmentally friendly handling of this waste. Over the past years a small part of gypsum waste generated from plasterboards was used for covering potash waste dumps, while most of the gypsum waste is land filled. Since summer 2011 this way of utilization is not allowed anymore in Germany.

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Probability based optimized design of concrete structures

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ABSTRACT

A significant amount of existing optimized methods are specialized in the area of structural design that considers all parameters as being deterministic (DBSO – Deterministic Based Structure Optimization): the reliability of a structure is expressed via the commonly used Partial Reliability Factor Method (PFM). The general deficiency of DBSO is the omission of uncertainties while determining loads, material characteristics, responses and the stability of the structure. Elimination of that deficiency enables the combination of design methods based on the Fully Probabilistic Approach (FPA) and optimized methods. This assessment is called Reliability Based Structure Optimization (RBSO).

In the presented case study a prestressed pole fabricated from spun concrete was designed via both methods. Two different concrete variants were used for the poles: C40/50 Reinforced Concrete (RC) and Reinforced Fiber Concrete (RFC).

The used objective function can take into account the economic and environmental aspects (acquisition cost, embodied energy and emissions of CO_2 and SO_2 released during the production of concrete elements) and minimizes all these aspects. The resulting multi-criteria task should be solved using weighted sums. In this study two alternative weighting coefficients were considered. In the first alternative (alt.1) only the cost of the pole was a decisive criterion, while in the second alternative (alt. 2) environmental aspects were also taken into account (weighting coefficients were set as follows: cost $\alpha_P = 0.5$; CO_2 and SO_2 emissions $\alpha_{CO} = \alpha_{SO} = 0.167$; embodied energy $\alpha_E = 0.167$).

The design was based on the assumption that the geometry of the pole is unchangeable (it is determined by the mold) and the number of prestressed wires was fixed (20 ø PN). Only the passive reinforcement was changed (the number *n* of rebars (*n* ø R) and the lengths of the bars).

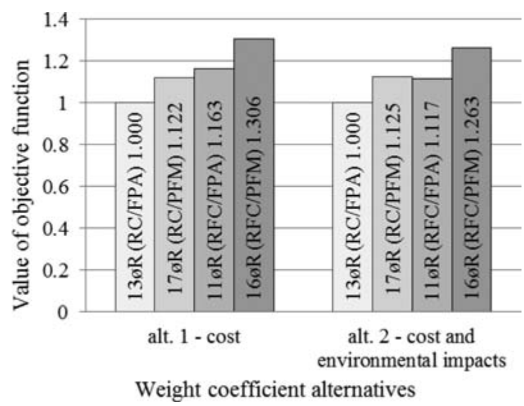

Figure 1. Comparison of designed poles.

A comparison of all designed alternatives can be found in the following graphs in Figure 1. When evaluating the target function, the result of the optimization of an RC pole via FPA (RC/FPA) with reinforcement 13 ø R in its critical cross-section is considered as a reference (because the individual terms of objective functions are expressed in different units).

The above graphs (Fig. 1) show that the method based on the FPA is more advantageous (more efficient) in terms of economic and environmental impacts. The use of reinforced fiber concrete meant a lowering of the weight of classical reinforcement used, though the weight saving did not compensate for the weight of the added fibers.

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Monitoring of ground-structure-interaction of an arch bridge

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ABSTRACT

This report describes the monitoring instrumentation at the abutment foundations of a deck arch bridge under certain alpine conditions. The observed object is a reinforced concrete arch bridge with an elevated deck across the Zauchen gorge, which, which slope gradient is around 40°. The main span of the bridge between abutments is approximately 60 m, its crown is about 14.5 m. The bridge was built between 2000 and 2001.

Based on various uncertainties concerning mainly the ground-structure-interaction in the section between the foundations and the ground, the Federal State Government of Carinthia initiated a monitoring system to measure this interaction and to analyze its consequences for the structure itself. The monitoring project has been coordinated by the engineering company 3P Geotechnik ZT GmbH. The data acquisition system consists of hydraulic load cells embedded between the abutment foundation and the ground to measure the development and distribution of stresses between structure and underground. Furthermore extensometers were installed to detect the displacements of the abutments and the strain in the arch crown. To consider the temperature, which influences the sensors as well as the structure itself, temperature gauges were embedded at the same section as the strain and pressure gauges inside and outside the structure. The data record covers the period from April 23th 2001, when concreting of the first arch segment took place, until July 27th 2002. Additionally bridge loading tests were performed on 24th of May 2001 just before opening to the traffic.

In 2010 the measured data where once more interpreted and controlled for their plausibility in a scientific paper work at the Carinthia University of Applied Sciences. In this work all measured life-phases: construction, testing and operation were simulated by different FE-Models of decoupled systems. In order to verify the design assumptions the monitoring results were compared with theoretic values based on the FE-Models of the bridge.

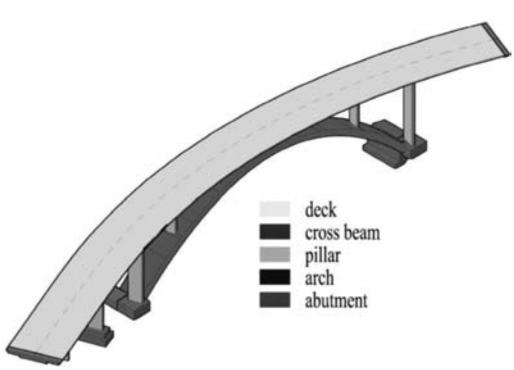

Figure 1. Perspective view of the concerned structure.

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Uncertainty in multivariate modelling: Main concepts and an application on fracture mechanics

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ABSTRACT

The assessment of a structure's resistance and performance is directly linked with reliability and safety issues, as well as with intervention decisions and their relative cost. This assessment can be enhanced by the use of probabilistic models, invoked by the random nature of load and resistance effects. Moreover, intrinsic properties of such models should relate to the commonly encountered dependencies among the input random variables. With respect to that, a significant advance towards effective modelling in current practice of structural reliability engineering has been achieved by integrating scalar indices, such as correlation coefficients. However, case may be that this approach overlooks, or even induces considerable uncertainties. Therefore, models which acknowledge and estimate this uncertainty type can serve the requirements for high reliability and robust decision–making. In the present study, the uncertainties pertaining to multivariate idiosyncrasy are discussed, and a generalised dependence structure paradigm is suggested in order to explicitly handle such uncertainties. The predictive potential of resulting models is demonstrated on an application on fracture mechanics, while the influence on contemporary assessment procedures is discussed. The corresponding numerical techniques can facilitate the inclusion of advanced correlation modes in the simulation methods for structural performance.

Current status in probabilistic structural and civil engineering seems to promote rather simple models regarding multivariate dependence. Such models are useful but not always accurate approaches, often failing to describe several natural phenomena in a realistic way (Ferson et al., 2004). In the present paper, a type of uncertainty, often disregarded in structural engineering, induced by multivariate dependence, is investigated. Copula functions are used in order to construct different multivariate models which rely on the same available information (Nelsen, 2006).

The structural system of interest comprising an anchor under tension, failing with concrete cone breakout, is depicted in Figure 1. Through the examination of the above system, represented in terms of resistance as a model with two covariates (Strauss, 2003), the uncertainty contained in the predicted failure probability is evaluated.

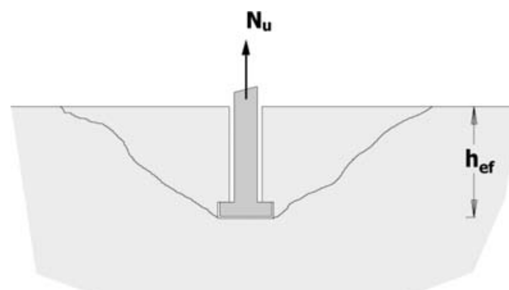

Figure 1. Single anchor failing with concrete cone breakout.

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RC precast bridges with joint-less deck: An Italian experience

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ABSTRACT

Integral Bridge Design Technique has recently increased its application in many countries, as a mean of life-cycle elongation of new civil structures but also as a replacement of existing.

In the context of the project sponsored by the Regional Highway Managing Institute (Autovie Venete), three reinforced concrete joint-less bridges were designed. The main purpose was to substitute three existing simply supported bridges enlarging the main carriageway from four to six lanes without interrupting the traffic. The bridges have a total skew deck width of over 41 m, a span length variable from 20 to 30 m (along the bridge axis), and the abutments are realized with a palisade of big diameter bored piles. By means of a continuous slab which extends from the deck into the abutments a joint-less connection has been realized in order to prevent structural deterioration and to withstand horizontal actions, meanwhile pre-cast simply-supported beams, supported by reinforced rubber devices, bear vertical loads.

A significant scale of parametric analysis was carried out, considering both static and seismic condition, aiming to understand the response of the structure to different soil characteristics, piles' length, piles' distance, deck span and deck inclination.

From results it is shown that some significant parameters allow to recognize the limit of this construction technique, and it is evidenced that in medium-high seismicity regions, the design of connecting slab and piles is guided by seismic limit state load combinations.

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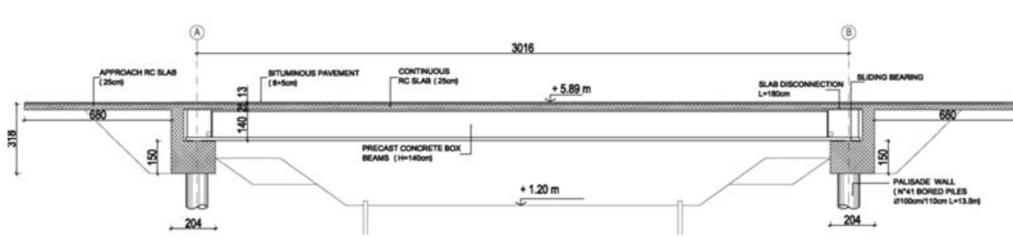

Figure 1. Joint-less Bridge on the River Musestre: Transverse section and plan.

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Life-cycle performance assessment of PSC bridges exposed to coastal zones under uncertainty: I. Theory

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ABSTRACT

In this first part of a two-part paper, an integrated computational methodology for the life-cycle performance assessment of Prestressed Concrete (PSC) bridges in coastal zones under uncertainty is presented. Specifically, the time-variant system reliability is used as performance indicator. A finite element-based approach for evaluating the life-cycle performance of PSC bridges is proposed. A computer program named CBDAS has been developed to perform this analysis. Statistical parameters of the main random variables involved in resistance, load and environment are determined based on the statistical data collected in China and other countries. Time-variant resistance of structural components and load effects are thus calculated by using Monte Carlo simulations associated with CBDAS and the statistical parameters of the random variables. Finally, the time-variant components and system reliabilities are evaluated. This first paper has three main sections.

(1) Assessment of life-cycle performance
Two essential problems involved in the life-cycle performance assessment of PSC bridges are discussed: (a) the computation of the critical times in the degradation process; and (b) the solution of some conventional mechanics problems encountered in the construction process and several durability mechanics problems associated with the degradation process, such as the deterioration of materials properties, the reduction of cross-sectional areas, and the variation of overall structural performance. CBDAS has been developed to perform this analysis.

(2) Random variables
The uncertainties in life-cycle performance assessment of concrete bridges stems from three sources: resistance, load and environment. In this section, the distribution type and associated statistical parameters of the random variables are investigated and determined based on the statistical data from laboratories, construction sites and primary documentations in China and other countries.

(3) Time-variant system reliability
The time-variant system reliability is calculated by using the following steps: (a) divide the overall structure into components; (b) evaluate the distribution types and statistical parameters of time-variant resistances $R(t)$ and load effects $S(t)$ for all components by using Monte Carlo simulations; (c) define the structural failure modes to realistically reflect the structure performance; and (d) Calculate the point-in-time system reliabilities using the software RELSYS.

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Life-cycle performance assessment of PSC bridges exposed to coastal zones under uncertainty: II. Application

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ABSTRACT

This paper, as the second part of a two-part paper, presents the application of the proposed life-cycle performance assessment approach to a Prestressed Concrete (PSC) bridge. The numerical example is a three-span PSC continuous box girder bridge. Ultimate and serviceability limit states are both considered. The following conclusions are drawn:

1. The mean values of the corrosion initiation time, cracking initiation time, and cover spalling time vary significantly along the cross-section of the PSC bridge. Reinforcing steels in some parts of the cross-section may begin to corrode at 20 years while in other parts may not corrode throughout the entire service life.
2. The concrete cover may be spalling only a few years after the initiation of steel corrosion, since the corrosion rate is very fast under chloride penetration.
3. The time-variant component reliability is significantly affected by the distribution types of the resistance and load effect.
4. The bridge system reliability index at ultimate limit state is very low at 100 years, due to a serious deterioration of the structural shear capacity resulted from the large area loss of stirrups in the web zone of concrete cross-section. Therefore, maintenance inventions are necessary to improve the structural shear capacity. The system reliability index for serviceability limit state reduces from 2.0 at the beginning of service life to about −1.0 at 100 years. Accordingly, maintenance actions that can delay the initiation of concrete cracking are necessary.

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Validation of diffusion models for life-cycle assessment of concrete structures

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ABSTRACT

For concrete structures the aggressive attack of environmental agents may lead to a progressive deterioration of the mechanical properties which makes the structural systems less able to withstand the applied actions. The damage process includes deterioration of concrete and corrosion of reinforcement, and a proper modeling of these effects is therefore essential to a life-cycle assessment of concrete structures.

In recent years, novel approaches to life-cycle analysis and design of concrete structures in aggressive environment have been proposed in deterministic and probabilistic terms (Biondini & Frangopol 2008). In this context, it is important to check the reliability and accuracy of models used to study corrosion phenomena and related effects induced by chloride ingress. Most observations indicate that the transport of this aggressive agent in concrete is diffusion controlled (Bertolini et al. 2004). For this reason the chloride ingress can be effectively modeled through the second Fick's law of diffusion, that is generally applied in the one-dimensional form. However, in most cases the diffusion process should be more properly described by considering two- or three-dimensional patterns of concentration gradients (Glicksman 2000).

In this paper, the accuracy and reliability of the one-dimensional (1D) modeling of diffusion and related damage is checked with respect to more accurate two-dimensional (2D) formulations. To this aim, the numerical solution of the diffusion problem is obtained by using a special class of evolutionary algorithm known as cellular automata (Wolfram 1994). This approach has been proposed in previous works and applied to life-cycle reliability assessment and design of concrete structures in aggressive environment (Biondini et al. 2004, 2006, Biondini & Frangopol 2009, Biondini 2011).

The proposed formulation is validated considering 1D and 2D problems for which the analytical solution is known. Subsequently, parametric analyses for the diffusion process on concrete cross-sections subjected to corrosion are performed by checking the accuracy of the 1-D approach with respect to more accurate 2D solutions. Based on a suitable degradation model for the corroded steel bars, the results in terms of damage are also compared. The results show that 1D diffusion models can lead to a significant loss of accuracy depending on the exposure conditions, the geometrical shape ratio of the cross-section, and the points where the concentration and damage are evaluated. As a consequence, a 2D simulation of the diffusion process at cross-sectional level is generally necessary for an accurate life-cycle assessment of concrete structures exposed to corrosion.

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Deterministic versus probabilistic reliability analysis of existing bridge structures

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ABSTRACT

To provide uniform standards for the assessment of existing road bridges the German Federal Ministry of Transport, Building and Urban Development (BMVBS) issued the "Guideline for recalculation of existing road bridges (short: Recalculation Guideline)" (BMVBS 2011). In this guideline current and former standards are reconciled with respect to load-bearing capacity, serviceability and durability. This paper gives a brief overview of the potential for the reliability assessment of bridges, which is now regulated more transparent and comprehensive then in the individual guidelines before.

The present article deals with the probabilistic reliability analysis of bridge structures in the framework of the Recalculation Guideline. The application of this procedure is allowed within step 4 of the investigation hierarchy in the Recalculation Guideline. The major advantage of the probabilistic method is that the operational failure of probability can be determined specifically. In comparison to the deterministic calculation the probabilistic method requires more computing time, but the reliability reserves of a bridge can not only be estimated, but also be quantified and used, in case the recalculation with other methods shows deficiencies of the bridge structure and limitations in the use of the bridge have to be specified.

The principle procedure is explained by using the example of an ordinary two-span bridge (post-tensioned T-beam). Firstly, a comparison between the results of the semi-probabilistic safety concept, e.g. according to Eurocode, and the probabilistic assessment is done, indicating a higher reliability index in regard to the shear force capacity of the bridge within the probabilistic analysis.

For the chosen example a parameter study on the deterioration of shear reinforcement has been conducted. In summary, 14% of the shear reinforcement can corrode in the considered bridge structure before the target reliability falls below the limit value of $\beta = 3.8$ in the Ultimate Limit State (ULS). Finally, the

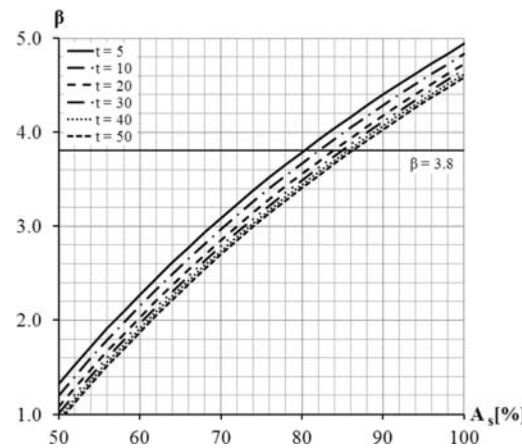

Figure 1. Reliability index β in dependence of shear reinforcement and a reduced remaining service life t.

consideration of a reduction of the remaining service life time has been investigated. On the basis of adjusted traffic loads for the varied observation periods an increase of the reliability index can be noted.

For the chosen bridge example figure 1 indicates an increase of the reliability index of $\Delta\beta = 0.25$, if for a non-damaged structure the service life t is reduced from 50 years ($\beta_{t=50} = 4.58$) to 10 years ($\beta_{t=10} = 4.83$). In some cases this correlation may help, if the target reliability index cannot be verified for a service life of $t = 50$ years, and a reduced service life has to be assessed.

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Production of pervious concrete by using construction and demolition wastes

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ABSTRACT

Pervious concrete is a new type of concrete with significantly increased water permeability, ensuring increased rates of drainage of rainfall. The environmental benefits arising from the use of such products are summarized to: the management of rainwater, the prevention of flooding and the enrichment of underground waters. The high porosity is achieved by removing a large percentage of fine aggregates from the mix.

Continuous building construction has caused a lot of damage to nature. More specifically, it affects the environment, directly or indirectly, during the entire life-cycle of the buildings, as well as during the life-cycle of their materials and components. Additional problems to the environment occur at the end of life of the buildings from the Construction and Demolition Wastes (CDW) that are gathered after demolition of the buildings

The present paper comprises an approach to the addition of CDW as substitutes for coarse aggregates in pervious concrete. The use of these materials will face the problems coming from their disposal and it will constitute one still alternative solution for the 500 million tons of construction and demolition wastes of Europe per year.

As far as this particular form of exploitation is concerned, Recycled Concrete Aggregates (R.C.A.) present a high porosity which, for the production of the conventional type of concrete, constitutes a disadvantage, however, for the production of pervious concrete, it constitutes the main desired property.

The purpose of this paper is to prove that recycled concrete aggregates are indeed an opportunity for the industries and that will be proved through the evaluation of the use of C&D Wastes as aggregates in pervious concrete and the summary of the environmental and technological benefits of this use.

More specifically, two types of aggregates have been used: construction and demolition wastes and conventional limestone aggregates. The produced pervious concrete samples are compared for their properties, such as water permeability, percentage of void content, compressive strength and abrasion behavior.

According to the results of this paper, it is observed that the incorporation of C&D Wastes leads to better abrasion behavior, and to a slight increase, in some cases, of the compressive strength. The behavior in water permeability is also examined and discussed.

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Extension of sample size in Latin Hypercube Sampling – methodology and software

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ABSTRACT

In many computer experiments the adequacy of a given sample to give acceptable estimates of desired statistical quantities cannot be determined a priori, and thus the ability to extend or refine an experimental design may be important. This can be done very easily in crude Monte Carlo sampling. Very often, though, running each realization (physical or virtual experiment) is very expensive. Therefore the variance reduction techniques such as the Latin Hypercube Sampling (LHS) represents a suitable option because it yields lower variance of estimates of statistical moments compared to the crude Monte Carlo sampling. However, in conventional LHS it is the necessary to specify the number of simulations (or physical realizations in the design of experiments) in advance. Unfortunately, in real life problems the sample size yielding stable and statistically significant estimations of output statistics is not known beforehand. If too small sample set is used, the analyst has to abandon the results and run new analyses with a larger sample set. It is thus desirable to start with a small sample and then extend (or refine) the design if deemed necessary. The extension would permit the use of a larger sample set without the loss of any of the already performed, and possibly quite expensive, calculations.

An algorithm is presented that overcomes the problem of conventional LHS. The first versions of the method was suggested by Vořechovský (2006) and it was later extended in Vořechovský (2010). It is based on a hierarchy of LHS-like samples which are proven to yield to smaller variances of results compared to the crude Monte Carlo. The subsets sampled by the proposed method can be merged together exploiting the property of variance reduction, yet retaining the sampling flexibility. The whole procedure of a cascade of LHS-like runs can be fully automated and the (automatic) stopping criterion might be e.g. the significance of output statistics or the maximum desired computational time.

A sketch of the coordinate system adopted is shown in figure 1.

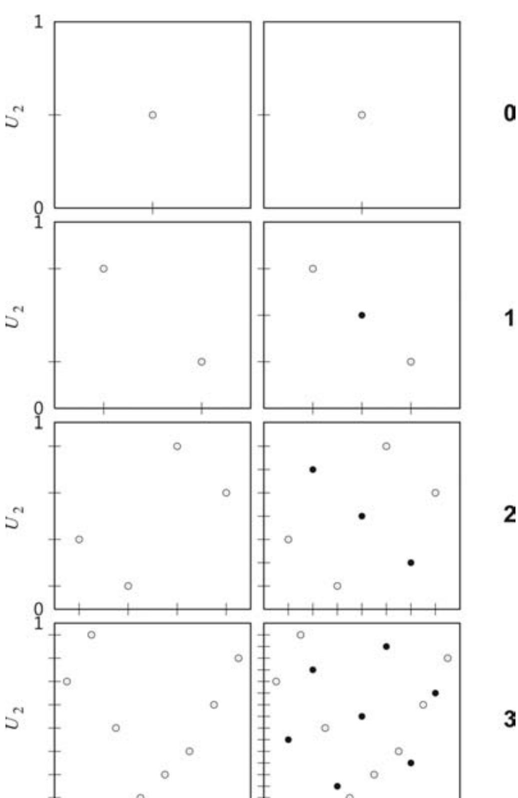

Figure 1. Sketch of the adaptive refinement of sampling probabilities.

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Prevent corruption – measures to increase integrity in organizations

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ABSTRACT

Each national or international enterprise should have an increased awareness of possible negative influences out of intentional crime, corruption, bribes or other influences. The results exposed in this article are the findings of different international studies ASFINAG BMG realized together with FH JOANNEUM, Graz, Stempkowski Baumanagement & Bauwirtschaft Consulting GmbH and PIARC.

The global aim of these surveys was the compilation of a funded analysis and the prevention of corruption in international enterprises in the road sector. Therefore it should be emphasized that on a global level, corruption is often a symptom of an ineffective governance framework for the wider public sector and society as a whole. The measures thus will be most effective when they are backed up with an explicit commitment from the roads sector, government and representatives of civil society at the highest level to a governance framework which promotes transparency, integrity, accountability and the rule of law.

Building on the findings of the survey a cyclical model was developed, which describes the procedure for preventing or tackling corruption on various levels and stages. This "cycle of integrity" comprises several stages whose reciprocal interactions and induced effects can be depicted in two opposing directions (Figure 1).

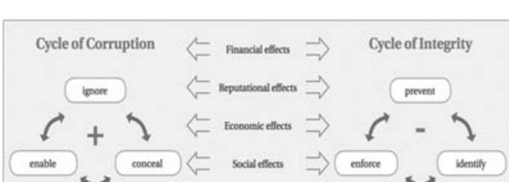

Figure 1. Cycle of Corruption – Cycle of Integrity.

Based on the cycle of corruption and integrity, findings from international surveys of PIARC and work by bodies such as Transparency International, a comprehensive toolkit of anti-corruption measures has been devised. Division of the essential anti-corruption measures into the relevant fields of prevention, identification and enforcement is characteristic of this toolkit.

Organizations can use it as a checklist (Figure 2) to encourage the implementation of active, coordinated measures that promote integrity and reduce the potential of corruption. Practical relevance is provided by selected examples of anti-corruption measures that are already successfully implemented within different organizations.

	Subject Areas	Measures	Description of Measures	Fully Implemented	Partially Implemented	Not Implemented	To be introduced	To be developed further
PART A - MEASURES for the PREVENTION of CORRUPTION								
1	Development of business ethics and anti-corruption strategy - awareness-raising and educational measures							
	Theory: In many cases there is a lack of awareness about the definition, nature and consequences of corrupt behaviour ("trivial offence"). Awareness-raising measures therefore need to be provided as a priority for all employees and if possible for all contractors.							
		Ethics guidelines	Ethics guidelines with summary of the organisation's mandatory (and legal) policies and practices in the areas of business ethics and anti-corruption applicable to all departments and all employees Clear definition of what constitutes corruption and rules of conduct in the context of encountering perceived or actual corrupt behaviour Basic understanding of the organisation's ethics philosophy Sources of further advice, information and support					
		Ethics workshops and other dissemination events	Regular completion of ethics workshops and other dissemination events for the training of employees and for the internal discussion of current issues, overview of the current regulatory framework and consequences (e.g. accepting gifts, damage to the organisation, damage to other market participants)					

Figure 2. Abstract of Toolkit of Anti-Corruption Measures (Checklist).

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Harvesting factor in hydropower generation

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ABSTRACT

Discussions on climate change as well as the European and national ambitions on promoting renewable energy sources, the importance of hydropower is increasing (Directive 2009/28/EC, 2009). In Austria this is carried out using the Austrian strategy, which is strongly focused on hydropower (BMWFJ, 2010). Technically approved arguments should be provided for the decision-making process regarding the development of possible hydropower projects compared to other energy sources.

The aim of this paper is to show the environmental impacts of a hydropower plant over its life-cycle in a holistic approach. Accordingly, a life-cycle assessment based on the bill of quantities of a small hydropower project (Figure 1) was carried out to calculate its environmental impacts.

To determine the entire environmental burden, it is furthermore necessary to consider the material-related in- and output flows of the chosen hydropower plant to build, operate and decommission it.

The Life-Cycle Assessment (LCA), the approach of the ÖNORM ISO EN 14040 (ISO, 2006) was chosen. Within the life-cycle impact assessment, data regarding global warming potential is gathered during the life-cycle of the hydropower station.

This means everything is taken into account starting from the construction with the used products (product stage and construction process stage) up to the maintenance works (use stage). This also takes into account the decommissioning of the power plant at the end of its life time (end of life stage).

To calculate the environmental impacts caused by the hydropower plant during its life-cycle, several indicators have been chosen according to the FprEN 15804. Subsequently the Ecoinvent database v2.2 (Swiss Centre for Life-Cycle Inventories, 2004) was used to calculate the environmental impacts and the cumulative energy demand.

To determine the harvesting factor, it is necessary to focus on the cumulative energy demand (Figure 2).

Referring to the third emissions trading period, the selected categories, especially including the Global Warming Potentia (GWP) throughout the entire life-cycle of the hydropower plant, have also been investigated in detail.

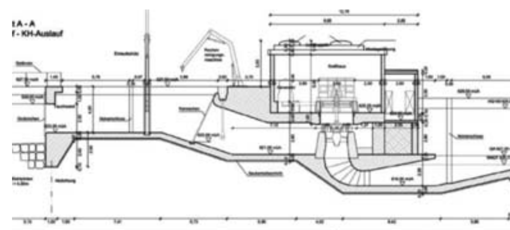

Figure 1. Cross section of the hydropower project.

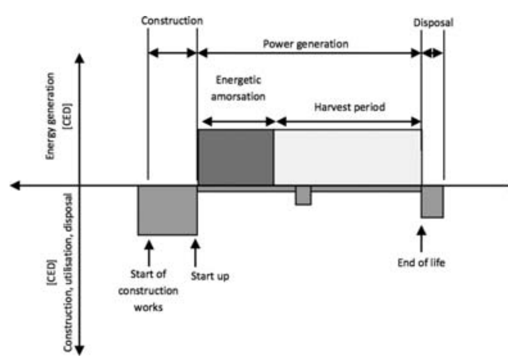

Figure 2. Illustration of determining the harvesting factor.

The environmental impact clearly shows that the use of hydropower is more sustainable than other energy production technologies due to its low impacts on global warming potential.

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Dynamic-based performance updating of a high-speed concrete railway bridge

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ABSTRACT

Structural model updating using experimental measurements has been in the focus of intensive research for over four decades and it continues to be a high-priority topic for the accurate prediction of structural performance (Friswel & Mottershead 1995, Calvi 2005, Yuen 2010). For high-speed lines under dynamic loads, the concrete can be submitted to significant loading effects, which may lead to a great change of the dynamic Young's modulus of the concrete (Bucknall & Marvillet 1999). The results of the finite element models and the on-site measurements may be different for concrete bridges due to the uncertainties of the parameters such as Young's modulus, etc. In addition, there are other kinds of parameters which can strongly influence the dynamic characteristics, like the stiffness of the foundation and the mass of the structure. Consequently, the need for taking uncertainties into account within the model updating process has been widely recognized. It leads to a wide research of randomness and uncertainty of the parameters for the concrete structure of the high speed railway lines.

This paper presents a probabilistic computational framework for updating the parameters of a structural model according to the dynamic results obtained from experimental measurements. The technique for assessment of a bridge structure is presented with uncertain system parameters from structural acceleration responses with measurement noise. On one hand, the probabilistic model is realized in the interface of ANSYS Probabilistic Design System considering several parameters which possibly influence the dynamic characteristics of the bridge. An analysis of sensitivities of the parameters determines the parameters that should be considered in the updating numerical model. On the other hand, structural health monitoring is considered to obtain the characteristics of the bridge experimentally based on the system identification technique-random decrement technique. The procedure of Bayesian probabilistic framework is undertaken to update the input parameters of the structures. A concrete railway bridge PRA24270 of the high speed line in the eastern of France is presented to illustrate the applicability of the proposed procedure.

A Bayesian network of this bridge has been built as shown in Figure 1.

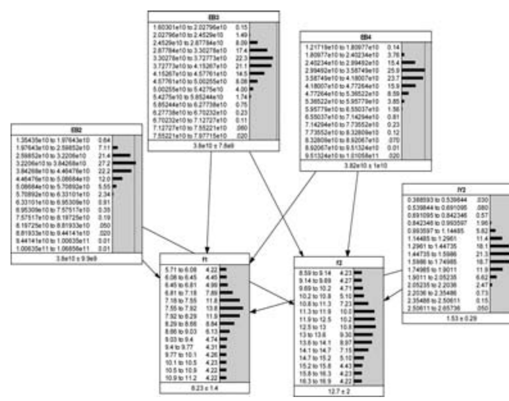

Figure 1. Bayesian network for one railway bridge.

The application of Bayesian network on this bridge shows that it is a powerful tool in the domain of civil engineering maintenance.

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The environmental impacts of concrete containing Nano-SiO_2 and typical concrete on global warming and fossil fuel depletion: A comparison

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ABSTRACT

Finding appropriate alternatives for the cement used in concrete, is one of the best solutions associated with improvement of concrete from an environmental view. Nano-SiO_2 with high pozzolanic nature has been considered as a supplementary cementitious material for part of cement content.

The purpose of this study is the environmental and economic assessment of concrete containing nano-SiO_2 by use of life-cycle assessment method and comparing the results with typical concrete.

In order to assess the environmental impacts, the Environmental Life-Cycle Assessment method is used. Furthermore, a Life-Cycle Cost Method is applied to do an economic assessment. Finally, the environmental and economic assessment results would be integrated by means of Multi-Attribute Decision Making method.

Results show that the use of nano-SiO_2 in concrete for a short-term period,including production process and design stage, is not environmentally recommended. But within service life, in case of using 1.5% nano-SiO_2, the environmental impacts reduced so significantly.

According to the economic evaluation results the cost of 1 m^3 concrete in case of using 1.5% nano-SiO2 in the base-year compared to typical concrete increased by 88%, which suggests that its short-term production is not justifiable. But the long-term economic evaluation shows that the life-cycle cost of concrete decreased by 10% in concrete containing 1.5% nano-SiO2.

With regard to the overall performance scores, concrete containing 1.5% nano-SiO2 suggests a better performance in long-term, compared to typical concrete though the difference is not considerable.

Since, the decision making process depends not only on long-term benefits but also short-term ones, and due to potentialtoxic risks of nano-SiO_2 and its unknown effects, the appropriate option must be selected more carefully by considering the slight difference between the overall performance scores of two type of concrete.

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Concrete sludge water recycling: An essential practice for the sustainability of a ready-mix concrete plant

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ABSTRACT

Sustainability issues have troubled the construction industry for the past few decades in several application fields. The production of low CO_2 emissions cement, the use of SCM's, as well as the reuse of recycled aggregates are a few of many well studied but also applied practices that lead to environmental friendly construction products associated with the cement and concrete technology. Even through this plethora of scientific outcomes, which are of environmental as well as legal importance for the industry, there are still issues that have not been thoroughly studied. This study concentrates on the management of the water required for a ready-mix concrete plant to function, while focusing on the need to recycle the water used to wash the concrete mixing trucks at the end of each shift.

The previously referred water forms a sludge water that stands out for two main characteristics: high concentration of dissolved and dispersed solids and a high pH value. Specifically, sludge water when exiting the mixing drums of the concrete mixing truck consists of about 10,000 ppm of total solids and has a pH value over 12 (usually 12.5), that prevents its deposition into the environment, since by international and local environmental law, materials with pH over 11.5 are hazardous for disposal.

In addition to the environmental impact that would be caused by the disposal of concrete sludge water, there is also an economical and ecological 'in the matter of natural resources preservation' problem the ready-mix concrete plant has to deal with; the excessive water consumption in order to meet its needs. It has been calculated that at the end of each shift there is an amount of approximately 200–400 kg of returned concrete on the inside of the concrete mixing truck which requires about 1500 L of water to be washed out. Adding this to the fact that to produce 9 m^3 of concrete 'a standard quantity carried in a mixing truck' 1600 L of water are needed, one can easily conclude that water recycling could lead to reduction of water consumption in half.

The difficulty in recycling sludge water lies in the fact that whilst the European and American standard for concrete mixing water allow the use of this sludge water for concrete production, there are national regulations, such as the Hellenic standard, that based on the water's chemical analysis, limit or prohibit its use. This forces the Greek ready mix concrete plants to recycle sludge water partially after mixing it with fresh water or neutralizing it, constraining them to recycle in minimum or zero ratios.

In this case, a statistical study is presented including data from Greek and European ready-mix concrete plants concerning their need in water, the nature of the recycled and the production water as well as the applied concrete mixing water standard, in order to underline the necessity of applying sludge water recycling.

Moreover, the chemical analysis of the recycled sludge water from numerous Greek ready-mixed concrete plants is listed which, combined with the results on workability and flexural and compressive strength of mortars produced with sludge water – either as it is or treated-, result in an undisputed recommendation of reusing sludge water as concrete mixing water.

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Combining resilience and sustainability in infrastructure projects

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ABSTRACT

The main goal of this paper is to present an original approach aimed at combining the emerging concepts of resilience and sustainability of infrastructure systems, bridges in particular. Resilience is a metric of the impact that the bridge has on the society when subjected to extreme events, such as hurricanes and earthquakes (Bruneau et al. 2003). On the other hand, sustainability analyses attempt to quantify the impact of the bridge on several issues of concern for the society (e.g. health, biodiversity, land use, etc.) under ordinary conditions (Brundtland 1987). Nowadays, both resilience and sustainability should be considered when a new infrastructure system is designed, built and managed. However, since they have always been treated separately, their combination presents challenges and requires trade-offs. In this paper it is proposed to merge resilience and sustainability analyses in a general probabilistic framework rooted in risk theory.

In fact, since both resilience and sustainability address the impact that an infrastructure system has on the society, at the most conceptual level they are very similar. The threshold that classifies the events accounted by one analysis or the other is the probability of occurrence. For this reason, the framework of risk analysis has been identified as the most appropriate, well-established, common ground to combine resilience and sustainability. With a generalization of the expected risk assessment equation, the impacts on the society associated with the various events during the life-cycle of an infrastructure system are weighted by their probability of occurrence (Bocchini et al. 2012):

$$I = \sum_{e \in E_r} P_e \cdot I_e + \sum_{e \in E_s} P_e \cdot I_e \qquad (1)$$

where I is the expected value of the life-cycle impact of the infrastructure on the community, E_r is the domain of events usually addressed by resilience (i.e. extreme events), E_s is the domain of events usually addressed by sustainability (e.g. construction, normal operations, maintenance, disposal), P_e is the probability of occurrence of event e, and I_e is the estimated impact on the community of event e. The proposed technique can be used to quantitatively compare and combine the results of resilience and sustainability analyses for decision making that accounts for the relative importance of "ordinary" conditions and "extra-ordinary" scenarios along the service life. The objective of decision makers should be the minimization of the overall impact, as provided by Equation (1).

Three heuristic scenarios are used to represent possible practical applications of the proposed approach.

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Epoxy-coated reinforcement – optimization of life-cycle costs?

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ABSTRACT

In order to ensure the serviceability of reinforced concrete structures in the German traffic system enormous annual expenditures are required. Beside frost-induced degradation of concrete, chloride and carbonation induced corrosion are the ultimate causes of many corrosion damages yielding costs of repair. To achieve an improved durability of reinforced concrete structures Epoxy-Coated Reinforcement (ECR) can be used. The assessment of the corrosion protection potential of such a system is the objective of a current research project. This paper presents cost-benefit analysis of ECR by the means of realistic simulated scenarios (inter alia full probabilistic service life predictions, practical maintenance varieties) over the whole service life of a structure. Solely based on economic considerations the necessary Extra Service Life by the use of ECR (ESL) compared to normal black steel (uncoated) is calculated (Figure 1). As a result of these findings a novel approach of a holistic Life-Cycle Management starting with the design phase is introduced.

With the cost-benefit analysis certain life-cycle costs of different maintenance strategies can be calculated over a fictional service life of a reinforced concrete structure or element. The combination of full probabilistic service life design (FIB 2006) with practical maintenance procedures (herein concrete replacement and concrete coating) and corresponding costs allows for the selection of proper building materials by the planner or the most cost effective maintenance strategy by the owner, respectively. The knowledge of the so called “best practice” also allows the comparison to other options or systems for an optimized maintenance (e.g. herein epoxy-coated reinforcement).

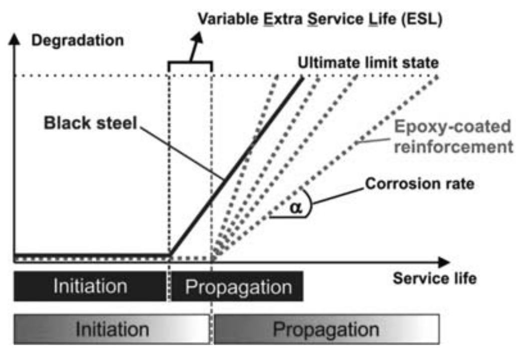

Figure 1. Possible corrosion protection effect of ECR compared to black steel (herein Extra Service Life – ESL).

Beside assumptions for maintenance strategies and their costs also the corresponding volume of the used material (concrete, reinforcement etc.) is ready for further processing. This allows for both, the calculation of the entire life-cycle costs and the estimation of the ecological costs (LCA – Life-Cycle Assessment). With a novel approach not only several types of concrete but also completely different maintenance strategies can be investigated and assessed under sustainable considerations.

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Poor life-cycle performance versus engineering design process

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ABSTRACT

Structure's life-cycle spans from its earliest concepts and outlines until it has reached the limits of its performance. Experience has shown that structural performance limits depend on more factors than may be apparent to the engineer at the conceptual stage of a project. In the case of buildings, projects that involve expansion, changes of a building's use or any other modification during a building's lifetime, may change the load patterns on the structure and affect its ability to withstand the new loads. In the case of bridges, factors such as a lack of maintenance, incorrectly determined temperature loads, the corrosive effect of deicing salts and poor conceptual design, can all diminish the structure's capacity and its ability to withstand environmental and other loads, and significantly reduce its life expectancy.

A poor life-cycle performance can have catastrophic consequences; Forensic Engineering investigates the causes of these consequences. Many extreme cases of structural failures frequently exhibit common characteristics, and the conclusions drawn from investigating the causes of these failures indicate that many of them could have been avoided if an appropriate mechanism had been in place to orient the design engineer towards a deeper evaluation beyond a limited procedure that follows the codes and standards used in the design process. The Province of Quebec is rich in "engineering challenges"; its winter weather conditions include abundant snow fall, freeze-thaw cycles, freezing rain, extreme temperature differentials and an abundant use of deicing salts. This part of Canada is also an active seismic region. The aim of this paper is to present the consequences of a poor life-cycle performance arising from limited engineering design considerations using examples of structural failures from the author's experience on the field of the Forensic Engineering, as well as some other commonly known cases. The figures 1, 2 and 3 show some of the cases discussed in the paper.

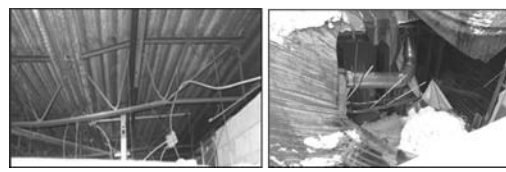

Figure 1. Failure due to increased snow loads.

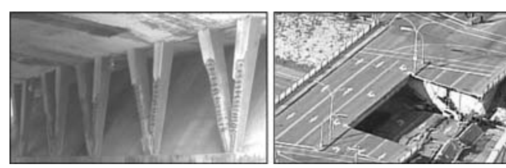

Figure 2. Failure due to the use of deicing salts.

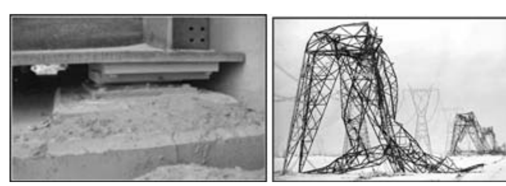

Figure 3. Failure due to temperature loads (left) and failure due to freezing rain (right).

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Life-Cycle and Sustainability of Civil Infrastructure Systems contains the lectures and papers presented at the Third International Symposium on Life-Cycle Civil Engineering (IALCCE 2012) held in one of Vienna's most famous venues, the Hofburg Palace, October 3rd - 6th, 2012. This volume consists of a book of extended abstracts and a DVD with 344 full papers presented at IALCCE 2012, including the Fazlur R. Khan Lecture, 10 Keynote Lectures, and 333 Technical Papers from 52 countries.

All major aspects of life-cycle civil engineering are addressed, including aging of structures, deterioration modeling, durable materials, earthquake and accidental loadings, sustainability, fatigue and damage, structure-environment interaction, design for durability, failure analysis and risk prevention, lifetime structural optimization, long-term performance analysis, performance-based design, service life prediction, time-variant reliability, uncertainty modeling, damage identification, field testing, health monitoring, inspection and evaluation, maintenance strategies, rehabilitation techniques, strengthening and repair, structural integrity, decision-making processes, human factors in life-cycle engineering, life-cycle cost models, project management, lifetime risk analysis and optimization, whole life costing, artificial intelligence methods, bridges and viaducts, high rise buildings, offshore structures, precast systems, runway and highway pavements, tunnels and underground structures.

This volume provides both an up-to-date overview of the field of life-cycle and sustainability in civil engineering and significant contributions to the decision making process for the purpose of enhancing the welfare of society. The aim of the editors is to provide a valuable source of information for anyone interested in life-cycle and sustainability of civil infrastructure systems, including students, researchers and practitioners from all areas of engineering and industry.

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