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Investigations of Mass Flow in the Existing Building Stock

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Summary

Within the framework of investigations regarding "building material usage and the primary energy input for residential buildings constructed at different times", 20 residential buildings of varying ages (typical examples of solid-wall type of construction) were examined with regard to their building material and mass composition. In addition, the primary energy contents of the buildings concretized in the form of building materials - the so-called accumulated energy expended in the production of the building materials (PEI_H) - was also determined. The purpose of the investigation is to obtain information regarding the composition of building materials in residential buildings dependent on their age, and thus gain knowledge for dealing with existing buildings.

1 The Composition of Building Materials of Buildings Constructed at Different Times

A mean value of 0.528 Mg/m³ gross cubic content is obtained for all examined buildings. Single family houses (mean value 0.592 Mg/m³ gross cubic content) tend to be heavier due to their higher proportion of building materials; the used building materials here normally enclose a smaller volume than for residential buildings for several families (with a mean value of 0.479 Mg/m³ gross cubic content). Over 90% of building material masses are mineral. Figure 1 shows the usage of building materials in residential buildings as changing with time as a mass percentage distribution. It can be seen that the composition of the building materials changes from one age group to the next. Whereas the concrete portion has increased since the mid 1920's, the portion of timber as an organic building material has decreased to under 5 %. Although the buildings of the age groups from 1918 to the present day are mainly examples of the solid-wall type of construction, the portion of block and brickwork walling decreases in this presentation. This illustrates the fact that the increased use of concrete is not only at the expense of the building material timber but also block and brickwork walling. In comparison, a continual increase in the use of inorganic materials such as glass and steel as well as insulation materials can be observed.

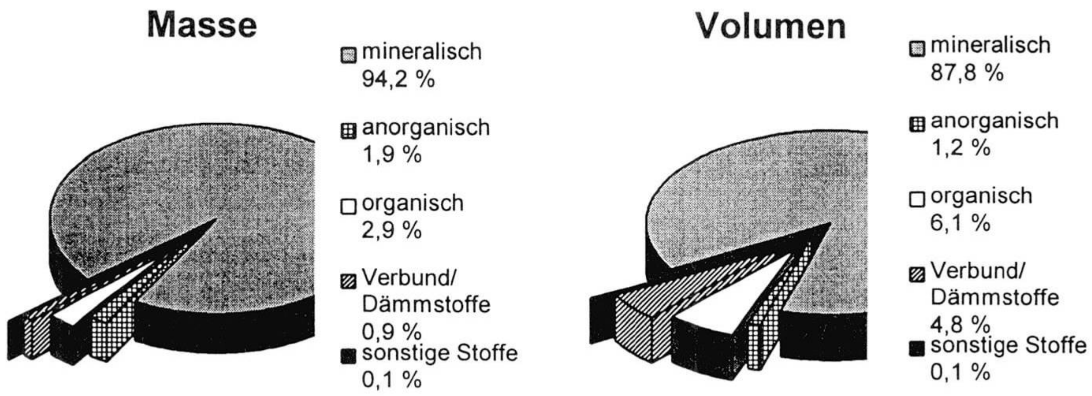

Fig. 1 Usage of Building Materials in Buildings of Various Age Groups as a Mass Percentage Distribution

The investigatin has shown that more than 80 % of the building material masses of the examined residential buildings can be apportioned to the loadbearing structure, and less than 20 % on the finishes. A displacement of the building material masses from the loadbearing construction to finishes is recognisable in the younger building groups. This emphasises the increased use of building materials in the area of finishes due to the increasing requirements made for thermal and sound insulation. The classifikation of the total mass of all examined residential buildings into the individual building element groups according to DIN 276 has shown the dominance of external walls and floor/ceiling construction each with a value of 31 % for multi-family houses, followed by internal walls with 21 %. For single family houses the proportion for external walls with 39 % is much more characteristic, instead the floor/ceiling construction portion is barely 21 % and the internal walls with 14 % are of less influence. Taking the external and internal walls together, it becomes apparent that the proportion of the walls for single and multi-family housing in total are almost identical with just over 50 % of the total mass.

2 Primary energy input of residential premises

The examination of residential buildings with regard to the cumulative production energy input (PEI_H) carried out on 20 buildings has shown that the basic primary energy input values for the building materials are not decisive but rather the building material mass, defined by the specific material density. The investigated buildings vary around the value PEI_H/mass = 2 GJ/Mg. But for example a building in the age group 1945 - 1955 is not only the heaviest but also has the highest PEI_H value; nevertheless however its relationship PEI_H/mass is more favourable than for buildings of the following building age groups. Overall it was ascertained that building elements with a heavy mass give a favourable level of the PEI_H/mass ratio; on the other hand a higher expenditure is required here for demolition and recycling. The elements of the building exterior are lighter, show however a higher building material energy level; on the other hand they contribute to the reduction of heating energy in the use phase.

3 References

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Monitoring of Apartment Buildings

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Summary

The subject of this paper is the preparation of the computer system for monitoring the of state of apartment building resources for cognitive purposes, for facilitating the technical exploitation of existing objects and for verifying accepted technical solutions affected by real conditions of use. The database was created on the basis of questioning performed by persons skilled in building know-how with the aid of a specially prepared questionnaire. Test analysis of the system was conducted for data obtained for 135 objects. It showed correct operation of the system.The computer system was based on the relation database according to the BDF standard. The software was prepared using the clipper language and the whole installation was set up on IBM PC type computer and DOS.

Structural concept

As a result of analysing the goals and tasks of the monitoring, the desired scope of monitoring was specified. The acquired data was divided into nine problem groups as follows:

1. Preliminary data: date of the investigation, name of person performing the investigation, access to object during the investigation, technical documentation.
2. General data: address of the object, owner of the object, year of construction of the object, essential modernisation, size of the object.
3. Use of the object: dwellings, other premises.
4. Construction data: type, damages, technical state.
5. Data about filling elements: non-bearing walls, openings, technical state.
6. Data about installations: range, technical state.
7. Data about protections: heat insulation, dampness insulation.
8. Data about environment: neighbouring objects, aggressive components of the environment.
9. Additional data in descriptive form.

Information about protection from heat and dampness refer to the insulating properties of construction partitions, but not in relation to energy consumed in the object.

The structure of the database is consistent on the basis of merit and formally with the dBase standard and user software was written using the clipper environment. In the questionnaire three type of answers occur:

- descriptive (e.g. address),
- questionnaire type (e.g. technical method of construction work: traditional, industrial, mixed),
- evaluated by linguistic variables (e.g. small cracks, substantial cracks and serious cracks)

This way of formulating answers causes that essential information on the technical state of the object is extracted - but it is still determined by a specifically qualified inquirer.

Methods of acquiring data

There are two main sources of acquiring data:

- questioning and
- the existing technical documentation

In general 135 questionnaires were filled out and stored in the computer using the appropriate program. The collected data concerned objects located in Warsaw and in its vicinity. The data contained information about differentiated buildings: old and new, small and big, being in different technical condition, and having different useful values.

For operation of programs general rules accepted for IBM PC type computers were followed. As a result of realisation of the system the tool for systematic data acquisition and analysis of existing apartment building resources were set up. In addition, principles of acquisition, classification, storing, processing and analysis were established. On the basis of the elaborated system the possibility of setting up unified procedures of data acquisition and creating the basis for establishing quality classification of buildings was formed.

Examples of the results of analysis are given in the Figures.

The percentages of the various types of floor slabs in the monitored sample (they are: p-type monolithic panels, monolithic slabs reinforced concrete slabs floor slabs made of small-scale elements, ribbed and wooden panels) are shown on Fig. 1.
Fig. 2 gives the percentages of cracks and other damages.
Performance, overload, ground settlement, thermal interaction, dampness, natural expansion joint and others.

Diagnoses of Large Panel Buildings in the Czech Republic

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Summary

General guidance for systematic investigation on technical conditions of large panel buildings. The results of a survey that has been already performed on different types of building systems in the Czech Republic proved the necessity to solve problem of safety of large panel buildings. This problem might be common to other Central and Eastern European countries. Although there is no evidence of structural failure of the system as whole large scale field investigation is urgently needed to prevent local failures of external structures. Remedial actions to prevent concrete deterioration must be taken. Detailed assessment is recommended before any structural alterations or refurbishment projects will start.

1. Methods of diagnostics

The high number of flats located in large panel buildings establishes the importance of systematic approach to:

a) Inquiry to obtain basic information on ownership, location, number of buildings and flats, structural systems and serious faults.
b) Field investigation and visual inspection. The methodology proposes a questionnaire that is based on all six essential requirements according to Directive 89/106/EEC. [1]
c) Structural analyses when visual inspection cannot provide all information for assessment of loadbearing structures. Effective software tool is available non-linear approach can be made when necessary.
d) Collecting data on technical conditions of buildings in regions.
e) Working out a database in which all important findings and other data must be arranged.

2. Results of survey

A large scale survey has not started yet. However some findings are already available.

2.1 Main problems

The main problems that influence safety and health of inhabitants are as follows:

- inadequate structural design that does not prevent *progressive collapse* of a structure in case of accidental loading (for instance gas explosions, impact loading)
- insufficient *integrity* of structures than can cause for example separation of external layers of sandwich panels, disintegration of concrete railing elements from a structure or disintegration of other parts of external structure exposed to environmental attack

- *corrosion* of reinforcement of external concrete structures can produce falling debris (concrete cover or even parts of concrete structure at least).
- inadequate construction details and application of materials, poor technological discipline of execution together with improper system of heating, ventilation and use of a flat can create an undesirable microclimate, its consequence is *unhealthy environment* (moulds are dangerous)

2.2 Other problems

Apart from the most serious problems listed above we are faced with other problems such as *cracks* in structural elements and joints, inadmissible *deformations* of floor slabs, a wrong design with regard to *volume changes* due to thermal effects, improper *tightness* of external envelope that causes air infiltration and water leakage, too *high energy consumption*, low level of *protection against noise*, poor level by products namely *sanitary facilities*

2.3 Causes of faults

There is a complex of items that can be considered as the main causes of the faults indicated above. These causes can be summarised as follows *inadequate technical standards* and lack of knowledge of designers when the first structures of this kind were designed, *time pressure* during execution, *poor quality of products, poor workmanship*, insufficient *quality inspection* on building sites, insufficient *quality of maintenance, inadvisable utilisation* of flats, gradual worsening of *environmental* conditions until year 1990 and too small concrete covers.

3. Evaluation of large panel structures

Czech Standard [2] covers all aspects of design of large panel structures quite well. Comparison of the archival design documentation with present requirements, shows that older panel structures do not satisfy general structural requirements. There might be doubts about proper function of joints.
However, the most structures enable wide range of alterations. For example:

- vertical extensions containing new roof apartments
- horizontal extensions, fixing of new architectural elements
- new ducts through floor slabs
- new openings in walls

4. Conclusions

According to all available findings from field investigations, non-destructive site tests, detailed numerical analysis and other assessments of different types of large panel systems it can be briefly concluded

- there is no real evidence of possible failure of any structural system as a whole
- there is potential risk of local failures
- process of ageing of large panel structures requires remedial actions
- large scale diagnostics procedure throughout the country is needed

References

[1] Council Directive 89/106/EEC Construction Products. Interpretative Documents Commission of the European Communities, Brussels 1993.

[2] ČSN 73 1211 - 2/1987 Design of concrete structures of panel buildings (in Czech).

Evaluation of Deterioration of Rural Buildings in Poland

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Summary

The Sudeten Region covers the south-west part of Poland. The rural development in this region, with distinct features of specific regional architecture, shows now a considerable degree of technical and functional deterioration being the effect of drastic negligence in repair and modernisation over the last five decades. Basing on the selected types of deterioration, an analysis of age distribution as well as assessment of wear and deterioration level of rural development in this region was presented. The analysis is limited to the area of Klodzko Valley located in the eastern part of the region under study. Also a scope of damages resulting from this year flood which dramatically worsened the state of existing building resources is indicated.

Keywords: rural buildings, deterioration of buildings, modernisation

1. Sudeten Region

South-west region of Poland, a Sudeten Region covering the Jelenia Góra and Walbrzych Voivodships, belongs to the most developed areas of the country, where the impact of slower economic expansion is particularly acute. This is a consequence of the fact that being rather insignificantly destroyed during the World War II this area was not financed in the field the repairs and modernisation to the same extent as the other regions of Poland. This caused that the real property in this area became worn in about 60% with consequential negative effects on the local social and economic relations, so that since the 70s this region is considered as an *„eminent problem region"*. Another cause which added to this was undefined property law and resulting from this the lack of responsibility for the technical state of existing buildings.

In the Sudeten Region there are the biggest building resources of the country but 75.8% of them were erected before 1945, including 34,6% erected even before 1918, what, in the view of permanent lack of repairs and modernization, has led to the avalanche deterioration. This process was particularly intensive in the eastern part of Sudeten Region, in Klodzko Valley (Grafschaft Glatz), where to the functional and technical deterioration of the building resources common for the whole region, the tragical effects of flood in July 1997 were added, which particularly severely affected this region making damages in the substantial part of municipal and rural resources situated on river banks.

2. Devastation of buildings in Klodzko Valley

The housing resources in Klodzko Valley amount to 201 176 dwellings, a 25,3 % of which is on the rural areas. The rural development exhibits so-called *technical deterioration of components and building itself* (due to age), so-called *functional deterioration* (loss of functional qualities) and damages caused by July flood.

From the analysis of technical deterioration of this development, it results that 54.4 of the buildings were erected before 1918 (including 47.3 % of masonry and 7.1% of wooden buildings) which are now at least 79 years old; 37.4 % of buildings were erected in the inter-war period (1919-1944), (including 35.2 % of masonry and 2.2 % of wooden buildings), which are now from 80 to 53 years old; 8.2 % of buildings were erected after 1945 (including 7.8 % of masonry and 0.45 % of wooden buildings). From this analysis if follows further that the wooden buildings, amounting to 9.3 % of total housing resources are in poor technical state and have and average deterioration level above 51% and only 0.4 % of buildings erected after 1945 shows the medium technical state, i.e. the deterioration within 18.6-34.4 %. 35.2 % of masonry buildings shows the medium deterioration within the range of 18.6-33.5 % while 47.3 % of them exceeded the stage of medium deterioration. The post-war development amounting to 9.7 % of total resources shows the medium deterioration (up to 18.2 %) and falls into the category I (good technical state). The technical deterioration above 51 % (poor technical state) was reached by the wooden buildings erected before 1918 and in inter-war period (9.3%). Deterioration within 31-50% (medium state) show the masonry buildings erected before 1918 while the buildings erected in the inter-war period and post-war wooden buildings (totally 82.9% of the housing resources on this area) are approaching this state. The buildings erected after the World War II show now deterioration up to 30

These have lost also their functional and aesthetic qualities. Due to age distribution of this development and lack of repairs 54.4% of buildings approach the limit of functional deterioration which is assumed to be 90 years. About 42% of dwellings are without bathrooms, 63% have no WCs and 28% are without sanitary system. Presently, this situation is even worse due to damages caused by flood in July 1997. A full scope of damages is now difficult to determine precisely since in the winter, the frozen water in soaked walls will surely cause additional damages.

Fig 1. Sudeten development destroyed by the flood in 1997.

Conclusion

The Sudeten Region with substantially deteriorated housing resources, after transformation of 90s connected with restoration of property law and responsibility of owners for the state of their property, is now under preparation to modernisation of the existing development basing on the programs which enable also to retain the regional character of local architecture.

Dynamic *in Situ* Tests on a Damaged Historical Building

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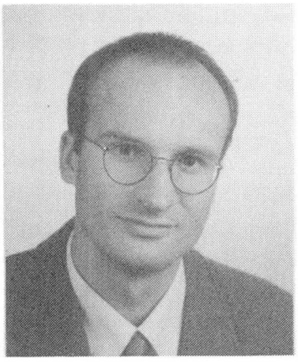

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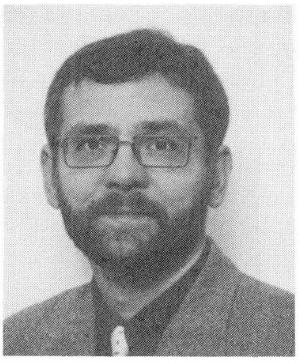

Summary

The numerical description of the structural behaviour of an existing, damaged structure is a challenging engineering task. The basis for the solution of this problem is the evaluation of the actual behaviour on site using advanced experimental and numerical techniques. In this study, the severely damaged bell tower of the church of Krölpa (Thuringia) was investigated. By utilizing an FEM-package several natural frequencies with the corresponding vibration modes, transfer functions and modal damping ratios were obtained from the measurement data. It became obvious that the dynamic behaviour of the structure depends not only on the frequency but also on the amplitude and direction of the exciting force. The presented methods appeared to be suitable tools for the identification of dynamic system properties even though difficulties in the numerical description of the nonlinear effects remained.

Keywords: structural dynamics, system identification, structural damage, experimental testing, nonlinear dynamics

1. Investigations on site

Fig. 1 : The bell tower, view from south-west

The appearance of the medieval Krölpa church tower is significantly characterized by its inclination which was caused by an uneven settlement of the ground. Today the tower is about 2.2 degrees out of the vertical and shows deep vertical cracks in the masonry of its four walls. These damages gave the reason for an investigation of the dynamic structural behaviour of the tower in its current state. It was intended to identify dynamic system properties as well as to model the structure with its specifics numerically.

The *in situ* tests were carried out in May 1997. A servo-hydraulic vibration generator was posed in the former place of the belfry in order to produce a harmonic exciting force. The frequency of the force was increased incrementally from 0.3 Hz to 9.7 Hz. The experimental programme consisted of several test series with the exciting force acting in different directions and with different amplitudes. The response of the system was registered by accelerometers which were placed in the corners of the tower in three levels.

2. Identification of system properties

From the stored time histories of the measured accelerations and the exciting force, the complex transfer functions were calculated as the ratio of the Fourier transform of the reaction and that of the excitation. Problems which are connected with the Fast Fourier Transformation algorithm were avoided by using an alternative method for the computation of the Fourier transforms.

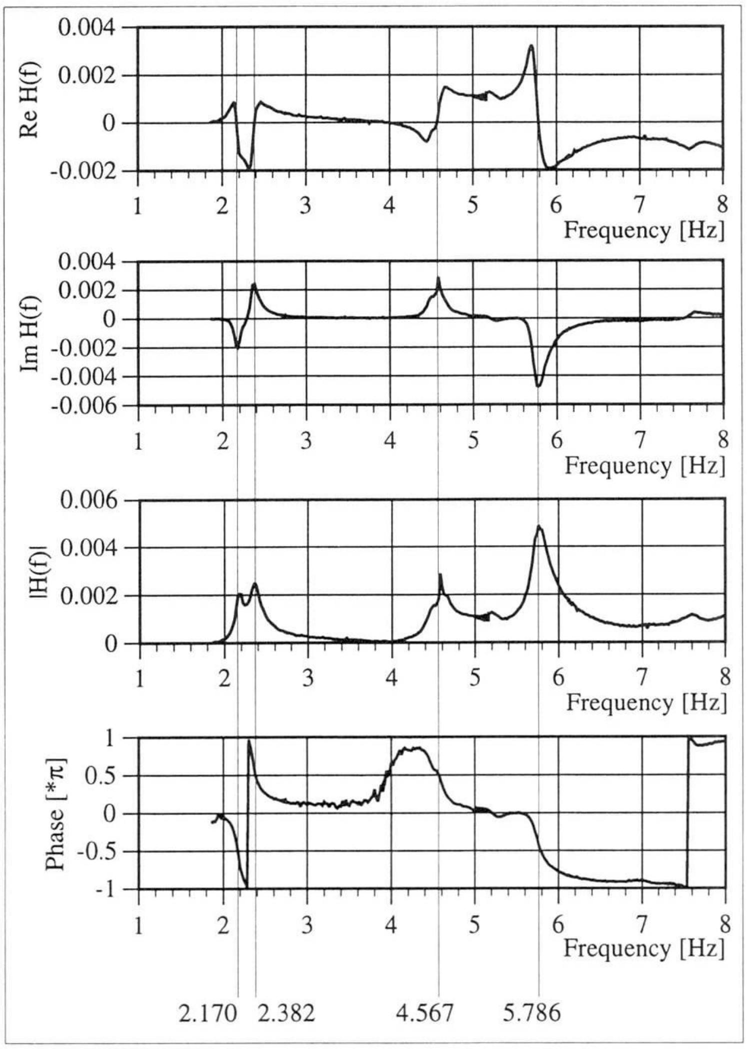

Fig. 2: Complex transfer function of acceleration obtained from the measurements (accelerometer at level B, NE-corner, N-S-direction, excitation E-W, max. F=2.0 kN)

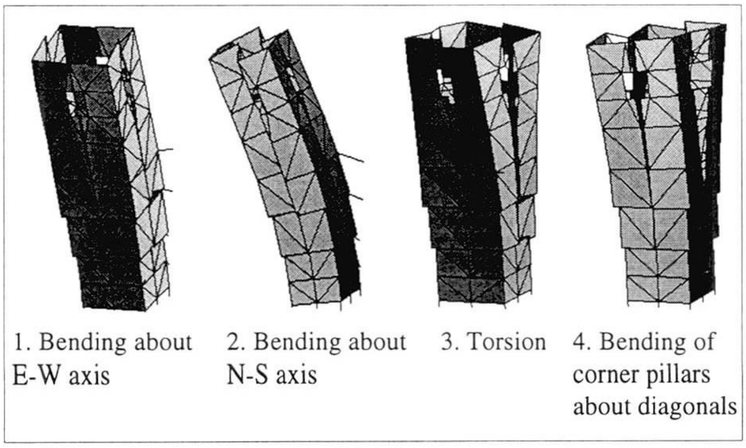

Fig. 3: First four identified mode shapes

The phase angles were directly determined from the complex transfer functions. The obtained curves allowed the identification of four resonances. Modal damping ratios for the resonance frequencies could be deduced from the phase angles at resonance.

A finite element model of the tower was created. By means of the method of dynamic condensation and the deflections obtained by integrating the accelerations twice four modes of vibration, which correspond to the resonances, were identified.

The results of the test series showed several nonlinearities in the dynamic behaviour of the bell tower. The resonance frequencies decreased with an increased amplitude of the exciting force. It could also be observed that the occurence of resonances depends on the direction of excitation.

3. Conclusion

Structural damages can lead to changes of the structural system and significantly alter the dynamic behaviour of a structure. The described techniques can be considered as a suitable method for the identification of dynamic system properties.

A number of nonlinear effects in the behaviour of the damged bell tower was observed. It can be concluded that the properties and the behaviour of the investigated structure strongly depend on the frequency, the magnitude and the direction of a load imposed on it. The numerical modelling of these phenomena remains the subject of further research.

Evaluation of Resistivity Measurement of Masonry by Numerical Simulation

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Summary

Geo-electrical resistivity maps are being used for the evaluation of brick and stone masonry. They provide information about the inhomogeneities and the geometrical boundaries of the structure. Only the information of the inhomogeneities is of interest for the evaluation of the internal condition of the structure. Therefore, the influence of the boundary conditions on the measurements needs to be eliminated. This paper discusses the comparison of laboratory measurements on a well-known physical model consisting of a sandbox of 2.5 m x 2.5 m x 0.35 m with embedded blocs and the results of a numerical model which simulates the measurements. The moisture content of the materials influences the measurements and is taken into consideration. The geometrical boundaries, the moisture content as well as the dimensions and localisation of the inhomogeneities are known characteristics which are used as input of the numerical model.

1 Experimental research - physical model

The physical model used in this research consists of an insulated sandbox with embedded blocs (fig 2). The physical model simulates a wall of which the geometrical boundaries, the localisation and dimensions of the inhomogeneities, the electrical properties and the moisture content of the masonry are well known. The inhomogeneities are simulated by insulated cubes with varying dimensions which are burried in the sand at a certain depth. The measurements are done with the electrode configuration of the Wenner-spread (fig 1).

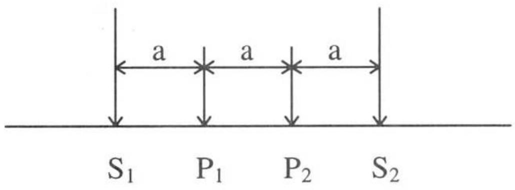

Fig. 1: Electrode configuration of the Wenner spread

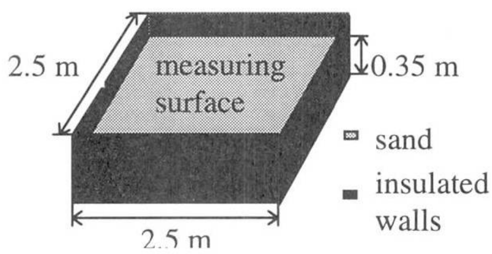

Fig. 2: Physical model: insulated sandbox

The material in the model is sand with a moisture content of 5%. Due to gravitational water movements the moisture content of the sand does not remain at 5 % over the whole model, but increases with depth. This means that the resistivity of the sand will decrease with depth. Previous research has shown that the moisture content has a big influence on the resistivity measurements. This influence is determined by taking cores out of the model and measuring the resistivity profile over the core length.

2 Simulations

The simulations are made using a finite element model. For the sandbox without inhomogeneities, we only refined the model in the vicinity of the measuring points (fig 3). Submodelling is used for the simulation of the sandbox with a cube placed at the centre.

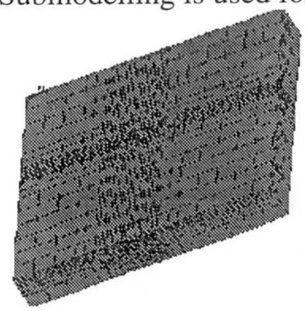

Fig 3: Finite element model

Some typical results of the measurements and the simulations are given in figure 4.

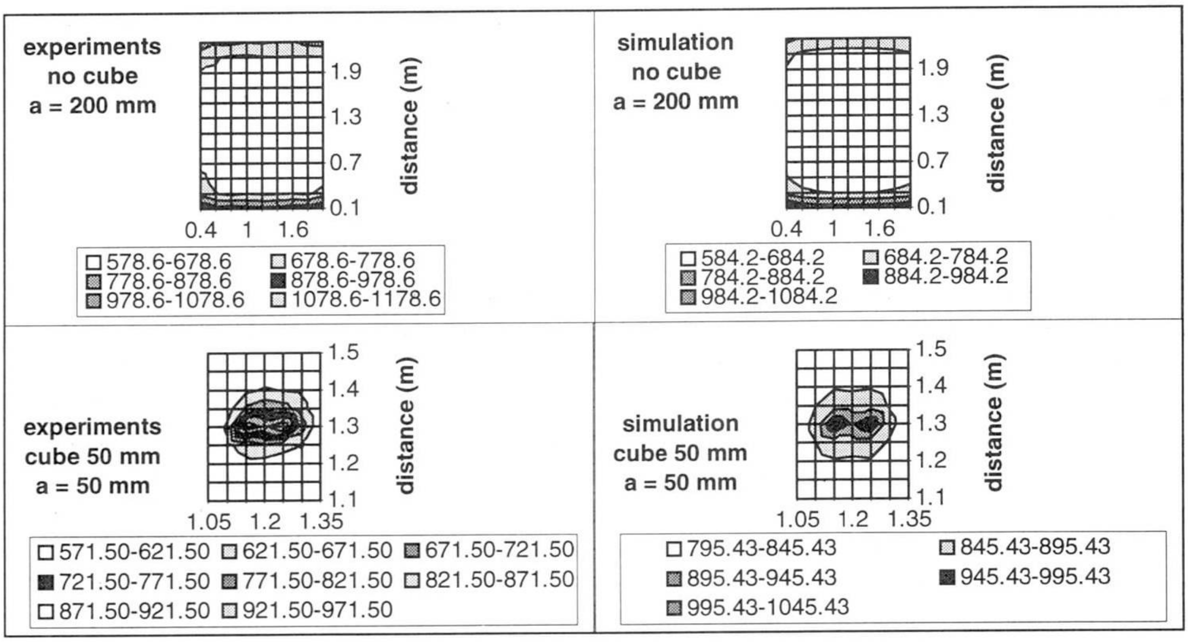

Fig 4: Comparison between experimental research and simulations (values in ohmm)

From this research we may conclude that the numerical model is able to simulate the influence of the geometrical boundaries and the inhomogeneities. It is accurate enough to be used for calibration and filtering of site measurements.

Modal System Identification Technique in Structural Damage Assessment

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Summary

A modal damage detection and assessment technique of multi-story, multi-bay framework is presented. The fundamental natural frequency and mode shape of the structure are the only modal information necessary in this approach. The *equivalent relative lateral story stiffness* ($R_{i,i+1}$) is chosen as a physical damage predictor parameter, which seems to be sensitive enough to offer the best information for occurrence, localisation and extent of the structural damage. It is assumed that the baseline parameter values are known. A data perturbation scheme is used both for the baseline and for the damaged structure, to establish the threshold above which damage can be confidently discerned from noise. To examine and illustrate the proposed damaged assessment technique, a numerical simulation study is performed and different damage types were considered.

Keywords: System identification theory, damage assessment, modal damage detection..

1. Introduction

One of the most challenging task for modern day structural engineer is to control the behaviour of the structure during its life time and, from time to time, to evaluate the structural damage, in order to enssure the safety of the construction. The system identification (SI) method seems to be the best tool to accomplish this task [1-3]. A variety of techniques for evaluating damage in structural systems have emerged and evolved in recent years. In this paper, a modal system identification technique is presented. The fundamental natural frequency and mode shape of vibration are the only modal measured information necessary to identify the occurrence, location and extent of the damage. The damage is defined as any change in the performance of the structure. For a structural system, such change means a reduction in its carrying capacity, which can be expressed by the reduction in flexural rigidity EI of the structural members. With this definition of damage, one must look for changes in certain physical parameters of the structure, between two time-separated moments. The choice of physical parameters, which have to be related to the measured modal characteristics of the structure, is of major importance. The *equivalent relative lateral story stiffness* ($R_{i,i+1}$) is chosen as physical damage predictor parameter, which seems to be sensitive enough to offer the best information for occurrence, localisation and extent of the structural damage. It is assumed that the baseline parameter values are known. The accuracy of such a model is generally impaired by the presence of random noises such modelling errors, unmeasurable disturbances, and measurement errors. A data perturbation scheme is used both for the baseline and for the damaged structure, to establish the threshold above which damage can be confidently discerned from noise. To examine and illustrate the proposed damaged assessment technique, a numerical simulation study on a three-story three-bay frame and a five-story two-bay frame is performed and different damage types were considered. We chose a simulation study over a real case study because our aim is to quantify the performance of the proposed technique, not simply to illustrate its use. Simulation is a method of controlled experimentation that allows such a quantification.

2. Conclusion

A new damage detection and assessment technique is presented. This study has shown that the chosen structural parameters, i.e. the equivalent relative lateral story stiffness, are sensitive even to not very large damage, and allows to localise the damaged stories. The damage localisation is based on an output-error parameter estimation procedure. The constitutive parameters of the structural system are estimated using a frame analysis model with known topology and geometry. The measured fundamental modal parameters, frequency and shape mode vector, are the only data necessary for damage detection and assessment, and the number of necessary measurement points is equal to the level number. The procedure was illustrated and tested using Monte Carlo simulation on two examples. The procedure of using noise perturbation on baseline structure proved to be a reliable method for assessing whether damage is detectable above the noise in the measurements. In all of the cases studied, the most severely damaged stories were always identified with great certainty.

Although the feasibility of the proposed procedure for structural damage assessment of multi-story multi-bay structure based on the measured modal data has been demonstrated, many related issues have not yet been addressed and additional research will be required.

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Inspection of Concrete Floor Slabs by Means of Dynamic Test

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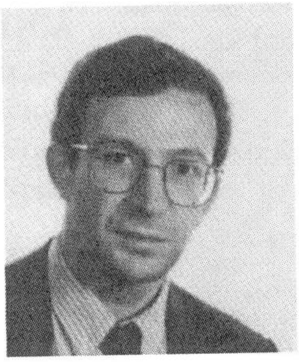

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Summary

The use of dynamic load tests as inspection technique for concrete floor slabs is presented as part of a more global method which considers the combined performance of static and dynamic load tests. The dynamic load test is mainly regarded as an auxiliary tool, inexpensive and very repeatable, which can be widely used over large roof surfaces or entire buildings in order to extend the coverage of the static tests, asses the homogeneity of the elements inspected, or to identify local structural abnormalities. Versatile tools of analyses for the numerical simulation of the dynamic response and for the parameter identification are needed.

Keywords: Non-destructive tests, floor slab, inspection, dynamic test, parameter identification,

1.- Introduction

Due to both technical and economical reasons, the conventional techniques of inspection today available for the assessment of floor slabs show significant inconvenients which actually constrain their applicability and practical possibilities. Among these techniques are the extraction of concrete cores or micro-cores for the mechanical testing, or the static load tests, both of which require to generate some previous deterioration and later repairs. In particular, the performance of the static loading tests may require the removal of some finishes or non-structural elements of the building, as pavements, partitions and ceilings; because of security reasons, scafoldings must be placed throughout all the floors below the one being tested, until reaching the foundation. These severe requirements mean a significant expense on preparatory labor and later repair. If the static loading test is carried out on one or more slabs in a building, there is still some uncertainty about the actual representativity of the measured response concerning the entire building.

In spite of its non-destructiveness, the measurement of the dynamic response of floor slabs subjected to an artificial excitation can provide very valuable information related to the mechanical parameters and state of preservation or damage. There is also the possibility to left untouched the finishes and furniture, and thus to dramatically reduce the cost and time required to execute such a test. However, being the dynamic test an indirect one -based on the measure of dynamic parameters to be correlated with more significant mechanical properties- it is by no means as reliable as the static loading test.

2.- Lay-out of the technique and parametrical studies

The methodology of inspection proposed is based on a special combination of static and dynamic load tests. The static test is considered necessary as a direct and reliable way to measure the loading capacity of the floor slab or, at least, to ensure its capacity to carry up to a certain amount of load. The dynamic test, fast, inexpensive and immediate, can be used to assess large roof surfaces or entire buildings and detect possible heterogeneities or unveil local abnormalities. Thus, the methodology considers two steps: (1) Executing both static and dynamic load tests on a few selected slabs, used to obtain their basic mechanical characteristics and, by comparison, calibrate the dynamic test, and (2) scanning the rest of the slabs through the systematic repetition of the dynamic load test. The numerous data obtained through the dynamic test can be used not only to assess the homogenity and representativity of the load tests, but also to estimate some of the mechanical parameter of the slabs through a parameter identification process.

The dynamic test is based on the measurement of the dynamic response of the slab after it is excited by means of a controlled impulse provided by a hammer of known weight. In the actual phase of the research, the first series of natural frequencies and the shape of the modes of vibration are used as main parameters for the identification process.

The interpretation of the measurements obtained through the dynamic test is achieved by comparison with the numerical results obtained by simulation, which requires a versatile tool of analyses. One of the challenges placed to the numerical analysis is found in the need to account for all the elements which can influence on the dynamic response of the slab: thin wall partitions, possible boundary conditions, influence of the rest of the structure, actual contribution of the concrete topping, mechanical action of the non-structural pavements, dead loads, placed on the slab, and other. At the moment, the procedure is being applied on one-way slabs with precast nerves . Some performed parametrical studies carried out into the present research illustrate the significant dependence which exists between the value of the natural frequency associated with the first flexural modes and factors such as the degree of cracking in the concrete section, the actual support conditions, and other. The influence of the actual boundary conditions at the ends of the slab span can be efficiently characterized through the ratio between frequencies corresponding to subsequent flexural modes of vibration. Also, the possibility to resolve the actual contribution to the stisffness of the concrete topping and the upper pavement from the dynamic response has been also shown in the study of different particular slabs . (See for the references in the 6 page paper).

3.-Real case examples

Two examples of real slabs studied using the presented technique are discussed. The first consists of a single spanned slab built with prestressed concrete precast semibeams, sustained on brick bearing wall, with span and thickness respectively measuring 3.65 m and 15 cm. The second case refers to a two-span one-way slab built with prestressed concrete precast full beams, sustained on a reinforced concrete frame, with pannels having span of 3.55 m and thickness of 22 cm. The studies allowed to conclude about the boundary conditions actually affecting the slabs, which were recongnized as a perfect clamping for the slab sustained on walls. On the other hand, the technique allowed to analyze the stiffening action of the non-structural pavements existing on the slabs, being characterized as non contributing in the first and fully contributing in the second. In the case of the continuous slab on framed structure, the sustaining system of transverse beams and columns had to be included in the model in order to obtain an adequate simulation of the dynamic response.

Mechanical Properties and Bond Behaviour of Corroded Deformed Bars

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Summary

An experimental investigation of the effect of corrosion on the mechanical properties and bond behavior of deformed reinforcing bars is presented. Tensile tests on corroded rebars through an accelerated process and bond tests on concrete specimens reinforced in all four corners were carried out. Results indicate that corrosion affects ductility of steel, decreasing bar elongation at maximum load. Too corrosion affects steel-to-concrete bond, but the presence of stirrups improves significantly bond behavior of corroded reinforcing bars.

1. Experimental Program

Fresh and corroded deformed bars with diameters of 12 and 20 mm respectively were tested in tension to determine yield strength, tensile strength and elongation at maximum load. For corroding the samples, a direct current of a 1mA/cm^2 density was impressed on the rebars immersed in 3% weight of NaCl solution or fresh water. These media reproduce, to a certain, the aggresive environment of a completely carbonated concrete and a chloride-contaminated concrete respectively. Several levels of corrosion were examined: no corrosion, corrosion associated to cross section losses from 7% to 20% and corrosion associated to cross section losses higher than 25%.

Pullout bond tests were carried out on 300x350x350 mm concrete specimens with 20 mm diameter bars arranged in the four corners. A cover to diameter ratio of 1.5 was examined and the embedded length was 14ϕ in all cases. The test method used simulates anchorage conditions in a part of a beam submitted to constant shear force, and allows to reproduce the most usual bond failure on reinforced concrete structures: splitting failure in the concrete cover (Fig.1). For corroding the rebars the concrete in which they were embedded was contaminated with 3% calcium chloride by weight of cement and later a direct current of a 0.1 mA/cm^2 density was impressed on each bar embedded in the pullout specimens. Several corrosion levels were reached producing crack widths ranging from 0.1 mm to 0.4 mm and cross section loss up to 5%. Various amounts of transverse reinforcement were also arranged: no stirrups, ϕ6 mm stirrups at 200 mm and ϕ8 mm stirrups at 10 mm.

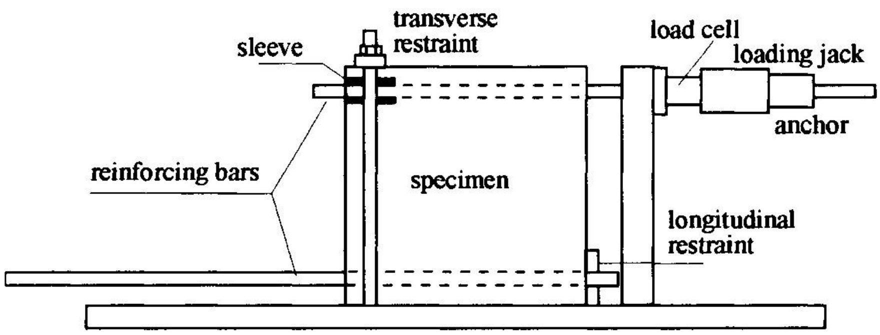

Fig 1. Bond test arrangement

2. Results and Discussion

The results of tensile tests hint yield stress and tensile stress do not decrease for cross section losses lower than 20%, corrosion range that includes the majority of actual cases of generalized corrosion. On the other hand, a systematic and important reduction of elogation at maximum load was observed both for rebars corroded in water with chlorides and those damaged in fresh water (Fig. 2). The results indicate reductions of elongation at maximum load of about 20% for cross section losses of 10%. When corrosion level achieves values about of 25%-30%, elongation at maximum load decreases 40% aproximately.

Pullout bond tests allowed to study the influence of bar cast position (top or bottom) and amount of transverse reinforcement on the bond strength of corroded rebars. As for the influence of the bar position, in noncorroded specimens the average values of bond strength for top bars were about 15%-20% lower than for bottom ones. Nevertheless, in corroded specimens a net diference between bond strength values of bottom and top bars was not observed, with the same specimen shape and corrosion level.

As for the influence of the amount of transverse reinforcement, the tests of specimens with no stirrups confirmed bond stregth is significantly reduced when corrosion takes place. On the other hand, the presence of stirrups improves considerably bond behavior of rebars damaged by corrosion. Thus, the presence of transverse reinforcement allows to count on residual bond strength at least equal to 1,6 Mpa, even for small amounts of stirrups.

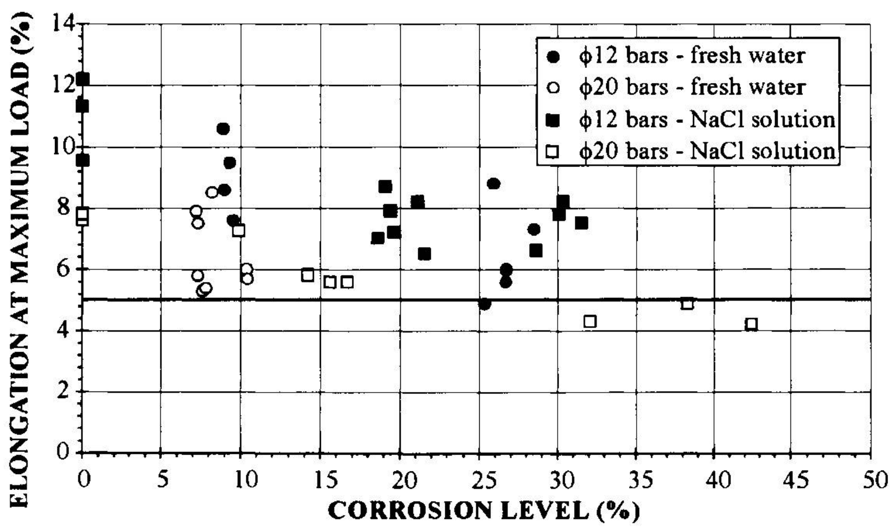

Fig 2. Relationship between elongation at maximum load and corrosion level

Locating Rebar Corrosion in Concrete Walls by Potential Measurements

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Summary

To localize corroding reinforcement in concrete buildings visual or sonic tests are not sufficient, because beginning corrosion cannot be recognized by these methods. By that means it is very likely that some areas starting to corrode will be missed - after a realized repair later damages may result. Therefore it is more useful to determine corrosion areas by electrochemical potential measurements.

Some application limits of the method of potential measurement have been examined. A special equipment has been developed to assess corrosion zones on the surfaces of concrete buildings.

1. Application Limits of Potential Measurements on Concrete Walls

The well-known method of potential mapping of bridges (fig. 1) is not transferable to concrete walls of buildings without problems, because the temperature and moisture of concrete walls is changing in a wide range, the concrete covering differs widely, concrete walls usually are surface finished by coatings or tiles, and often components of galvanized steel are used in concrete walls. Therefore the influences of the following parameters on the potential measurements have been examined:

- temperature and moisture of the concrete,
- thickness of the concrete covering,
- concrete surfaces with or without paints or coatings (fig. 2 and 3),
- components of zinc galvanized steel in concrete.

It could be shown that potential measurements do not depend on the temperature of the concrete but they were only possible between 30 and 90 % relative humidity of the surrounding atmosphere. Thick concrete coverings make it difficult to find small corrosion areas; fair-faced concrete walls coated e.g. with a thin acrylic paint are no problem (fig. 2), but on concrete surfaces with exposed aggregates by washing it is difficult to contact the cement grout (fig. 3). The interpretation of potential measurements in concrete walls with components of zinc galvanized steel is very difficult or impossible.

2. Measuring Equipment and Practical in-situ Tests

Coming from one half cell (cp. fig. 1) a more practical measuring equipment was developed: 6 CSE half cells were assembled in a frame, this enabled only one person to handle the equipment (fig. 4).

3. Conclusions

The reliability of the potential measurement method could be confirmed successfully by practical use on concrete walls.

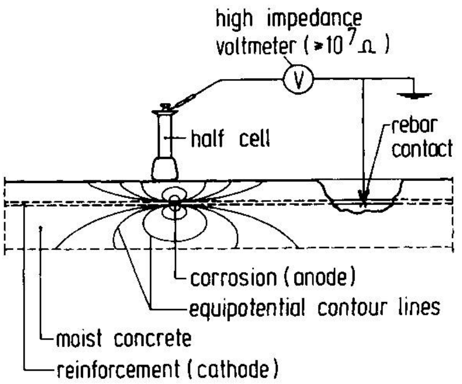

Fig. 1 Scheme for potential measurements with a half cell on the surface of a reinforced concrete member

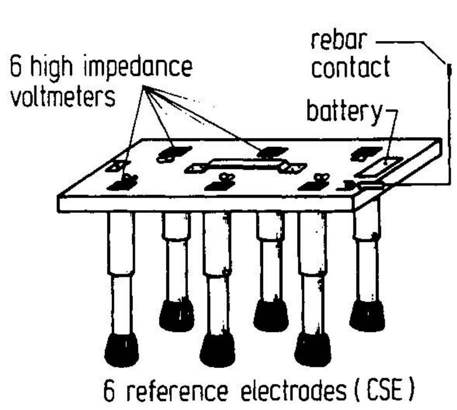

Fig. 4 View of the developed frame with 6 CSE half cells

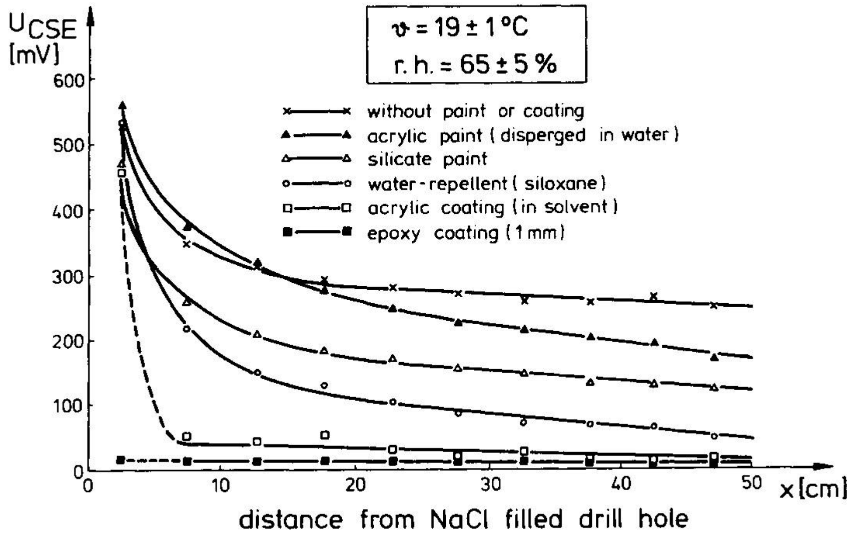

Fig. 2 Dependence of the potential U_{CSE} on the distance x to the NaCl filled drill hole for different paints or coatings on the concrete surface

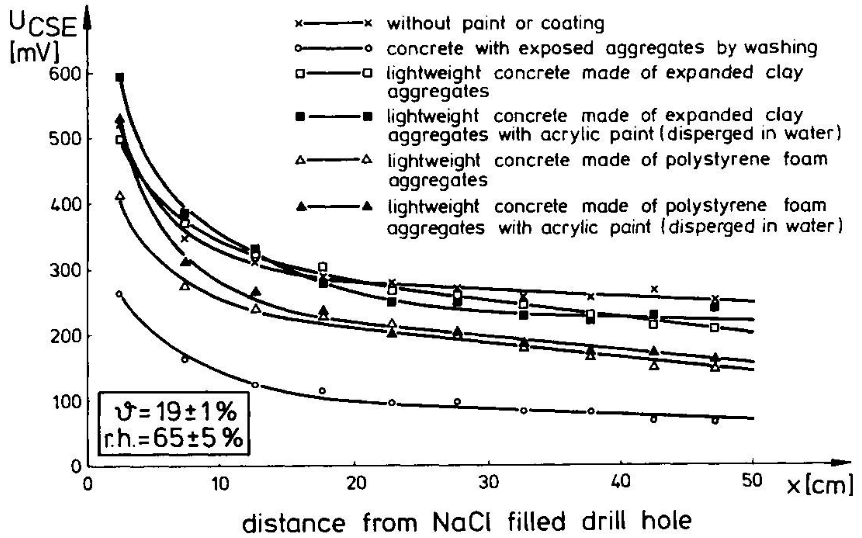

Fig. 3 Dependence of the potential U_{CSE} on the distance x to the NaCl filled drill hole for different lightweight concretes with or without thin acrylic paint

Dynamic Tests and Verification of Mechanical Properties of a Panel Building

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Summary

The calculated compressive stresses in the foundation base of an eight storey panel building due to standard wind load were compared with experimental results. The corresponding value of the compressive stress was obtained simultaneously with the displacement amplitudes of the roof from the ambient response of the building, which was also used to identify the predominant natural frequencies. This technique can help to assess the ambiguous mechanical behaviour in the event of redesign and rehabilitation of panel buildings.

Keywords: vibration mode shapes, dynamic tests, strain, panel building, machanical behaviour

1. Introduction

Most panel buildings were erected in the framework of the so called "collective production of flats" in the Czech Republic during the socialist period. The production quality of particular elements and the quality of their assembly on site was regularly different from the design parameters. Moreover, theoretical knowledge about the behaviour of this kind of concrete structures was insufficient, especially at the beginning of the period.

The survey of the condition of the bearing system, which was carried out at the Faculty of Civil Engineering CTU Prague on randomly chosen buildings in the housing estates of Prague between 1955 - 1970, showed, beside other results [1], the following facts:

- cracks 1 - 2 mm in width between the internal walls were found in 42% of the panel buildings
- similar cracks more than 2 mm in width were found in 32% of the buildings.

In comparison with the original design, the cracks lead most frequently to a partial loss of the stiffness of the vertical joints between the panels, which can considerably change the static scheme of the structure. The use of the stiffness of the vertical joints according to original documentation and drawings in the redesign, without knowledge of the loading history, could cause a substantial bias. The resulting stress of the structure depends on the rigidity of the joints between the panels [2]. An assessment of the real condition of all of the joints is normally impossible, and the expert's estimation will consider only the extreme limits. As in the case of new buildings, the not negligible uncertainty of the analysis is also caused by simplification of analytical models and by the uncertainty of input data [3-5].

We have considered the system of panel buildings with transversal walls, where the horizontal load-carrying capacity to wind load is critical from the reliability point of view, in some cases. The horizontal load can cause considerable compressive stresses in the base of panel structures which are braced in the longitudinal direction only by a single longitudinal wall.

The experiments described below were proposed in order to elaborate data on the true mechanical behaviour of panel buildings in the horizontal direction. The aim of the experiments was to make an experimental evaluation of the sensitivity of the compressive stresses due to the horizontal movement of the structure. The horizontal movement of the structure was measured from the ambient vibrations of the structure simultaneously with the stresses in the base of the structure.

Fig. 1: Photograph of the tested panel building

The Panel Building

The panel building that was chosen for the purposes of experimental verification (Resovska street, No. 22, 23, 24) was accomplished in the late seventies in Prague. It is an eight storey building consisting of three equal sections in a row, which are firmly attached to each other (see fig.1). The bearing system is formed by transversal walls 6,0 m apart and one 6m-long longitudinal wall in each section (see fig.2). The joints between the panels (0.19 m thick reinforced concrete slab-elements) consist of concrete and reinforcement. According to the visual survey some of the joints are weakened by hair-thin cracks. The front walls hang on the transversal walls and are not designed for load carrying purposes.

Dynamic Characteristics as Indicator of Structural Integrity

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Summary:

Detecting structural state of historical monuments and their adequate strengthening is very complex problem and requires cooperation among various specialists. Knowing the real state of historical monuments is very important before and after the strengthening and can only be established experimentally. Dynamic tests can reveal us the structural state without any damage to the structure. Usage examples of various dynamic tests, performed in order to determine dynamic characteristics and to conclude about the structural behavior and state, are outlined. Measured dynamic characteristics are then used to formulate precise mathematical model on which simulations of the extreme actions can be done. By collecting and statistically analyzing the measured data broader knowledge about the previous building methods can be gained and, many times, learned on the knowledge of old builders.

Keywords: historical monuments, structural state, dynamic tests, test-analysis corelation

1. Ferhad-Pasha Mosque

Experimental results indicated poor structural state and they were used to establish a real distribution of mass and stiffness along the height and to attribute earthquake forces, according to the real stiffness distribution. By knowing the measured results, reliable mathematical model could have been established and further analytical studies were made. Seismic strengthening was necessary and the chosen strengthening method did not disturb the mosque's historic value and was simple enough for construction.

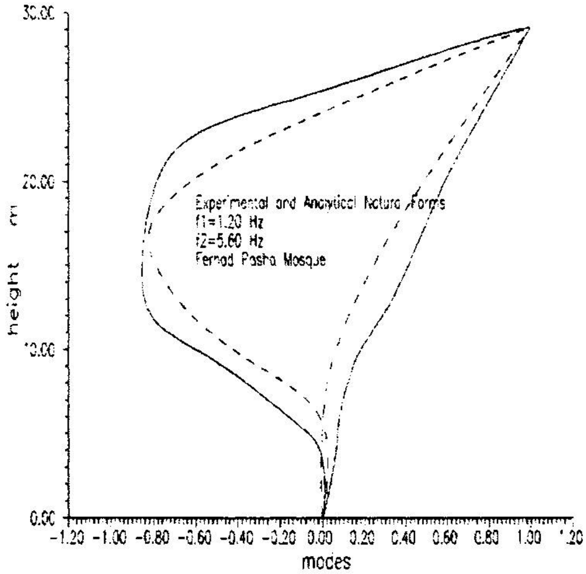

Fig. 1 Calculated and measured mode forms

2. Retaining Wall Structure on the Maria Luisa Road

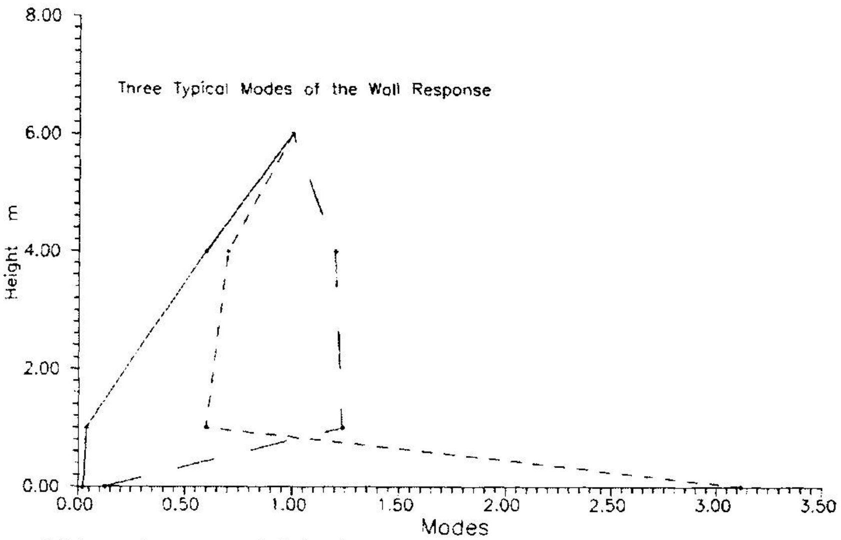

Fig 2. Three typical measured mode forms

Dynamic testing of the wall sections was performed as the only applicable method for evaluation of the structural state. Two methods were applied: (a) impulse tests and (b)transfer function test. As a result of both methods, wall response to dynamic excitation along the height was obtained, and the mode shapes of vibration could have been established. From the measured mode shapes, three main cases of the wall displacements could have been established and stability of the wall sections estimated on the ground of the measured quantity:

3. Mostar's Old Bridge

The measured mode shapes revealed that for vertical loading bridge behaves as an arch but of a smaller span, while for the perpendicular direction it behaves as a beam with fixed supports. Based on the experimental results, analytical model was modified until a satisfactory correlation among measured and analytical results was achieved.

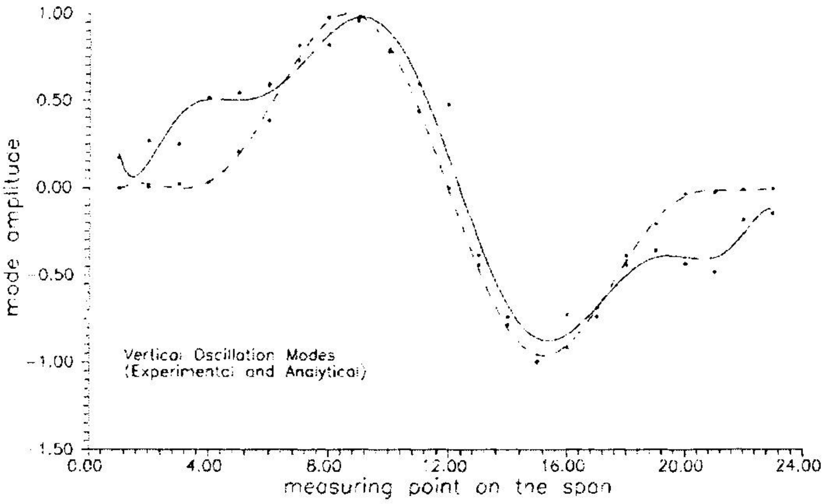

Fig. 3 Vertical mode shapes

Good correlation is obvious, and such mathematical model could be used for structural analysis and to check the influence and meaning of the various strengthening methods.

Dynamic characteristics reveal the most important structural data and represent the "personal" card of the structure. Changes in: measured mode shapes indicate change in strength distribution and reveal the real strength distribution; damping characteristics indicate change in the homogeneity; natural frequencies indicate change in the overall structural state.

Evaluation Prior to Repair of Industrial Reinforced Concrete Structures

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Summary

Tall and large-size reinforced-concrete structures, like high chimneys, cooling towers, silos, and other similar industrial buildings, need periodical diagnosis and repair, particularly when used in hard industrial conditions. Many kinds of damages result from technological reasons but, as a rule, the attack of polluted atmosphere from outside and/or from inside is always significant. As repair works on large and hardly accessible surfaces are costly the role of proper diagnosis, damage classification and, consequently, selection of repair methods is particularly important. In the paper, some examples of procedures are presented, as well as results of the investigations.

Keywords: concrete structures, concrete chimneys, cooling towers, corrosive actions, destruction of concrete, diagnosis of structures, concrete silos.

1. Introduction

Tall reinforced-concrete structures for heavy industrial use were usually erected without any finishing of concrete surfaces. Rough surfaces exposed for long time to direct influences of polluted atmosphere were step-by-step chemically and/or mechanically damaged. Statistically, the chemical destruction is dominating and the physical, mainly temperature effects are in the second row. In many situations, the walls for long time have been attacked from both sides, by external atmosphere and by destructive actions from inside. The speed and the range of deterioration depended on the intensity of corrosive actions but even more so on quality, mainly tightness of concrete, and the size of concrete cover over reinforcing bars.

Now, we know that maintenance and inspection of concrete structures should be regular and preventive but, as recently as 20 - 30 years ago the opinion about indestructibility of concrete was very common. Today, we also know the reasons of deterioration, and - more or less - we know how to eliminate these actions. Nevertheless, we are still looking for simple and objective methods of diagnosis, relatively not expensive tests, and selection of effective repair methods.

2. Evaluation of the state of structures

Unlike in bridges and road structures or marine structures, where the chloride penetration is the main corrosive agent, in industrial structures carbonation and sulfation processes are the most common problem. In real structures, the chemical test of carbonation degree and depth is the main indication how far is corrosive process, and what kind of protection is necessary to extend the lifespan of concrete structure. The most popular is determination of pH-reaction and the values pH≤9.0 are considered as unsafe degree of destruction process. Second step of chemical investigation includes more precise tests, e.g. determination of sulfate ions (SO_4^{2-}). The content of these ions over 1.5% of concrete mass is recognised as dangerous. To assess the thickness of layers with advanced carbonation and sulfation the drilled cores from a structure should be taken, (best of all throughout the full thickness of a wall), and cut into slices to be chemically tested.

In tall and large structures such tests are difficult, and sometimes even impossible, during the normal use of buildings. On the other side, the tests have to be done in many places, particularly along the whole height of walls. Unfortunately, in the same structure we may find places with carbonated layer of about 3-5 mm only, while few metres higher this layer is even 30 mm thick. It depends on the quality of concrete, particularly permeability of surface. As the number of drilled cores is limited, the specimens should be taken from characteristic places indicated on the basis of analysis of the course of erection and history of the structure use.

3. General description of typical cases

Large reinforced-concrete structures used for many years and not controlled properly are in majority damaged due to various external and internal influences. Each case requires exact diagnosis, classification of damage degree and selection of adequate repair methods.
In some kinds of structures the corrosive influences are typical (chimneys, cooling towers) while in others (containers, silos, tanks) the destructive actions and their results are much more individual. Generally, apart from careful inspection, the diagnosis should be supported by chemical tests. Particularly, the effect of deleterious industrial environment on carbonation rate and acid attack should be tested, as well as other technological internal pollutions.

High chimneys for large power plants were usually designed in the past as a single tube with the diameters chosen for the maximum emission from all boilers working with full rating. In reality, such a situation happened seldom or never. As a result, chimneys originally designed as "hot" were exploited all the time as "cold". Sometimes, additional reasons, like introduction of more efficient filters, seriously reduced the temperature of combustion gases previously expected.
It was the common reason of serious attack of aggressive agents, but the degree of corrosion varied significantly along the height of internal surface. The results of comparative tests of six chimneys, with height from 120 m to 300 m were the experimental material for the conclusions.

The characteristic shape of hyperboloidal cooling towers and relatively thin walls influenced on specific damages in these structures. Continuously wet atmosphere inside and high humidity outside the shells, in presence of aggressive gases (CO_2, SO_2) are the main reasons of concrete destruction. As a result of carbonation of $Ca(OH)_2$ and washing out of $CaCO_3$ (visible white outflows on the surfaces) the reduction of pH-reaction is a common phenomenon. The tendency of higher degree of calcium crystallisation in concrete (reduction of CaO) at internal surface is typical for the upper parts of shells. The results from tests on three cooling towers with heights 75 m, 90 m, and 96 m gave the experimental data for the general assessment.

Evaluation of the Behaviour of Corroded Beams

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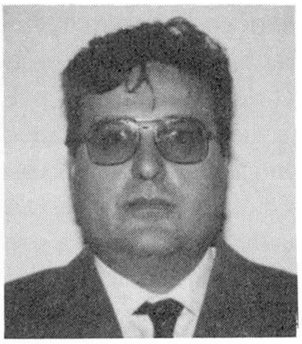

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Summary

To analyse the behaviour of corroded beams, in the paper a model based on damage function D.F. is shown linked to a global damage coefficient D.C. of the beam that may be obtained by means of dynamic tests.

1. Analysis of damaged beam

In the r. c. beams, the damage due to reinforcement corrosion may cause cracking of concrete around the bars both at the compressive and tensile zone of section [1]. Cracks due to rust create a *compressive softening* effect [2] in the concrete with an influence on the strength and ductility and load-deflection response of the concrete beam. The damage of concrete subjected to tension by rust causes a reduction of stiffness that has consequence on the dynamic behaviour of the beam [3]. The comparison between natural frequency of an undamaged beam, $\omega_{0,r}$, and a damaged beam due to corrosion, $\omega_{1,r}$, can define the state of damage if there are significant changes in the dynamic characteristics.
In the paper, we consider that the beam has a stiffness uniformly reduced on the length by diffused cracks due to corrosion. This situation is, for example, characteristic of cantilever beams. In this case, it is sufficient to use only one value of frequency to evaluate the undamaged and damaged state of the beam by means of global value damaged coefficient D.C. :

$$\text{D.C.} = 1 - (\omega_{1,r}/\omega_{0,r})^2, \quad r=1 \tag{1}$$

In the Fig.1, undamaged and damaged r.c. sections are shown with own strain - stress diagrams. The global coefficient of damage D.C. expressed in (1), considering the bending stiffness in undamaged $(EI)_0$ and damaged state $(EI)_1$, is defined in this way:

$$\text{D.C.} = \frac{(EI)_0 - (EI)_1}{(EI)_0} \tag{2}$$

$$M = \chi_1 \, (1 - \text{D.C.}) \, (EI)_0 \tag{3}$$

respectively for the curvature χ_1 and bending moment M in a damaged section (Fig.1).

To define the modified strain - stress state in the section, we assume a linear function D.F. which reaches the value D for $\eta = -k_1$:

$$\text{D.F.} = D \cdot (1 - \frac{k_1}{2 \cdot \alpha}) - (\frac{D}{2 \cdot \alpha}) \cdot \eta \qquad (4)$$

where $\eta = \frac{y}{d}$.

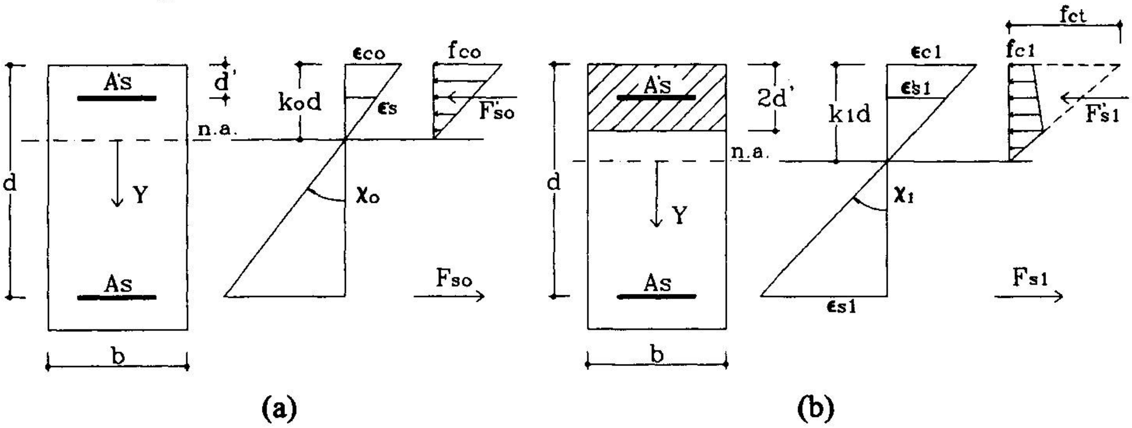

Fig. 1 - (a) Undamaged section ; (b) Damaged section

The value of D may be related to the strain ε_{c1} of the extreme fiber of compressive concrete of damaged section :

$$D = \gamma \cdot \varepsilon_{c1} \cdot E_c \qquad (10)$$

using a coefficient γ enclosed between 0 and 1.

In the damaged section of Fig. 1b, considering the compatibility and equilibrium relations, we obtain two relations in the unknown parameters γ, k_1 and D.C.. These relations together to the (1) permit to evaluate the stress-strain state in a r.c. damaged section.

2. Conclusions

The deterioration of r.c. beams due to corrosion reduces the bending capacity of the section with increase of curvature and deflection. The corrosion especially influences the strength of compressive concrete by means of the biaxial stress state due to the rust on the bars. Using dynamic tests it is possible to estimate the state of damage expressed by a global coefficient D.C.. Above we have defined also a linear damage function D.F. to analyse the stress distribution in the damaged compressive concrete.

3. References

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Repair of a Swimming Pool after Design Errors

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Summary

Due to its specific features, designing of indoor swimming pools and baths requires great experience from both: the designer and the contractors, otherwise the mistakes committed in the phase of designing or erection contribute to various defects. The paper illustrates some characteristic and common errors in designing and execution of indoor swimming pools which have induced emergency situations and the need of making costly repairs.

1. Introduction

Building of indoor swimming pools requires great experience from the designer and the contractors. Unfortunately, the designing of such facilities is often entrusted to persons who have an inadequate practice in this matter. Things being as they are, the designer, whilst designing the facilities is not well aware of the specific problems and circumstances which are vital for usefulness, durability and reliability of the facilities to be constructed. Considering a poor workmanship, frequent lack of appropriate engineering supervision and an usual poor quality of building materials, practically inevitable is a high failure frequency of such constructions.
The paper exemplifies some characteristic and common errors and irregularities in designing, which have lead to critical situations with many indoor swimming pools in the Upper Silesia. All facilities have been built in the period of last 25 and required costly repairs in order to eliminate great building damages, such damage being occurred after a drastically short period of use.

2. Design mistakes

2.1 Mistakes in designing of basin of the swimming pools

Reinforced concrete, monolithic basins of the swimming pools usually constitute spatial, statically indeterminate load-carrying structures. The brief foredesigns adopted by the designers at the stage of structural analysis to reduce the real structure to the equivalent ones i.e. the plain bar & plate structures very often produce too big errors. As a result of oversimplifications and of the errors in adoption of equivalent static diagrams, loads and bar rigidity factors, the values of the internal forces in the some of elements of the main frame structures are underrated. Neglecting of certain components of the real construction when adopting equivalent calculation schemes constitutes the other, but also quite typical example of errors committed at the stage of construction designing. The effects of such approach appear as the defects and failure conditions of the structure.

2.2 Faulty solution for expansion joints

The use of expansion joints between the basin structure and the surrounding roof is foreseen as a rule for the reinforced concrete basins of swimming pools. Sometimes, the designer has foreseen the execution of expansion joints between the basin and the roof structure components, but consequently has not used them for the finish layers. The gap created at the level of the overflow gutter was as wide as 30 mm. In order to eliminate the damage completely, necessity arose to remove away the finish layers surrounding the basin crest and remake them, ensuring a hermetic expansion joint of the newly laid finish.

2.3 Errors related to the shrinkage

In the real reinforced concrete structures of the basins of swimming pools often shrinkage cracks occur. As a result of the shrinkage, at the side walls of the basin there are developed vertical, through cracks. The shrinkage effect has been developed under the circumstances of limited deformation freedoms resulted from a monolithic linkage of the walls to the bottom plate, where the shrinkage phenomena had been established a considerable long time ago. It is typical situation, when the walls of basin have been embedded in concrete much later after the bottom of the swimming pool had been made

2.4 Improper solutions for deck roof structures or outer walls

There are many damages of the swimming pool buildings connected with faulty solutions of the deck roofs and walls, especially outer walls. Most of swimming pool halls have steel girders of the hall with a suspension roof of steel and timber structure When there is no adequate ventilation and protection, the corrosion has attacked improperly protected steel girders and suspension rods of the hall roof. Practically, the wooden components of the roof has been biologically destroyed. Moreover, some common mistakes were committed and they still exist as it concerns the structural-material solutions for the outer division walls of the swimming pool structure. It applies for blink walls as well as for those provided with window systems. Often, when designing, inadequate attention is paid to the aspects related to the building physics.

3. Conclusion

The authors hope for the fact, that the examples of frequent failures of swimming pools presented in the paper induce the future and actual designers towards a penetrating approach and a careful analysis of these facilities. Moreover, investors should try to make a proper choice of designers and specially to ensure for a professional, independent and may be a multistage verification of the design.