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Deterioration of Reinforced Concrete Structures under Normal Conditions

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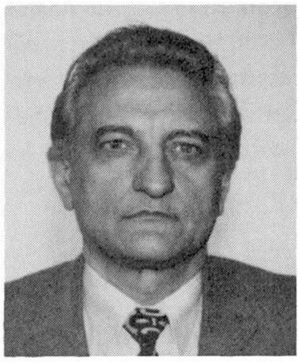

Corneliu Bob, born 1939, received his civil engineering degree from University "Politehnica" of Timisoara in 1961 and PhD in 1970. He is currently professor of Reinforced Concrete Structures and head of the National Research Institute INCERC- Timisoara branch. He published many papers and some books on various aspects of civil engineering.

Summary

The theoretical model, established by the author, as well as the experimental determinations were used assessing the stage of reinforcement corrosion in existing structures. A useful model which does not need experimental determinations is offered in this paper. A satisfactory agreement was obtained concerning the depth of carbonation and chloride penetration between the theoretical model and experimental results. The corrosion process period is analyzed, too. The minimum concrete covers, specified in different norms as well as in accordance with author's model, are also presented.

1. Introduction

In order to make quantified statements as to the service life of a member or a structure it is necessary to know the factors that affect the service life. The durability aspect is to be examined with reference to the failure or deterioration mechanisms, which are: reinforcement corrosion, chemical attacks, freeze-thaw bursting, alkali-aggregate reactions, fatigue, erosion. From these only reinforcement corrosion, freeze-thaw bursting and alkali-aggregate reactions are really important for reinforced concrete structures under normal conditions. The durability of concrete structures depends both on the resistance of the concrete against physical and chemical attack and on its ability to protect embed steel reinforcement against corrosion. In a climatic region the most common damage is the corrosion of the reinforcement adjacent to the exposed surface. The concrete structures examined in Romania by the author have had some deterioration of the component parts due to of reinforcement corrosion as main factor.

2. A quantitative model of reinforcement corrosion

In a reinforced or prestressed concrete element a so-called passivation layer is formed on the surface of reinforcing bars. This passivation layer may ,however, be attached by the surrounding concrete environment, so that electrical potential differences may develop along the bars: electrochemical corrosion will then take place. The progression of deterioration of an element with time is described by initial period and corrosion process period. A numerical calculation method for both initial period (time until deterioration start) and corrosion process period (time of deterioration) is presented. The author of this paper has suggested a formula for the average value of the depth of carbonation and for the chloride ion penetration as factors of initial period. The corrosion process rate of the reinforcement is also presented.

The proposed formula takes into account: the binding capacity of the cement type, environmental conditions, surface concentration and permeation properties by concrete compressive strength. The use of this parameter was suggested because of: the concrete compressive strength is a conventional quantity and its value depends on a multitude of factors, among them, the quality and content of the cement, the water-cement ratio, the aggregate characteristics, the casting conditions etc.; the concrete compressive strength is the major criterion when assessing the quality of a concrete class for the design of a new concrete structure as well as for judging of a concrete structure which has to be renovated.

3. Experimental determinations

Some concrete structures with different duration of service life and deterioration of component parts have been examined and proposed for rehabilitation. The stage of carbonation and/or chloride penetration in reinforced and prestressed concrete elements were theoretical and experimental assessed. For experimental assessment the concrete was extracted from the structure elements with an electric drill from different depth and was stored in small and air-tight boxes till laboratory analysis. The analysis were performed by phenolphthalein test and pH methods.

Theoretical assessment is based on the formula and data presented. Such quantitative model is very useful for engineers in judging the concrete structures which have to be obsolete and are then radically renovated or demolished as the bridge of 80 years old. The compressive strengths, used in formula for average depth of carbonation or chloride penetration, was established with nondestructive methods. On the other hand, the strength values were also used for the structural analysis of the buildings.

Data presented points out the importance of two parameters for the durability of the reinforced concrete structures under normal conditions: the concrete cover and concrete strength. The minimum cover to reinforcement, in mm and concrete quality for durability are presented.

4. Conclusions

A quantitative model of reinforcement corrosion for both time until deterioration starts (initial period) and time of deterioration (corrosion process period) is presented in the work. The model suggested is an analytical tool for the diagnostic guide and control of the concrete structures; only a few parameters easily to be obtained are necessary. On the other hand, experimental determinations were made on several and various elements of the concrete structures. A satisfactory agreement was obtained between the quantitative model and the experimental data. As a result of the authors studies an important conclusion for design is pointed: the importance of the strength and depth of the concrete cover for the durability of the reinforced concrete structures under normal conditions

Reliability Analysis in Structural Masonry Engineering

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Luc Schueremans, born 1970, graduated as a civil engineer at KU Leuven in 1995. Since 1995, he is a researcher at the Building Materials Division of the Civil Engineering Department of KU Leuven. His research focuses on the development of a reliability method for the structural evaluation of ancient masonry structures.

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Summary

This paper presents a probabilistic method to evaluate the reliability of structural masonry elements. This methodology is meant as a decision tool in the restoration process to determine whether of not structural strengthening by grouting or other methods is needed. The proposed reliability analysis method will be illustrated on three structural masonry problems. The first illustration focusses on the theoretical aspects and calculates the local probability of failure of a masonry shear wall. Two case-studies deal with the practical goals in masonry engineering problems: the reliability analysis of a masonry sewer system and the reliability of the façade of the St.-Amandus chapel. This research is part of a complete program on structural strengthening of ancient masonry and grout design, in which risk analysis, grouting and non-destructive tests are considered.

1. Introduction

Research in the Reyntjens Laboratory on ancient masonry deals with different aspects of restoration and renovation of masonry structures : diagnosis, (non)-destructive testing methods, strengthening and repair. As consolidation and strengthening of ancient masonry are always expensive procedures, it is of utmost importance to decide whether or not these interventions are required. For this purpose, a reliability analysis using a FORM-algorithm is performed. This is illustrated on tested shear walls, reported in literature. A simplified analysis is presented to calculate the probability of failure of a masonry sewer system. In the third example, the global probability of failure of an out of plumb standing façade is calculated.

2. Local probability of failure of masonry shear panels

A reliability analysis is performed to calculate the reliability index β or the local probability of failure p_f of masonry shear panels. Attention is paid to the applied methodology, the algorithm (FORM : First Order Reliability Method), the different failure modes and corresponding limit state functions, figure 1, and the probability distributions for the basic variables in these limit state functions : the masonry material properties and stresses in the masonry due to external loads. The probability of failure p_f or the reliability index β, is calculated in different points of the masonry shear panels. The obtained results are plotted in contour graphs. These provide a visual interpretation of the local probability of failure or of the reliability index.

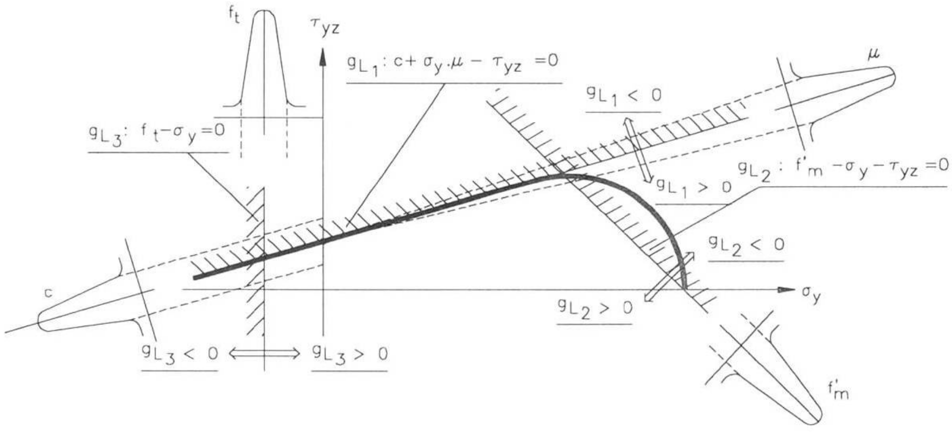

Fig. 1: Linearised "Cap-model" for unreinforced masonry shear walls

3. Sewer System

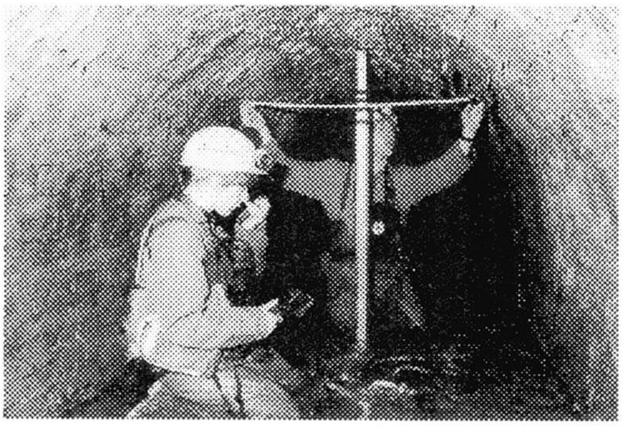

Fig.2: Sewer system Schiffweiler - Neuland

The method was applied in a restoration project to calculate a first value for the safety level of a masonry sewer system, figure 2. In this application, reaching the compressive strength of the masonry was the only ultimate limit state considered: $g(x_1,x_2) = x_1-x_2$. The calculated probability of failure amounts: $p_f=\Phi(-\beta)\cong 10^{-3}$.
Whether such a safety level is acceptable or not is a socio-economical problem. The probabilistic analysis provides a quantitative measure of the safety that can be used to compare different alternatives.

4. The façade of the St.-Amandus chapel

In the case study of the Saint-Amandus chapel at Erembodegem, the global probability of failure of an out of plumb standing wall is calculated. To save the authenticity of the chapel it was decided to minimize the (semi)-destructive test program. Because of the leaning forward of the façade, it was decided to monitor the evolution of the cracks, deformations and eccentricities. Supplementary, a reliability analysis of this structural element was performed, to assess the remaining safety. Therefor, it is required to be able to evaluate the reliability based on limited data: the geometry and the measured eccentricities.
The results are outlined in a graph: the measured eccentricities are plotted on the x-axis, the reliability index β is plotted on the y-axis. That enables to judge the remaining safety, using the eccentricities as a single input parameter.

Reliability-Based Evaluation Concept for Everyday Use

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A graduate of ETH Zürich, in 1989 Peter Tanner joined ICOM (Steel Structures) of the Swiss Fed. Inst. of Technology, Lausanne where he worked in the fields of fatigue and fracture mechanics. Since 1992 he has worked with consultants, before joining IETcc in 1996. His research fields are durability of concrete structures and reliability of existing structures.

Summary

This paper describes an approach for the development of a method for considering actual loads and resistance during the evaluation of existing building structures. The method is based on the use of a live load model and partial safety factors for loads and resistance, calibrated by applying reliability methods, which can be determined as a function of site characteristics. The method enables the accurate evaluation of existing building structures for which the degree of uncertainty related to loads and resistance can be reduced compared to that assumed by design codes. Due to this reduction, acceptable reliability may be verified even for structures that are damaged or deteriorated, thus avoiding the need for strengthening or live load restriction.

Keywords: structural reliability, probability of failure, assessment, probabilistic analysis, deterministic analysis, site characteristics, model updating, calibration

1. Motivation

When assessing the safety of an existing structure, the information is different from that available during design, because many characteristics may be measured from the structure under consideration which, at the time of its design, were just anticipated quantities. It is always possible to improve the level of accuracy for the load and resistance models, which are needed for the assessment, by collecting more data about a particular structure. In most cases, the cost of updating of information by collecting site data is outweighed by a significant reduction in the cost of intervention: possible consequences of an over-conservative evaluation of an existing structure include unnecessary live load restrictions, strengthening or demolition.

The most accurate way for an engineer to consider actual load and resistance would be to carry out a probabilistic analysis using site data. However, this is a time consuming process, involving a considerable understanding of probabilistic methods, and is possibly not aimed at the practising engineer for everyday use. A simplified deterministic method for the assessment of structural safety should therefore be available, based on the same partial factor formulation adopted in codes for structural design.

Applying reliability methods, a procedure for the calibration of site specific deterministic load and resistance models for the assessment of existing building structures is proposed in the present paper.

2. Calibration of site specific load and resistance models

The calibration of site specific load and resistance models for deterministic assessment is based on the *axiom* that a correct application of the current codes results in a safe structure. Partial safety factors, which can be introduced in a deterministic assessment of an existing structure, are derived for action effects and resistance. In the case of self-weight, permanent actions and resistance, the obtained partial safety factors are presented according to the type of structure, as a function of the coefficient of variation used when modelling each variable. Partial safety factors used in a deterministic assessment are thus based on the coefficient of variation of the corresponding variable. This represents the change in the associated uncertainty (due to the collection of site specific data) in relation to the models that are assumed to lie behind the rules of codes. For the purpose of obtaining a simple set of values for practical evaluation of a particular type of structure, it is proposed to determine maximum factors as a function of only the coefficient of variation of the corresponding variable (Figure 1). It would however be possible to make more distinction between different types of specific use, the type of load effect, and possibly even the span length. For live loads, it is proposed that the calculated partial safety factors are presented according to the type of specific use as a function of the tributary area only.

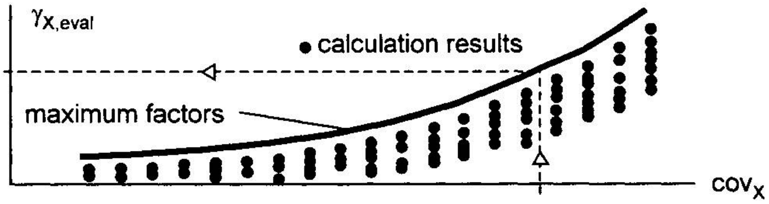

Fig. 1 *Schematic representation of partial safety factors for deterministic assessment, $\gamma_{X,eval}$, as a function of the coefficient of variation of the corresponding variable, cov_X*

3. Deterministic assessment with site specific models

Partial safety factors, which are attributed individually to the basic variables in a Limit State Function, derived by applying reliability methods, can be introduced in a deterministic evaluation of an existing structure, using the partial factor formulation adopted in design codes. These factors ($\gamma_{S,eval}$ and $\gamma_{R,eval}$ for action effects and resistance, respectively) are to be used together with the actual nominal values for action effects, $S_{eval,nom}$, and resistance, $R_{eval,nom}$, as described below. The requirement for structural safety in its simplest form can be expressed by the following condition:

$$\gamma_{S,eval} \cdot S_{eval,nom} \leq \frac{R_{eval,nom}}{\gamma_{R,eval}}$$

In the case of self-weight, permanent actions and resistance, site specific data is used to determine the coefficient of variation of the corresponding variable and its actual nominal value (mean values for permanent actions and cross-sectional properties, characteristic values based on a 5% fractile with a confidence level of 75% for material properties). The partial safety factor to be used in a deterministic assessment can be selected as a function of the associated uncertainty, represented by the coefficient of variation determined from site data as mentioned before. Figure 1 shows schematically the relationship to be used for selecting a partial safety factor, $\gamma_{X,eval}$, as a function of the measured coefficient of variation, cov_X. Variables that are not measured should be considered along with those of the default models (nominal value and partial safety factor) prescribed in current design codes. Live loads can vary significantly with the tributary area. Therefore, the live load model (characteristic value of the equivalent uniformly distributed load and partial safety factor) to be applied in a deterministic assessment is selected as a function of site characteristics, which can be represented by the specific use of the building and the tributary area for the element under consideration. The tributary area can be determined from available information about the structure, updated by visual inspection.

Nondeterministic Assessment of the Structural Performance

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Gheorghe Cristea, received his civil engineering degree and PhD from the Technical University of Civil Engineering-Bucharest, where he is now professor of Applied Computer Science. His teaching and research activities deal with Computer Aided Structural Engineering, with emphasis on Artificial Intelligence and Fuzzy Logic.

Summary

The structural parameters of the damaged structures, usually, cannot be assessed by precise values. The paper describes a method for the analysis of structures having the parameters defined by nondeterministic values. Using nondeterministic algebra operations rules, the method computes directly the nondeterministic values of the displacements and stresses. The confidence functions of the input parameters are processed to derive the confidence functions of the structural response. The method, based on general principles, has a broad application, for all the types of practical structural analysis.

1. Nondeterministic models

The nondeterministic approach can give qualitative and quantitative information about the confidence in the computed response of the structure. Starting from the nondeterministic values of the input data, the nondeterministic values of the displacements and stresses are derived. The nondeterministic data are defined by the crisp support x and the confidence function $\varphi(x)$. The confidence function $\varphi(x)$ and its definition interval $[x_{inf}, x_{sup}]$ can be defined according to the following nondeterministic models: the random model, the fuzzy model and the heterogeneous model. The random model is effective if the probability density function can be derived from a large enough number of sampled values. The fuzzy model has a broad applicability because it does not depend on the number of sampled values for the nondeterministic variable. The unified approach of the two above models, the random model and the fuzzy model, gives the possibility to use random and fuzzy variables together, during the same processing.

2. Nondeterministic computing method

The assessment of the confidence function is one of the most important features of the nondeterministic structural analysis. In the case of random variables, the confidence function, represented by the probability function is derived starting from the relative frequencies. In the case of fuzzy variables the confidence function is defined by the following methods: (1) The prototype method; (2) The method of the relative membership.
To perform a nondeterministic algebraic operation it is necessary to find out the deterministic support and the confidence function of the operation result.
The deterministic support issues from the following operations: (a) for discrete operands x, y:

$z = x \; op \, y$; (b) for intervals, J: $J_Z = J_X \; op \, J_Y$, where the result of the operation is the margins of the J_Z interval.

The confidence function is approximated on subintervals, by piece-wise linearisation. For two subintervals of the operands x and y, the linear functions are $f(x) = a \cdot x + b$; and $f(y) = c \cdot y + d$, respectively. The confidence function, $f(z)$ results according to specific composition laws: (a) for random operands, $f(z) = f(x) \cdot f(y)$; (b) for fuzzy operands, $f(z) = \lambda_x \cdot f(x) + \lambda_y \cdot f(y)$; λ_x, λ_y are weight functions, specific to the operations.

3. Example

The method application is exemplified for the nondeterministic analysis of a reinforced concrete plane frame (fig. 1). The frame was subjected in the past to strong earthquake motions which caused partial damages. The existing cracks, especially at the members' ends decrease the stiffness, so that the end-connections behave like partial hinges. Because the stiffness value of members and connections is uncertain, they were defined as nondeterministic variables (fig.2, table 1). As a result, the member stresses have nondeterministic values(fig. 3, table 2). According to the Romanian Code P100-91, the interstorey drift condition, is: $\Delta_r / H_e \leq 0.0035$. The condition checked at the first floor, for the values of the displacements is as follows:

$\Delta_r / H_e = \{0.748\backslash 0.00419, 0.849\backslash 0.00304, 0.803\backslash 0.00229, 0.736\backslash 0.00209, 0.698\backslash 0.00201\} \leq 0.0035$

The result is: 0.849*TRUE*. Finally, the drift condition is satisfied if $0.849 \geq \mu_{\min}$, where $\mu_{\min}$ is the minimum value required for the confidence degree of the condition.

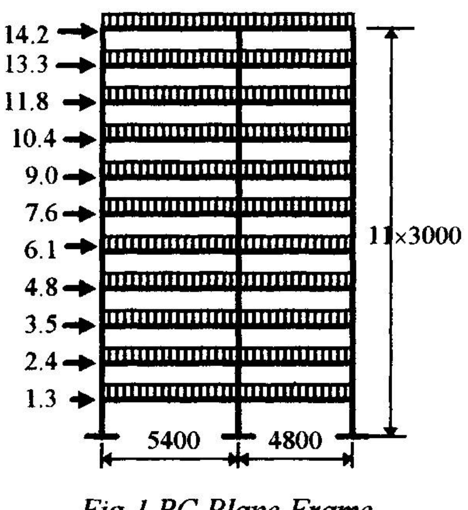

Fig.1 RC Plane Frame

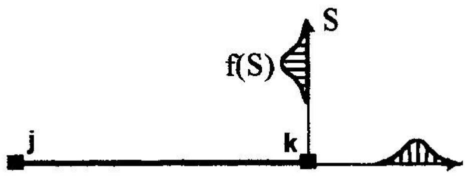

Fig.2 Nondeterministic Stiffness

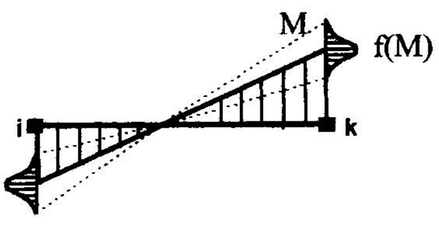

Fig.3 Nondeterministic Bending Moment

Table 1. Nondeterministic Stiffness.

Member		Values at the interval boundary				
1	f(S)	0.689	0.797	0.895	0.741	0.653
	S	123435	98132	91540	87243	83356

Table 2. Nondeterministic Bending Moment.

Member / Joint		Values at the interval boundary				
1	f(S)	0.742	0.863	0.826	0.773	0.692
4	S	169.3	156.2	143.4	139.2	127.6

Evaluation of the Load-Carrying Capacity of Structural Members in Existing Buildings by Proof Load Testing

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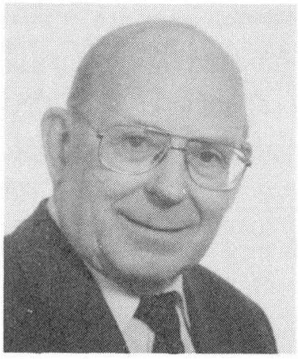

Gerhard Spaethe, born in 1932 received his civil engineering degree from the Technical University Dresden in 1958. He was at last head of the department of structures and deputy director in the Institute of Rehabilitation and Modernisation of Buildings in Berlin. In October 1997 he retired.

Summary

The first part of the paper reports about experimental work testing prestressed concrete slabs by a self-securing system consisting of a hydraulic device and the measuring-equipment plotting deflections and strains on-line on the computer screen. Tests of connections in multilayer external walls are mentioned. The verification of structural safety after proof load testing is considered. Applying the Bayesian procedure for the determination of the characteristic value (5 %-fractile) of the resistance, taking into account prior informations before the test and additional information by test results, the verification of the structural safety can be carried out by the limit state method.

1. Experimental Method

If great uncertainty in the theoretically calculated load-carrying capacity (resistance) exist and if safety reserves are expected in structural elements of existing structures an experimental test in situ can be very useful.

Modern developments in hydraulic loading devices and in computer based on-line measurement technology have improved the experimental possibilities for proof load testing.

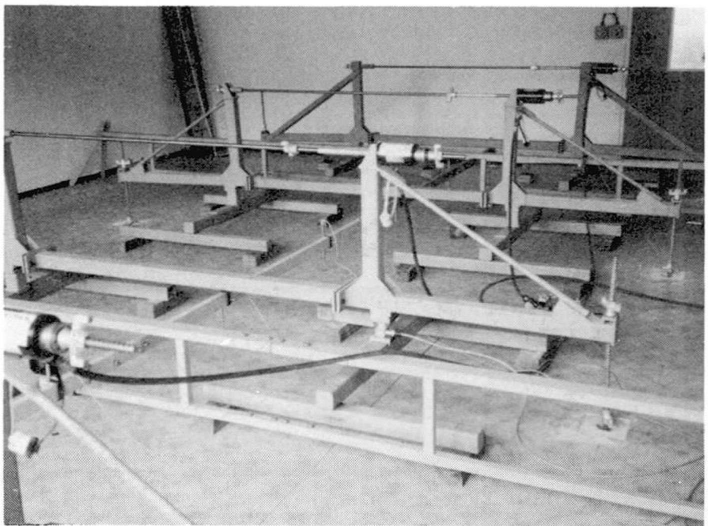

Fig.1 Set up for proof load testing of slabs

Fig. 1 shows a set up for tests of slabs built as a component system in existing buildings. Tensile forces are induced in the upper chord of a truss. The compressive forces at the bottom are distributed by a leverage system to get quasi-equally distributed loads. The reactive forces are induced into neighbouring walls or into the support of the slabs in the floor below.

The results of the measurement of the forces, the deflections and the strains in steel and concrete are shown on line at the computer-screen . The experimenter can so directly control the state of the structure during the proof load test and guarantee, that the linear, elastic area is not exceeded. So a self-securing system is available which avoids damage to the tested structure entirely or almost entirely.

2. The determination of the characteristic value of the load carrying capacity with the Bayesian procedure

The practical verification of the structural safety by the limit state method with partial safety factors is based on the characteristic value, which is defined as 5 %-fractile of the load-carrying capacity of the structural element. If a reliable value of the characteristic value is available by the proof load test than the safety check can be carried out by the principals of limit state design in a well know manner.

The Bayesian method is applied to determine the characteristic value as 5 %-fractile of the load carrying capacity by a proof load test, taking into account prior information by calculation or engineering judgement before the test.

It is shown that in the case of a transformation from a tested element to a not tested one from the same population the posterior distribution function of the resistance R is

$$F_R''(r) = \int_{-\infty}^{+\infty} F_R(r|\vartheta) f_\Theta''(\vartheta) d\vartheta \quad = \frac{1}{c} \int_{-\infty}^{+\infty} F_R(r|\vartheta) \prod_i (1 - F_R(s_{pi}|\vartheta)) f_\Theta'(\vartheta) d\vartheta$$

and in the case of a direct testing

$$F_R''(r) = \begin{cases} \frac{1}{c} \int_{-\infty}^{+\infty} \frac{\left(F_R(r|\vartheta) - F_R(s_{p_j}|\vartheta)\right)}{1 - F_R(s_{pj}|\vartheta)} \prod_i (1 - F_R(s_{pi}|\vartheta)) f_\Theta'(\vartheta) d\vartheta & if \quad r > s_{pj} \\ 0 & if \quad r \le s_{pj} \end{cases}$$

4. Conclusions

From the results of an example the following conclusion can be drawn:

- The higher the proof load is in a successful test, the higher is the characteristic value of $R_{0,05}$.
- The characteristic value $R_{0,05}$ increases with increasing number n of tests.
- The characteristic value is greater if the structural member is directly tested than in the case of conclusions from tests of other elements from the same population.
- The more diffuse the prior information is, the more effective is the experiment.
- In the case of an exact prior information only the direct testing gives an effect.
- If the structural element is tested directly and successfully with high load-levels the prior information becomes irrelevant.

These results are in good agreement with the engineering-experience. Bayesian procedure gives the possibility to combine information about the load carrying capacity R by calculation before the test with new information by testing and gives a rational basis for the verification of structural safety by the limit state method. With these experimental and theoretical methods existing structures often can be saved and used for a longer residual service life.

Assessment of Structural Safety of Prefabricated Buildings

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Summary

In the course of time, the load-bearing system of multi-storey prefabricated buildings is exposed to loads with variable, alternating and cyclic components (temperature, moisture, wind, ground vibrations, dynamic traffic impacts, sound wave impacts etc.). In relation to the ratio of the permanent and alternating load components, the construction and reinforcement of bearing joints and bonds, degradation of structural properties of the joints and bonds occurs due the impacts of variable unidirectional, as well as alternating loads resulting in lower structural safety of the load-bearing system and consequently in its decreased residual service life. Practical examples testify to the time factor impact on the degradation of the structural qualification of the load-bearing system. A reliable reconstruction design of prefabricated buildings has to be based on numerical assessment of the load-bearing system considering residual joint rigidity.

The properties of load-bearing structures are characterised by the so called load-bearing qualification which may be defined as the ability of a structure, or a load-bearing system to fulfil the required load-bearing functions from the point of view of ultimate bearing capacity and functionality under static and dynamic loads and other impacts causing mechanical states of stress or deformation and strain.

Load-bearing structures are exposed to: the impacts of vertical and horizontal loads, climatic load impacts (wind, snow, temperature, moisture), the impacts of changes in the footing bottom shape, dynamic traffic impacts, rheological impacts, as well as chemical, biological and other impacts.

During the service life of buildings, these impacts may be visibly displayed by the appearance of failures. In assessing failures it is necessary to distinguish between technological defects or failures which are often associated with prefabricated structures and structural failures caused by some of the above mentioned impacts. A major part of failures of prefabricated structures are those caused by temperature and moisture.

The intensity and character of the resulting loading effect change in relation to the intensity and ratio of individual loads in a given time. Apart from decisive vertical loading effects due to the weight of load-bearing and finishing structures which may be specified as constant, permanent, unidirectional loads with negligible dependence on time, load-bearing systems are exposed to short-term or variable impacts, both of unidirectional and alternating character. These are, above all, temperature, moisture, wind impacts, variable components of operable loads, ground vibrations, dynamic traffic impacts, sound wave impacts etc. These impacts cause that individual parts of the structure are, in the course of time, exposed to loading with a variable, alternating and cyclic component. In relation to the ratio between the constant and alternating component, the construction and reinforcement of the joint, degradation of static properties of joints exposed to the impacts of variable loads may occur in time lowering the structural safety of the load-bearing system or affecting the service life of the load-bearing system.

The relevance of defects and failures discovered during a structural and technical investigation may be assessed on the basis of structural, as well as constructional and physical evaluation based on a truthful computational model of the structure, a computational model of loads and a material model of the structure. In this relation the properties of materials and structures have to be considered as variable quantities, depending on time and environment. Without the knowledge of their time-dependent behaviour, the problem of durability and reliability of load-bearing systems cannot be solved. The changes of properties in time are most frequently caused by variable unidirectional and alternating loads, corrosive and degradation processes (physical, chemical and biological impacts).

A reliable reconstruction design of a prefabricated building is based on numerical evaluation of the structural qualification of the load-bearing system considering the lowered rigidity of bearing units' joints, or the so called residual rigidity of joints suffering from mechanical failures during investigation. This procedure always has to be respected if failures caused by cyclic temperature and moisture impacts are in question. Corresponding structural modifications and rehabilitation of the damaged units and joints have to be assessed with regards to the history of loading to avoid repetitive occurrence of failures.

The aim of numerical analysis is to determine the so called critical points of the structure (load-bearing system) which are of vital importance to the structural safety and reliability of the system. Defects and failures occurring at these points belong mostly to the category of serious failures of the load-bearing structure requiring, as a rule, immediate and extraordinary measures (such as temporary structural support, structure lightening etc.).

Among serious defects and failures there are all defects and failures lowering, in a significant way, the ultimate bearing capacity and rigidity of the load-bearing system. A gradual development and propagation of failures (cracks, disintegration etc.) create redistribution of internal forces from damaged points to unaffected areas. If there are no reserves in the structure to compensate for the increased loads due to this type of redistribution, local or overall failure (collapse) of the structure (system) may occur. Among these serious failures there are prominent continuous cracks in vertical joints of walling units, cracks and ruptures at the joints between walling and floor units, prominent continuous cracks in longitudinal joints of floor units, impaired and insufficient ultimate bearing capacity of walling and floor units.

In safeguarding 3D rigidity of the load-bearing system and its resistance to the effects of extraordinary loads leading to breakdown condition of the building, principal importance is attributed namely to horizontal and vertical reinforcement installed in the units and interconnected at joints, or reinforcement placed in the units' joints, i.e. bracing and sealing reinforcement. Prefabricated structures with a non-existent or insufficient dimension and implementation of this reinforcement show a small range of elastoplastic and plastic deformations, they are liable to the appearance of failures and they are not sufficiently safe in relation to extraordinary loads of breakdown type.

Causes of Failures and Methods for Repair of Weather Panels

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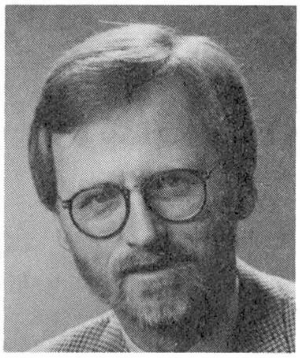

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Summary
Based on investigations of his own the author concludes that many façades do no longer fulfill the requirements of stability. Before applying an External Thermal Insulating Composite System (ETICS) on Large Panel Structures it has to be checked if the external "weather panels" are stable and durable especially with regard to the additionally caused hygrothermal forces. Hints are given for detecting structural failures and their causes. For the rehabilitation and stabilization of curtain walls and weather panels a number of solutions in the form of bolt or corbel structures are available.

1 Structural Concept

Newly erected buildings appear in all their glory; unfortunately this cannot be preserved for long. Within a few years first signs of aging appear that are mainly caused by weather. The relevant actions were often unknown to the designer, or even inaccessible. Today details for the design of façades, e.g. the hygrothermal actions, are better known. The design, the manufacturing, and the construction of façades require special knowledge, especially regarding multi-layer sandwich panels.
The designer has to take into consideration that

- the thickness of the weather panel according to the design is often only 6 cm,
- the stresses from the dead weight of the structure are concentrated at only a few locations where they are transferred to the substructure.

Therefore it is clear that the required precision for the manufacturing is within a range of millimeters. Furthermore the transfer of the dead load causes deflection forces and tensile stresses in very thin members, which require special care when placing the required reinforcement.
Many façades do not fulfill the requirements of stability according to the present state of knowledge. Considering the difficult production of filigree concrete members lots of buildings call for immediate action. This paper compiles hints to help controllers detect potential defects. Principles and systems for the rehabilitation and stabilization of weather panels are described.

2 Causes of failure

Since the 1960s residential and public buildings in the former German Democratic Republic have been predominantly built as large panel structures. With the beginning of the 1970s the 'Wohnungsbauserie 70' (WBS 70) was standardized. The room-size units were manufactured by collective combines so that it could happen that precast members from different production plants were delivered to one construction site.
It is necessary to describe the production of a sandwich panel in order to explain actual dimensional deviations. The panel was produced in a horizontal position. The sandwich panel

was manufactured in three steps. As a rule the load carrying layer was poured first; after having applied the insulant the casting of the weather panel was done. If it was done the other way round, it could happen that through the weight of the load carrying panel the insulant was pressed into the weather panel, the concrete of which was not yet hardened. This led to significant thickness variations of the weather panel. Sometimes the fresh concrete passed through the joints of the insulant thus forming 'concrete bridges' between the weather panel and the load carrying wall.
Devices for the subsequent stabilization of weather panels require an official agrément. The applicants had to measure and evaluate the actual thickness in various buildings where the target thickness was 60 mm. The mean value of 57 mm was in the range of usual tolerances. The 5%-fraktile of this sample was 40 mm. An extreme value was 80 mm.
In the draft of a letter by the Ministry of Building and Construction of the GDR to the manager of the collective combines, dated January 1989 [1] it can be read that "... the investigation on the weak points of the weather panels revealed a poor and alarming quality performance of sandwich panels." This evaluation was based on a paper by K. Ritter [2] describing 27 typical mistakes.
Another deficiency was the use of steel without corrosion protection for the load carrying anchors. In sandwich panels anchors with a welded "normal steel - stainless steel" joint were found, e. g. as detailed in design documents by the collective combine of Rostock. There are no basic objections against this kind of connection if the welded parts and the structural steel are protected against corrosion. With regard to the dimensional it is difficult to verify an appropriate corrosion protection. A comprehensive inventory of the actual layer thickness can be obtained where new façade elements are mounted. In this case holes have to be drilled for fastening the new façade with anchor bolts. In Jena these measurements revealed weather panels with a thickness down to 25 mm. Partially the reinforcement of the weather panel was exposed and already corroded.
It can be stated that load carrying anchors made of stainless steel are a **necessary but not sufficient** requirement for durability. The actual thickness of the weather panel plays an important role as well. Panels that are partially too thin have to be secured additionally; where the thickness is insufficient it might be necessary to replace the panel. Panels whose thickness is less than 40 mm require subsequent stabilization. The required stability of such panels cannot be verified due to the lack of secondary reinforcement and the insufficient anchorage length of the load carrying anchor within the weather panel. Usually on these panels new lightweight units are applied. The additional load has to be transferred into the load carrying wall without impairing the overall stability. A spot check-like control of a few panels is not sufficient because the quality of precast units from different manufacturers may differ considerably. **A conclusion from 'n' to 'n+1' is not acceptable. A comprehensive securing appears to be more appropriate, both technically and economically, rather than an extensive building diagnosis.**

3 Symptoms for early damage detection

Easily noticeable corrosion damages indicate serious defects. Besides the characteristic color of rust concrete pieces spalled off due to corrosion pressure are also easily detectable. A settling of the weather panel due to lack of load carrying capacity produces compression and protrusion of the sealing compound or cracking of the upper edge sealant. The lateral joints show diagonal cracks. But also varying widths of the joints –unless they were caused during the erection process- horizontal or vertical recesses indicate flaws. In a particular case the weather panels had settled in such a way that the quarry stone cladding underneath became load carrying and sheared off.
It can be concluded that often the combination of several symptoms indicate a damage. The diagnosis should be established by experienced engineers.

Long - Term Behaviour of Reinforced Concrete Panel Walls

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Summary

Most of multifamily buildings constructed in large panel technology in Poland have outer façade precast layer fixed by special steel links. These buildings are actually thermally inefficient. There are also noticed certain failure of the façade plates. Additional thermal insulation requires evaluation of strength and technical conditions of the external plate and links coupling with the bearing precast wall.
Paper presents some results of investigation and analysis of outer precast panel in large panel building constructed in OWT system in Białystok.

1. Introduction

Within last four decades in Poland were constructed tenfold thousands of multistories RC large paneled buildings. Most of the buildings have RC composite three layered external wall where outer façade plate is suspended with steel links to the structural bearing plate.
New requirements in respect of energy saving solution proved that these buildings are thermally inefficient, besides some problems related to safety of the external façade plates arose recently [1], [3].
Evaluation of strength and technical condition of the outer façade plates and steel links coupling with the bearing part of the wall became indispensable in view of currently realized additional thermal insulation. Paper presents results of investigations and assessment of technical state of the outer RC façade plate in multifamily buildings constructed in large panel system OWT in Białystok.

2. Results of testing

768 wall large panels were investigated directly in the buildings where twelve suspension steel links ϕ12mm coupling outer façade plate with bearing part of the wall were uncovered and tested.

Twelve insignificantly damaged wall panels prefabricated within period when buildings were constructed still remained on the stockyard were investigated in the precast yard.
The following incorrectness, defects and failures of the built-in RC façade plates in investigated buildings were detected and discovered:

- at about 40% of investigated panels had visible cracks width of ~1.5mm and external failure of concrete,
- damaged corners of the panels and uncovered already corroded reinforcement,
- significant loss of granulated grit externally covering the façade plate,
- varying thickness of the outer plate differing in 2.5 - 5.0 cm,
- heterogeneous not uniform and porous structure of concrete,
- low quality of concrete class evaluated in the range of 7.5 - 12.5MPa,
- delamination of concrete in the façade plate,
- insufficient thickness of cover layer for reinforcing wire at the thermal insulation side. In limiting cases no cover at all,
- initial phase of corrosion in uncovered reinforcement of façade plate,
- reinforcement and link bar ϕ8mm anchoring coupling link ϕ12mm are directly placed on thermal insulation without any cover of concrete,
- partial melting or completely melt out polystyrene insulation around the coupling links ϕ12mm (~25% of investigated links),
- concrete plugs around the coupling links on all thickness of the thermal insulation layer.

Some evident defects of anchoring and coupling links were discovered:

- 40% of investigated links have no protecting cover of concrete from outside,
- the thickness of corrosion in coupling links were 0.5 mm,
- deviation of coupling links ϕ12 from vertical position were at about 30°,
- coupling links were placed and arranged inconsistently to the design arrangement,
- bars ϕ8mm linking and anchoring suspension hangers ϕ12mm were often placed directly on the thermal insulation, hence there's no required bond of surrounding concrete.

Resembling the above defects were detected in large panels in the precast yard. Remarkable in many investigated panels lack of cover for reinforcement at the thermal insulation side or inadequate its thickness were noticed in many outer façade plates.

3. Conclusion

- construction and materials defects of RC façade plate create different working condition than those foreseen on the stage of design,
- detected degree of corrosion in the coupling links does not reduce their bearing capacity,
- poor quality of concrete in the façade plate, its delamination and low strength creates inadequate anchorage of coupling links,
- insufficient concrete cover of reinforcement in façade plate, cracked and damaged concrete can lead in result to withdrawal of reinforcing wire mesh and failure of the plate,
- the façade plate left in existing state may lead to continuously progressive corrosion of reinforcement and concrete and destruction of all wall panel in result,
- additional thermal insulation improves thermal and moisture conditions in the panel but it cannot be anchored in the façade plate,
- large panelled wall should be investigated and analysed in respect of future anchorage and its technical state before construction of new façade layer with additional thermal insulation.

Fatigue Life of Railway Bridge Welded Joints

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ABSTRACT

In the period from 1956 to 1990 the Metal Structures Department of the Technical University of Szczecin conducted X-ray examination of butt welded joints in the mains girders of 155 operated bridges. The most dangerous weld defects, i.e. cracks, were detected in 34 bridges at 437 radiographs. All the cracks but one were found in the bridges built before 1960.

Within the work presented in this paper an analysis of strength of the welded joints with cracks was carried out for 9 plate girder railway bridges of span between 14,5 m and 31,4 m. There were 1408 radiographs taken altogether for those bridges; as much as 67,2% of the radiographs were counted among R4 and/or R5 defectiveness class which are inadmissible for the new bridge structures in the light of the relevant Polish Standard's rules.

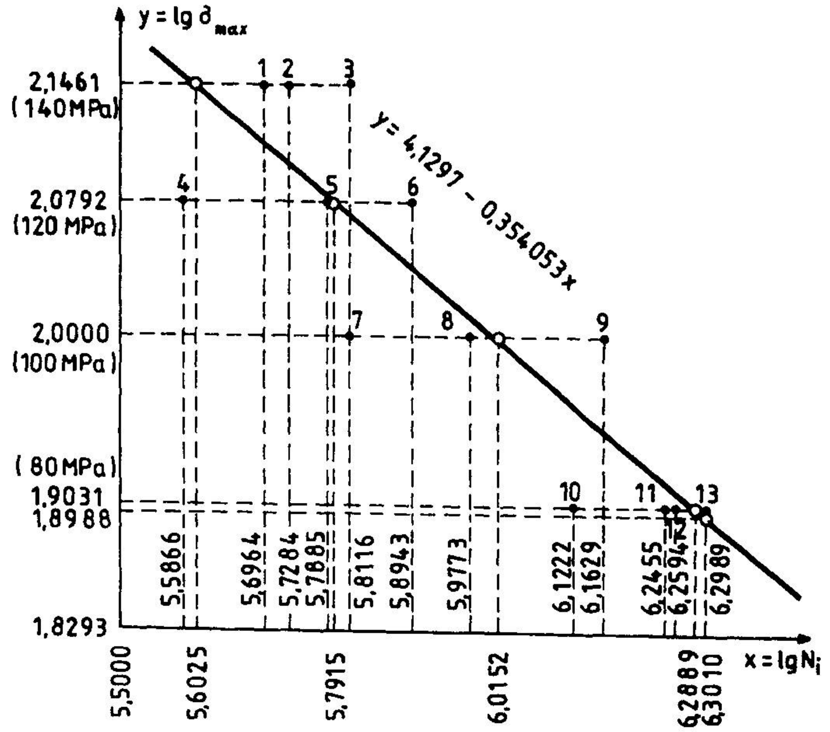
Each flange butt joint was covered one-sidedly with rhomboidal plates situated on the internal surface of the flange. These cover plates had a thickness of 8 mm and they were 80 to 160 mm wide and 160 to 240 mm long.

The maximum global stresses in particular flange joint, calculated from unfactored dead and service load varied from 42,9 MPa to 74,4 MPa whereas within the zone just beyond the straps these figures ranged from 51,9 MPa to 84,2 MPa (design value, without stress concentration effect taken into account).

At the Metal Structures Department of the Technical University of Szczecin, some laboratory tests were carried out on the model of a welded joint that corresponded to the design solution of the bridges flange joints subjected to the strength analysis. The specimens, sized at 180 mm x 12 mm x 720 mm, had the X-type transverse butt weld reinforced one-sidedly with two rhomboidal cover plates (straps) 70 mm x 4 mm x 120 mm. The X-ray examination of the specimen detected a medium continuous lack of penetration in all the welds, as well as numerous medium and small slag inclusions.
The tests were carried out under pulsating tension at 4 levels of the nominal stress: 140, 120, 100 and 80 MPa, by means of a pulsator at frequency of 500 cycles per minute. The cycle stress ratio ($R = \sigma_{nin} / \sigma_{max}$) was equal to $R = 0{,}1$. On the basis of the test results, by means of the least square method, the following regression line equation was formulated:

$$y = 4{,}1297 - 0{,}354053\ x$$

where: $x = \lg N_i$, $y = \lg \sigma_{max}$, N_i - number of cycles to the specimen failure , σ_{max} - maximum value of stress, in MPa.
The value of mean safe fatigue strength derived from the regression line equation for $N_i = 2 \cdot 10^6$ cycles was $Z_{rj} = 79{,}2$ MPa, and for $N_i = 10^5$ cycles was $Z_{rj} = 228{,}8$ MPa.



Regression line derived from fatigue tests

The analysis of the unfactored stresses calculated at joints under the dead and service load as compared to the fatigue limits obtained from the laboratory test as well as from the literature data was a decisive factor in the way of treating the joints with cracks.

4 bridges were reinforced with additional cover plates riveted to the flanges. The welds with cracks in 5 other bridges, though inadmissible in bridge structures, were decided to be left without reinforcement - which was based on the relatively small value of the computed stresses, lower than the safe fatigue strength obtained from the laboratory tests. At the same time a periodical X-ray inspections of the joints were recommended. The subsequent, three-times-repeated inspections of these bridges did not show any changes or development of new cracks, and this despite a great number of train passages which was estimated at $1{,}6 \cdot 10^6$ cycles within a 50-year period of bridge operation.

The following conclusions may be drawn up from the described examination, laboratory tests and calculations:

- The results of X-ray examination conducted in situ have shown that it is economically unjustifiable to evaluate the load capacity of operated bridges making use of the same regulations and standards as in designing new structures. In the standards concerning testing the existing bridges the loads should be close to real weights which actually act on the bridge under consideration. The X-ray inspections also indicated that a defect within a joint, even as sharp as crack, can be admissible if it does not develop during the structure operating.
- All the specimens tested became cracked within parent material at the plane of sharp ends of rhomboidal cover plates which apparently reinforced the joint. The investigations confirmed recommendations applied till now concerning the inexpedience of the use of welded joints with straps in dynamically loaded structures.
- The observations concerning the place of cracks initiation suggest the advisability of conducting periodical external inspection of the welded joints with rhomboidal cover plates.

Condition Investigation Prior to Renovation Process - a Systematic Approach

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Summary

To determine the condition of a building or a structure by an investigation periodically during its lifespan is an essential part of maintenance. The methods to carry out a condition investigation are not, however, very advanced. Obviously, there are problems in recognising the phases in the investigation process as well as in detecting the points to be investigated. A systematic procedure to carry out a condition investigation as a whole is presented in this paper.

Keywords: Maintenance, Renovation, Condition investigation, Deterioration, Damage, Repair

1. Introduction

The condition investigation of a building or a structure is an essential part of its maintenance. The condition governs for example the serviceability of the structure and the repair options available in the maintenance as well as the repair costs to be expected.

Nowadays, the role of condition investigations has been acknowledged among house owners and consultants to some extent, but the procedures to carry it out are not very systematic or advanced. Especially the procedure to select the items to be investigated is usually more or less irrational. The procedure described in this paper is based on the development work carried out in some 100 condition investigation projects by the authors since the late 1980's.

2. Basic Principles of Condition Investigation

The basic aim of a condition investigation is to produce information about the factors affecting the performance of the structure and consequently the options for its maintenance for the client. A systematic condition investigation aiming at this consists of fairly simple and clear phases as discussed later. The phases are not totally separate and successive, but rather partly overlapping.

The first phase is to study what kind of a structure there is under investigation. This means to find out what kind of a structural system the structure has and what are the materials it has been constructed of. This information can be gathered from original construction documents and by a visual inspection. It is important that different types and parts of structures and identical structures under different exposure conditions are distinguished from each other and that they are also investigated as separate groups of objects.

The second phase is to recognise what kind of problems may exist in the structure. This is considered on the basis of the type of structure and materials in it as well as the exposure conditions. The problems may be caused either by different kind of deterioration mechanisms or by malfunction of structures, for example problems with moisture. The list of potential problems can then be utilised directly as the list of items to be investigated. It is, however, essential to evaluate the mutual importance of various problems. The factors related to the safety and health of residents and other users (like bearing capacity of the structure and the safety of fixings) are naturally the most important items to be investigated carefully.

The third phase is to consider the feasible techniques for the maintenance and repair of the structure. This is important to do as early as possible because different remedial techniques require different amount and type of information of the structure to be repaired. These alternative measures have to be also re-evaluated from time to time during the investigation process whenever there becomes new information available about the condition.

The fourth phase is to gather objective information concerning the deterioration processes and malfunction of the structure. The information can be gathered usually by four ways: firstly by studying the construction documents which give information mostly concerning the vulnerability of the structures, secondly by a visual inspection which gives information of the minimum extent of the damage (all the damage is not visible), thirdly by different kinds of in-situ tests and measurements and fourthly by taking samples and by different kinds of laboratory tests. As many different methods as possible should be utilised in examining each separate problem in order to improve the reliability of the results. The gathering of information should be done in the ways which are also representative and statistically reliable.

The fifth phase of the investigation is to carefully analyse the information gathered during the earlier phases. Practically, this means seeking answers to the following six questions: what kinds of problems exist in the structures, what is the extent of each type of damage and malfunction, what is the stage of each damage and malfunction, what are the reasons for the problems noticed, what kind of effects do the problems have on the structure itself or on the users of the building and finally how the damage or malfunction will proceed in the future.

The sixth phase is to prepare a report in which the results are presented for the client. The report should not consist only of measured values etc. but rather of practical conclusions concerning the alternative practical measures for the client to manage with the structure. There are usually several options for repair, and all these methods should be evaluated shortly in the report.

3. Concluding Remarks

A method to carry out a systematic condition investigation is presented. The method is general and fairly free from limitations as far as the object of the investigation is concerned. Therefore, the method can be utilised in numerous different cases in civil engineering where there is need to determine the condition of a structure and find the appropriate options for maintenance or repair.

The procedure described consists of rather simple phases which are quite easy to perform. The point of this method is how the simple parts are connected into a systematic procedure.

Durability Assessment of Prestressed Concrete Buildings in Bucharest

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Summary

During the period 1995 - 1997, the Building Research Institute - INCERC Bucharest worked out several studies having the object of the inspection and diagnosis of prestressed concrete structure durability for ten cinema buildings, radio concert hall and circus hall. The main aim of this studies consisted in the assessment of the prestressed concrete structural members built more than 30 years ago (1959 - 1969) and to identify the deterioration phenomena with the effect on durability.

1. Inspected Buildings

1.1 Cinemas

The inspection was applied on six cinemas of 800-900 seats capacity, designed and constructed in 1961-1963. The roof of these cinemas was conceived in composite cast in place vertical box grid (honey comb) with upper (partial precast) and lower slabs. The prestress was applied in two or three directions. Other group of four inspected cinemas of 500-900 seats capacity and built in 1963-1969 were conceived with the roof structure composed of prestressed concrete beams placed in one direction.

1.2 Radio Concert Hall

The roof of the Radio Concert Hall with a plan trapezium shape and transverse dimensions 17.4 to 38.0 m / 44.85 m long was built in 1957-1958. Roof structure was conceived in 7 reinforced concrete archs with prestressed ties. Upper side of the roof is composed by slabs and longitudinal beams supported by the archs through variable reinforced concrete bars.

1.3 Circus in Bucharest

The main circus hall built in 1959-1961 was covered by waved cupola of 60.6 m diameter and 13.5 m height, composed by 16 double curved waves, supported on 16 contour columns and prestressed polygonal ring.

2. Inspection Results

2.1 Cinemas

Inspection of the structural members of the ten cinemas roofs was restricted by the impossibility to observe all details subject of potential deterioration.

Results of the experimental tests on concrete
Experimental in situ tests shown a compact concrete cover, with carbonation depth of 2-4 mm. Other experimental tests performed on concrete cub samples provided from the same concrete during construction shown a porosity value of 4.6-5.5% and water absorbtion of 1.51-1.86% with a good bond matrix – aggregate, in each case failure taking place in aggregate. Also carbonation depth was of 3-7 mm on these samples, with a pH value of 12.
Results of the experimental tests on reinforcements
The width of concrete cover corresponds to the environmental conditions, except some isolated zones placed on lateral faces of beams where on limited areas the passive reinforcement bars appears visible at surface and having superficial corrosion. All other tests shown a good protection of reinforcement (no corrosion) by a compact and alkaline concrete cover.

2.2 Radio Concert Hall

Experimental tests on the ties, including inspection on the state of cracking, destructive tests on concrete cover and metal sheath until the post-tensioned wires, test on alkalinity, non-destructive tests on concrete strenght, shown: a depth of carbonated concrete cover not exceed 5 mm, existing cracks on the one tie only were caused either by shrinkage or more likely by technological procedures and have the width of max. 0.2 mm. The destructive tests on concrete cover until the post-tensioned wires shown clean wires covered by compact and alkaline cement grout placed in metal sheath with no traces of corrosion

2.3 Circus Hall

As well as in the case of cinema roofs presented above, the inspection of Circus roof structure and especially of the reinforced concrete cupola from bellow was hindered by false ceiling.
Results of the experimental tests on concrete
The depth of carbonated concrete cover not exceed 2-3 mm.
The concrete appears as compact and there are no segregations.
The existing cracks placed at the inner side of the main reinforced concrete cupola are old, inactive and have the width of max. 0.2-0.3 mm.
Results of the experimental tests on reinforcements
The examinations made at the inner side of the main cupola shown that there are no visible reinforcements and traces of corrosion.
The destructive tests on concrete cover shown clean reinforcements covered by compact and alkaline concrete.
The observations on the aspect of exterior surfaces of the joints (the zones for anchorages of post-tensioned tendons) of prestressed ring did not shown degradation phenomenons (cracks, traces of corrosion).

Conclusions

The investigations regarding durability of some cultural prestressed concrete buildings shown the necessary and the importance of extended (in-depth) inspections which are the same time more difficult due to the presence of false ceiling, finishes and exterior isolation.
Only when a complete access to all elements susceptible to corrosion is available, it is possible the correct diagnosis of damage causes and to allow the application of measures for reducing / elimination of these causes and repair / strenghtening. So it is important to plan an inspection during the repair period of ceiling, finishes or exterior isolation.
One of the main requirement for new buildings, regarding their durability, is to provide, in the design stage, the access to structural members and their joints if they are susceptible to suffer deterioration in time.
Also, it is necessary to provide periodical inspections to be perfomed by a specialist in the behaviour of the prestressed concrete structures.

Durability of Protecting Layers on Steel Cladding Sheets

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Summary

The necessity of improving heat insulation in existing buildings, especially those erected before 1991, arose as a result of changes of the thermal insulation standards as well as from the tendency decrease heating costs. This may be realised in several ways. One of the methods frequently used is applying cladding steel sheets with painting protective layers. Observations show that their durability is limited. Maintenance carried out at proper times can reasonably elongate the service life of cladding sheets.

Keywords: durability, facade, heat insulation, residential buildings, cladding sheets, corrosion, protection layers, profiling.

In the last few years, requirements for the insulation properties of external building walls were raised in order to decrease heat loss in buildings. Heat insulation is being performed in many existing buildings. One of the methods of insulating the external walls is using mineral wool insulation shielded from the external side by profiles made of steel sheets. This method was used particularly in the case of high multifamily buildings, in which, apart from energy savings, fire safety considerations are also important.
The durability of the sheets is determined by three essential factors:

- the quality of protecting layers on the surfaces of the sheets,
- usefulness of the sheets for processing,

- corrosion aggressiveness of the environment.

Protection layers on typical sheets consist from two elements: zinc coating and lacquers coating.
Only sheets with a thickness of zinc not less than 275 g/m^2, which corresponds to 20 μm from each side or Al-Zn alloys of the same thickness are accepted in Poland for making facade surfaces.
In the framework of work performed in the Building Research Institute investigations were carried out for more than one hundred objects exploited in various environments.
Investigations were carried out for residential, industrial and municipal objects. The range of damages of the protection layers and the degree of the environment aggressiveness were determined.
Investigations were performed in characteristic spots of facade sheets: bends of flat surfaces of sheet edges of cuttings and in mechanically damaged places. On the basis of investigation results obtained according to the classification given below, the relative shortening of the service life of coating in different places of the sheet profiles were determined. The results are given in table 1.

Table 1

Relative shortening of the service lives of coating on profiles from cladding sheets (in relation to the durability of the coating on a flat surface)

Position on the profile	Relative durability of coatings
Flat surface	1
Sheet bending arising at profile forming	0.7 -0.9
Cut edges of elements	0.5 - 0.8
Mechanical damage of protection layer: scrapes, scratches, indents	0.3 - 0.5

As can be seen from the Table the durability of facades are effected not only by the quality of coating but also the method according to which profiles are made. In practice deferent profiles occur, for which the bending radiuses are very small. In these cases indications of damages to the organic coating are observed in just a few years.
Exploitation investigations carried out in Poland were performed for sheets used for less than 20 years. The sheets were usually covered with an acrylic paint layer, practically not in use anymore. The results of the investigation consisted of determining the estimated durability of sheets given for different aggressiveness of the environment.

Table 2

Estimated durability of protection coating on sheets

Degree of aggressiveness of the environment	Durability in years
Very weak corrosion interaction	30 - 50
Weak and strong corrosion interaction	8 - 20
Strong corrosion interaction	1 - 4

Cladding sheets are finding broader use in Poland. Many investors are interested in making facades from such sheets. Up to now the sheets were considered as a product of great durability. However the investigations have shown that the durability of the sheets is limited to approx. 20 years. Extending at the durability can be achieved by applying specially developed renovation coverings. The durability is defined by the durability of the coating itself on flat surfaces and methods of making profiles and cutting sheets.

Repairing and Extending the Lifespan of Chimney under Severe Conditions

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Summary

The main chimney of the one and only heat-generating plant located on the island at the seaside was erected with initial faults of concrete shell. During the period of over 20 years the structure has been examined several times. As a result of specific localisation the chimney was exposed to severe conditions of wind, moisture, and salt attack. First time, after several years of use the emergency repair works were necessary, and in the next few years other serious repairs have been applied. Finally, the problem of radical strengthening appeared in 1997, as the existing plant must work for the next five years, at least. Description of diagnosis, recent repair methods, and final strengthening are presented in the report.

Keywords: concrete structures, concrete chimneys, extending the lifespan of structures, repair and reconstruction, strengthening of structures.

1. Description of the structure and recent repairs

The cylindrical reinforced-concrete chimney 80.5 m high, with internal duct diameter of about 3.20 m, was built by means of original method of double slip-form. The idea of the method was to erect the external shell from ordinary concrete, 0.22 m thick, and simultaneously, the internal shell from refractory concrete, 0.14 m thick. Between these two shells the insulation 0.12 m layer from granulated slag was provided.

The results of the experimental use of double-slip-form were rather poor. Serious faults on the surface of external shell, as well as on the internal surface of lining were noticed very soon after erection. After seven years of chimney use the first repair was undertaken. The vertical cracks up to 3 mm width were recorded in the lower part of external shell. To strengthen the chimney, particularly around the flue openings, the steel hoops along the whole lower half of the origin shell were introduced from outside. Epoxy resin was used for injection filling of main cracks.

Five years later, much worse situation was observed during inspection. The vertical cracks in the lower part exceeded in some places 20 mm, and the length of cracks was 15 m or more. It was the result of thermal influences due to destruction of internal lining and insulation layer, as well as external action of severe atmospheric influences on cracked concrete surfaces.
The strengthening of the chimney up to the level of 35 m (in form of R.C. shell, 0.12 m thick), new lining at the bottom part, and extension of hoops up the top of chimney were introduced. Such a kind of strengthening was possible because the foundation slab was relatively strong and supported by 39 piles 12.0 m long. The additional mass of concrete in the lower part of structure was advantageous for the improvement of dynamic response of the structure.

2. Present reconstruction

As the heating plant have to be in use for about five years (prior to erection of new one in another place) two possibilities about the chimney have been taken into account. First one was demolishing of the structure and erection of new multi-channel steel chimney on the former foundation. The second one was to extend the lifespan of the existing structure by means of strengthening of its upper part. On the basis of the feasibility study, considering the economical aspects, the second way was decided. The strengthening of structure should not increase significantly the weight of total chimney. Therefore, the most popular method of introduction of the new outer shell was not useful in this case. On the other side, the real time of suitable assembling of strengthening members was up to half a year due to climatic conditions.

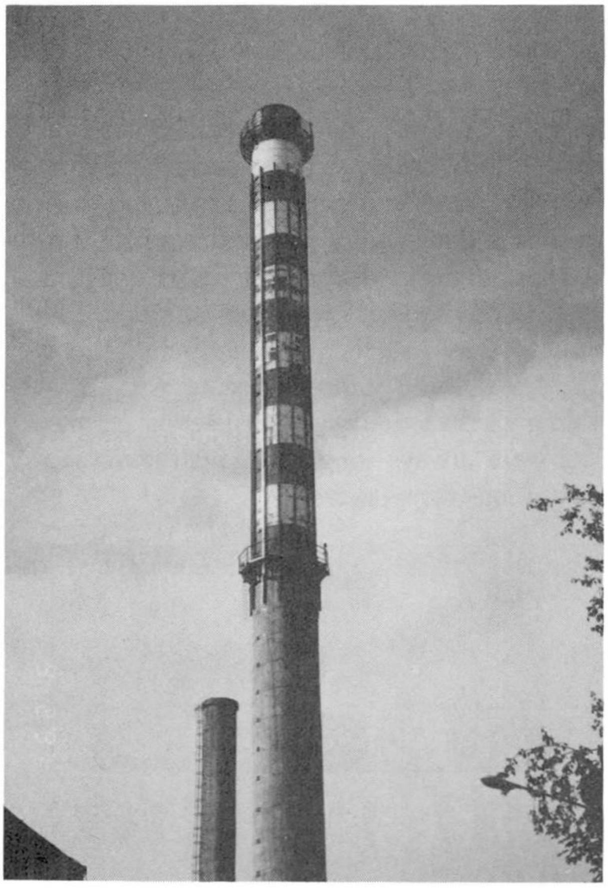

Fig.1: Overall-view of strengthened chimney

Finally, the steel external structure consisting of 12 columns and system of crossing tie-bars was designed. The vertical columns were fixed in the reinforced concrete shell of the lower part, made from the strong concrete. It created the kind of braid which was matched exactly to the real shape of structure. To ensure the possibility of simple and safe work, without heavy cranes, the elements of columns were divided into segments about 4.0 m long. All the assembly connections were bolted. High strength friction grip bolts were used in joints. The base of the braid was provided in form of strong ring with twelve arms anchored in the lower concrete part. Before fixing the anchors the layer of epoxy mortar was used to adjust the rigid ring to the existing concrete set-off. It was the only "wet" work during assembly. After preparation of steel members and trial assembly in the workshop, all the strengthening structure was erected in about six weeks with few breaks according to weather conditions. Another four weeks took the finishing of the surfaces of concrete and final painting.In this particular case the method of strengthening was assessed as over two times cheaper than the new external concrete shell.

Building Construction and Maintenance in Croatia

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Summary

In the first part of the paper, there is the description of the pre - transition situation with the analysis of construction, maintenance and the conditions of using the apartment buildings data. The property structure is also analysed as well as the space owner's attitude towards the rent payment and building maintenance. In the second part of the paper, there is a short survey of the housing fund demolition during the war in the Republic of Croatia. The third part of the paper gives the analyses of the privatisation conditions as well as their influence on the building infrastructure which has already been constructed. In this part, there are some information about the model and the structure of the privatised apartment buildings. The authors mentioned some law changes and the new model of maintenance building infrastructure.

1.Pre-Transition Period

During this period construction of new buildings had to symbolise the advancement of society, for which the ruling political party took credit. Housing buildings in particular played a key role in the philosophy which stated "every worker has to have a flat". Every year waiting lists were created and workers were ranked according to their employment and social status and then money for purchasing flats or crediting was distributed accordingly. It was common practice for companies and organisations in large cities to purchase flats and then to distribute them to employed personnel and their families with lifetime rights for usage of the real estate, with only a minimum charge of 10-30 DEM / month / flat. Building construction was consistently oriented on multi storey city housing buildings. Waiting lists were always long and construction was never able to keep up with the necessary demand for dwellings for workers.

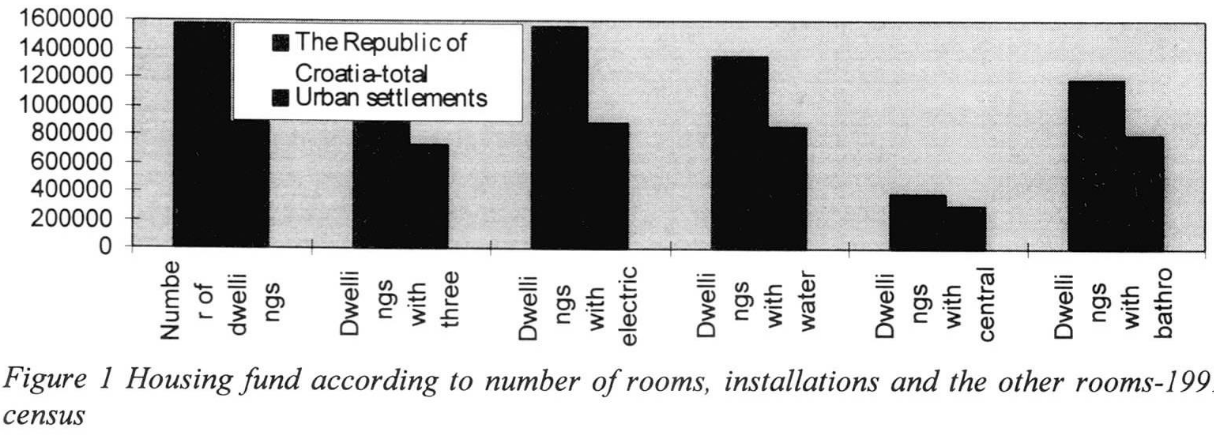

Figure 1 Housing fund according to number of rooms, installations and the other rooms-1991 census