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NUCLEAR ENERGY AGENCY
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

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OECD-NEA WORKSHOP ON THE EVALUATION OF DEFECTS, REPAIR CRITERIA & METHODS OF REPAIR FOR CONCRETE STRUCTURES ON NUCLEAR POWER PLANTS

Hosted by GRS at the DIN Institute in Berlin, Germany

10th-11th April, 2002

English - Or. English

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COMMITTEE ON NUCLEAR REGULATORY ACTIVITIES

The Committee on Nuclear Regulatory Activities (CNRA) of the OECD Nuclear Energy Agency (NEA) is an international committee made up primarily of senior nuclear regulators. It was set up in 1989 as a forum for the exchange of information and experience among regulatory organisations and for the review of developments which could affect regulatory requirements.

The Committee is responsible for the programme of the NEA, concerning the regulation, licensing and inspection of nuclear installations. The Committee reviews developments which could affect regulatory requirements with the objective of providing members with an understanding of the motivation for new regulatory requirements under consideration and an opportunity to offer suggestions that might improve them or avoid disparities among Member Countries. In particular, the Committee reviews current practices and operating experience.

The Committee focuses primarily on power reactors and other nuclear installations currently being built and operated. It also may consider the regulatory implications of new designs of power reactors and other types of nuclear installations.

In implementing its programme, CNRA establishes co-operative mechanisms with NEA's Committee on the Safety of Nuclear Installations (CSNI), responsible for co-ordinating the activities of the Agency concerning the technical aspects of design, construction and operation of nuclear installations insofar as they affect the safety of such installations. It also co-operates with NEA's Committee on Radiation Protection and Public Health (CRPPH) and NEA's Radioactive Waste Management Committee (RWMC) on matters of common interest.

COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

The NEA Committee on the Safety of Nuclear Installations (CSNI) is an international committee made up of scientists and engineers. It was set up in 1973 to develop and co-ordinate the activities of the Nuclear Energy Agency concerning the technical aspects of the design, construction and operation of nuclear installations insofar as they affect the safety of such installations. The Committee's purpose is to foster international co-operation in nuclear safety amongst the OECD Member countries.

CSNI constitutes a forum for the exchange of technical information and for collaboration between organisations which can contribute, from their respective backgrounds in research, development, engineering or regulation, to these activities and to the definition of its programme of work. It also reviews the state of knowledge on selected topics of nuclear safety technology and safety assessment, including operating experience. It initiates and conducts programmes identified by these reviews and assessments in order to overcome discrepancies, develop improvements and reach international consensus in different projects and International Standard Problems, and assists in the feedback of the results to participating organisations. Full use is also made of traditional methods of co-operation, such as information exchanges, establishment of working groups and organisation of conferences and specialist meeting.

The greater part of CSNI's current programme of work is concerned with safety technology of water reactors. The principal areas covered are operating experience and the human factor, reactor coolant system behaviour, various aspects of reactor component integrity, the phenomenology of radioactive releases in reactor accidents and their confinement, containment performance, risk assessment and severe accidents. The Committee also studies the safety of the fuel cycle, conducts periodic surveys of reactor safety research programmes and operates an international mechanism for exchanging reports on nuclear power plant incidents.

In implementing its programme, CSNI establishes co-operative mechanisms with NEA's Committee on Nuclear Regulatory Activities (CNRA), responsible for the activities of the Agency concerning the regulation, licensing and inspection of nuclear installations with regard to safety. It also co-operates with NEA's Committee on Radiation Protection and Public Health and NEA's Radioactive Waste Management Committee on matters of common interest.

Foreword

The Committee on the Safety of Nuclear Installations (CSNI) of the OECD-NEA co-ordinates the NEA activities concerning the technical aspects of design, construction and operation of nuclear installations insofar as they affect the safety of such installations. In 1994, the CSNI approved a proposal to set up a Task Group under its Principal Working Group 3 (recently re-named as the Working Group on Integrity of Components and Structures (IAGE)) to study the need for a programme of international activities in the area of concrete structural integrity and ageing and how such a programme could be organised. The task group reviewed national and international activities in the area of ageing of nuclear power plant concrete structures and the relevant activities of other international agencies. A proposal for a CSNI programme of workshops was developed to address specific technical issues which were prioritised by OECD-NEA task group into three levels of priority:

First Priority

- Loss of prestressing force in tendons of post-tensioned concrete structures
- In-service inspection techniques for reinforced concrete structures having thick sections and areas not directly accessible for inspection

Second Priority

- Viability of development of a performance based database
- Response of degraded structures (including finite element analysis techniques)

Third Priority

- Instrumentation and monitoring
- Repair methods
- Criteria for condition assessment

The working group has progressively worked through the priority list developed during the preliminary study carried out by the Task Group. Currently almost all of the three levels of priority are effectively complete, although in doing so the committee has identified other specific items worthy of consideration. By working logically through the list of priorities the committee has maintained a clarity of purpose which has been important in maintaining efficiency and achieving its objectives. The performance of the group has been enhanced by the involvement of regulators, operators and technical specialists in both the work of the committee and its technical workshops and by liaison and co-operation with complementary committees of other international organisations. The workshop format that has been adopted (based around presentation of pre-prepared papers or reports followed by open discussion and round-table development of recommendations) has proved to be an efficient mechanism for the identification of best practice, potential shortcomings of current methods and identification of future requirements.

SUMMARY

OECD-NEA workshop on the evaluation of defects, repair criteria & methods of repair for concrete structures on nuclear power plants

OECD-NEA IAGE held an international workshop on the evaluation of defects, repair criteria & methods of repair for concrete structures on nuclear power plants in Berlin, Germany on April 10-11, 2002. Through 2 technical sessions devoted to Operational Experience and State of the Art and Future Developments, a broad picture of the status was given to a large audience composed by 54 participants from 17 countries and International Organisations. 21 papers have been presented at the Workshop.

The objectives of the workshop were to examine the current practices and the state of the art with regard to the evaluation of defects, repair criteria and methods of repair for concrete structures on Nuclear Power Plants with a view to determining the best practices and identification of shortfalls in the current methods, which are presented in the form of conclusions and recommendations in this report.

This workshop on the evaluation of defects, repair criteria and methods of repair for concrete structures on Nuclear Power Plants is the latest in a series of workshops.

The complete list of CSNI reports, and the text of reports from 1993 on, is available on http://www.nea.fr/html/nsd/docs/

Acknowledgement

Gratitude is expressed to GRS, Germany for hosting the Workshop at the DIN Institute in Berlin. In particular, special thanks to Mr. Helmut Schulz and Dr Jurgen Sievers, and also Mrs Brunhilde Laue and Mrs Schneider for their help.

Thanks are also expressed to chairmen of the sessions and to the Organizing Committee for their effort and co-operation.

Dr Leslie M Smith	**BEG(UK) Ltd**	**(UK) Chairman**
Prof Pierre Labbé	IAEA	(International)
M. Jean-Pierre Touret	EdF	(France)
Herr Rüdiger Danisch	Framatome ANP GmbH	(Germany)
Mr James Costello	USNRC	(USA)
Dr Dan Naus	ORNL	(USA)
M.Eric Mathet	OECD-NEA	(International)

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10th and 11th April, 2002
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Volume 2

SESSION B: STATE OF THE ART & FUTURE DEVELOPMENTS
Chairman: Dr. Naus, ORNL (US)

Addressing concrete cracking in NPPs.

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Abstract

The phenomenon of concrete cracking is one of the most frequently encountered deterioration at NPPs as it has been shown by a wide Survey of NPPs performed by IAEA in 1994-95

It can be due to a multitude of causes such as the normal ageing process (shrinkage, creep, prestressing force loss) as well as exposure to the environment (temperature variation, moisture, freeze/thaw, etc)

The above mentioned Survey has also shown that in 64% of cases, no action was taken or required. It became also obvious that there is a lack of guidance as when remedial actions are needed.

The paper describes, with the help of a Flow Chart, the various stages to be considered, from the first step of identification of cracks, to the definition of causes, evaluation of extent of damage, evaluation of effect/implications (safety, reliability) , to the final step of deciding if repair action is required.

Finally, based upon a wide literature survey the paper proposes in a Chart format, Criteria for addressing concrete cracks in NPPs., when taking in considerations all these factors.

General

Reinforced concrete structures deteriorate in various ways due to the normal ageing process (shrinkage, creep, prestressing cables relaxation) and/or impact from aggressive environment (temperature, moisture, cyclic loading)

The rate of deterioration will depend on the component’s design, material selection, quality of construction, curing, and aggressiveness of the environment.

The experience gained from an international survey on ageing of Nuclear Power Plants initiated by IAEA in 1994-95 [2] was that concrete cracking was the most frequent form of degradation. At the same time the Survey has shown that in 64% of cases no action was required.

Various reasons could explain this lack of action which could have unpleasant implications.

In-service inspection techniques are available that can indicate the occurrence and extent of such an ageing or environment-stressor related deterioration. Periodic application of these techniques as part of a condition assessment program can indicate the progress of deterioration. The results obtained from these programs can be used to develop and implement remedial actions before the structure attains an unacceptable level of performance. Depending on the degree of deterioration and the residual strength of the concrete component, the remedial measures may be structural, protective, cosmetic, or any combination of these.

A plant service life management program would normally include measures for detection and monitoring (inspection, instrumentation, reporting, mapping, etc,), assessment of extent of damage (measurement) and type of damage (impacting safety, reliability), technicalities (possibility to repair, cost, duration), scheduling (prioritization), and selection of a repair method.

However the world literature as well as codes and standards ,if not confusing, at least are lacking clear indications when to proceed with repair, taking in consideration all aspects involved.

This paper discusses the process which should lead to the selection of an effective repair method and proposes, based upon worldwide standards and literature, criteria which should lead to the decision whether to repair or not concrete cracks, after the cracks have been identified and evaluated, addressing the entire range of aspects involved.

Lead way to the selection of a repair method

Definition of Cause

Once cracks have been identified through periodic inspection or instrumentation, their location and dimension recorded and evolution monitored, the next step is the definition of causes, since no repair should be undertaken before the cause of cracks has been identified.

Concrete ageing, external factors or simply the aspect of the cracks could provide information about causes. In each of these three categories there are a number of specific indicators (Chart#1).

(i) Cracks due to concrete ageing .

One category of cracks is the result of the ageing phenomenon of the concrete per se, and with time cracks will multiply or increase in width, depth or length, thus giving an indication of the status of ageing of that concrete.

The reasons for cracking which are time dependent could be:

- shrinkage/creep
- prestressing loss of the cables
- reinforcing bars corrosion.

The creep of the concrete will be present in particular in prestressed concrete structures and the associated loss of prestressing force viz cable relaxation should always be considered as a potential cause of cracking, while this cause will not be present in conventionally reinforced concrete.

(ii) Cracks due to external factors.

Another category of cracks is the result of the various external factors independent of the concrete ageing but associated with ageing since they have an acceleration effect upon it.
In this category fall cracks resulting from the effect of exposure to the environment :

- freeze/thaw cycle,
- chloride penetration,
- carbonation,
- aggressive environment (e.g.atmospheric pollutants, acid rain, fog, sea/ground water exposure.)

-cycling loading (e.g. mechanical/thermal),
-construction defects (e.g.low concrete quality, excessive permeability, alkali reactive aggregates-AAR, early formwork removal),
-design criteria (allowing for concrete in plastic/non-linear state, thus with acceptable crack formation in the tension zone, insufficient knowledge of some long term acting design parameters),
-thermal gradient
-accidents (e.g. loss of coolant-LOCA, fire, earthquake)
-excessive testing, (e.g. containment leak rate test)

(iii) Cracks aspect

The location or pattern (direction) off cracks could also provide indication about the reason of their formation.

To recognize, viz differentiate between all these causes and heir related effect, the world literature and in particular the extensive work and publications produced by RILEM and IAEA can provide ample information. [1], [3], [4], [5], [6], [7], [8], [13].

Extent of Damage.

Two aspects need to be considered in the process of identification of the extent of damage i.e. the cracks dimension and their evolution.(Chart #1).

(i)Cracks dimension.

There are three parameters which need consideration, i.e. the width, length and depth of the cracks. These parameters are usually monitored and IAEA Survey [2] has shown that, world wide,
- the crack width was recorded by 73.3% of the stations,
- the length was recorded by 74% of the stations,
- the depth was recorded by 21.4% of the stations.

Recording crack width and/or depth is important since the crack size can affect the corrosion of the reinforcing, which in turn will amplify the effect of the freeze/thaw cycles and the ingress of other aggressive agents (e.g. chlorides) to the proximity of the reinforcing bars. Various authors have rated the significance of the crack width based upon these aspects as shown in Table1, [3], [7]. From Table1 it appears that crack widths between 0.1mm and 0.4 mm are considered acceptable, depending on the environmental conditions.
However it does not address the complexity of the cracking phenomenon with all aspects involved.

(ii)Cracks activity

Recording crack length is important in order to determine their evolution, since this will indicate if cracks are active, i.e. the process accountable for the cracks is still in effect and will have to be identified and addressed, while no change in the crack length (dormant cracks) could indicate that the process has stopped and may not be age related.

There are one time cracks like those due to an accidental loading or construction defects and which will not further develop, and cracks originated by time dependent factors like concrete “ageing per se” (e.g. carbonation) or environmental factors (e.g. chlorides, freeze/thaw).

To the first category belong the Dormant cracks while to the second category belong the Active cracks **(**Table2), [3], [8].

This classification is important when assessing the urgency for repair or the repair method. Also in the case of Active cracks the cause should be first eliminated, before a repair is undertaken.

Table 2 however is limited to a few cases and can not provide sufficient guidance.

Impact of Damage.

The next step of the process consists of establishing the impact the cracks can have upon the concrete member in particular and upon the structure in general, since this will affect the decision whether to proceed or not with repairs. This step involves consideration of the following;

Structural member ranking.

The effect of cracks will depend upon the structural member ranking which should take into account the importance and function of the member based upon its structural and functional role which will indicate to what extent the safety or functional performance are affected. Finally this will lead to the necessity and urgency to proceed with repairs

A few methodologies were suggested for ranking nuclear plant components, i.e. in the US according to a program elaborated at the Oak Ridge National Laboratory

[7],[9] ,or in the UK [10], or in Canada [11], and by RILEM [8].
These should provide the basis for each NPP to develop their own methodology .

Assessment.

Before selecting a type of repair, the last step in the process is the assessment of the necessity to proceed with repairs or not. This should be based upon a Cracks Acceptance Criteria

The best attempt so far was made by C.J. Hookham in 1995 taking in consideration the crack dimension and environmental factors (chloride penetration and depth of carbonation), as shown in Chart [6].

Going with the complexity of the crack phenomenon one step farther is the Cracks Acceptance Criteria presented in this paper and as shown in a chart form in Figure 1 and 2.
The two charts include the following parameters for consideration:

-crack width (range 0.2 mm to 1.0 mm),
-type of environment (mild or aggressive),
-cracks activity (active or dormant), and
-depth of chloride penetration or carbonation (low or high).

The values set are based upon a review of the world literature and codes and standards referenced in this paper.

The crack width is closely related to the urgency of repair since this represents an open path for the aggressive agents to reach the reinforcement.

The upper limit is 0.2mm beyond which repairs are not required under any circumstances while beyond the lower limit of 1.0 mm, repairs are required in all cases. For in-between values indication are given in the two charts.

The aggressiveness of the environment will also affect the decision. Thus an aggressive environment (e.g. sea shore location with high chloride content, proximity to air pollutant industries, high freeze/thaw cycle, etc) will increase the limit and associated repair urgency factor.

Regarding the cracks activity, for the dormant ones the necessity of repair will be less stringent as reflected in the two charts.

The depth of chloride penetration and/or carbonation are indicators of how close the initiation of reinforcement corrosion is. Maximum permissible chloride contents, as well as minimum recommended reinforcement cover requirements have been provided in codes and guides. The threshold of acid-soluble chloride contents reported by various investigators which could initiate steel corrosion ranges from 0.15 to 1.0% by weight of cementitious materials, whereas code limits range from 0.2 to 0.4%. [3],[7],[13]. In the two charts (Fig.1 and 2) the limit selected is 0.4%.

As earlier indicated the cracks have also to be considered in connection with the importance of the structural member affected which will come from the Plant Components Ranking. For a high ranking component (Figure 1) the urgency of repairs will be grater than for a low ranking one (Figure 2).

Recommendations.

Each NPP should develop a concrete component ranking as well as a database (history) of crack repairs performed and their effectiveness.

An experienced civil engineer familiar with concrete ageing, should be associated with the entire assessment process described.

Repair should proceed only after the cause has been determined and the repair should include the elimination of cause.

Concluding remarks

The paper has detailed the various steps to follow when dealing with concrete cracks, from the time of their identification to the decision making of when and how to proceed with their repair, with Chart#1 as a guide through the various steps involved.

Figure 1 and 2 are the Acceptance Criteria to follow at the end of the assessment process when all required crack parameters are known, to provide the answer if and when remedial action is required.

The selection of repair materials and method were not within the scope of this paper.

However such information, when required, can be found in the world literature or from the experience of other similar NPPs using inter communication grids like WANO or COG since most Utilities will have a database of repairs performed and their effectiveness[11].

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Table 1: Permissible crack widths to prevent corrosion of steel reinforcement. [3],[7]

Author	Environment factors	Permissible width, mm
Rengers	Dangerous crack width	1.0 to 2.0
	Crack width allowing corrosion within 1/2 year saline environment	0.3
Abeled	Structures not exposed to chemical influences	0.3 to 0.4
Tremper	Found no direct relation between crack width and corrosion	
Boscard	Structures exposed to a marine environment	0.4
de Bruyn	Found no direct relation between crack width and corrosion	
Engel and Leeuwen	Unprotected structures (external)	0.2
	Protected structures (internal)	0.3
Voellmy	Safe crack width	up to 0.2
	Crack allowing slight corrosion	0.2 to 0.5
	Dangerous crack width	over 0.5
Bertero	Indoor structures	0.25 to 0.35
	Normal outdoor exposure	0.15 to 0.25
	Exposure to sea water	0.025 to 0.15
Haas	Protected structures (interior)	0.3
	Exposed structures (exterior)	0.2
Brice	Fairly harmless crack width	0.1
	Harmful crack width	0.2
	Very harmful crack width	0.3
Salinger	For all structures under normal conditions	0.2
	Structures exposed to humidity or to harmful chemical influences	0.1
Wastlund	Structures subjected to dead load plus half the live load for which they are designed	0.4
	Structures subject to deal load only	0.3
Efsen	Exterior (outdoor) structures exposed to attack by sea water and fumes	0.05 to 0.25
	Exterior (outdoor) structures under normal conditions	0.15 to 0.25
	Interior (indoor) structures	0.25 to 0.35
Rusch	Ordinary structures	0.3
	Structures subjected to the action of fumes and sea environment	0.2

Table 2: General guide to repair options for concrete cracking. [3],[8]

Description	Repair Options	Perceived durability rating (1-5*)	Commentary
Dormant pattern or fine cracking	Judicious neglect Autogenous healing Penetrating sealers Coatings HMWM or epoxy treatment Overlay or membrane	4 3 2 3 2 2	Only for fine cracks Only on new concrete Use penetrating sealer for H_20, Cl resistance Use coating for abrasion and chemical resistance Topical application, bonds cracks For severely cracked areas
Dormant isolated large cracking	Epoxy injection Rout and seal Flexible sealing Drilling and plugging Grout injection or dry packing Stitching Additional reinforcing Strengthening	1 3 4 3 4 5 4 3	Needs experienced applicator Requires maintenance Requires maintenance
Active cracks	Penetrating sealer Flexible sealing Route and seal Install expansion joint Drilling and plugging Stitching Additional reinforcing	3 3 3 2 4 4 3	Cracks less than 0.5 mm Requires maintenance Use for wide cracks Expensive May cause new cracks May cause new cracks May cause new cracks
Seepage	Eliminate moisture source Chemical grouting Coatings Hydraulic Cement dry packaging	2 1 4 4	Usually not possible Several applications may be necessary May have continued seepage May have continued seepage

* Scale from 1 to 5, with 1 being most durable.

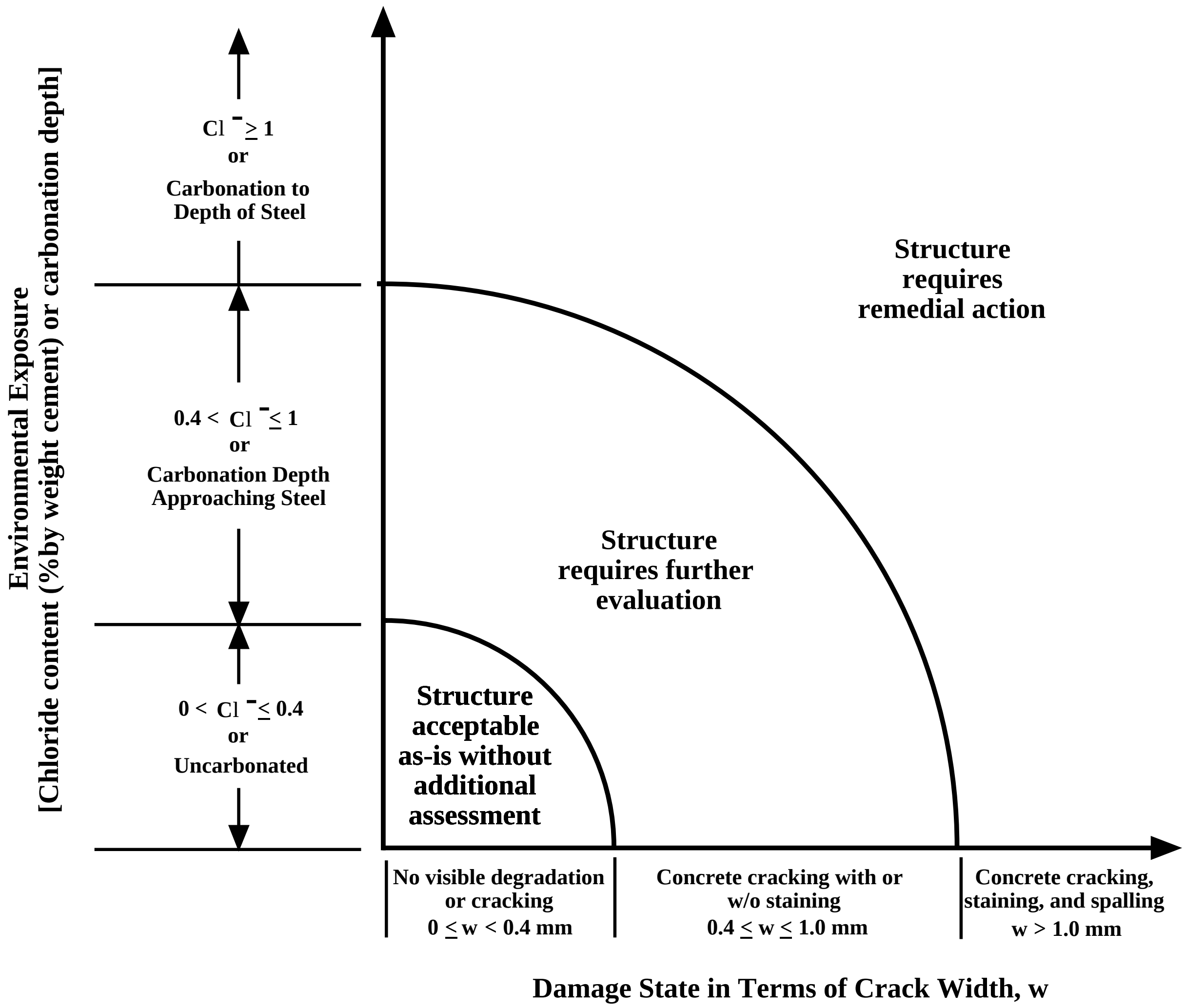

FIG. 7.3. Damage state chart relating environmental exposure, crack width, and necessity for additional evaluation or repair [Ref. 7.6].

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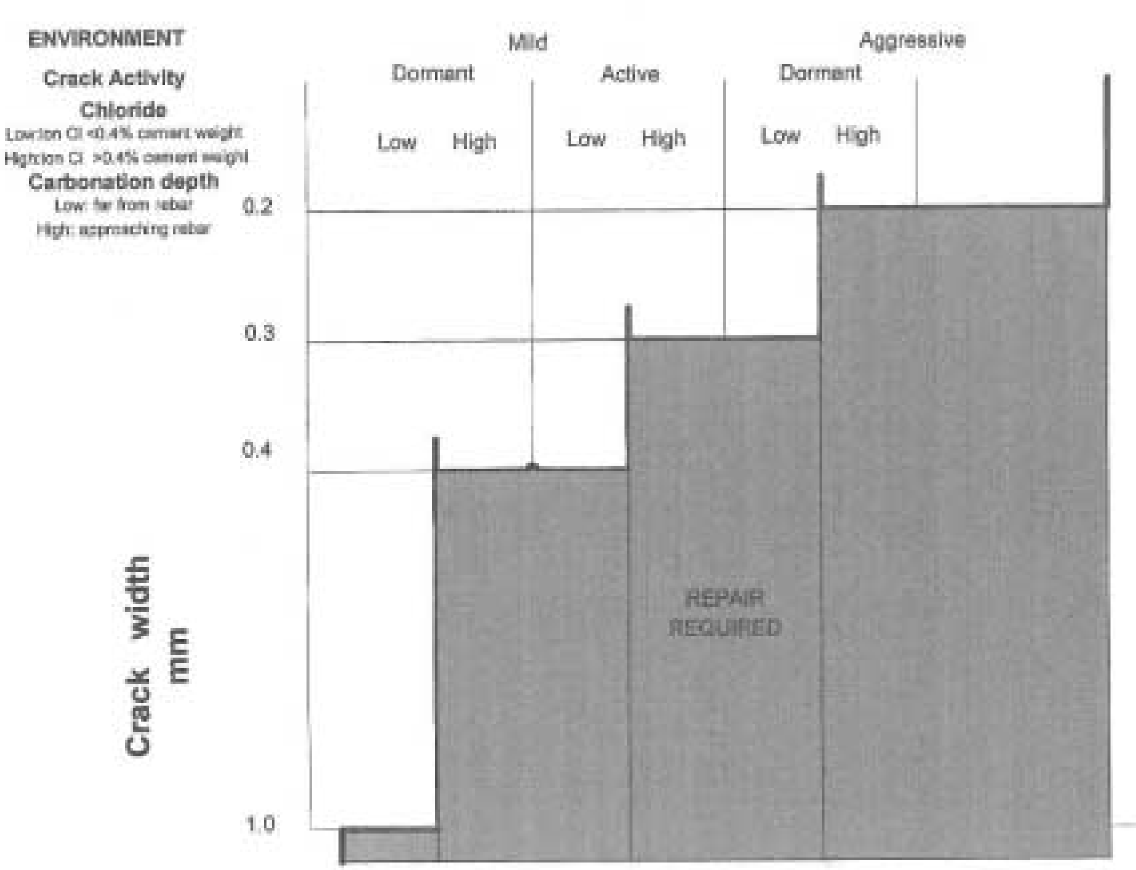

Fig. 1 Criteria for crack repair
High ranking equipment.

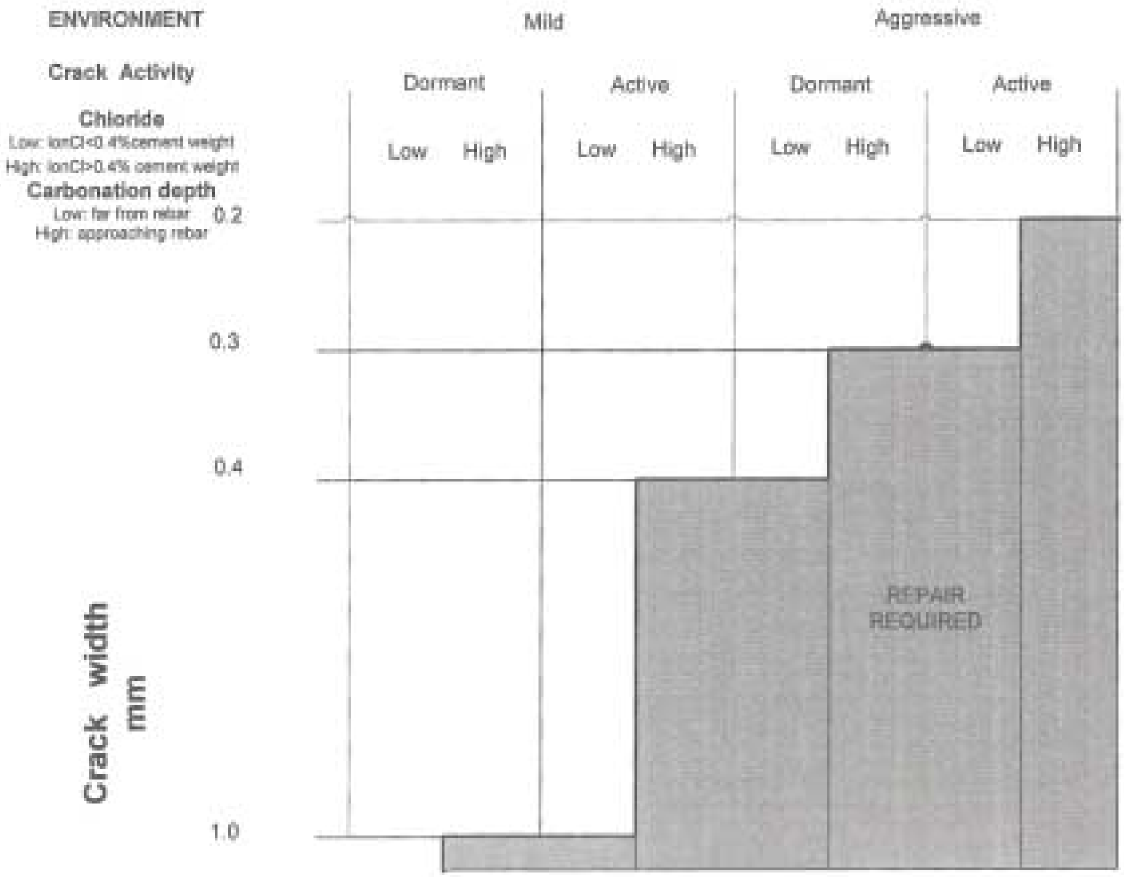

Fig. 2 Criteria for crack repair
Low ranking equipment.

Investigation of the leakage behaviour of reinforced concrete walls

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1 Introduction

Information about the leakage behaviour of the containment in case of an accident is of decisive importance for the verification of nuclear power plant safety. Assuming a core melt accident with failure of the reactor vessel in a pressurized water reactor, high internal pressures accompanied by temperatures well over the water boiling temperature can occur. During loss of the primary coolant a large quantity of steam develops. In case of resulting cracks through the entire thickness of a pre-stressed containment wall without liner an air-steam mixture enriched with aerosols may be released.

Regarding the leakage of pure air through cracked concrete walls there are a couple of investigations introducing correlations based on crack width and pressure differential, like [7], [12] for instance. The leakage behaviour of water through cracked concrete walls has already been investigated as well [3], [9]. Investigations on the leakage of steam and air through narrow, idealized cracks were performed in [1]. However, corresponding knowledge about the leakage of air-steam mixtures through real cracks was missing world-wide so far.

2 Conception of the experiments

The leakage observation can be divided in mechanical processes determining cracks through the wall and the thermo-hydraulic processes. The development of crack patterns and average crack widths can be calculated quite well using available numerical procedures like the finite element method [5]. Contrary to the mechanical processes it's difficult to describe the parameters governing the thermo-hydraulic processes inside the cracks like roughness, temperature, heat transfer and condensation.

Aim of the project was to analyse the thermo-hydraulic process of water-air leakage with condensation through known crack patterns. In contrast to other investigations on large scale model containments like MAEVA [8], it was not intended to simulate the integral behaviour of the containment as a whole but of a representative section of the containment wall.

Integral tests were performed on specially developed specimens with complex and realistic crack patterns. Due to the applied axial load during cracking the induced cracks are of almost uniform mean widths. The thermo-hydraulic load used for the tests followed a severe accidental scenario based on the design scenario for a core melting accident of the EPR (Fig. 1).

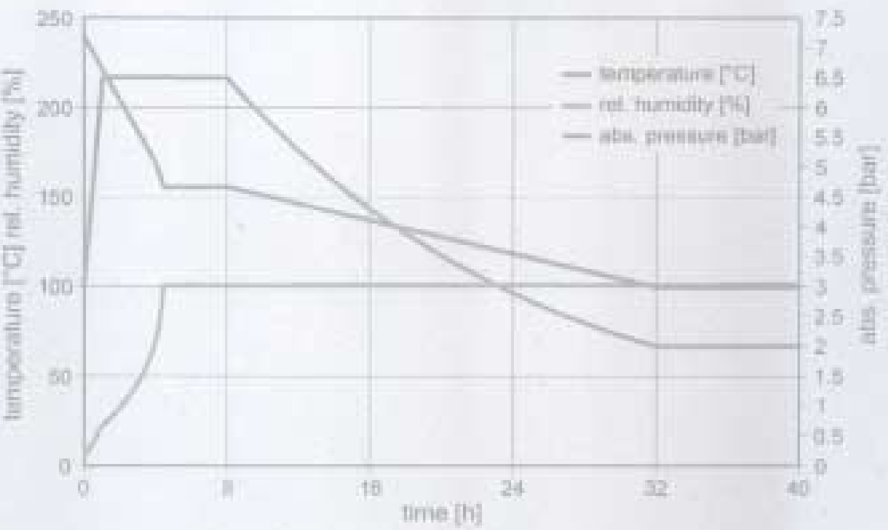

Fig. 1: Test scenario

3 Testing facility

3.1 Mechanical set-up

The mechanical part of the testing facility consists of a load frame, two abutments, 12 hydraulic jacks, a pressure chamber above the specimen and a control room underneath. The mechanical set-up is shown in Fig. 2 and Fig. 3. The abutment for the anchorage and the abutment for the hydraulic jacks are connected together with the load frame and form a closed load path between the jacks and the abutments.

Additionally the load frame holds down the reinforced concrete cover of the pressure chamber and works as an abutment for pressure cushions sealing the cracks at the side of the specimen. Above the specimen is a steel made insulated pressure chamber, fixed by a reinforced concrete cover and bolted down to the load frame. The pressure chamber has an area of 3,28 m² and a volume of about 1 m³. The air-steam-mixture enters the pressure chamber at one side. After leaking through the specimen the outflow of condensed water and air is trapped in a control room. The amount of leakage is measured after the control room.

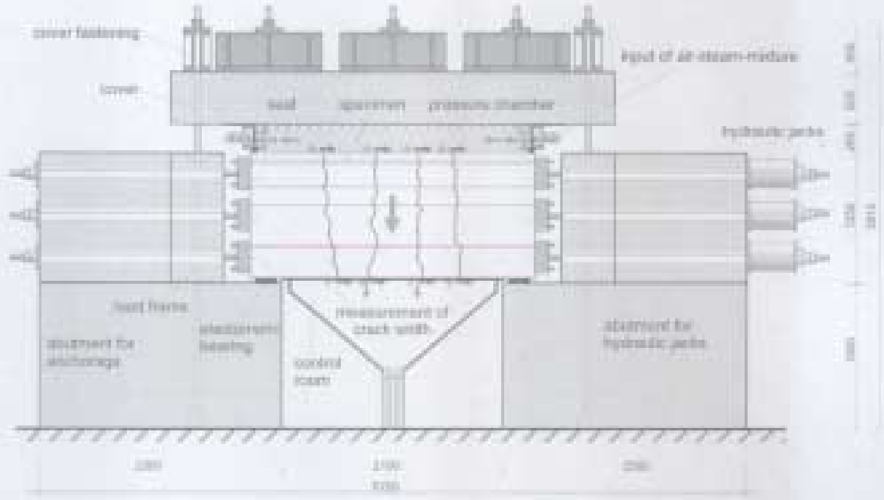

Fig. 2: Vertical cross section

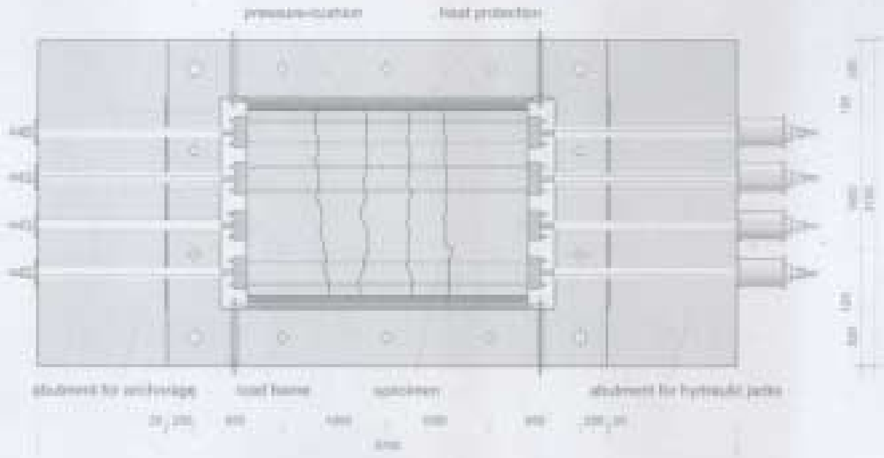

Fig. 3: Horizontal cross section

In order to ensure the desired pressure, temperature and humidity conditions and to avoid unwanted condensation, a minimum flow through the pressure chamber is established by opening an additional bypass valve opposite the mixture inlet.

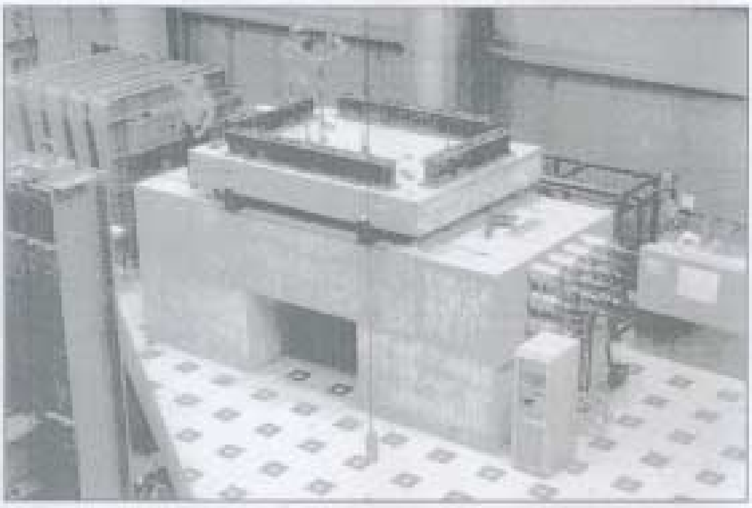

Fig. 4: Total view of the testing facility

3.2 Thermo-hydraulic set-up

Aim of the design process of the thermo-hydraulic set-up was to achieve stable air-steam mixtures matching complex, highly time dependent accidental scenarios. To fulfil the predefined accidental scenarios it is necessary to regulate the parameters temperature, partial pressure of steam and partial pressure of air. These three parameters describe the physical state completely at any time. The production principle of the air-steam-mixture is shown in Fig. 5. Unlike temperature, the parameters partial pressure air and partial pressure steam are not available for directly measurement. It is necessary to define the system state in equivalent measurable parameters. Instead of the partial pressures the total pressure and the relative humidity are measured and taken as a control parameters.

The main parts of the air-steam mixture process are the compressor, boiler, static mixer, air heater, steam super-heater and 3 pneumatic valves.

The process can be divided in the air channel, the steam channel and the air-steam-mixture channel on the inlet side of the specimen, a bypass channel on the outlet side and the control room to collect leakage. The measurement and control system of the mixture production is realized using five control loops for temperature, pressure, relative humidity and minimum flow.

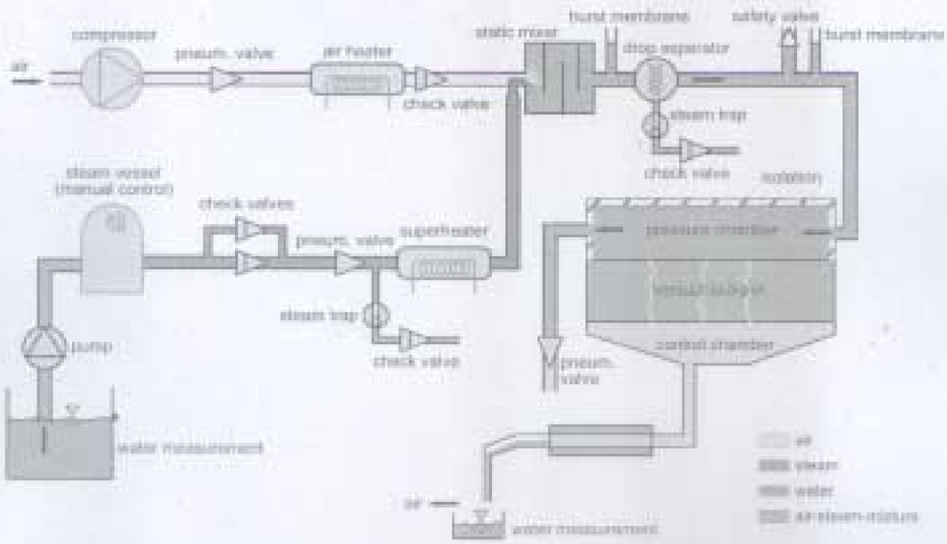

Fig. 5: Air-steam-mixture production principle

Temperature	100-162 °C	saturated steam-air mixtures
	100-250 °C	superheated steam-air mixtures
	20-250 °C	pure air
pressure	1-6.5 bar absolute	
relative humidity	1-100 %	
max. steam capacity	500 kg/h	

Tab. 1: Capabilities of the air-steam-mixture process

3.3 Specimen

The specimen are reinforced concrete slabs with dimensions of 2,7 m x 1,8 m x 1,2 m. These dimensions are based on the following boundary conditions:

- Thickness of 1,2 m is equivalent to the design wall thickness of EPR
- The maximum force available for axial tensile cracking of the specimen limits the cross section area and determined the width to 1,8 m
- To achieve an observation area of 1,8m x 2,0m a total specimen length of 2,7m is needed to allow load introduction being completed outside the observation area

Two different types of specimen with same overall dimensions have been developed. The first type has only longitudinal reinforcement. This allows a free development of the crack pattern along cross section. The reinforcement layout is shown in Fig. 6.

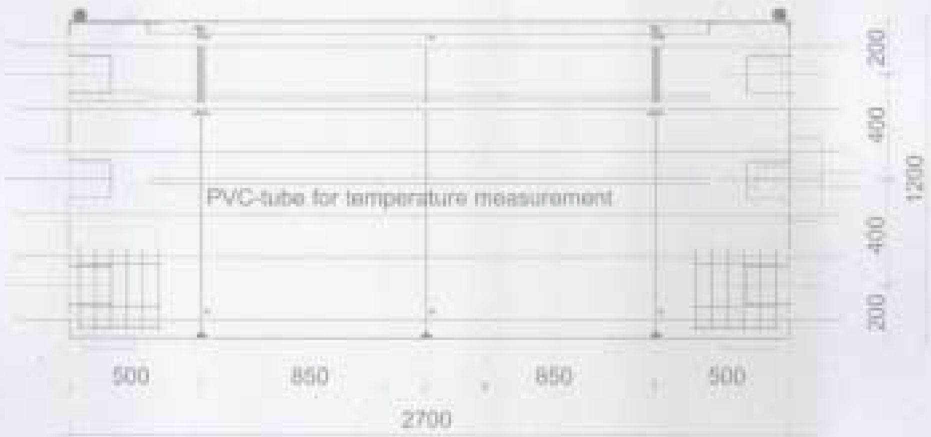

Fig. 6: Vertical section specimen type 1

The second specimen type is equipped with ducts and a surface mesh reinforcement equivalent to the EPR containment design. Fig. 7 shows a longitudinal section through the second specimen type with 2 layers of longitudinal ducts on the external side and one layer of transversal ducts above. A reinforcement mesh is located near both surfaces for crack distribution.

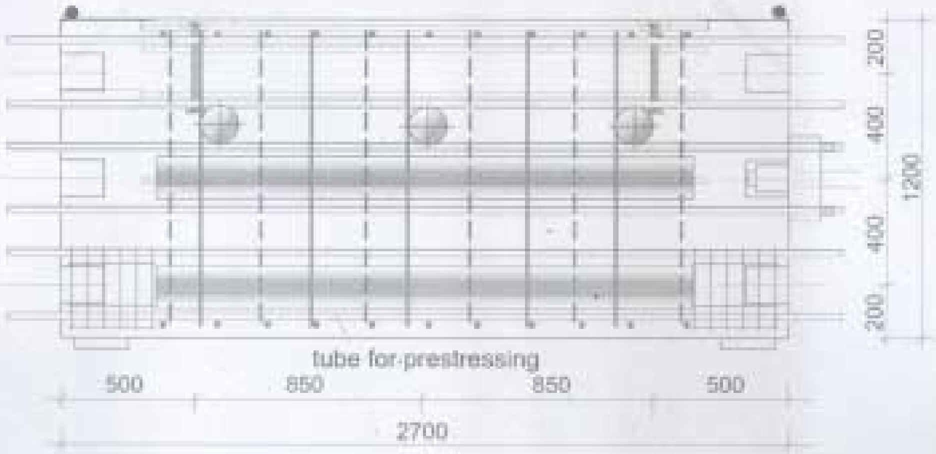

Fig. 7: Vertical section specimen type 2

The longitudinal reinforcement bars of both specimen types are used only for load introduction. The cracks are induced by applying axial tension with the hydraulic jacks. During the tests the external crack width is adjusted by changing the applied load of the jacks and can be modified independent from the pressure inside the pressure chamber.

4 Results

So far, two type 1 specimens and one type 2 specimen have been produced and tested. The first specimen served only to calibrate the air-steam-mixture process.

number of experiment	crack length [m/m²]	mean crack width at external side [mm]	crack widths during experiment [mm]		total leakage [litre H_2O]
			internal side, top (min)	external side, bottom (max)	
VK2/1	ca. 3,15	"0"	0,007	0,07	0,105
VK2/2	ca. 3,15	0,15	0,007	const. 0,15	~ 200
VK3/1	ca. 2,75	"0"	0,005	0,07	1,9
VK3/2	ca. 2,75	0,30	0,11	const. 0,30	~ 440

Tab. 2: Overview of the performed experiments

4.1 Preparation of specimen 2

The second specimen is described for reference of the established testing procedure. It is a type 2 specimen with ducts and a surface mesh reinforcement.

The cracks were induced 139 days after casting applying a maximum axial force with the hydraulic jacks of 7000 kN. The crack pattern obtained is shown in Fig. 8.

Displacement transducers were mounted after crack induction onto the top and the bottom surfaces in order to measure the crack widths during the thermo-hydraulic scenario. If the crack width on the external side varied, it was adjusted by changing the load applied by the hydraulic jacks.

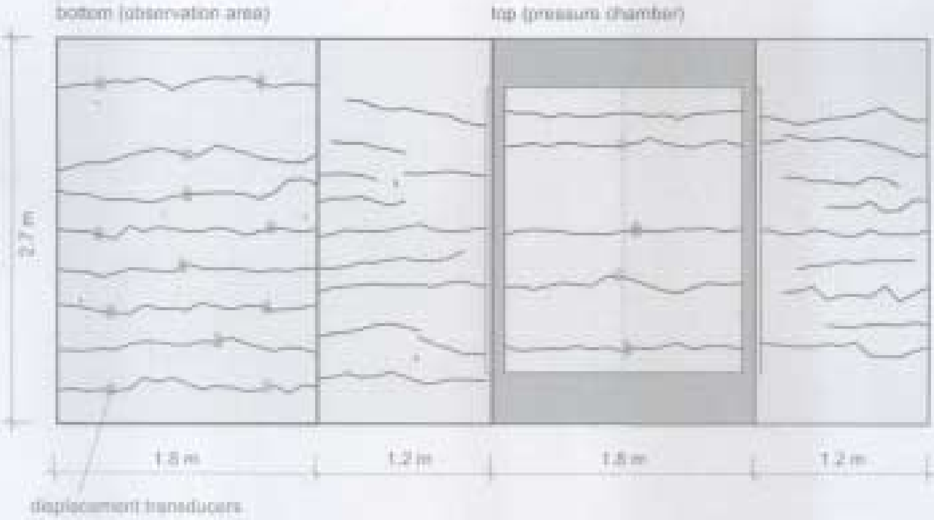

Fig. 8: Crack pattern of specimen 2

4.2 Tests with specimen 2

The first steam test VK2/1 was performed with closed cracks without axial load in order to simulate a cracked wall with remaining crack widths only. During the test with closed cracks there was hardly any leakage.

For the second steam test VK2/2 the average crack width was adjusted to 0.15 mm. The crack width at the bottom surface was kept constant during the whole 40 hour scenario by changing the applied axial load. The crack width change at the top surface side of the specimen followed the temperature inside the pressure chamber (Fig. 9).

The measured leakage rate is shown in Fig. 10. For a better comparability the leakage is based on a unit crack length of 1 m, a pressure of 1 bar and a temperature of 0 °C. The steam condensed entirely inside the cracks and no steam outflow was observed. The total amount of leakage during the whole 40 hour scenario was about 200 l of water.

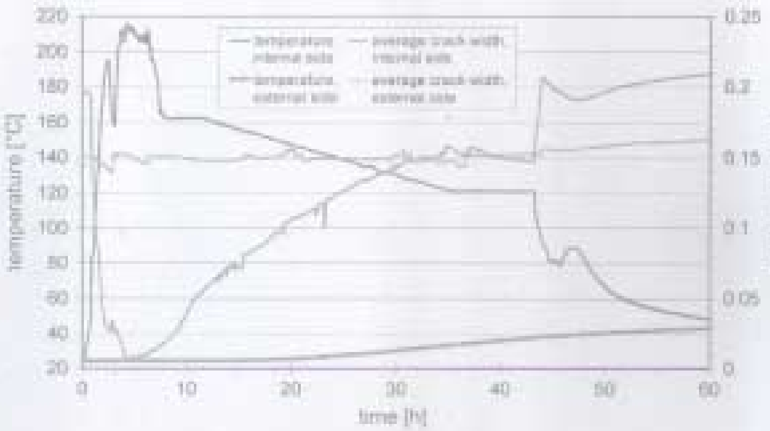

Fig. 9: Temperature and crack width specimen 2/2

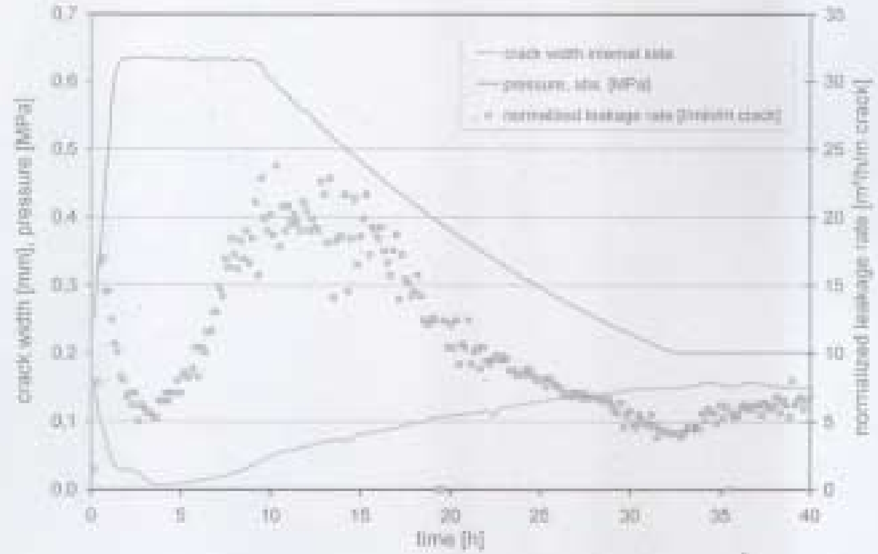

Fig. 10: Leakage of specimen 2/2

5 Conclusions

So far four 40 hour tests were performed with 2 different pre-cracked specimen types. During tests with closed cracks no measurable leakage was found. During tests with average crack widths of 0.15 mm and 0.30 mm water outflow was observed only. The steam condensed entirely inside the specimen. The test results can be used for a first estimation of the integral containment-leakage behaviour for a given crack pattern.

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The Development of a State-of-the-Art Structural Monitoring Instrumentation System for Nuclear Power Plant Concrete Structures

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Abstract

This paper describes the development of a state-of-the-art diverse monitoring system for application to existing concrete structures on Nuclear Power Plants. The system has the capability to monitor surface behaviour or, in cases where surface effects are not the most critical, sub-surface behaviour can be monitored with minor modification to the installation arrangement.

The system uses instrument clusters with both fibre optic and AC-LVDT transducers designed to monitor small structural displacements accurately. By simultaneously measuring the structural response using transducers of different types, type-based errors may be eliminated and the system reliability enhanced.

In order to determine the type of instrumentation to be used, a research project was undertaken to evaluate the performance of available equipment and the practicality of its installation. This paper describes the work carried out under the research project and the development of the system to the practical installation stage.

Introduction

Historically, emphasis has been placed on the use of instrumentation to validate design and analysis assumptions and for initial structural integrity testing and it has been normal for the first structure of a new design or series to be extensively instrumented with subsequent structures receiving less attention. Although instrumentation has been used for long term monitoring this has often been as a result of the continued operation of systems installed for commissioning and structural integrity tests, which have then been adopted for long term monitoring purposes. It is now generally accepted that the installation of structural monitoring systems at the time of construction will provide useful information for the lifetime management of nuclear power plant structures and the detection of ageing effects.

The OECD-NEA workshop on the instrumentation and monitoring of concrete structures [1] considered this in some detail. There is now a perceived need to address the ageing of concrete structures. To this end, the use and acceptance of instrumentation techniques has increased with time and more reliance is being placed on such techniques.

The workshop recommended that, while techniques are available to monitor the ageing and performance of structures and new systems are available which may be retrofitted, improvements be made in retrofitting instrumentation and in relating it to existing instrumentation. It is extremely difficult to replace instrumentation that was installed at the time of construction. Additionally, the usefulness of installed instrumentation is dependent on the accuracy and reliability of the sensors used. The usefulness of instrumentation systems has also been improved by developments with regard to computer management systems and databases.

The Current Project

British Energy identified a potential need to be able to measure small changes in structural displacement in reinforced and, particularly, prestressed concrete structures using retro-fitted instrumentation. This paper describes the development of a state-of-the-art diverse monitoring system for application to existing concrete structures on Nuclear Power Plants. The system has the capability to monitor surface behaviour or, in cases where surface effects are undesirable, sub-surface behaviour with minor modification to the installation arrangement. The specification required the investigation of an instrumentation package chosen to include established and innovative technologies, and instrumentation diversity and redundancy.

This paper describes the development of a state-of-the-art diverse monitoring system for application to existing concrete structures on Nuclear Power Plants. The system has the capability to monitor surface behaviour or, in cases where surface effects are not the most critical, sub-surface behaviour can be monitoredwith minor modification to the installation arrangement.

Programme of Investigations

The project was initiated in April 2000 and was undertaken at the University of Strathclyde in three stages with project milestones as follows:

<u>Stage 1</u>

> Investigation and definition of the monitoring requirements. Preparation of proposals for the instrumentation required. This work was completed by mid - May 2000.

<u>Stage 2</u>

> Laboratory investigations at the University of Strathclyde and preparation of an Interim Report on the outcome of the investigations. This work was completed in late-December 2000 and an Interim Report was issued in January 2001. A recommendation of the Stage 2 work was that an extension to Stage 2 be granted to investigate the installation problems associated with the use of Automatic Crackmeters penetrating down below the surface of the concrete some 50 to 100 mm.

<u>Stage 2 (extension)</u>

> The Stage 2 (extension) was undertaken during April and May 2001. This work involved developing two types of Automatic Crackmeters, installing them in specially constructed reinforced concrete beams and testing their measurement efficiency.

<u>Stage 3</u>

> The Final Report set out the performance and accuracy of the instrumentation and included recommendations for the instrumentation system to be used on NPPs.

Results

In Stage 1 of this project the normal operational conditions on Nuclear Power Plants were assessed and the most likely types of instrumentation to effectively undertake the monitoring were identified. In Stages 2 and 2(extension), evaluation of these instrument types and the methods of installing these were investigated (Plate 1). Stage 3 involved the preparation and submission of the Final Report.

The Stage 2 investigations were designed to measure vertical distortion, in-plane deformations / strains and temperatures on a centrally loaded, simply supported 3m x 1m x 100mm lightly reinforced concrete beam. The instrumentation package was chosen to include established and innovative technologies, instrumentation diversity and redundancy. The range of instrumentation employed is shown in Table 1. The data on the claimed resolution, frequency of measurement, mounting and development stage of each instrument are shown in Table 2. The details of the installation methodology, tuning/ commissioning and data collection technique associated with these instruments are given in Table 3. The layout of the instrumentation employed on the beam is given in Fig.1 and illustrated in Plate 1. In addition to these instruments, ambient air temperature was measured using PRT's above, below and at the side of the beam. The objectives and details of the testing regime are shown in Table 4. Incremental vertical distortion test data was obtained from two separate tests, involving four stages of short-term loading with a single stage of unloading. In addition, tests were conducted to compare the analogue and digital outputs from the fibre optic (Fabry-Pérot interferometer) instruments. The reason for undertaking these tests was to prove that analogue signals could be obtained from the fibre optic instruments, as this had not been established previously. The main findings of the testing undertaken under Stage 2 were:

- Vertical and in-plane deformations could be measured by the instrumentation employed to an accuracy of +/- 0.01mm, (+/- 10 microns).

- In-plane strains could be measured to an accuracy of +/- 2 microstrain.

- Temperature could be measured to an accuracy of +/- 0.1 o C.

- AC- LVDT's and fibre optic deformation gauges were the preferred instruments to measure deformations.

- Whilst highly accurate and possibly extremely useful in the future, some further development work was required on the TMS laser system before it could be employed for structural monitoring.

- VW strain gauges and fibre optic strain gauges were the preferred instruments for directly measuring strain.

- Digital outputs from the instrumentation signal conditioners were preferred as they allowed much greater flexibility in data collection and networking. However, it was shown that analogue outputs may be obtained with similar accuracies to digital outputs.

- Instrumentation should not be surface mounted to avoid superficial cracking and de-bonding problems.

- The use of Automatic Crackmeters should be considered to measure in-plane deformations / strains. The pillars and instrument fixings should be manufactured from a temperature compensated alloy such as Invar (Fe/Ni alloy) to reduce potential temperature effects.

- The temperature variations at the location of each instrument should be measured and temperature compensations / corrections applied.

It was recognised that potential installation problems have a major influence on the choice of instrumentation and consequently on the measurements to be made. Indeed, it was realized that the need to create a horizontal reference plane against which out of plane distortion could be measured, was not practical at many NPP locations. Thus measurement of out of plane distortion was not recommended and the choice of instrumentation was limited to those instruments that could measure in-plane deformations and strains.

It was identified that superficial cracking of the concrete, free surface effects and de-bonding of instruments on the surface of the concrete could result in mis-leading data being collected. Thus it was agreed that the instrumentation packages to be used would require to be capable of measurement at a sub-surface level. It was recommended that sub-surface deformations and strains should be measured using both conventional electrical and optical fibre instruments. It was decided that measurements should be made at a depth of 50 to 100mm below the top surface of the concrete to avoid surface effects influencing the data collected.

The study recommended that sub-surface deformations should be measured using Automatic Crackmeters consisting of pillars drilled and fixed into the concrete with the change in distance between pairs of pillars measured using external AC electrical LVDT’s and fibre optic deformation gauges. The types of instruments should be similar to those used in the Stage 2 investigations, however, they should be individually temperature monitored to check for local temperature variations on the top surface of the concrete.

Sub-surface strains should be measured using temperature compensated VW strain gauges and optical fibre strain gauges fixed within Automatic Crackmeters.

Ambient air temperatures immediately above the concrete surface and concrete temperatures at 25 to 50mm below the surface should be measured.

Although the correlation between analogue and digital outputs from the signal conditioners was good, it was recommended that digital outputs should be the preferred form as:

- Links between instruments and signal conditioners were simpler.
- Site specific problems of mutual interference and external interference were avoided.
- Networking of instrumentation using RS422 / RS485 links was possible.

The Stage 2 (extension) investigations were designed to measure in-plane deformations / strains and temperatures on a centrally loaded, simply supported 3m x 1m x 150mm reinforced concrete beam. The instrumentation packages used were designed to provide examples of possible combinations of established and innovative technologies with instrumentation diversity and redundancy.

The two instrumentation packages used were:

- A four instrument package, consisting of a VW strain gauge (subsurface), a fibre optic strain gauge (subsurface), an AC LVDT (external) and a fibre optic deformation gauge (external), and,

- A two instrument package, consisting of an AC LVDT (external) and a fibre optic deformation gauge (external).

The arrangements of the instrumentation packages are shown in Plates 2 and 3 and the layout of the instrumentation was as shown in Figure 2.

Test results were obtained from a nine stage loading and single stage unloading test undertaken over seven days. These indicated that for lightly loaded and unloaded conditions there were identifiable variations of the outputs, (“instrumentation noise”), of +/- 0.002 mm (+/- 2 microns) and +/- 1 microstrain. In addition, creep of the concrete could be identified over time. Thus the main findings of the testing undertaken were:

- The two types of instrumentation packages could be installed on the beam efficiently.
- Owing to the use of subsurface instruments, the installation of the four-instrument package involved a great deal more drilling than the two-instrument package where the instruments are external to the concrete surface and drilling is only required for the fixing posts..
- The detailed design of the instrument packages and the adjustment /calibration of the instruments required great care to ensure that the instruments were not damaged during installation and operated efficiently in place.
- To calculate the strains in the concrete from the LVDTs and fibre optic deformation gauge output data, the gauge length was taken as the distance centre to centre between the vertical pillars.
- To calculate the strains in the concrete from the VW and fibre optic strain gauge output data, the gauge length was taken as the distance centre to centre between the vertical pillars and the strains recalculated on this basis. All data were efficiently recorded using digital outputs, (i.e. using the recommended / preferred method of Stage 2).
- In plane deformations could be measured by the LVDTs and fibre optic deformation gauges to an accuracy of +/- 0.01 mm, (+/- 10 microns).
- Background noise in the LVDTs and fibre optic deformation gauges was at a level of +/- 0.002 mm, (+/- 2 microns).
- In plane strains could be measured by the VW and fibre optic strain gauges to an accuracy of +/- 2 microstrain.
- Background noise in the VW and fibre optic strain gauges was at a level of +/- 1 microstrain.
- Creep occurring in the concrete over time was reflected in a gradual change in the outputs from the instrumentation.

Final Recommendations

Instrument packages consisting of a VW strain gauge, a fibre optic strain gauge, an AC-LVDT and a fibre optic deformation gauge should be used. The strain gauges should be temperature compensated and be embedded into the concrete to a depth of 50 to 100mm using a shrinkage compensated cementitious grout and an epoxy “jacket” to isolate the instruments from local effects. The AC-LVDT and fibre optic deformation gauge should be mounted above the surface, PRTs should be embedded at 25 and 50 mm to measure concrete temperatures. PRTs should be mounted next to the bodies of the AC-LVDTs and fibre optic deformation gauges to measure local temperatures and so allow temperature corrections to be made.

It should be noted that the instrumentation package gauge length must be precisely determined to allow the effective gauge length of the strain gauges to be calculated and so their strain outputs corrected. The instrumentation package gauge length is used to calculate strains from the LVDT and deformation gauge outputs.

An alternative reduced package may be used where space is restricted. This should consist of an AC electrical LVDT and a fibre optic deformation gauge placed one above the other between two vertical pillars. The instruments should be placed just above the top of the concrete surface. The vertical pillars should be embedded into the concrete to a depth of 50 to 100mm. PRTs should be embedded at 25 and 50 mm to measure in concrete temperatures and a PRT should be mounted next to the bodies of the AC-LVDT and fibre optic deformation gauge to measure local temperatures and so allow temperature corrections to be made. It should be noted that the instrumentation package gauge length must be precisely determined to allow calculation of the strains from the AC-LVDT and deformation gauge outputs.

Specified types of VW and fibre optic strain gauges, AC-LVDTs, fibre optic deformation gauges and PRTs which have been proven to be accurate and reliable for the purpose of monitoring deformations and strains at the very low levels required should be employed in the instrumentation packages.

The installation method should be very carefully specified to provide holes in the concrete into which the instrumentation packages will fit. Thus close tolerances should be specified on hole sizes, spacing and orientation.

The instrumentation packages should be designed and constructed to close tolerances to ensure:

- The instruments are not damaged during fabrication.
- The instruments can be calibrated before installation and, at least for the AC-LVDTs and deformation gauges, after installation.
- The instruments can be easily set to any specified point in their range prior to installation.

The grouting of the instrumentation packages into the concrete should be very carefully specified and controlled. (The size of the holes in relation to the size of the vertical posts must take account of the grouting process).

The instrument package and associated cabling should be very carefully protected from damage after installation.

Calibration of the instrumentation should be very precisely undertaken at a range of movement and range of temperatures appropriate to the operational conditions expected in service.

The output from the instrument signal conditioners should be digital. This provides much greater flexibility in data collection, allows networking and avoids site-specific interference problems. The use of the analogue signals is possible if other factors preclude the use of digital signals. The signal conditioners to be used should be those specifically designed by the manufacturers of the instrumentation for the various instruments. They should be connected to the instruments with the appropriate cable type and length. The cabling should have the minimum possible number of joints/connections.

The conditioned digital signals should be collected and processed in a PC using specialist data collection and processing software with the capacity to collect, process and store data at an appropriate rate for a specified period. The PC thus requires to possess sufficient processing and storage capacity.

“Instrumentation noise” and evidence of creep in the concrete over time, are likely to be a feature of the outputs from all the instruments. Thus it will be necessary to monitor structures in order to identify their “normal engineering behaviour”.

In-service Performance

A number of the two-instrument external instrument packages have been installed on actual NPP structures and these are giving very good results with levels of noise and accuracy comparable with the laboratory tests.

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TABLE 1 - INSTRUMENTATION EMPLOYED

GROUP	TYPE	MANUFACTURER	SPECIFICATION
VERTICAL DISTORTION	MECHANICAL DIAL GAUGE	JOHN BULL	MANUAL READING ±0.002 mm
	ELECTRICAL DISPLACEMENT	R.D.P. GROUP	RANGE ± 2.5 mm LINEARITY ± 0.1%
	FIBRE OPTIC DISPLACEMENT	FISO TECHNOLOGIES INC	RANGE 25 mm RESOLUTION 0.2 MICRONS ACCURACY 0.08% OF FULL SCALE
	TELBIP LASER	TUNNEL MANAGEMENT SYSTEMS	RESOLUTION ± 0.01 mm RANGE ± 12.5 mm WITH MIN. MEASUREMENT FREQUANCY OF 20 SECS
IN-PLANE DEFORMATION/ STRAIN	MECHANICAL DEMEC	JOHN BULL	MANUAL READING ± 0.0025 mm
	STANDARD STRAIN GAUGE	SHOWA MEASURING INSTRUMENT CO LTD	½ BRIDGE (120Ω) CONDITIONED TO BALANCED FULL BRIDGE
	V.W. STRAIN GAUGE	GEOKON (STRAINSTALL)	ACCURACY 2% OF FSR RESOLUTION 0.1 MICROSTRAIN RANGE 2500 MICROSTRAIN SENSITIVITY 0.5 TO 1 MICROSTRAIN
	ELECTRICAL STRAIN GAUGE	(AS ABOVE)	½ BRIDGE (350Ω) STRAIN LIMIT 5% OF GAUGE LENGTH
	FIBRE OPTIC STRAIN GAUGE	FISO TECHNOLOGIES INC	RANGE 10,000 MICRO STRAIN RESOLUTION 0.1 MICRO STRAIN
TEMPERATURE	PRT	PEAK SENSORS LTD	100 OHM AT 0°C TEMP RANGE -70°C TO 500°C ACCURACY 0.1°C
	FIBRE OPTIC	FISO TECHNOLOGIES INC	ACCURACY 0.3°C RANGE -50 + 250°C
	V.W. COMPENSATORS	GEOKON (STRAINSTALL)	RANGE - 20 TO + 80°C INCORPORATED IN VW GAUGE

TABLE 2 - DETAILS OF INSTRUMENTATION

GROUP	TYPE	CLAIMED RESOLUTION	FREQUENCY OF MEASUREMENT	MOUNTING	DEVELOPMENT STAGE
VERTICAL DISTORTION	MECHANICAL DIAL GAUGE	± 0.002 mm	5 MINS	MECHANICALLY MOUNTED ON CROSS BEAM	STANDARD PRODUCTION
	ELECTRICAL DISPLACEMENT (LVDT)	± 0.005 mm	5MINS (15 MIN AVG)	MECHANICALLY MOUNTED ON CROSS BEAM	STANDARD PRODUCTION (HIGH SPEC. LINEARITY)
	FIBRE OPTIC DISPLACEMENT (LVDT)	± 0.002 mm	"	MECHANICALLY MOUNTED ON CROSS BEAM	STANDARD PRODUCTION (NEW TECHNOLOGY)
	TELBIP LASER and CAMERAS	± 0.01 mm	4 HZ (AVG 1 MIN)	MECHANICALLY MOUNTED ALIGNED LASER and CAMERAS	IN DEVELOPMENT STAGE
IN-PLANE DEFORMATION/ STRAIN	MECHANICAL DEMEC	± 0.002 mm	MANUAL	DEMEC STUDS FITTED TO UNDERSIDE OF BEAM	STANDARD PRODUCTION
	STANDARD STRAIN GAUGE	± 0.1 MICROSTRAIN	5 MINS (15 MIN Average)	EPOXYED TO UNDERSIDE OF BEAM	STANDARD PRODUCTION
	V.W. STRAIN GAUGE	± 0.1 MICROSTRAIN	5 MINS	SURFACE MOUNTED TO UNDERSIDE OF BEAM	STANDARD PRODUCTION
	ELECTRICAL STRAIN GAUGE	± 0.1 MICROSTRAIN	5 MINS	SURFACE MOUNTED TO UNDERSIDE OF BEAM	STANDARD PRODUCTION
	FIBRE OPTIC STRAIN GAUGE	± 0.1 MICROSTRAIN	5 MINS (15 MIN Average)	EMBEDDED at CENTRE at 25 mm	STANDARD PRODUCTION (but NEW TECHNOLOGY)
TEMPERATURE	PRT	± 0.01°C	5 MINS (15 MIN Average)	EMBEDDED at 25 mm EMBEDDED at 50 mm SURFACE MOUNTED (TOP BOTTOM & EDGE)	STANDARD PRODUCTION
	FIBRE OPTIC	± 0.01°C	(AS ABOVE)	EMBEDDED at 25mm at CENTRE	STANDARD PRODUCATION (but NEW TECHNOLOGY)
	V.W. COMPENSATORS	± 0.01°C	5 MINS	INCORPORATED IN VW STRAIN GAUGE	STANDARD PRODUCTION

TABLE 3 - INSTALLATION OF INSTRUMENTATION

GROUP	TYPE	METHODOLOGY	TUNING/COMMISSIONING	DATA COLLECTION
VERTICAL DISTORTION	MECHANICAL DIAL GAUGE	MECHANICALLY MOUNTED BY ADJUSTABLE BRACKETS PROVIDED WITH STRIKER PLATE	NONE	MANUAL READING
	ELECTRICAL DISPLACEMENT (LVDT)	(AS ABOVE)	BENCH TESTED TO CHECK LINEARITY SET TO CENTRE ZERO (ELECTRICALLY) SET RANGE V SPAN (ELECTRICALLY)	ANALOUGE SIGNAL CONDITIONED AND TRANSFERRED TO PC DIGITALLY VIA RS232
	FIBRE OPTIC DISPLACEMENT (LVDT)	(AS ABOVE)	BENCH TESTED TO CHECK LINEARITY	(AS ABOVE)
	TELBIP LASER and CAMERAS	LASER UNIT INDEPENDENTLY MOUNTED LASER PENTAPRISMS (FREE) TELBIP TRANSDUCERS MOUNTED ON SLAB	LASER POSITIONED with PENTAPRISMS and CAMERAS ACCURATELY LINED IN AND FIXED TO SLAB	TELBIP OUTPUTS (ANALOGUE) FROM THE TMS CENTRAL MANAGEMENT UNIT CONDITIONED OUTPUT FED TO PC VIA RS232
IN-PLANE DEFORMATION/ STRAIN	MECHANICAL DEMEC	DEMEC STUDS EXPOXYED TO UNDERSIDE OF BEAM	(NONE)	MANUAL
	STANDARD STRAIN GAUGE	FOIL STRAIN GAUGE EPOXYED TO UNDERSIDE OF BEAM	OUTPUT CONFIGURED. SCALE + RANGE CONDITIONED IN RDP 600 UNIT	GAUGE OUTPUT (ANALOGUE) FED TO RDP 600 UNIT + TRANSFERRED VIA RS232 TO PC
	V.W. STRAIN GAUGE	FITTED BY STRAINSTALL ENGINEERS JOB NO. 84924	COMMISSIONED BY STRAINSTALL ENGINEERS	SIGNAL DELIVERED VIA GEO-LOGGER and DATA TRANSFERRED TO PC
	ELECTRICAL STRAIN GAUGE	(AS ABOVE)	(AS ABOVE)	(AS ABOVE)
	FIBRE OPTIC STRAIN GAUGE	EMBEDDED IN BEAM at 25mm on CENTRE LINE	(NONE)	OUTPUT TRANSFERRED TO PC VIA DISPLAY/CONDITIONING UNIT
TEMPERATURE	PRT	PRT'S EMBEDDED VERTICALLY IN BEAM AT DEPTHS. 25mm & 50mm PRT'S PLACED TOP BOTTOM & SIDE OF BEAM	RANGE, SPAN and CORRECTION FACTOR SET WITH KEITHLY 2700	PRT OUTPUTS (ANALOGUE) SCANNED & LOGGED BY KEITHLY 2700 TRANSFERRED VIA GPIB IEEE TO PC
	FIBRE OPTIC	FIBRE OPTIC PRT EMBEDDED at 25mm ADJACENT TO FIBRE OPTIC STRAIN GAUGE	(NONE)	OUTPUT FED TO FISO DISPLAY/CONDITIONING UNIT & VIA RS232 TO PC
	V.W. COMPENSATOR	FITTED BY STRAINSTALL (PART OF VW STRAIN GAUGE)	COMMISSIONED BY STRAINSTALL	SIGNAL LOGGED and CONDITIONED BY DATA TAKER and TRANFERRED TO PC

TABLE 4 - TESTING REGIME

OBJECTIVE: *TO ASSESS BY INCREMENTAL LOADING AND UNLOADING THE RESPONSE OF VARIOUS TYPES OF INSTRUMENTATION ATTACHED TO A TEST SLAB OVER VARIOUS TIME PERIODS.*	
METHOD OF LOADING:	INCREMENTAL LOADING AND UNLOADING OF TEST SLAB WHILST LOGGING OUTPUTS FROM ALL INSTRUMENTATION. THIS IS TO PROVIDE COMPARISONS OF INSTRUMENTATION ACCURACY. LOADING BY STANDARD LEAD WEIGHTS EACH 26 kg (APPROX).
SHORT-TERM LOADING TESTS:	4 STAGE LOADING AND ONE STAGE UNLOADING OVER APPROXIMATELY 1 HOUR EACH LOAD INCREMENT OF APPROXIMATELY 52 kg.
LONG-TERM LOADING TESTS:	SUSTAINED LOADING OF APPROXIMATELY 100 Kg FOR 4 DAYS

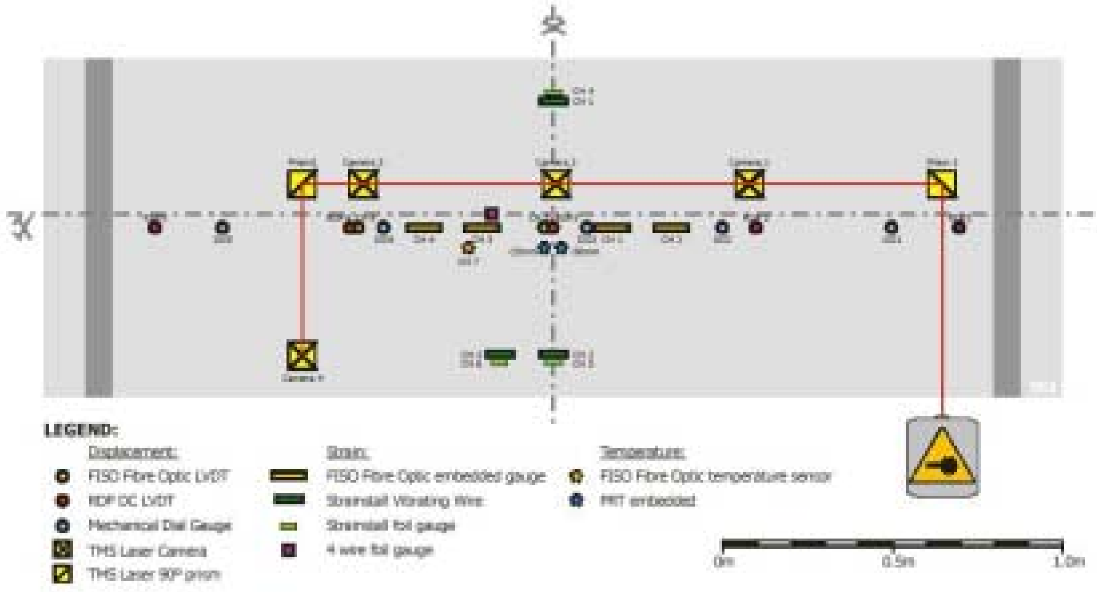


Figure 1 Layout of the instrumentation employed on beam 1

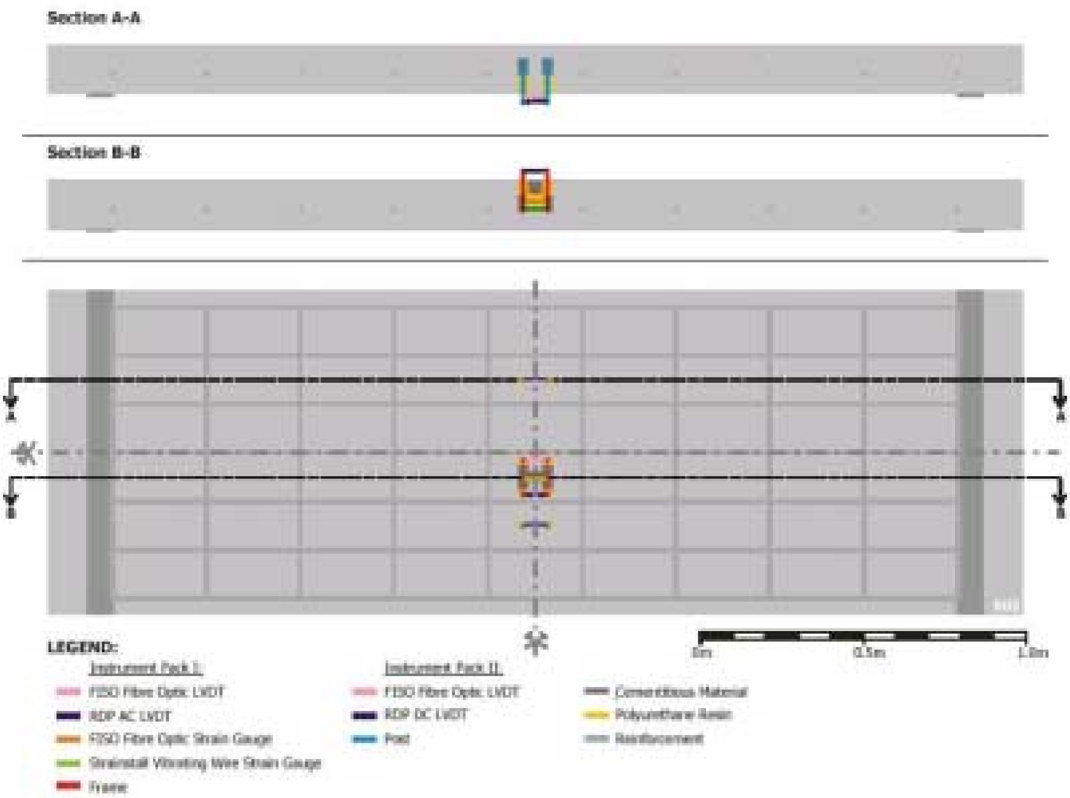
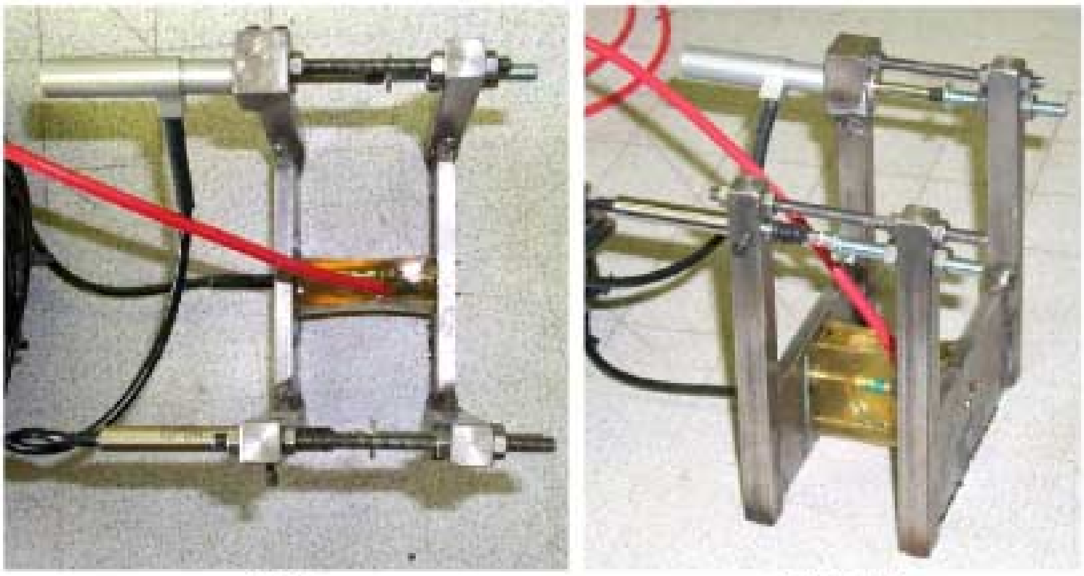


Figure 2 Layout of the instrumentation employed on beam for stage 2 (extension)

Plate 1 General layout and set-up of the instrumentation employed on beam for stage 2



(a) Plan

(b) Isometric

Plate 2 Transducer arrangement for the four instrument package

Plate 3 Transducer arrangement for the two instrument package

Ageing and Static Reliability of Concrete Structures under Temperature and Mechanical Loading

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Abstract

The contribution presents some aspects of the static reliability of concrete structures under temperature effects and under mechanical loading. The mathematical model of a load-bearing concrete structure was performed using the FEM method. The temperature field and static stress that generated states of stress were taken into account. A brief description of some aspects of evaluation of the reliability within the primary circuit concrete structures is stated. The knowledge of actual physical and mechanical characteristics and chemical composition of concrete were necessary for obtaining correct results of numerical analysis.

Studied problems were divided into a number of fields and worked out in details:

1. Verification of contemporary physical and mechanical characteristics of concrete (input parameters of the FEM models).
2. Checking of the concrete microstructure and verification of the grade and kind of possible microstructure changes.
3. Experimental verification of the boundary conditions from point of view of the temperature field and the radiation stress.
4. Setting up a mathematical model of the structure for an examination of the interaction of temperature and static stresses (finite element method, software ANSYS) in two alternatives:
   a) Macro-model representing the essential part of concrete structures in the proximity of the reactor,
   b) Model of extremely stressed parts of the concrete structure (a part of the macro-model).

1. Introduction

The nuclear power plant Dukovany (EDU) has been in use under reliable operation for more than 15 years. Within the programme Harmonisation whose aim is to ensure high-quality and safe operation of this nuclear power plant at least until the year 2025 so for this reason there has been disposed a great number of tasks concerning various areas. Actual static reliability of concrete structures is besides others one of the problem of the power plant building part. Considering the fact that concrete structures have to be functional, safe and reliable for substantial time period after the operation of the nuclear power plant there have been worked out large-scale procedures and models for evaluation of particular - for the nuclear power plant reliability – dominantly important structural parts. The problems of evaluation of the concrete structure reliability are solved from the experimental and theoretical point of view.

2. Experimental part

Concretes of the load-bearing structures of the primary circuit are effected not only by the mechanical stress but also by the moisture stress. Moreover, these concrete structures are also subjected to long-period influence of high temperatures during their lifetime. Due to the present temperature and moisture stresses, new crystalline formations inside the concrete structure can develop (e.g. 11 A-tobermorit) and so one part of evaluation works was focused on observations of the actual physically- mechanical characteristics of concrete. The experimental part of the work was especially concentrated on the following areas:

- Determination of the distribution of temperature field on the surface of the concrete structures which serves as the boundary conditions for temperature field (calculated by the means of mathematical models)
- Determination of the distribution of moisture field inside the concrete structures. It is used as the constraint condition for moisture field determination by the means of mathematical model
- Determination of the actual physically-mechanical characteristics
- Determination of different degradation effects on the concrete structure of the primary circuit.

2.1. Measurements in situ and laboratory tests

There were carried out

- Measurements of concrete moisture (by gravimetric and by neutron method),
- Determination of the physically mechanical characteristics of concrete by laboratory tests on samples obtained by drilling (volume mass, modulus of elasticity, stress-strain diagram under the temperature stress, characteristics of temperature and moisture expansivity, determination of the content of soluble boron salt in concrete etc.),
- Evaluation of radionuclide level activity of concrete,
- Measurements of thickness within the non-hermetic internal protective lining in the zones of the moisture content measurement of concrete by the neutron method.

Concrete samples were drilled from predefined areas and according agreed schedule within the period 1997-2001 (diameter of the samples was 100 mm). The other samples were fragments of the external surface of concrete under the protective lining used for determination of the concrete structure moisture.

The range of measured moisture values within the observed period was roughly the same. However, significant differences were among the moisture values measured on the same areas. This proved considerable changes of the moisture conditions - Fig. 1. It is possible to state that the moisture migration inside concrete occurs during the time period. From more measurements carried out in the identical areas within one shutdown it is evident that the moisture varies in time dependency.

Following physically mechanical concrete characteristics of the primary circuit were found at the analyses

- Volume mass (density, thickness) under natural conditions ρ 2192 to 2330 kg/m^3
- Compression strength f_c 48.1 to 60.8 MPa,
- Modulus of elasticity E_c 25.2 to 33.4 GPa.

Furthermore, the temperature and moisture expansivity characteristics were measured and the stress-strain diagrams of concrete were verified (including the decreasing branch) under the temperature stress.

Considering the fact that there were prescriptions and records available concerning the evident concrete tests within the time of the nuclear power plant construction, it was possible to compare the origin physically mechanical characteristics with the ones after 15 years operation. No significant differences were found.

2.2. Conclusions of the experimental part

From the experimental tests of the steel and concrete samples and from the measurement in situ it is evident that

- temperatures of the concrete structure exceed 100^{o} C in some areas,
- the migration of moisture inside concrete demonstrates itself within the time of shut-down, which was found out by comparison of the moisture in the identical areas of the RC load-bearing structure at the block 2,
- amount of boron was found in concrete by physically chemical tests,
- in the course of the nuclear power plant operation there have not appeared any substantial physically-mechanical characteristic changes of concrete within the observed structures of the primary circuit,
- the incidence of pitted corrosion was found in the samples of non-hermetic protective lining (it does not have any static importance but it only forms the protection of concrete against the contamination at purification during the shut-down),
- the appearance of artificial radio-nuclides in the samples of concrete taken from the structure of the primary circuit was found. The radionuclide content in samples is so low that it does not influence the physically mechanical properties. Also from the point of view of the State Supervision of Nuclear Safety (SÚJB) classification there is not dealt with any emitter,
- the appearance of CSH-gels and 11-A-tobermorit in the samples of concrete that only demonstrates the present existence of the increased moisture and temperature at the time of its origin.

3. Modelling of the structure behaviour

Consistent with the observation of the structure behaviour of all four blocks of the nuclear power plant it was stated that

- all EDU blocks can be considered to be identically thermally stressed and that is why it is not necessary to distinguish the temperature stresses of the individual blocks. The differences of temperature about ±5 C in the extremely stressed areas cannot be considered as substantial,
- in some areas of the concrete structure the temperature of 50^{o}C is exceeded in long terms. It is the temperature that causes the decrease of the concrete strength in accordance with the standard •SN 73 1201-86,
- locally variable moisture was found in the concrete structures close to the reactor.

3.1. Types of loading

3.1.1. Radiation loading

The detailed analysis of the radiation influence on the concrete structures reliability was carried out - detailed theoretical summary can be found in [4], [5]. According to the American standard ANSI/ANS-6.4-1985 [6] the radiation influence on the primary shielding is minimal on the condition that the density of the temperature flow of energy does not exceed 10^{10} MeV . cm^{-2} . s^{-1}= 16 W m^2. According to our carried out calculations, the density of the temperature flow of energy at the primary concrete shielding entry is 714 times lower than the value considered by the USA standard. The literature states that the radiation flow of one mW cm^{-2} results into the temperature increase of concrete by approximately 1.5 C. On this assumption (and at the validity of the linear dependence between the temperature and energy of the falling down radiation), the temperature increase on the surface of the primary concrete shielding of the reactor would be 0.0036^o C.

On the base of these calculations it is possible to neglect the influence of the internal sources of the temperature inside concrete initiated by the radiation on the temperature field values when it is solved by the FEM method.

3.1.2. Moisture loading

Considering the fact of random moisture migration inside the concrete structure, the influence of the moisture expansivity on the state of stress of the structure was neglected at simplifying modelling. Up today, there have not been available reliable time-dependent changes of the moisture during the tests.

3.1.3. Other types of loading

The limiting importance for the state of stress evaluation, function reliability and durability of the concrete structure has the distribution the temperature field inside the concrete structure and by this generated the state of stress (of course, in the co-action with the mechanical, radiation and moisture stress). That is why we solved the problems

- relating with the specification of a mathematical model of the parts of the primary circuit structures, which are based on:
  1. definition of the boundary conditions of temperature field within the mathematical model of the primary circuit structures,
  2. theoretical analysis of the determination of the concrete structure referential temperature. The aim was to formulate the theory needed and to define the referential temperature field within the solved structures in dependence on their thickness,
  3. final solution of the problem considering the influence of random temperature sources inside concrete initiated by absorption of neutron radiation and by gamma emission,
  4. complementation of some missing data relating to geometry, material characteristics and the composition of some structural parts.
- application of the specified model for solution of the particular situation defined after the discussion with the nuclear power plant workers,
- evaluation of the actual static reliability of the concrete structures within the above stated situations.

3.2. Model of the Structure

Based on the experimentally obtained data (the boundary conditions of the temperature field), following amendments on the mathematical model of the primary circuit structures were carried out taking into account the comments issuing from the discussion with some EDU departments

- Geometry improvements of the modelled structure (more exact model).
- Physically-mechanical characteristics improvements of some materials in accordance with results of the carried out experimental tests.
- Specification of the boundary conditions of the temperature field according to the measured results (non-contact thermometers, thermo-vision, standard and non-standard measurements).
-

Following calculations were carried out:

- calculation of the temperature field distribution,
- calculation of the state of stress generated by the temperature field in the steady state,
- calculation of the mechanical state of stress under the static load,
- calculation of the total state of stress (under the temperature and static load).
-

Following design states were taken into account:

- STANDARD: the standard operating situation that corresponds to the steady (time-independent) behaviour of the concrete structures during testing procedures when the influence of the reactor starting-up operation is not substantial (stationary problem). This situation is defined by

  - the boundary conditions of the temperatures measured (non-contact thermo-meters,
  - thermo-vision, standard and non-standard measurements performed on the modelled part of the structure),
  - the dead load, the technology load (machinery parts loading the structures) and by the boron content of a reservoir,
  - consideration of the creep influence on the state of stress generated by the temperature stress (the temperature loading is assumed as the long-terms one).
  - LPT 30: the hypothetical operating situation that corresponds to the steady behaviour of the concrete structures during the testing procedures and that is defined by
  - the dead load, the technology load (machinery parts loading the structure) and by the boron content of a reservoir,
  - final definition of the temperature boundary conditions that correspond to the temperature field increasing values on the measured ones. There is assumed: the temperature increase in the reactor zone by 300 C, zero temperature increase on the boundary of the solved situation (solved part) and the linear course of the temperature increment among the boundary values defined by above mentioned two items,
  - the consideration of the creep influence on the state of stress generated by the temperature loading.
  - KPT 30 : the hypothetical operating situation that corresponds to the steady behaviour of the concrete structures during the testing procedures and that is defined by:
  - the dead load, the technology load (machinery parts loading the structure and by the boron content of a reservoir,
  - final definition of the temperature boundary conditions that correspond to the temperature field increasing values on the measured ones. It is assumed: the temperature increase in the reactor zone by 300 C, zero temperature increase on the boundary of the solved part and the quadratic approximation course of the temperature fields of the concrete structures

- KPT 30 KR: the hypothetical operating situation that corresponds to the steady of behaviour of the concrete structures during the testing procedures and that is defined as the same as the situation KPT 30 with the only difference when the short-time temperature stress is assumed (e.i. the creep influence on the state of stress generated by the temperature stress is neglected).
- LPT 30 KR: the hypothetical operating situation similar to LPT 30. The short-time temperature loading is assumed.

  For details see [1], [2], [3].

3.3. Conclusions of the experimental part

From the works that were carried out in the years 1998-2001, which dealt with the problems of the concrete structures reliability on the primary circuit, can be concluded:

according to the carried out evaluation of the static reliability in accordance with the Czech standard •SN 731201-86 at the steady temperature operating regime can be stated that all the structural parts meet the requirements of the calculated stresses for following combinations:

- situation STANDARD (dead load + technology + temperature in the steady operating regime)
- dead load + technology + temperature in the steady operating regime increased by 30^0 C in the reactor zone (linear course, loading situation LPT 30),
- dead load + technology + temperature in the steady operating regime increased by 30^0 C in the reactor zone (quadratic course, loading situation KPT 30).

- satisfactory compliance among the results of the numerical temperature field solution and measured values was found.

4. Conclusion

By solving of the described interactive problem we can

- obtain values of the deformations and internal forces of the concrete structure
- analyse the influence of
  - the radiation, temperature and static stress acting on the structure
  - the physical non-linear behaviour of concrete
  - the shrinkage and creep of concrete.

The results of analyses can serve as the base for correct computing of time reliability and lifetime prognosis of the concrete load bearing structures.

It was confirmed that the set-up mathematical models would be possible simply to extend for the cases using the simulation method and for evaluation of the concrete structure reliability in which some of the input data are considered as random. At present, input statistic data are gathered for application of these models. But the substantial complication will be – regarding the extent of the solved task - the time demand of the calculation, but even this problem can be solved – [7], [8]. Application of the non-linear modelling is another sphere of the result accuracy specification.

In regard to the extreme loading (mechanical, temperature, moisture and radiation), owing to importance of the concrete structures and in view of extremely high demands on the reliability (that must be regularly controlled during the nuclear power plant operation and even after its closing as well), there appears as the only possibility of the experimental monitoring combination of important properties of a concrete structure with numerical verifying of the structure actual reliability by the help of mathematical modelling (with regard to the actual physically-mechanical characteristics) in the future. In connection with this method of the reliability conclusive evidence there are being prepared:

- application of the totally reliable access for the reliability evaluation of the concrete structures (mathematical modelling, simulation methods – input data considered as random values/field
- preparation and completion of a complex of tests for fast models of ageing (degradation processes, complex of referential samples).

Acknowledgements

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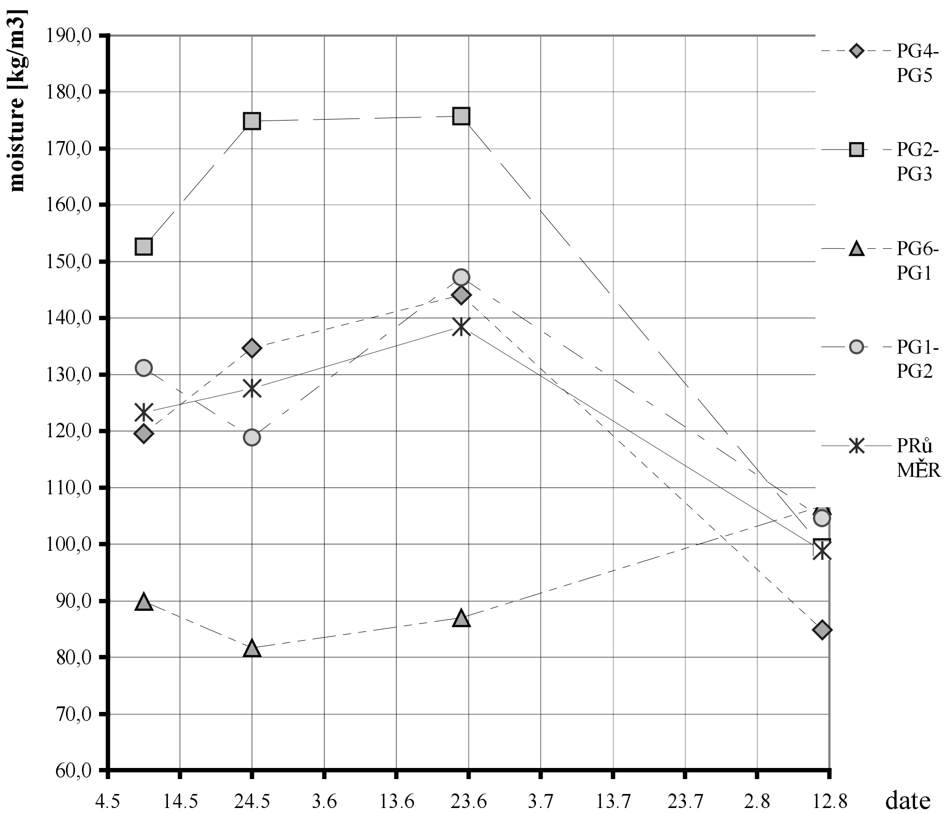

Fig. 1: Time-dependence of moisture on a tested area

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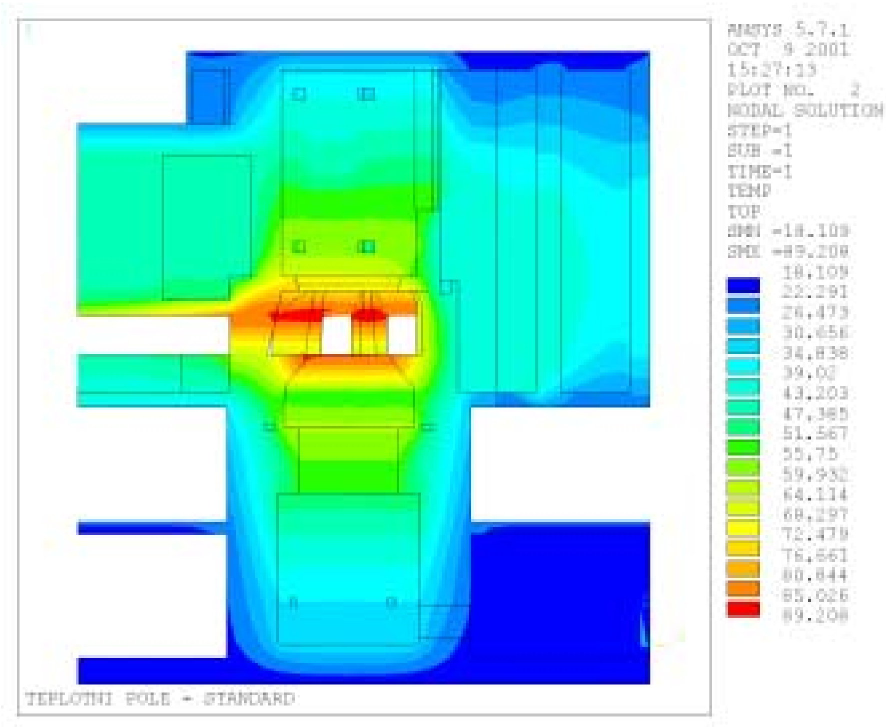


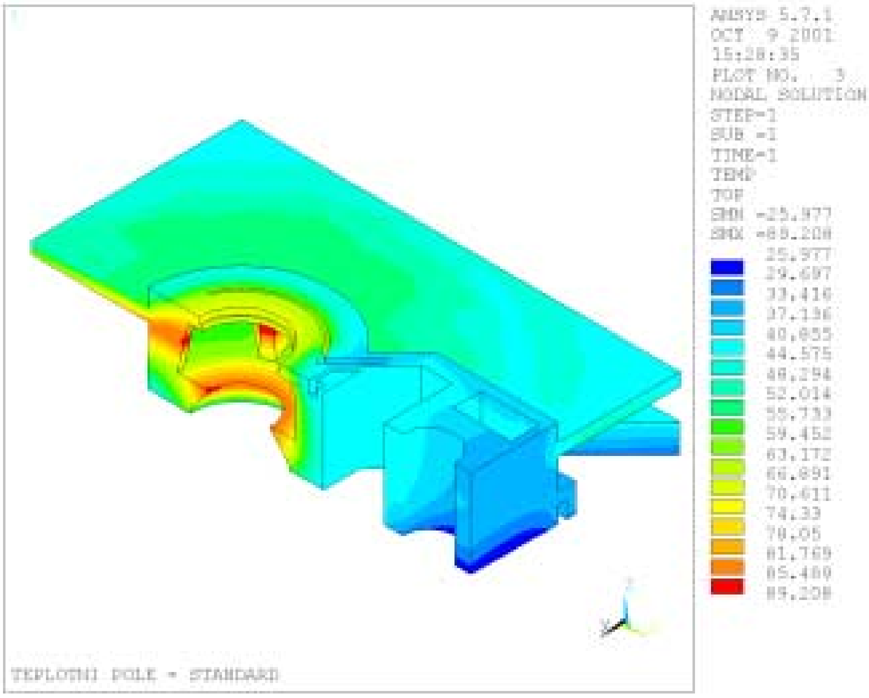
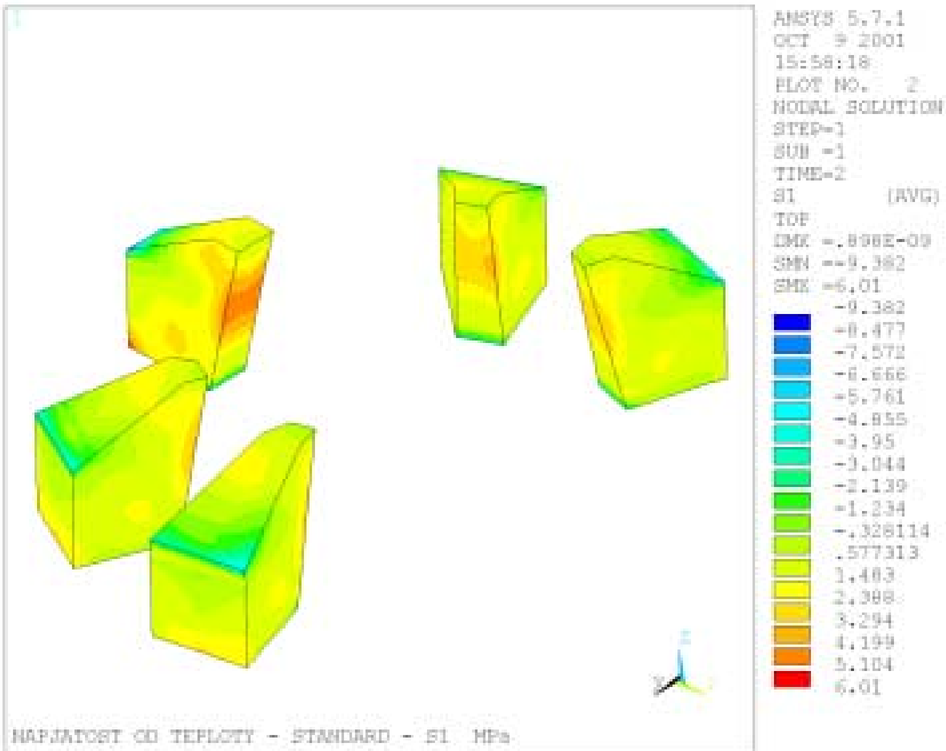


Fig. 2: Distribution of the temperature field on the concrete structures surface



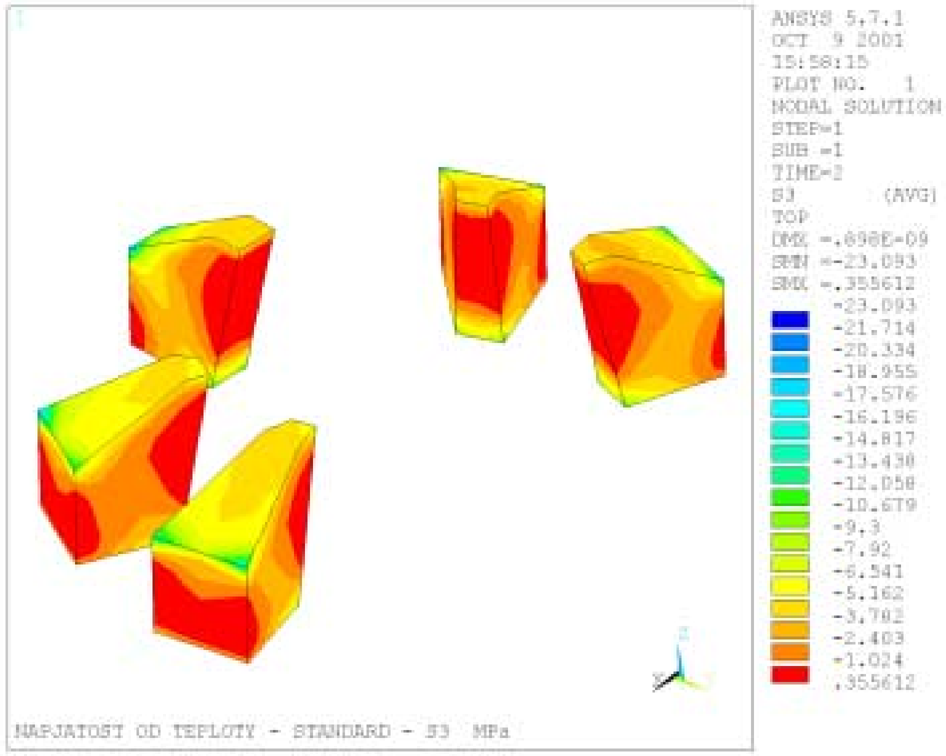
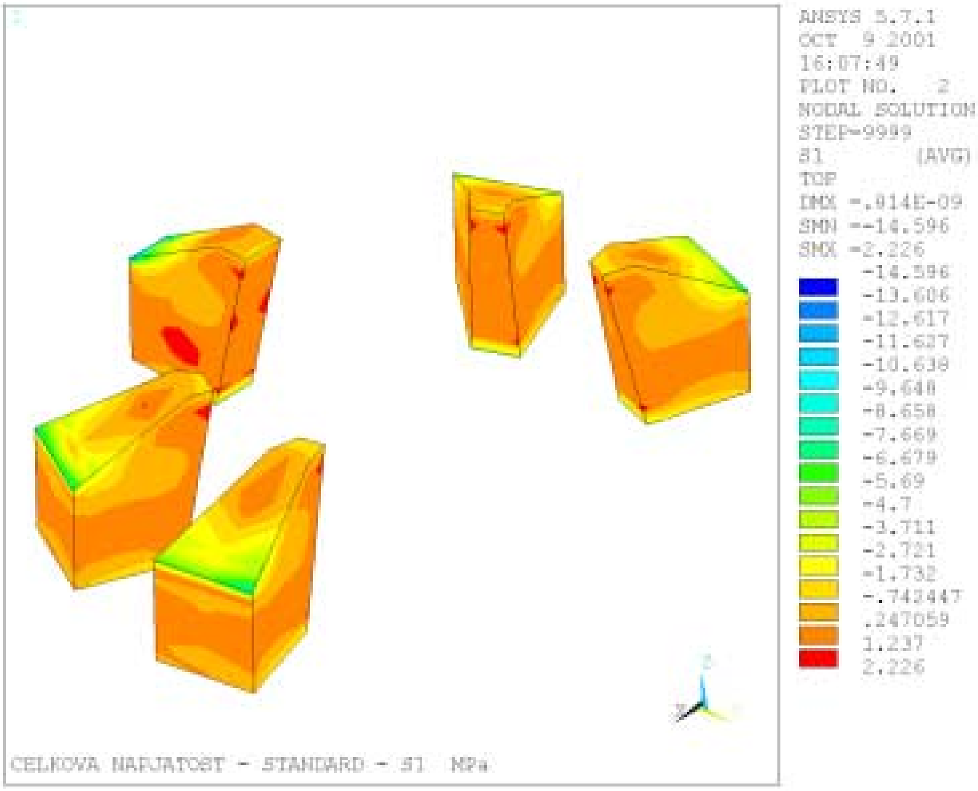


Fig. 3: State of stress (principal stress) of the concrete cantilevers initiated by the temperature loading



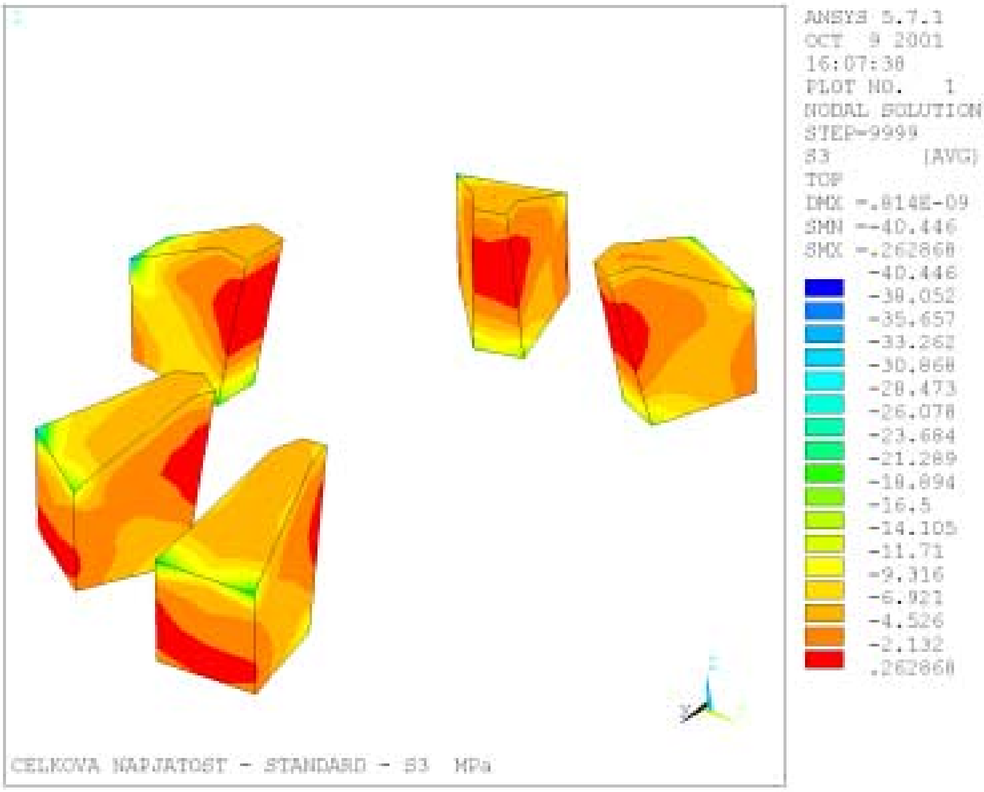


Fig. 4: Total state of stress (principal stress) of the concrete cantilevers (interaction of mechanical and temperature loading

Efficient management of inspection and monitoring data for a better maintenance of infrastructure

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Keywords: management, monitoring, inspection, maintenance

Abstract

In North America, Europe and Japan, government agencies and large private owners are now facing the challenge of maintaining, with limited resources, large stocks of vital structures like traditional and nuclear power plants, Cooling towers but also highways, railways, bridges, dams, harbors, industrial facilities etc… These structures are representing a large amount of money, have not been designed to be easily repaired or replaced, and are getting older and more vulnerable.

People involved in structure management have developed extensive technical methods and tools to monitor the condition of the structure and establish the diagnosis. Each authority has been developing is own inspection maintenance procedures, taking into account their specificity, their different priorities, safety requirements, resources and range of competence.

In most cases visual inspections are used to detect deteriorations, to rank structures, define priorities, estimate repair costs, etc… These visual inspections require to record, report, analyze and store for years large quantities of data (inspection records, drawings, photos…) and it is easy to get lost in the clerical work. Moreover a number of decision steps (inspection record, ranking of defects, long-term analysis) are still highly subjective and can greatly affect the quality of the final diagnosis.

An inspection-based management software system has been developed to optimize this process and provide decision-makers with objective information on the condition of the infrastructure. The system is a comprehensive management system which integrates: database of structural defects, on-site computerized record, analysis, maintenance, diagnosis, repair and budgetary functionalities.

This paper describes the basic functions and benefits of the system.

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1. LIMITATIONS OF CONVENTIONAL INFRASTRUCTURE MANAGEMENT PROCESS

The characteristics and limitations of, still widely used conventional structure management process are listed below:

- Design data (drawings), inspection data, detailed investigation data and repair data are not stored in a single system.
- Inspection frequency for a given structure is based on the type and age of the structure. It is rare that the date of inspection is based on the results of the previous inspection.
- Before inspection, inspectors must prepare inspection drawings – very often original design drawings are not available and inspectors must spend time to make new drawings.
- During inspection: the inspectors take hand-written notes of the defects. Inspectors usually do not bring with them the heavy reference manuals.
- Back in office the inspectors copy the deterioration onto the structural drawings, along with their dimensions and characteristics. Sometimes these data are stored electronically (excel sheets and CAD drawings).
- In accordance with the inspection manual, a ranking indicator or a comment is affected to each deterioration. The inspectors then establish reports that are transmitted to the engineers in charge of the analysis.
- The engineers receive several reports from different inspectors. They may have difficulties with inconsistent data, inhomogeneous ranking systems, unreadable handwriting or confusing dimensions. However on the basis of these reports the engineers must estimate the condition of the structures and recommend actions for maintenance or repairs.
- When needed, detailed investigations are performed by specialized consultants and specialized contractors propose repair solutions.
- Maintenance and repair costs are then presented to decision makers.

After reviewing the above points, it becomes clear that even with a clear inspection manual and an efficient organization, conventional infrastructure management process allows too much room for subjectivity and conventional infrastructure management is expensive.

2. OBJECTIVES OF THE INSPECTION-BASED MANAGEMENT SOFTWARE

The inspection-based management software has been developed with the following objectives:

- improve the overall efficiency of the maintenance process,
- reduce the cost of maintenance process at all steps,
- build a comprehensive database system which integrates all steps of the maintenance process (inspection preparation – inspection – reporting – analysis – repair – budget),
- facilitate the task of inspectors,
- assist engineers in the compilation and analysis of large quantities of data,
- allow easy access to all data at any step of the engineering and decision process,
- provide decision makers with valuable and objective information on the condition of the infrastructure, on which they can base and justify their decisions.

3. BASIC DESCRIPTION OF THE INSPECTION-BASED MANAGEMENT SOFTWARE

The inspection-based management software consists of several components specifically designed to handle the tasks of each people involved in the maintenance process (Table 1).

Software components	Tasks	Designed for
Infrastructure Management	- database of detailed information on structures (drawings, design-construction-inspection-repair data), - ranking of structures according to preset rules, - management of the database, - budgetary tools, - scheduling of tasks, - management of repair works	- operators of structure, - consultants, - specialized inspection companies
Inspection Management	- preparation of inspections, - transfer of data from mainframe to mobile inspection units	- inspectors, - specialized inspection companies, - consultants
Inspection	- on site recording of deterioration (on pen-touch light computers) reference available	
Photo-based inspection	- time-effective survey of deterioration based on photos	
Report	- reporting of site-records, - automated standard report	
Analysis	- advanced analysis functions, - detailed investigation, - detailed repair definition	- engineers, - consultants, - contractors

Table 1: Components of the inspection-based management software

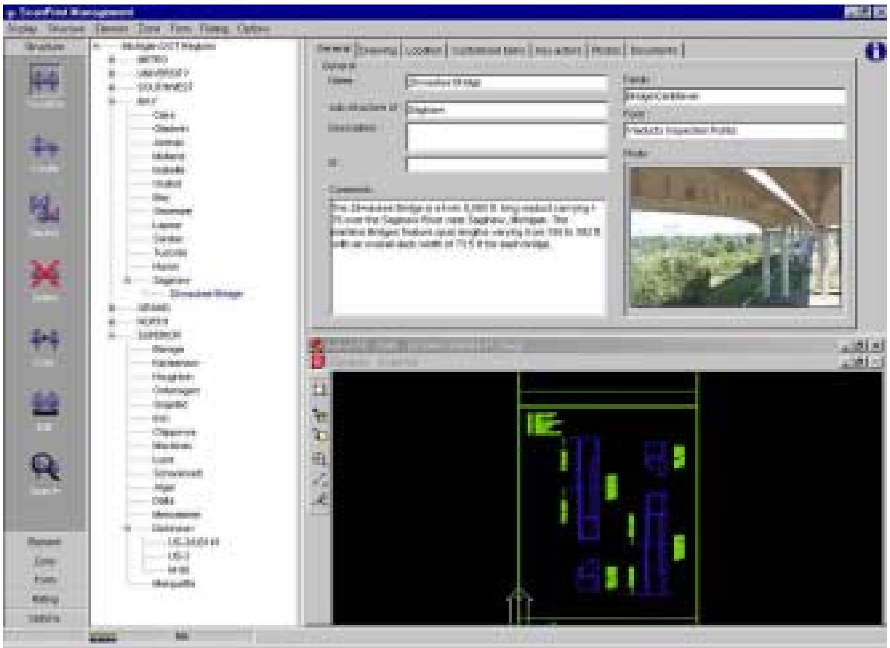

Fig.1: General information tab

3.1 Infrastructure management software

With the infrastructure management software, operators can organize their inventory of structures. The system allows to build a database including:

- general information about each structure (design, construction, location, pictures, drawings – see Fig.1),
- detailed check-lists for each structural component (see on Fig.2 an example for expansion joints),
- damage criteria for ranking of deteriorations (see Fig.2 where limit values for joint opening are defined),
- catalogue of repair solutions and corresponding costs and durations (see Fig.3).

The software has been built so that all parameters can be changed and adjusted to the specific usage of each industry/administration/operator. For example: the limit criteria for crack opening is smaller in nuclear containment vessels than in highway tunnels. Such a limit criteria can be set in accordance with the corresponding regulations.

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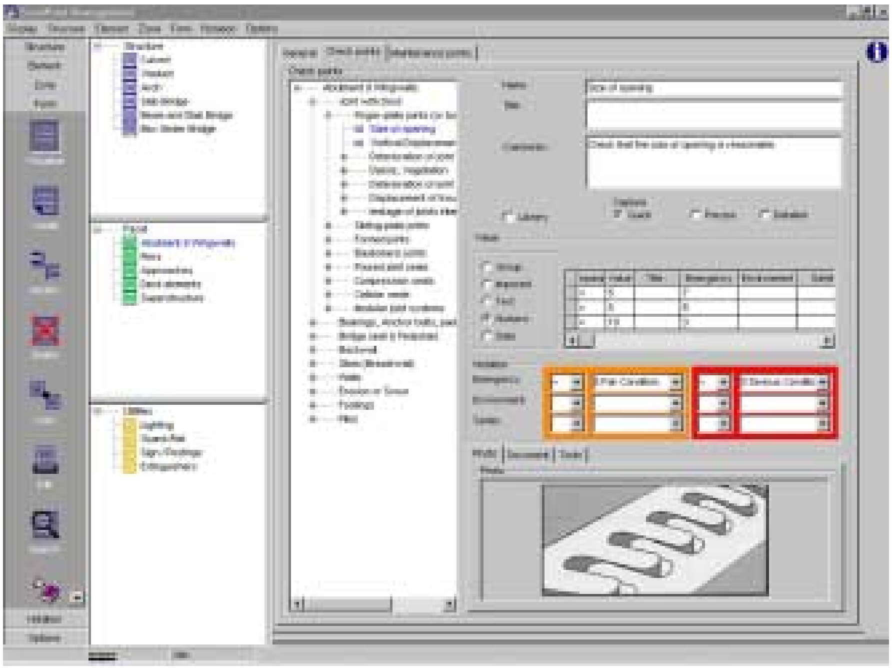

Fig.2: Definition of check-points and damage criteria for expansion joints

Fig.3: definition of tasks, with corresponding costs and scheduling

3.2 Inspection software

The inspection software is designed to simplify the task of inspectors and ensure coherent and regular deterioration surveys and maintenance records (Fig.4). Running on pen-touch computers (Fig.5), the inspector can access at any time:

- the inspection reference manual,
- the drawing of the inspected structure,
- the history of the deterioration that was recorded in past inspections,
- the maintenance check-list for each structural component.

When the inspector detects a deterioration, he draws with the pen directly on the computer screen the shape of the deterioration. He can use a reference of more than 200 deterioration types classified in families. In accordance with the inspection manual, the software then requests the inspector to measure and record a certain number of parameters to describe the deterioration (dimensions, color, humidity level…). He may also want to take pictures of the deterioration. The software saves all this information in the database:

- the deterioration type,
- its graphical representation in the CAD drawing,
- its dimensions and other specific parameters,
- any picture of the deterioration.

Because these data are linked together in the database, it will be easy to access and sort such information at any step of the maintenance process, even years after the inspection.

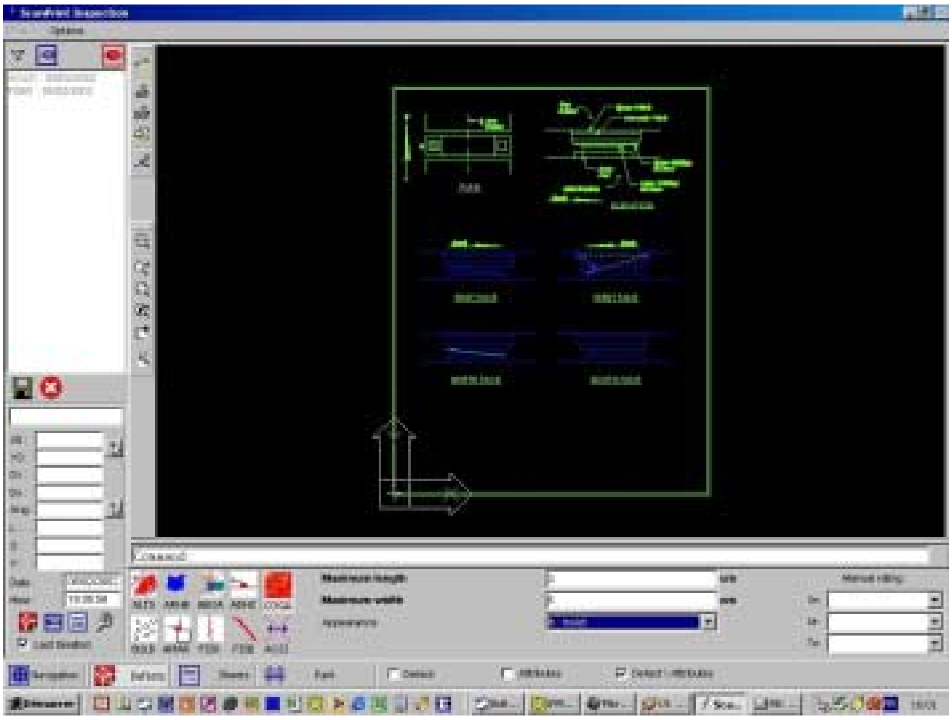

Fig.4: Site inspection software running on pen-touch computers

3.3 Photo-based inspection software

In some cases when access is difficult, survey of deterioration can be done using high quality digital pictures. The photo-based inspection software has the following functions:

- correction of the lens deformation of the picture,
- deformation/scaling of image,
- on-scale insertion of image data into CAD drawing,
- highlighting of typical deteriorations (rebar, cracks…)

Photo-based inspection is often a cost-efficient alternative for the inspection of large structures (dams, cooling towers, etc…)

Fig.5: Site inspection using pen-touch computers

Fig.6: Photo-based deterioration survey

3.4 Reporting software

Immediately after an inspection has been performed, the inspectors can edit standard reports with the reporting software:

- output of drawings with deteriorations (Fig.7),
- tables,
- pictures.
-

The process is automated and the output format can be adapted.

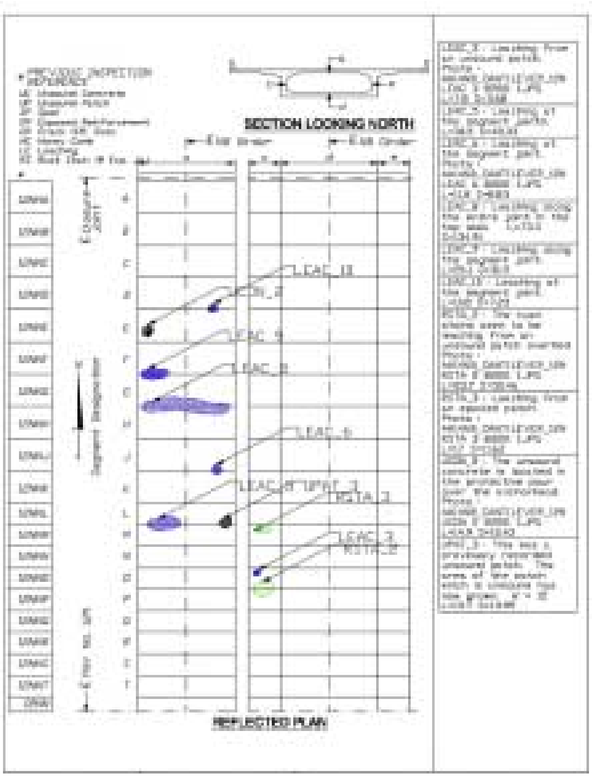

Fig.7: Standard output

3.5 Analysis software

Designed for engineers in charge of the analysis of data, the analysis software consists of a set of tools for:
- browsing and sorting of data,
- evolution of one deterioration or a group of deteriorations,
- comparison of similar structural components,
- evolution of the condition of one structure, evolution of the stock of structures,
- identification of deterioration to be repaired in priority,
- bill of quantities for repairs,
- assistance in the diagnosis.

4. BENEFITS AND DIFFICULTIES

The direct benefits are cost reduction and improved efficiency due to:

- organized database of consistent data (structural, inspection, maintenance and repair data),
- easier access and sharing of information,
- time savings for preparation of drawings, reporting and analysis of data,
- long-term management.

Whereas the main difficulties in implementing the system are:

- staff need to be computer literate,
- drawings that exists only on paper must be digitalized (scanner) or redrawn with CAD,
- older data should be input in the new system.

The system is therefore easier to implement on recent structures because CAD drawings are available and older data is smaller: the system has been used from the beginning on the Tagus estuary crossing in Portugal (Vasco de Gama bridge).

However, the extra work required to input paper-based drawings and older data can be recovered through cost and time savings at all steps of the process. During the inspection and analysis of the Zilwaukee bridge (a twin 2.5 km-long precast segmental viaduct carrying I-75 over the Saginaw River, Michigan, USA), the system proved to be very cost-effective. Every day the inspectors sent the data by email for review by the Project Manager and the analysis team, located 3,000km from the bridge. Data from previous inspections was later inputted electronically so that it can be compared, sorted and visually displayed along with new data.

5. CONCLUSION

Structure management is an increasingly important concept for structure owning authorities or private companies around the world today. Each owner has been developing its specific health condition indicator system, allowing to express the structural condition of a structure by some quantitative measure, to monitor durability, safety and to decide at what point action needs to be taken.

A management software that would integrate all the steps of this maintenance process can dramatically optimize its efficiency through easier management, storage, sharing and analysis of the structural information. While such a concept is not new, this paper presented in detail the capabilities and benefits as they were observed in actual large scale implementation of the inspection-based management software.

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AGING PROCESS OF A GOOD CONCRETE DURING FORTY YEARS

Dr. Peter Lenkei

Pécs University, College of Engineering (Hungary)

A prestressed concrete truss (Fig. 1) was used to cover an uranium ore processing (concentrating) hall in a nuclear industry plant. The environment of the hall was slightly aggressive, containing sulphuric acid and carbon dioxide. Due to the uranium oxid dust all the equipment and the reinforced concrete structures were covered with a special paint coating for easier decontamination.

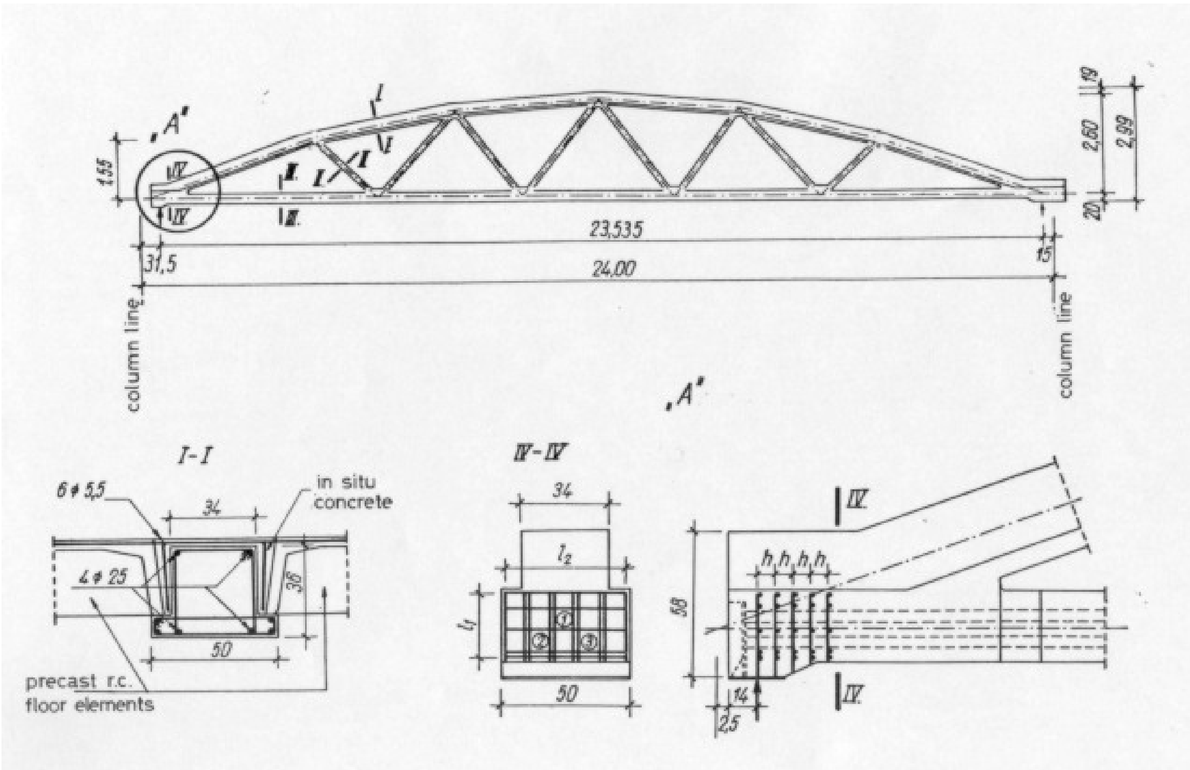


Fig. 1 The lay-out of the truss

After 40 years of service the uranium ore mining was terminated and the hall was demolished (Fig. 2).

Fig. 2 The partly demolished hall

During the life span of the structures several investigations were made. Destructive tests (DT) were made after construction in 1960 (200*200*200 cubes), in 1990 and in 2000 thorough non-destructive tests (NDT) were carried out. Finally, after demolition another DT was made, core samples were taken from the truss. The results converted to 150*300 cylinder strength are shown on Fig. 3.

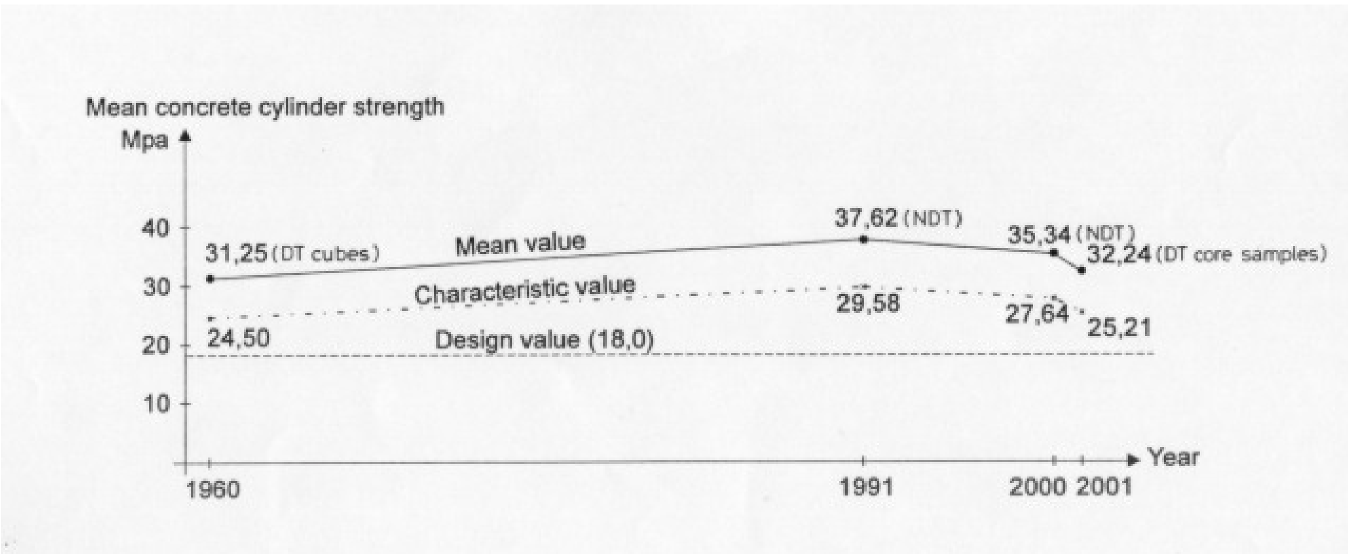


Fig. 3 Concrete strength variation in time

Likewise after the demolition parts of the prestressed bottom chord of the truss were surveyed and neither on the prestressed wires, nor on the other reinforcement traces of corrosion could be find (Fig. 4).

Fig. 4 Surface of prestressing wires and rebars

The results demonstrated, that the concrete aging process was characterized by a definite increase over the initial strength and even after 40 years the concrete strength was 3% higher of the initial strength. Most probable the NDT gave a little higher results over the true strength values. Neither visible cracks or carbonization, nor reinforcement corrosion were detected.

CONCLUSIONS

1. Initially good quality concrete and reliable concrete cover, with sufficiently maintained paint coating could guarantee the long term life span of prestressed concrete structures.

2. Even the correct NDT may slightly overestimate the concrete properties.

SESSION B: STATE OF THE ART &FUTURE DEVELOPMENTS (Continued)
Chairman: Mr. Jean-Pierre Touret, EdF, (France)

THE USE OF ACOUSTIC MONITORING TO MANAGE CONCRETE STRUCTURES IN THE NUCLEAR INDUSTRY

Marcel de Wit, Gilles Hovhanessian
Advitam

ABSTRACT

Concrete and steel are widely used in containment vessels within the nuclear industry. Both are excellent acoustic transmitters. In many structures tensioned wire elements are used within containment structures. However, tensioned wire can be vulnerable to corrosion. To reduce the probability of corrosion sophisticated protection systems are used. To confirm that the design strength is available through time, extensive inspection and maintenance regimes are implemented.

These regimes include tests to confirm the condition of the post-tensioning, and pressure tests (leak tests) to verify the performance of vessel.

This paper presents an acoustic monitoring technology which uses widely distributed sensors to detect and locate wire failures using the energy released at failure. The technology has been used on a range of structures including post-tensioned concrete bridges, suspension bridges, buildings, precast concrete cylinder pipelines (PCCP) and prestressed concrete containment vessels (PCCV), where it has increased confidence in structures and reduced maintenance costs.

Where the level of ambient noise is low then SoundPrint® acoustic monitoring can detect concrete cracking. This has been shown in PCCP pipelines, on laboratory test structures and also in nuclear structures. The programme has shown that distributed sensors can locate internal cracking well before there is any external evidence.

Several projects have been completed on nuclear vessels. The first has been completed on an Electricité de France (EDF) concrete test pressure vessel at Civaux in France. The second at the Sandia PCCV Test Vessel in Albuquerque, New Mexico, USA, which involved the testing of a steel lined concrete vessel. The third was on a PCCV in Maryland, USA.

Acoustic monitoring is also able to monitor the deterioration of post-tensioned concrete structures as a result of seismic activity. Summary details of a case history are presented.

1. INTRODUCTION

Continuous acoustic monitoring has been used since 1994 to monitor failures in bonded and unbonded tendons in post-tensioned structures, where it has shown major benefits in confirming the performance of structures, increasing Client confidence and reducing maintenance costs. To extend the application of this technology to the monitoring of concrete cracking required that the effectiveness of the principles and methods was evaluated for each structural type.

For acoustic monitoring technology to function in a particular environment it must be shown that the signals generated by cracking can be detected above general noise levels and distinguished from events which are not of interest. Furthermore, to assess the structural implication of each event it is generally important to be able to locate the source of each emission. Provided with high quality data of this type, the engineer can appraise a structure with knowledge of the actual failures in damaged elements, and their location, in the entire structure over the monitoring period. The alternative, to base the assessment on a physical inspection at a sample of locations, leads to uncertainty when for practical and economic reasons the number of inspection points is limited. Monitoring the entire structure may also reveal failures not detectable by a conventional investigation.

In many applications the acoustic data is transmitted over the Internet for processing and analysis. After processing and quality control checks, the data can be made available on a secure section of the SoundPrint® website, allowing owners rapid independent access to their database of results.

SoundPrint® acoustic monitoring systems have also been placed on structures, which are in active seismic zones. Rapid status reports on wire failures / structural damage allows Owners and Regulators to assess the condition of a structure within a few hours.

The technology is useful in providing cost-effective long-term surveillance of both unbonded and grouted post-tensioned containment structures. This paper shows how the technology can also monitor the cracking of concrete structures which are subject to low levels of ambient noise.

2. DEVELOPMENT OF CONTINUOUS ACOUSTIC MONITORING

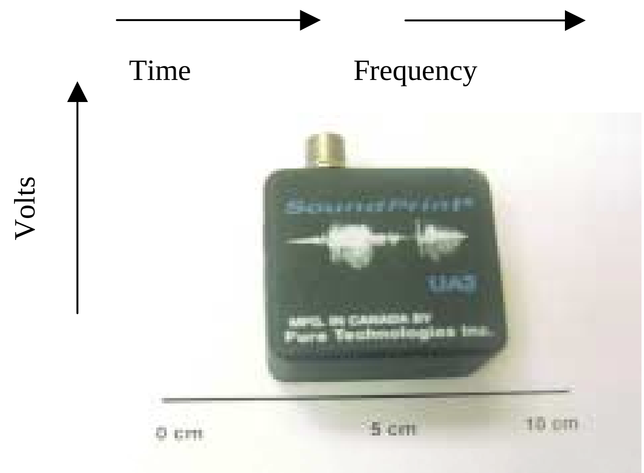
The principle of examining acoustic emissions to identify change in the condition of the structural elements is not new. However, until recently, continuous, unattended, remote monitoring of large structures was not practical or cost-effective. The availability of low-cost data acquisition and computing hardware, combined with powerful analytical and data management software, resulted in the development of a continuous acoustic monitoring system called soundprint®, which has been successfully applied to unbonded post-tensioned structures in North America since 1994.

Corrosion of the steel strands in these post-tensioned structures has become a concern for designers and owners. As with grouted post-tensioned bridges, the extent of corrosion is not known, primarily because of the difficulty of identifying corrosion due to the inaccessibility of the corrosion sites, the lack of external evidence and the limited spatial coverage of intrusive inspections.

The SoundPrint® system uses the distinctive acoustic characteristics of wire breaks to separate them from other acoustic activity on a structure. Using a combination of instrumentation, data acquisition and data management, it is possible to identify events, as well to locate the failure and time of failure.

This concept allows the non destructive identification of broken strands, so that these strands can be replaced periodically as part of a long term cost effective structural health programme. In addition, an understanding of the condition of the steel wire elements allows the life of the structure to be extended.

A typical system includes an array of sensors (Figure 1) connected to an acquisition system with coaxial communication cable. The sensors are broadband piezo-electric accelerometers fixed directly to the concrete slab. Sensor locations are chosen so that an event occurring anywhere on the slab can be detected by at least four sensors. Sensor spacings range from 1 per 60 square meters for fully grouted slabs up to 1 per 100 square meters for ungrouted tendons. Multiplexing techniques are able to acquire data from many hundreds of channels on 32 acquisition channels.



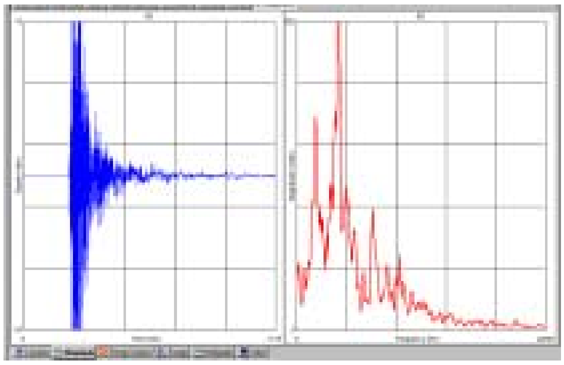


Figure 1 – Standard sensor for buildings, bridges and parking structures

Figure 2 – Time domain and frequency spectrum plots of wire break detected by sensor 10.0 m. from event

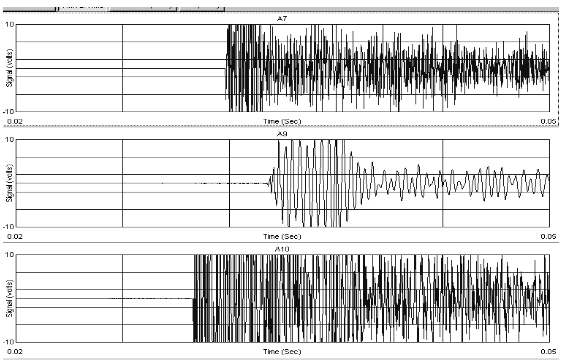


Figure 3 – Time domain plot showing relative arrival time of signal at different sensors

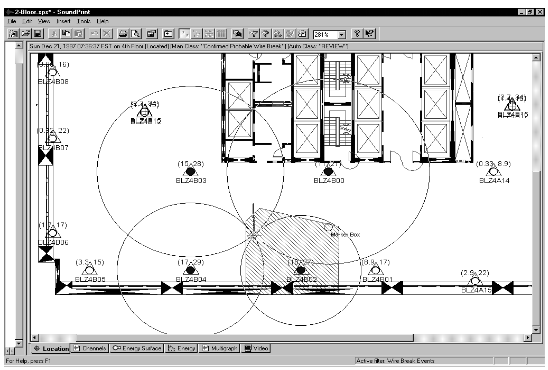


Figure 3 – Time domain plot showing relative arrival time of signal at different sensors

Using several characteristics of the acoustic events including frequency spectrum it is possible to classify wire breaks and to reject environmental noise.

By analyzing the time taken by the energy wave caused by the break as it travels through the concrete to arrive at different sensors, the software is able to calculate the location of the wire break, usually to within 300 – 600 mm of the actual location. Independent testing showed the system to be 100% correct when spontaneous events classified as “probable wire breaks” were investigated. Figure 2 shows a typical acoustic response to an unbonded wire break at a sensor 10.0 m from the break location. Figures 3 and 4 illustrate how the system locates events.

SoundPrint® site systems download all data automatically using the Internet to the Calgary processing center. This allows the cost of data transfer to be minimized. All data can be viewed by the owners team directly on the Pure Technologies secure web site. This allows the owner to review areas of concern in parallel with the generation of routine reports. Various levels of alarms can be triggered semi-automatically using e-mail, automatically voice activated phone alarms, etc.

Presently, over 300,000 square meters of unbonded post-tensioned slab in twenty structures, five bridges and almost 100km of large diameter water pipe are being simultaneously monitored. The analytical software is capable of automatically generating reports summarizing the time and location of wire breaks and other significant events. The operating efficiency of the system over the monitoring period is also recorded.

3. MONITORING OF WIRE BREAKS IN GROUTED POST-TENSIONED BRIDGES

Acoustic monitoring has been used in a wide range of applications including suspension and cable stay bridges (reference J.F. Elliott), and pipelines (reference Mark Holley).

The technology has also been applied on many post-tensioned concrete bridges as described at this conference (reference Carlyle, Adkins, Youdan).

4. MONITORING OF CRACKING DEVELOPMENT IN CONCRETE STRUCTURES

Description of Concrete Projects

During the UK TRL grouted post-tensioned bridge evaluation program, the developers of the acoustic monitoring system, Pure Technologies Ltd (Pure) and TRL had the opportunity to evaluate the application of the method to crack development in a partially hollow reinforced concrete beam specimen. This specimen was tested with three-point loading.

Dr. Walter Dilger of the University of Calgary provided access to a flat post-tensioned slab specimen. The slab was 5 m by 10 m by 150 mm thick supported by 3 columns and tested in shear.

Electricité de France allowed access to a large-scale model prestressed concrete containment vessel at Maeva being tested with internal pressure. This vessel has part of the perimeter wall instrumented for crack detection. This specimen had previously been tested to the same pressures used in the current experiment.

At the Sandia National Laboratories in Albuquerque, New Mexico, a ¼ scale test vessel was pressured to the full ‘failure’ test pressure in a modification of a standard leak rate test. The objective was to monitor concrete cracking, tearing of the liner, and gas leakage.

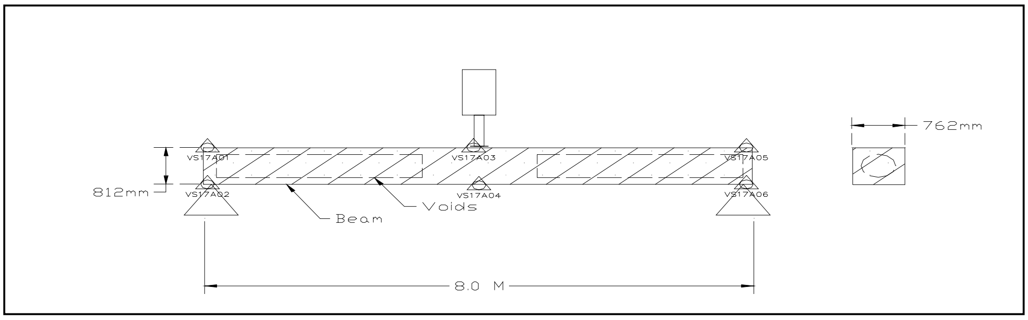
Finally in Maryland, US an operating PCCV was tested during an Integrated Leak Rate Test to determine if any wire failures were recorded.

Transport Research Laboratory, UK

A partially-voided reinforced concrete bridge beam was loaded as shown in Figure 5.

Microphones / accelerometers were installed at six locations on a voided beam specimen. Stress was applied as three point loading. Emissions were noted at all loads and continued throughout the test.

Figure 5 - Arrangement of Test Specimen VS17



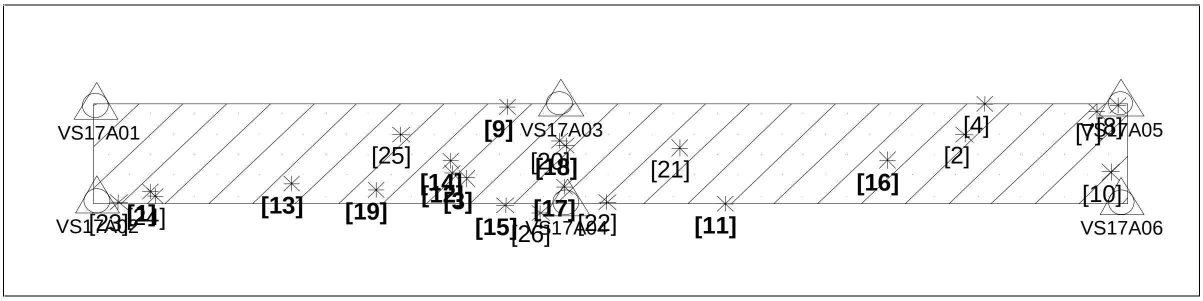


Figure 6 – Location of Events from 0 kN to 30 kN (0 to 6,700 lbf)

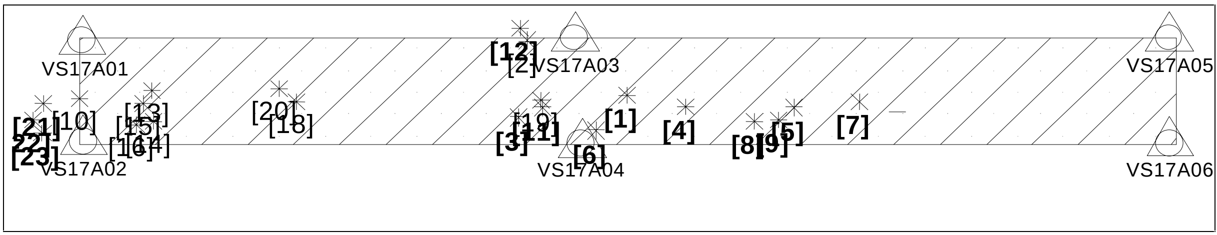


Figure 7 – Location of Events from 30 kN to 100 kN (6,700 lbf to 22,500 lbf)

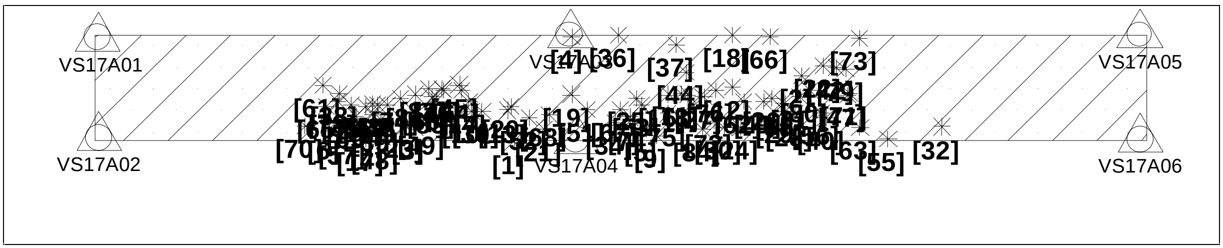


Figure 8 – Location of Events from 100 kN to 200 kN (22,500 lbf to 45,000 lbf)

Results

As used here, ‘cracking noise’ means the generation of acoustic events associated with the propagation of cracks, some of which were not visible. Amplification through the data acquisition system produced audible cracking noise throughout most of the test. No sounds were heard or recorded during periods when displacement had stopped. Acoustic events were located as plotted in Figures 6, 7 and 8.

It was noted several times during the test that the locations of cracks could be determined from the acoustic data before the cracks became visible. On most occasions, the operators of the acoustic equipment were able to direct researchers to the area where cracks had occurred, resulting in the visual confirmation of cracks at those locations.

Crack Monitoring at University of Calgary

Procedure

Ten accelerometers were attached to the underside of the test slab. The slab-column arrangement is shown in Figure 9 and sensor locations are shown in Figure 10. Lateral motion of the slab was commenced and the resulting cracking events were heard and recorded. Upon first loading of the specimen, a very large number of small emissions were heard. This is known as the Kaiser. This effect describes the generation of acoustic events coincident with initial load sharing and redistribution when a concrete specimen is first loaded to a given level. Subsequent unloading and reloading to the same level will not produce new acoustic events until the previous maximum load is exceeded. The rate of occurrence of these emissions is estimated at between 10 and 100 per second in the specimens tested at the rate of loading used. A sample of the time-domain data is shown in Figure 11. Each graph represents the output of one sensor.

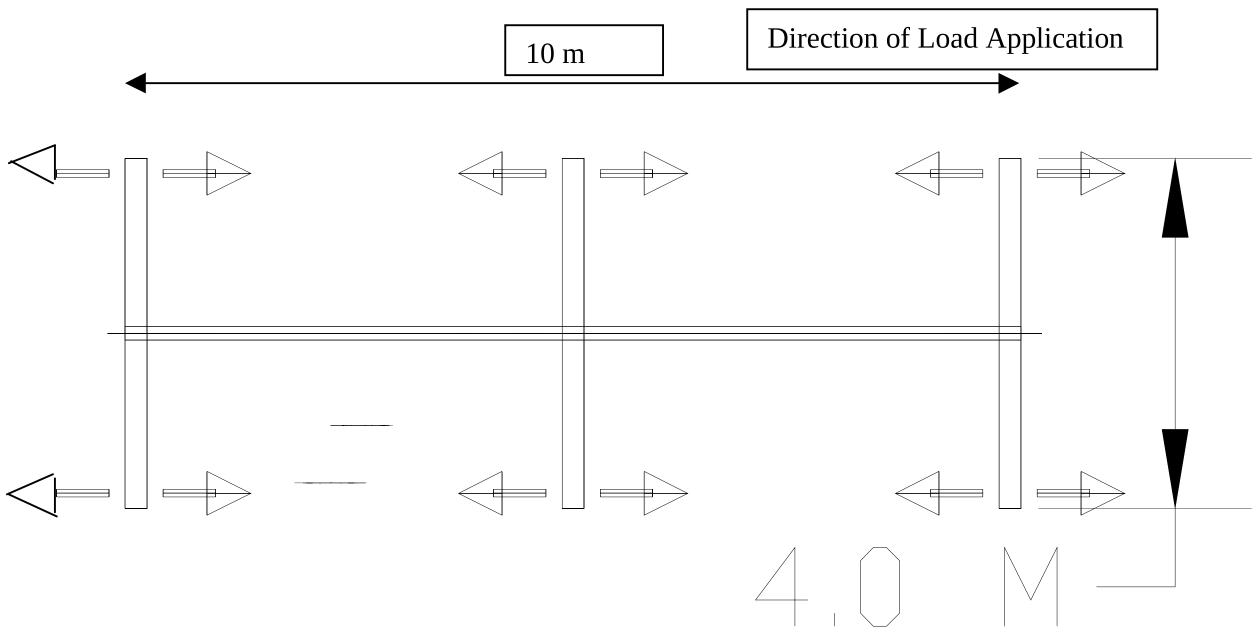


Figure 9 - Slab Arrangement – Specimen UCS1

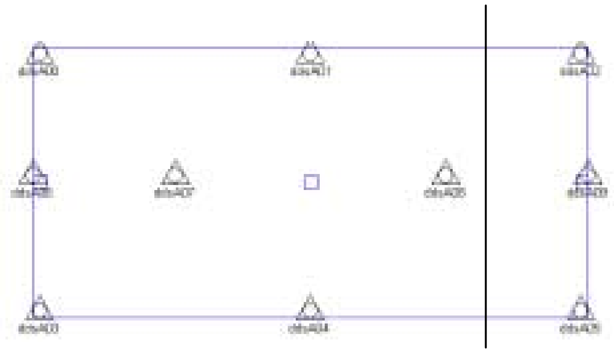


Figure 10 – Sensor Locations

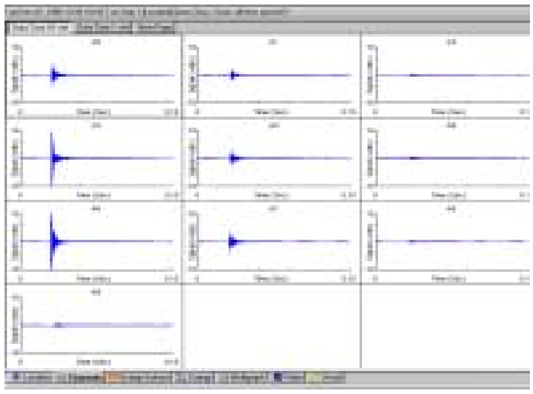


Figure 11 – Time Domain Graphs

Results

Amplification through the data acquisition system produced audible cracking noise throughout most of the test. No sounds were heard or recorded during periods when displacement had stopped. The locations of these events are shown in Figure 12.

As was the case with the TRL test, it was possible to direct researchers to the location of cracking before the cracks were visible. The locations of cracks identified by the acoustic system coincided with the confirmed locations as determined by the researchers.

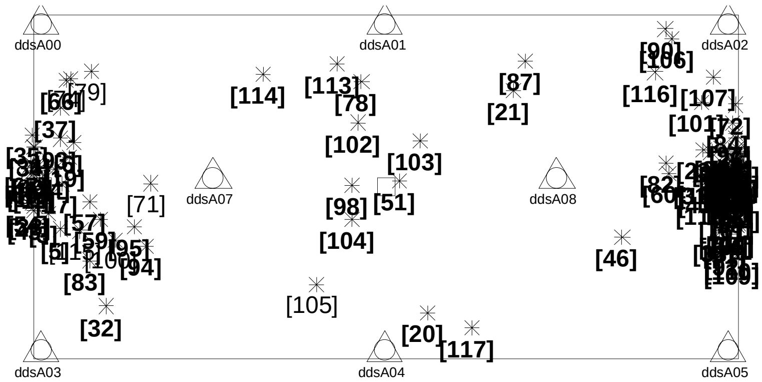


Figure 12 – Event Locations

Maeva Model Containment Vessel

The Maeva vessel was built for other purposes relating to internal pressure testing. The vessel consists of a cylindrical wall with an internal diameter of 16.0 m (52.5 ft.) and an external diameter of 18.4 m (60.4 ft.). The floor and roof of the vessel consist of concrete slabs connected by four columns each containing sixteen x 75 mm (3 in.) high-strength steel Macalloy bars. The concrete wall is enclosed by a watertight steel bulkhead. Instrumentation has been installed on two panels of the vessel to confirm the ability of the acoustic monitoring system to detect cracking of the concrete as pressures change. The programme includes a medium pressure test to 5.66 bars (82.1 psi) (which has been completed) and a high-pressure test to 10 bars (145 psi) (which is to be completed later). Because the vessel had previously been pressurized to a greater pressure than was used in the medium pressure test, the vessel was not expected to produce the Kaiser effect cracking.

Sixteen sensors were attached to two contiguous wall sections of the vessel in the pattern shown in Figure 13. The sensors were attached to the outer perimeter of the vessel in the annular space between the inner and outer wall. The sensors are connected to a data acquisition unit located in a building 200 m (656 ft.) from the structure.

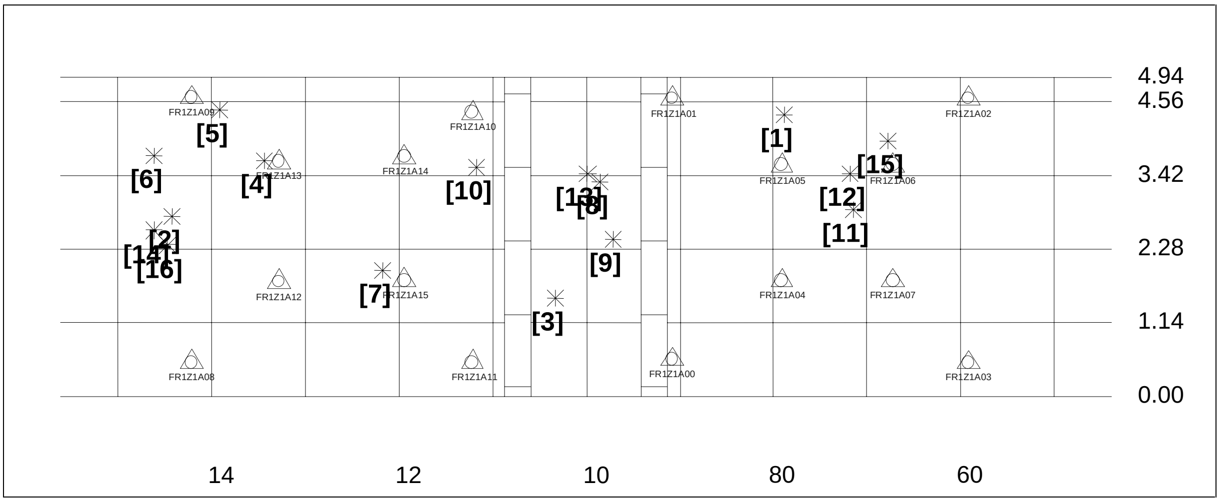


Figure 13 – Sensor Layout and Location of Acoustic Cracking Events on Maeva Vessel Test No. 3, June 1999

<u>First Results</u>

Sixteen acoustic cracking events were detected and located. Of these, events numbered 2, 6, 14 and 16 occurred outside the area monitored and therefore could not be accurately located. Locations of events are shown on Figure 13. The largest cracking events were detected at locations 1, 5, 6, 12 and 15. Comparison with a known impact from a Schmidt hammer suggests that some of the cracking events released approximately 1 Joule of energy. Event #1, the first large event, occurred at an internal pressure of 3.01 bars (42.66 psi). Maximum pressure achieved was approximately 5.66 bars (82.09 psi). Events 15 and 16 occurred after the pressure began decreasing, at 5.65 bars (81.95 psi) and 5.46 bars (79.19 psi) respectively. Time domain and frequency spectrum plots for Event #1 are shown in Figures 14 and 15.

To confirm the relationship between acoustic events and cracks it is common to correlate the visible surface evidence of cracks and the measured locations of the acoustic cracking events. This process is limited by the fact that cracking events recorded by SoundPrint® may reflect cracks that are internal to the structure. Also a visual inspection will normally only record cracks that are of the order of 0.2 mm (0.008 in.) wide, whereas SoundPrint® will record cracks finer than this. For this test vessel two additional factors are relevant. The first is that no crack survey was taken before this latest phase of the testing commenced, and the vessel had previously been taken to higher pressures than were used in this phase of testing. The second is that the internal surface of the concrete had been treated with a thin layer of epoxy, which would have the effect of reducing the clarity of the any crack survey. However, a preliminary crack survey was carried out after the medium pressure tests and this survey will be repeated after the high pressure tests.

The results of the first phase of acoustic monitoring have confirmed that the system is working satisfactorily, can identify acoustic cracking activity and is able to locate the source of such cracking events.

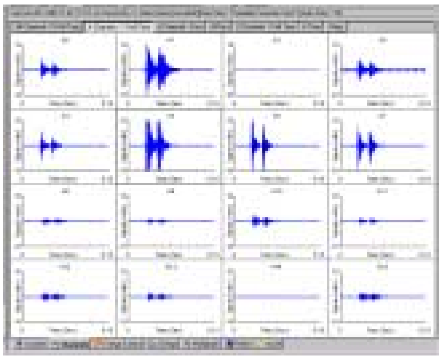


Figure 14 - Time Domain Plot of Sensors Responding to Crack #1.

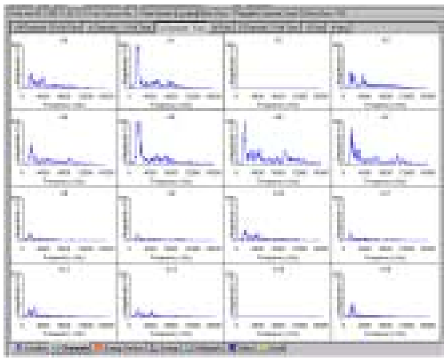


Figure 15 - Frequency Domain Plot of Sensors Responding to Crack #1.

<u>Outline Details of the Sandia Test</u>

In a US project sponsored by MITI and the National Research Council (NRC), a one-quarter scale model of a steel lined containment vessel has been constructed at Sandia National Laboratories in Albuquerque, New Mexico. The model is approximately 16.5 m (54 ft.) high and 11 m (36 ft.) in diameter. The objective of the work was to validate numerical simulation methods by comparing measured to calculated responses well into the inelastic regime, up to and including failure. The intention was to map the development of strains and eventual damage as the pressure in the vessel is brought above the design pressure of the vessel. Although more than one thousand strain and other gauges were installed on the vessel, much of the vessel was not monitored by strain gauges. Acoustic monitoring was being installed on the entire vessel wall area as part of the programme to detect tendon failures and with the hope of monitoring concrete cracking and liner tearing/leakage. The system has been specially configured to stream data offsite to a back-up computer incase the onsite unit is destroyed during the test. Preliminary details of the test are reported by Hessheimer[5]. Results of this test will be published first by the NRC in 2001. See Appendix A for update to June 2001.

<u>PCCV Maryland</u>

This vessel is approximately 42m diameter reinforced with two sets of circumferential tendons. A total of 204 tendons are present; a normal tendon is comprised of 90No. 6mm wires.

The vessel was monitored with 36 distributed acoustic sensors, which were placed on the external concrete surface. The vessel was tested during an Integrated Leak Rate Test. Surveillance of the vessel was by on-site monitoring in real time during the test. During the ILRT zero wire breaks were recorded. To confirm the performance of the acoustic monitoring system events were generated using impacts with a similar energy (and a similar acoustic signature) to a wire break. Impacts made at anchorage cans, were correctly identified and located. This was quite an achievement as some of the cans were less than 1m apart. See Appendix A for update to June 2001.

5. SEISMIC MONITORING

In numerous applications Owners need to know the effect of a seismic event on their structures. This is particularly important in seismically active zones and in the nuclear sector where seismic standards are rigorous. In California where there is a statutory requirement to report on the effect of a seismic event within 24 hours, SoundPrint® acoustic monitoring has been used to provide rapid and relevant details to the Owner.

East Bay Municipal District of Southern California - Brookwood Reservoir

Above ground prestressed water storage tanks are common in many areas of the world, including California. Failure of these tanks due to corrosion or other factors can be catastrophic. To investigate the usefulness of long term acoustic monitoring as a management tool, the East Bay Municipal District of Southern California commissioned the installation of a monitoring system on the Brookwood reservoir, a 10 million litre (264,00 US gal) capacity tank in the Walnut Creek area East of San Francisco.

As part of the commissioning process, two individual wires were corroded to failure and the results monitored with the system installed there. Both wire breaks were detected and located successfully within 300 mm (1 ft.) of the actual location.

At 18:06 on 17 August 1999 there was a Mercalli 5 earthquake in the Bay Area of California. At 23:04 SoundPrint® recorded 1 wire break at the water tank resulting from the earthquake some 30-km (19 mi.) distant. Details of this minor damage were e-mailed to the owner within hours. With this data the owner was able to report quickly, and positively, recording only minor damage. With the proven success of the system the owner is planning to extend the number of structures monitored.

If many sites were monitored, the owner can also use the rapid notification capability of the system to direct emergency repair teams to the areas where most damage has occurred in the event of a large earthquake.

6. SUMMARY

Continuous remote acoustic monitoring has been used successfully to determine the time and location of wire breaks in prestressed structures. Testing of the technique as a method of detecting cracking in concrete structures subject to loading has been carried out on post-tensioned and reinforced concrete structures of different configurations. On two of the three structures tested, the locations of cracks identified by the monitoring system were confirmed by visual inspection. Crack development was detected by the monitoring system before cracks became visible.

The SoundPrint® acoustic system has been shown to provide reliable continuous remote monitoring capability and the ability to determine the times and locations of both prestressing failures, concrete cracking and liner tearing/leakage. These capabilities are useful in providing cost-effective long-term surveillance of unbonded and grouted post-tensioned containment structures, and of cracking within the concrete itself where ambient noise levels are low.

SoundPrint® has been shown to be a valuable management tool where rapid monitoring of seismic response is required.

ACKNOWLEDGEMENTS

The authors thank TRL and the UK Highways Agency, Dr. Walter Dilger, University of Calgary, and Electricité de France for use of these data. The authors wish to thank the many individuals and agencies who contributed to the gathering of these data.

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Concrete Properties Influenced by Radiation Dose During Reactor Operation

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Technical Counselor
Secretariat of Nuclear Safety Commission

ABSTRACT

The radiation dose effects on the physical, chemical and mechanical properties to the biological shielding concrete of the Japan Power Demonstration Reactor (JPDR) were studied to obtain useful information for the plant life management strategy of commercial nuclear power plants. The JPDR was passed 25 years from the construction and performed 957 days operation and the total reactor operating time 14,230 hours. The cumulative radiation dose rate on the biological shielding concrete of the JPDR was estimated as equivalent with the one that from the current commercial nuclear power plant after operated 40 years. High radiation dose is a special unique environment for the concrete structures in nuclear facilities. The evaluation of the radiation dose effect to the concrete structures in the nuclear power plants is important factor for the plant life management strategy considering aging of the concrete properties during plant operation. Usually, studies on concrete properties influenced by the radiation were performed under the test condition of short term and high irradiation rate. The test results under the condition of long term and low irradiation rate for the concrete are rarely exist. This study was conducted using the actual concrete samples from the JPDR biological shielding concrete obtained when the plant was decommissioning. The maximum fast neutron and gamma ray at the reactor side surface of the biological shielding concrete are 1.11 × 10E+18 n/cm2 and 4.77 × 10E+18 Gy, respectively, at the level of the reactor core. The test results showed that the compressive strength of the concrete samples were not decreased by the radiation exposure which was rather shown the tendency to increase along with the fast neutron fluencies within the test range to 10E+17 n/cm2. The test results showed the biological shielding concrete with steel lining have good durability in the test range of radiation exposure dose rate in spite of affected with the heat generation within the shielding concrete by the neutron and gamma ray flux.

INTRODUCTION

The evaluation of the radiation dose effect to the concrete structures in nuclear power plants is important factor for the plant life management strategy considering aging of the concrete mechanical properties during plant operation. Usually, studies on the concrete properties influenced by radiation exposure were performed under the accelerated test condition of short term and high irradiation rate. Test results under the condition of long term and low irradiation rate for the concrete using the actual concrete specimens extracted from operating nuclear power plants are rarely existing. The study of radiation dose effects on the physical, mechanical and chemical properties of concrete was performed using the actual biological shielding concrete sampled by coring from the Japan Power Demonstration Reactor when the plant was decommissioned. The Japan Power Demonstration Reactor (JPDR) was the first nuclear power generation reactor of the rated reactor power 45 MW in Japan that was passed 25 years from the construction at the 1986 and performed 957 days operation that the total reactor operating time was 14,230 hours when the plant was on the decommissioning. The cumulative radiation dose rate on the biological shielding concrete of the JPDR was estimated as equivalent with the one from the current commercial nuclear power plant after 40 years operation. High radiation dose is a special unique environment for the concrete structures in the nuclear facilities. The tests were conducted considering the various environmental state conditions of the radioactivity and the heat generation caused by the neutron exposure, gamma-ray exposure and massive concrete hydration in the biological shielding concrete of the JPDR.

OUTLINE OF THE JPDR CONCRETE

The biological shielding concrete of the JPDR was constructed with the Portland cement concrete as the mixing strength 35 MPa from the March to November 1962. The mixing proportion is shown in Table 1. The maximum thickness of the concrete was 3.0 m and had a lining of 13 mm thick steel plate on the reactor side surface and epoxy paint finishing on the outer side surface. Cooling pipes in order to reduce the thermal heat by radiation exposures and guide tubes of neutron monitoring were installed in the reactor side concrete.

Table 1 Mixing proportion

WATER TO CEMENT RATIO	SLUMP (cm)	UNIT WEIGHT(kg/m³)		FINE AGGREGATE (kg/m³)	COARSE AGGREGATE (kg/m³)	AIR-ENTRAINING AND RANGE WATER REDUCING ADMIXTURE (%)
		WATER	CEMENT			
0.47	7.5	139	290	653	1240	0.3

NOTE ; Cement : OPC , Fine Aggregate : Land Sand , Coarse Aggregate : Nakagawa River

The location of the core samplings were selected from the level of the reactor core as the high irradiation concrete, and the upper and lower distant level from the reactor core as the low irradiation concrete in order to clarify the influence of the radiation environment and the concrete placing. Figures 1 and 2 show the sampling location and the concrete core sample.

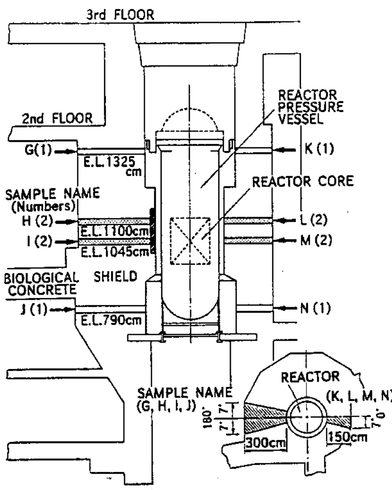


Fig. 1 Sampling location of the concrete core

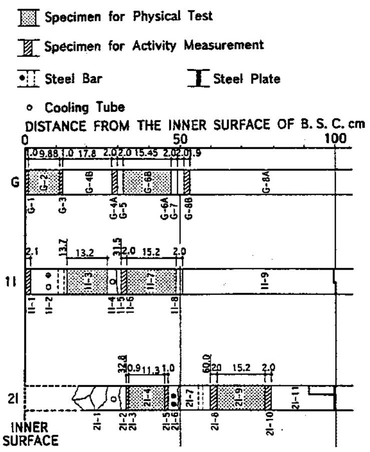


Fig. 2 Concrete core sampled

EVALUATION OF THE ENVIRONMENTAL CONDITIONS

Influencing environmental conditions to the biological shielding concrete are radiation exposure and heat of hydration in the hardening of the massive concrete after the placing. Figure 3 shows neutron flux and exposure dose distribution in biological shielding concrete at the level of reactor core obtained by calculation using computer code ANISN-JR. Neutron exposure dose for concrete samples used for the strength tests were evaluated by the calculation of the neutron flux and exposure dose distribution to the 2 dimensional R-Z cylindrical column model using the computer code DOT 3.5. Distribution graph of the

Fast neutron flux (E>0.11 MeV) and thermal neutron flux (E<1.85 eV) were obtained by the calculation using the computer code DOT 3.5. Neutron flux at the test specimens were decided by the distribution graph and the neutron exposure dose for the concrete samples were calculated multiplied with the converted dose for the rated reactor power operation time of the JPDR. Since the error of the calculated value of the thermal neutron flux from the measured one become larger according with the distance apart from the core center to upper or lower directions the calculation was corrected using the radioactivity of Eu-152 measured. Figure 4 shows comparison of the radioactivity of Eu-152 by the calculation and the measurement.

The maximum neutron irradiation dose rate to the biological shielding concrete were estimated that the fast neutron exposure rate was 1.11 × 10 E+18 n/cm2 and the thermal neutron exposure rate was 4.75 × 10E+17 n/cm2 at the reactor side concrete of the reactor core level. Figure 5 shows the gamma ray flux distribution in the biological shielding concrete obtained by calculation at the level of the JPDR reactor core. The maximum gamma dose rate obtained 4.77 × 10 E+8 Gy by the flux converted to the effective dose and multiplied with the total rated reactor power operation hours. Figure 6 shows the calorific value distribution that was calculated one dimensional transportation analysis using the computer code ANISN-JR generated by the total neutron and the total gamma ray in the biological shielding concrete at the level of reactor core. The calorific value in the biological shielding concrete at the reactor side biological shielding concrete was obtained 3.0 × 10 E-4 w/cm3 by neutron exposure, and 5.68 × 10 E-3 w/cm3 by the total of primary and secondary gamma-ray exposure. The contribution to the maximum calorific value in the biological shielding concrete that was obtained 5.98 × 10 E-3 w/cm3 by the total neutron and the total gamma-ray was largely contributed by the gamma-ray.

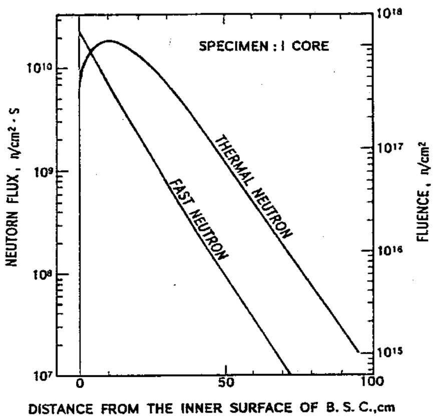


Fig. 3 Neutron flux and exposure dose distribution in the biological shielding concrete at the level of reactor core

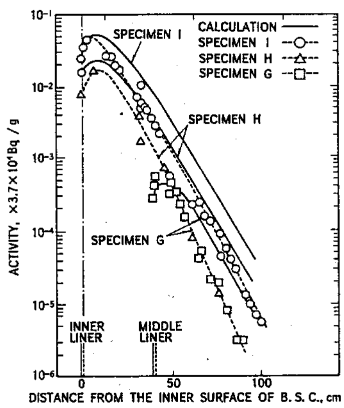


Fig. 4 Activity distribution in the biological shielding concrete at the level of reactor core (Eu-152)

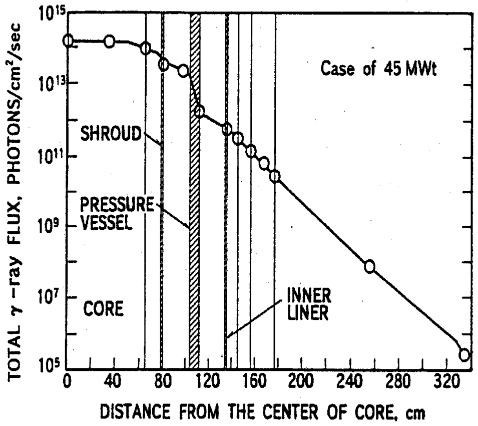


Fig.5 Distribution of total gamma-ray flux at the level of reactor core

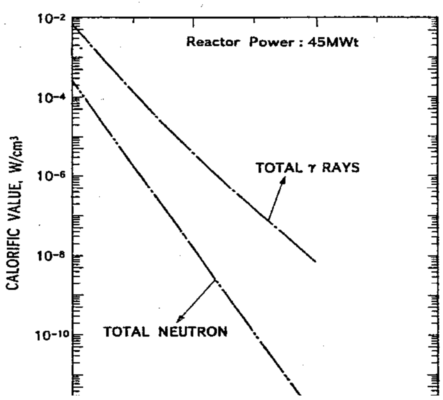
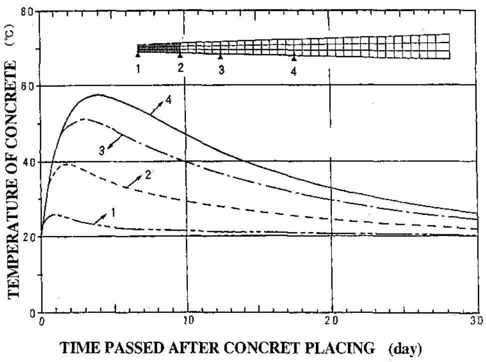


Fig.6 Calorific value distribution generated by total neutron and total gamma-rays at the level of reactor core

The distributions of the concrete temperature for both case of the hydration heat during cure after concrete placing and the radiation exposure heat during the operation were calculated. The results were shown in the Figures 7 and 8, respectively. As shown in the figures, biological shielding concrete were thermal influenced by the heat of hydration in the early stage and heat of radiation exposure during the reactor operation.



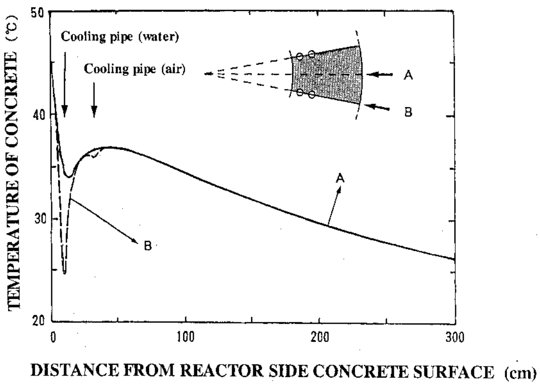


Fig. 7 Temperature distribution of the biological shielding concrete by the heat of hydration at the placing

Fig. 8 Temperature distribution of the biological shielding concrete in the operation

METHOD AND RESULT OF THE TESTS

Test on the mechanical properties of the irradiated concrete

In the tests on the mechanical properties of the concrete, compressive strength, tensile strength, modulus of static elasticity, and Poisson's ratio were investigated. The test specimens were shaped the core samples in the size of 8 cm diameter and 16 cm height for the tests of compressive strength, modulus of elasticity and Poisson's ratio, and in the size of 8cm diameter and 8cm height for the tensile strength tests. The concrete specimens were cured in the water 24 hours before the tests. The modulus of static elasticity were evaluated by the stress-strain ratio at the one third of the maximum stress on the stress-strain curve obtained by the compression gauge and the strain gauge in the compressive strength tests. The Poisson's ratio was evaluated from the strain ratio of lateral to longitudinal in the linear strain range.

Figure 9 shows the distribution of the compressive strength along with the depths from the reactor side to outer side concrete. The compressive strength of the concrete core samples was distributed from the range of 29.4 to 53 MPa (average 44 MPa) and the average was 20 % larger than the strength of mixing proportion. The compressive strength showed the tendency to increase along with the fast neutron fluence increased in the range from 1×10E+13 n/cm2 to 1×10E+17 n/cm2 when it was looked in the relationship between the compressive strength and the fast neutron fluence calculated as shown in Figure 10. Since the compressive strength was influenced by the over burden of the placing height of the flesh concrete at the construction, the compression strength were converted to the concrete placing height at 0 cm.

In the previous study, the compressive strength did not decrease in the range of the radiation dose rate 2×10 E+18 to 2×10E+19 n/cm2 but when the irradiation dose rate is increased to over the 5×10E+19 n/cm2 the compressive strength is decreased significantly, and also the compressive strength of the concrete have been said to decrease by the Gamma irradiation accumulated approximately over 10E+9 Gy (Hilsdorf, H.K. et al.). In our tests, however, the maximum fast neutron and gamma ray dose rates at the reactor side concrete are 1.11×10E+18 n/cm2 and 4.77×10E+18 Gy, respectively and it could not confirm the tendency.

Figure 11 shows the relationships of the modulus of static elasticity and neutron fluence. The influence of the neutron fluence to the modulus of elasticity was not shown as the figure shows. The relationship of the modulus of static elasticity and compressive strength were largely distributed around the AIJ curve of the relations as shown in the Figure 12.

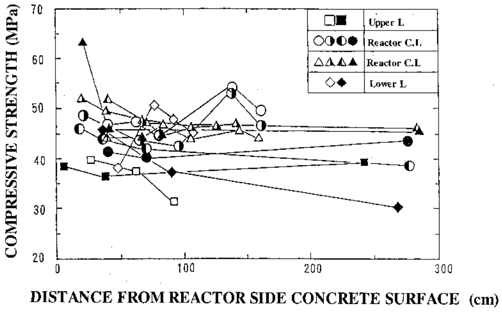


Fig. 9 Compressive strength along with concrete depth

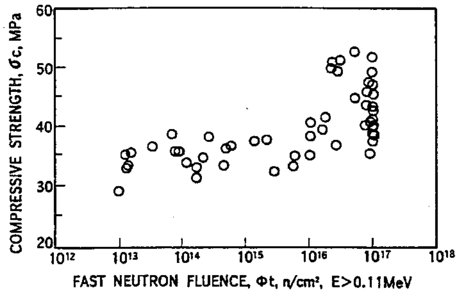


Fig. 10 Compressive strength distribution along with fast neutron fluence

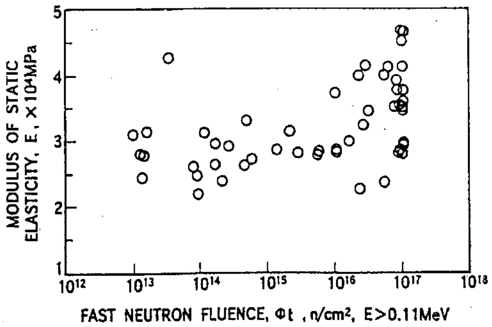


Fig. 11 Modulus of static elasticity along with fast neutron fluence

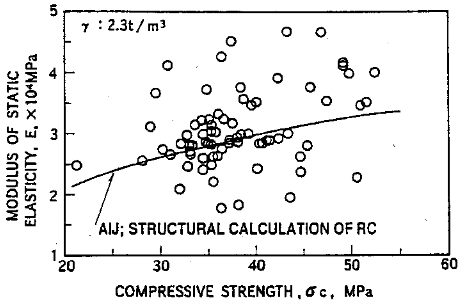


Fig. 12 Relationship of compressive strength and modulus of static elasticity

Figures 13 and 14 shows the relationship between Poisson's ratio and fast neutron fluence and the relationship between tensile strength and fast neutron fluence, respectively, obtained by the strength test of the concrete core samples. The influence of the fast neutron fluence to the Poisons ratio and the tensile strength were not shown in the irradiation range of the test samples.

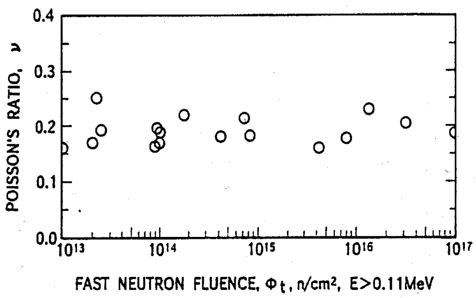


Fig. 13 Poisson's ratio along with neutron fluene

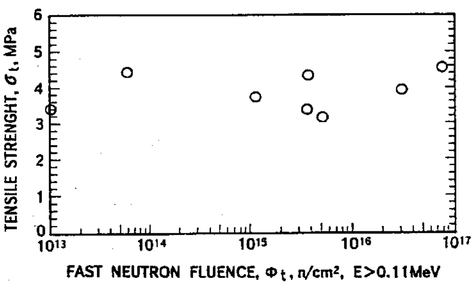


Fig. 14 Tensile strength along with fast neutron fluence

Test on the chemical properties of the irradiated concrete

In order to investigate what influences were affected to the concrete microstructure components by the irradiation, the tests of the chemical properties of the concrete core samples were performed. Tests items were chemical element analysis, X-ray diffraction analysis, scanning electron microscope observation, porosity measurement, water of crystallization measurement, and differential heat analysis.

Chemical element analysis: The analyses were performed regarding the nine principal elements of Si, Ti, Fe, Al, Mn, Ca, Mg, Na, and K by the method of the Inductively Coupled Plasma Spectrometry. The oxidation products of the nine principal elements were matched almost the same between the concrete samples of reactor side and outer side as expected.

X-ray diffraction analysis: The analyses were performed using the mortar specimens crushed into the diameter less than 45 that were placed uniformly on the glass plate and the diffraction angles were measured. The spacing of the crystal faces were obtained input the refraction angles into the Bragg's condition and the crystals of the specimen were identified comparing with the standard samples. The influences of the radiation dose effect to the crystallization in the concrete microstructures were not shown from the diffraction pattern.

Scanning electron microscope: The observations were performed on two scanning field for one mortar specimen using the mortar made by roughly crushed the concrete specimen after vacuum drying in scales up to 500, 1000, and 3000 times larger by the scanning electron microscope. From the scanning electron microscope observation at the pore where hydration crystal growth was observed the specimen of reactor side showed the large growth of the needle crystal than outer side.

Porosity measurement: The micro-pore size of the mortar extracted from the crushed concrete specimen were measured in the range of 60 to 99,000 + using porosity gauge by the penetration method of pressurized mercury into the mortar in the pressure range from 0.9 to 2000 kg/cm2. The micro-pore diameter distribution of the reactor side concrete was distributed to the small size region than the outer side concrete. The peak of the micro-pore distribution of the both side concretes were shown at the diameter 130-250 +.

Bound water measurements and differential thermal analysis: They were performed using the micro-crushed concrete specimen to the size of 45 . The quantities of the water of crystallization were obtained from the differences after heated the micro-crashed specimen one hours each at the temperatures of 105 , 400 , 650 and 950 by electro heater. The quantity of the free water measured from the loss of weight at the heating temperature 105 in the reactor side concrete showed 12% larger than the outer side concrete. The differential thermal analysis results were shown not much difference between the reactor side and outer side concrete.

Comparison tests on the mock-up concrete

The test concrete were exposed the thermal condition by heat of hydration during the massive concrete placing and the heat of irradiation during the reactor operation. In order to clarify the influences of the irradiation effect and the environmental thermal effect, reference tests were performed on the physical, mechanical and chemical properties with the same test items using mock up specimens simulated the environmental thermal conditions that are the heat of hydration and the radiation exposure heat as to become the same conditions of the actual concrete samples. The test specimens were made to represent four cases of environmental conditions as shown in the Figure 15. The test cases represent the environmental conditions of the actual concrete that are massive concrete cured in an air as case 1, massive concrete cured in an air and radiation exposure as case 2, normal concrete cured in a water as case 3, and normal concrete cured in an air as case 4.

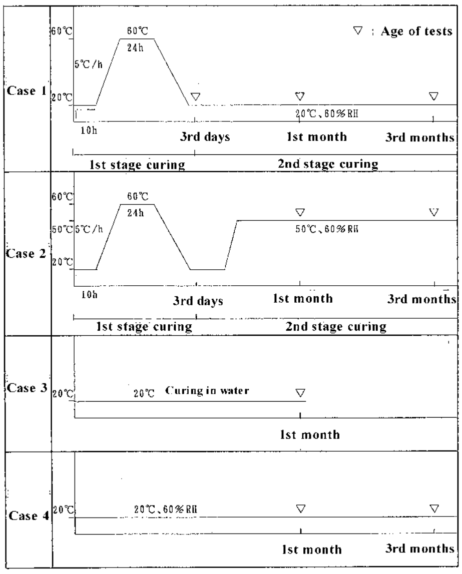


Fig. 15 Curing conditions of the mock-up test cases

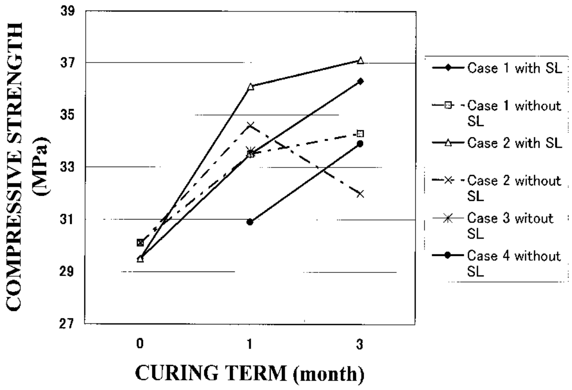


Fig. 16 Compressive strength of the mock-up concrete tests

The test results showed that the effect of the steel lining brought good cure conditions for the concrete in the both cases 1 and 2 to increase the compressive strength along with longer the curing term as shown in the Figure 16. While the compressive strength of the mock up concrete without steel lining showed the increasing rate of the long term strength became slower, especially, it were decreased significantly in the case 2 by the influences of long term heating.

The scanning electron microscope and the X-ray diffraction analysis test results showed not much difference between the cases 1 and 2, but the porosity test results showed the pore size distribution of the both cases 1 and 2 were shifted to the larger size in the case of without steel lining while in the case of with lining it is not changed. The heat of hardening of massive concrete as represented the curing condition in the early stage of the cases 1 and 2 were increased the strength generation at the age of one month as large as the same age strength of the case 3. Then the heat of second stage in the case 2 to represent the condition of the heat by radiation in the operating stage was affected to the concrete without lining to dry the concrete slowly and evaporate the water content, coarse the hardening body organization, and increased the total volume of the micro pores, that seemed to made the strength decrease at the age of 3 month than at the age of one month. The specific gravity of the concrete with steel lining did not change according with the curing term but the concrete without shield lining decreased according with the curing term increased in the both cases 1 and 2.

CONCLUSION

The evaluation of the radiation exposure dose effect to the concrete structures in nuclear power plants is important factor for the plant life management strategy considering aging of the concrete properties during plant operations. The study using the actual concrete samples from the JPDR biological shielding concrete was performed when the plant was decommissioning. The major results of the study is summarized as follows, The maximum fast neutron and gamma ray dose rates at the reactor side of the biological shielding concrete are 1.11 × 10E+18 n/cm2 and 4.77 × 10E+18 Gy, respectively, at the level of the reactor core which is almost the same level as the one after 40 years operation of current commercial nuclear power plants.

The biological shielding concrete are categorized two featured environmental conditions influenced to the material properties that are 1) reactor side concrete with steel lining, and, 2) outer side concrete without steel lining. These conditions were generated relatively high temperatures of concrete by the heat of hydration of massive concrete in the early stage and the long-term heat by radiation in the reactor operation.

The generation of heat within the biological shielding concrete by the hydration, neutron and gamma-ray exposures caused large influences to the concrete properties and it were appeared as the variations of the compressive strength, the modulus of static elasticity, and the pore size distributions. Poisson's ratio was not shown the influence by the fast neutron dose rate.

The influences of the radiation exposure to the microstructures of concretes were appeared mainly in the behavior of the water contents by the long term heating of radiation exposure and it caused the water dissipation slowly from the outer side concrete without lining. While, the reactor side concrete showed good durability for the water dissipation by the steel lining even though the decomposition of water contents by neutron exposure as suggested from the high radioactive tritium generation in the reactor side concrete.

The compressive strength distribution of the biological shielding concrete were matched the tendency to increase along with the fast neutron fluencies increasing within the test range to 10E+17 n/cm2. The negative effects of the radiation dose to decrease the compressive strength of the concrete were not appeared within the dose range of the test samples to 10E+17 n/cm2.

Scanning electron microscope observed the large crystal growth of the ettringite in the reactor side concrete. In generally, the hydration of the Portland cement is said that aluminate phase C_3A and gypsum $CaSO_4$-2H2O generate the ettringite C_3A-3$CaSO_4$-32H_2O in the early stage and then it is changing to the monosulfeto C_3A-3$CaSO_4$-12H_2O. Large growth of ettrigite is said to swell the concrete organization and to obstruct the strength generation. The large crystal growth of the ettringite in the reactor side concrete is contradict with the general tendency. It is suggesting that there are still unknown factors about the influences of the radiation exposure to the concrete properties.

ACKNOWLEDGEMENT

This report was summarized the past study reports performed by the research people in the Japan Atomic Research Institute and the Kajima Corporation when the JPDR was decommissioning. The participants in the study were greatly appreciated.

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The use of composite materials in the strengthening of nuclear concrete structures

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Abstract:

Due to the normal ageing of the civil structures, after more than 20 years of plants in operation, for the oldest, Edf has examined all the capabilities of evaluation and repairing available on the "market". The composite system appeared very interesting and his use is now a major concern for EDF as well for the leak tightness recovering in concrete structures or strengthening of metallic or concrete structures.

EDF has investigated composite liner materials in order to evaluate their feasibility as a replacement for metallic liners and whether they can be added to unlined prestressed concrete walls in order to improve their leaktightness.

Several liner systems were selected and qualified at the laboratories of EDF in France. The composite liner systems typically consist of a matrix of vinyl ester or epoxy resins in which different layers of glassfibres are embedded. The application to the concrete surface is performed manually, layer by layer.

The tests at MAEVA were carried out in order to demonstrate the suitability of such composite liner systems. Up to march 2000 the following tests had been performed at MAEVA : three test series in air and vapour at design pressure and liners proposed by the companies Max Perlès (epoxy),

Based on the results obtained during the different test campaigns the composite liner systems were improved with respect to their adhesion to the concrete and resistance against alkaline water.

On the other hand, the applicability of composite system has been studied with French company Freyssinet in order to expand the solution range for repairing or strengthening concrete structures. The general idea is to substitute to traditional steel reinforcement, located inside the concrete, coated layers, compound with composite fibres (like carbon or similar material), located on the appropriate outside concrete face. This system has been used several times for strengthening masonry walls, which wasn't designed against earthquake, for strengthening steel vertical cylindrical tanks against dynamic buckling failure mode, for strengthening cooling towers presenting evolutionary cracks at the top of the shell. This methodology has been validated both by experimental approach combined with numerical simulation. Moreover, different working group have been constituted in the general context of civil works in France in order to provide design rules, applicable to all concrete structures.

1. Use of composite liners for leaktightness improvement of concrete structures

The watertighness improvement by the use of composite liner is common for structures as water tower but is innovating in the nuclear industry. In the 70's, composite coatings has been applied in CANDU plants; a similar concept has been considered in the EPR project for the containment, wich is of the same type used for CIVAUX 2 (double-walled unit, dynamic containment and High Performance Concrete) seeRef[9].

The safety objective for the internal wall of the containment is to guarantee that the releases are limited to 1% per day of the mass of gas contained in the containment under accidental conditions. This corresponds to a design pressure (leak-rate expressed as a volume) that is clearly more severe than for an N4 unit (1.5% with a lower design pressure).

This objective must be met without the use of a leaktight liner. Improvements have therefore been made to the N4-type containment, particularly on the design plan by imposing more severe criteria (for creep, stress limitations, etc) in the case where the accidental situations lead to reinforcement of the steels and prestressing around the equipment hatch, and with use of High Performance Concrete and a new prestressing system (55C15 units). Furthermore, the rapid combustion of hydrogen must be taken into account in the scope of the EPR project.

The composite liner provides margins for beyond-design conditions : the application of such a liner on the most permeable areas or those susceptible to being permeable %has therefore been envisaged to improve the leaktightness of an internal wall in an accidental situation.

An initial specification of the qualification profiles established during the "Basic Design" phase has allowed :

- A certain number of liners to be preselected.
- An experimental qualification campaign to be conducted, constituted by "elementary" and/or "complex" tests in laboratories, completed by industrial applications.

The possibility of qualification of such a liner has been considered favourable following the results of different test campaigns which are, however, to be improved (adhesion capacities and liner crack-bridging to be optimised, definition of a new criterion of the crack width to be bridged, limitation of the thermohydraulic test for the liner surface, optimisation of the Severe Accident sequence to define the temperature curves with respect to liner qualification).

On the basis of results acquired from the BD (Basic Design) phase, and those from the optimisation studies, the qualification requirements have been refined in the BDOP (Basic Design Optimisation Phase).

2. Test programme

2.1. Description of situations considered

The product qualification tests are defined in the context of the EPR project, in accordance with different types of situations perceived and corresponding safety requirements for the liner (air leaktightness greater than that for concrete alone, solidarity to support during leaktightness test and thermohydraulic test under accidental conditions, non-liberation of debris under the pressure and temperature conditions of an accidental situation).

As the accident can occur at any moment, the most severe situations for the liner are those when the accident takes place either at the start or at the end of the lifetime of the nuclear power unit (60 years).

The complete qualification programme includes in particular :

- Physico-chemical identification tests (density, dry extract, ash content, infrared spectra), mechanical test (fresh state, aged state), and determination of the vitreous transition temperature. No criteria are specified for these characterisation tests,
- Tests of aptitude for use in the BR (reaction to fire, PMUC),
- Functional tests for fresh state (adhesion, permeability under tension, resistance to cracking, resistance to thermohydraulic test, heat behaviour),
- Study of behaviour under irradiation (ageing irradiation corresponding to a normal service of 60 years, accident irradiation).

- Functional tests after ageing (permeability after ageing and accidental irradiation, crack-resistance tests after ageing and accidental irradiation + thermohydraulic test).

With respect to system selection, an initial prequalification test phase has proved to be necessary.

Taking into account the irradiation period, the prequalification tests are conducted on fresh-state products without ageing nor accidental irradiation.

2.2. Specifications of the prequalification tests

The liner generally being composed of several layers, the tests are conducted on the complete system (with the finishing coat if specified in the supplier's technical sheet), following the configurations representing actual functioning situations as best possible.

The concrete support samples are made from High Performance Concrete (composition of CIVAUX 2 type), conforming to the hypothesis taken for the material of the internal containment.

2.2.1. Test descriptions

2.2.1.1. Identification

Physico-chemical, mechanical and vitreous transition temperature identifications are characterisation tests allowing the consistency of the manufacturing quality of the products used for the production of the liner to be checked, during subsequent in-situ productions (type of identity card for the product).

2.2.1.2. Functional tests

a) Types of tests

In order to check the aptitude of the liner to satisfy accidental and post-accidental design conditions, the functional characteristics measured for the prequalification tests are the resistance to thermohydraulic test, leaktightness, resistance to cracking, adhesion to the support and heat behaviour.

All tests are performed on the liner in its fresh state and without irradiation, some tests being conducted both before and after the thermohydraulic test. Depending on the type of test, steel and/or concrete supported specimens or liner on its own are planned.

b) Test chronology

The functional tests are conducted in the order specified below :

1) Before thermohydraulic test :
   - Adhesion
   - Permeability without tension
   - Resistance to cracking
   - Heat behaviour

Following this test, at least one specimen is subjected to the thermohydraulic test, and the others to an adhesion test.

2) Thermohydraulic test :
   - Adhesion
   - Permeability without tension
   - Resistance to cracking

3) Test of aptitude for use in the BR :
   - Reaction to fire
   - PMUC
   - Contamination/decontamination

These tests will be performed subsequently if the results of the previous phase (functional tests) are satisfactory.

2.2.2. Criteria

No criteria are specified for the identification tests.

Due to the role it performs in the containment, it is advisable to check that the composite liner is capable of assuring its leaktightness function and that it remains integral and attached to the wall (at least on the areas previewed to this effect) for all configurations.

2.2.2.1. *Thermohydraulic test*

No alteration involving blistering, cracking/decladding, flaking, checking, cracking or rusting must be observed during the visual examination perfomed on specimens immediately after their removal from the autoclave.

2.2.2.2. *Cracking strengthening*

It is important that the shear resistance with respect to the sample is lower than the lowest resistance measured during the tensile test on the liner by itself (in the intermediate sense to the principal directions of the fibres).

A lower limit of fresh-state crack-bridging of 1.5 mm is required, taking into account the deformation envelope adopted for this containment in High Performance Concrete.

After the thermohydraulic test, no criterion is specified but the results are recorded until rupture.

2.2.2.3. *Adhesion*

The acceptable adhesion value for a fresh state liner is greater than 2 MPa, or the resistance of the support.

After the thermohydraulic test, it is important that the liner remains attached to its support in the areas of attachment on the concrete and the metal. A value of 0.5 MPa after the thermohydraulic test is specified.

2.2.2.4. *Permeability*

The formula giving the limit value of the rate is calculated in such a way as to limit the leaks during a test in air or an accidental test.

For permeability tests in air on the liner alone, there are the following criteria:

- In fresh state : the maximum rate is 0.15 $Nl/h/m^2$
- After thermohydraulic test : the maximum rate is 1.5 $Nl/h/m^2$

2.2.2.5. *Heat behaviour*

No debris liberation must be observed and the liner must remain in place on the facing of the containment.

No criteria are specified for the thermohydraulic test or the adhesion tests, these tests being conducted for informative purposes.

2.2.2.6. *Reaction to fire*

During these reaction to fire tests, the maximum classification index M2 is required for the liner, just as the index F2 represents smoke opacity.

2.2.2.7. *PMUC*

The liberation of sulphur and halogens must be less than 1000 ppm (measured by the leaching test).

2.2.2.8. *Contamination/decontamination*

The decontamination aptitude must be greater than or equal to 85 % and the susceptibility to contamination less than 20 %.

3. Strengthening Of Structures With CFRP In Nuclear Plants – Tests And Applications

3.1. Introduction

One of the most important challenge of EDF for the next years is to provide or to prolong the duration of life of its facilities and particularly of nuclear plants. To succeed, an unavoidable problem to be solved is the maintenance of the integrity and the functionality of the Civil Engineering structures. The need for repairing such structures is always due to at least one of these three factors: natural ageing of materials, a weak point towards rules or initial design and the evolution of requirements due to changes in Safety Rules or to developments of new calculation methods. EDF has examined all the capabilities of evaluation and repairing solutions available on the "market" for the ageing of its plants. The composite system appeared very interesting for metallic or concrete structures strengthening. In this paper, one of the composite system is first presented and its use in nuclear facilities briefly described. More precisely, it concerns the strengthening of masonry walls, which were not designed against earthquake, and the strengthening of steel vertical cylindrical tanks against dynamic buckling failure mode.

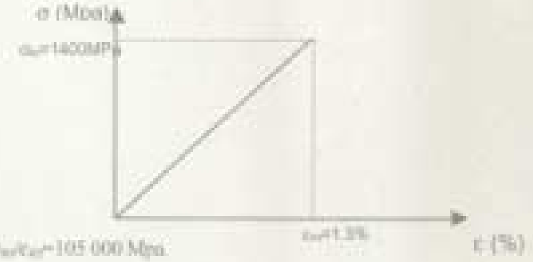
3.2. The composite system for the structures strengthening: Carbon Fibre Reinforced Polymer - TFC®.

TFC® is made up of carbon fibres and synthetic resin for their soaking and sticking. The resin is produced by ATOFINDLEY company. The fibres by SOFICAR and TFC® is a FREYSSINET product. It is composed by 40% of fibres and 60% of resin. The characteristics of these constituents lead to the TFC® following minimum guaranteed mechanical characteristics :

AVERAGE THICKNESS	0.43mm
TENSILE AT RUPTURE	1400Mpa
MODULUS	105GPa

The theoretical behaviour of strengthening by TFC® is based on the French reinforced concrete rule, BAEL 91, using Limit States concepts. TFC® is considered as steel reinforcement with its specific constitutive law. For the case of masonry wall strengthening, the classical hypothesis are preserved: masonry in traction is neglected, sections stay plane, TFC® grips to masonry…

The repairing design uses the following idealised constitutive law for the TFC®.



Young modulus $E_f = \sigma_{fu}/\varepsilon_{fu}$ = 105 000 Mpa.

σ_{fu} et ε_{fu} represent respectively the guaranteed tension and deformation at TFC® rupture.

3.3. Validation of the seismic strengthening by TFC® – Example of RC walls strengthening and applications in nuclear plants

Many tests whose aim were to qualify TFC® system for different configurations or behaviours were carried out during the last four years. They cover test on beams, columns, bearing walls submitted to bending or shear. All these tests allowed FREYSSINET to validate TFC® system and to write a technical rules approved by the Control French company SOCOTEC. The main tests supported by EDF concerning seismic behaviour are briefly presented in this part of the paper. Some of them were directly linked to an application of a strengthening in nuclear plants such as in Fessenheim.

3.3.1 'SAFE' Tests

The 'SAFE' programme (Structures Armées Faiblement Elancées – Stocky reinforced structures) aimed to characterise by experimentation the dynamic behaviour of load-bearing shear walls constructed from reinforced concrete and with a low slenderness ratio. The experimental programme carried out in the European laboratory ELSA in Ispra Italy, consists of a series of 13 pseudo-dynamic tests on models loaded in pure, in conjunction with behaviour tests on the materials used to construct these models. The last test was a replica of the eighth one ; this complementary test allowing the application of a reinforcement to both faces of the wall by horizontal and vertical bands of the TFC®. The results have shown that the strengthening improved the ultimate capacity of the wall by 47%.

Figure 2.1: SAFE wall

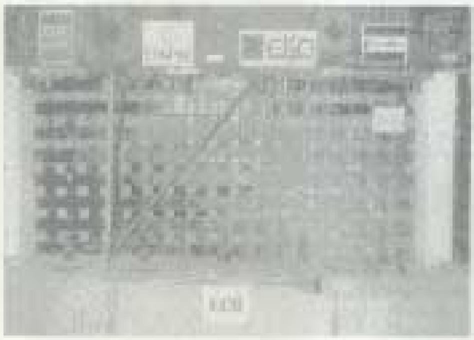


Figure 2.2: SAFE wall strengthened by TFC®

3.3.2. « CAMUS » Tests

Reinforced concrete bearing walls with limited reinforcement ratio are commonly used in France for the building structures. The purpose of CAMUS program was the experimental characterisation of such reinforced concrete slender walls dynamic behaviour submitted to bending. Two models were tested on the shaking table of the CEA (Commissariat à l'Énergie Atomique) in Saclay, France. The first model was damaged until its collapse for a seismic level of 0.7g and then repaired with TFC®. The repaired model withstood the same signals which led to failure of the non-repaired structure. The collapse of the repaired model happened at 0.8g. This improvement is substantial considering that the strengthening was not preventive but applied on the damaged model whose many sections had reached yielding and some steel longitudinal reinforcements were broken.

Figure 2.3: CAMUS model on Saclay shaking table

3.3.3. Applications of TFC in Nuclear Plants

This part presents two original applications of TFC® in Nuclear Plants. As a matter of fact these applications are not standard such as some general applications on RC beams, columns or slabs in which TFC® typically replaces steel reinforcement. They concern the strengthening of masonry walls against out of plan failure and steel vertical cylindrical tanks against dynamic buckling failure mode.

3.3.3.1. *Strengthening of masonry walls*

For this application the problem was not the seismic behaviour of masonry walls themselves which are not considered important for the safety of the plant but the potential damages which can be caused to important equipements by the fall of breeze-blocks due to the out of plan failure of masonry walls. This problem has concerned few nuclear plants such as Fessenheim which were designed before the appearance of special requirements for masonry walls against earthquake. In the French rules PS92, geometric criteria are specified for masonry walls and the out of plan stress is limited at 0.3Mpa on the tense face of a non reinforced masonry wall. These criteria were not filled for some walls of the oldest plants in France (The more recent plants are designed with reinforced masonry walls specially against earthquake)

To qualify the use of TFC® on masonry walls a test program was carried out in SNCF (Société Nationale des Chemins de Fer) laboratory. More precisely, the aim of this program was to study the global behaviour of masonry walls with different strengthening configurations such as different space between TFC® bands or TFC® anchorage configurations, and to validate a theoretical model for the strengthening calculation.

The experimental program consisted of 4 tests: one for the reference (non strengthened wall – Figure 2.5.a), and three other tests of walls strengthened as shown in the figures 2.5.b to 2.5.d. The testing bench is illustrated by the figure 2.4.

Mechanical characterisation tests on the blocks with coating and paint lead to a tensile at rupture of 0.5Mpa based on SATEC tests.

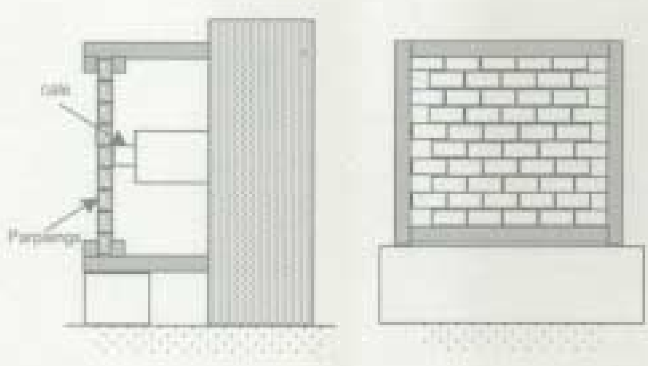


Figure 2.4: Testing-bench for out of plan masonry wall behaviour

The results of the tests are summed up on the figures bellow and discussed after. Note that for the first one, test 0, the limit conditions were different and the equivalent value of failure load (if the limit conditions were the same as for the other test) is noted Q_{rupt0}^*. For the other tests i=1 to 3 the failure load are noticed Q_{rupti}.

Figure 2.5.a

Q_{rupt}=50kN - Q_{rupt}'=19.5kN

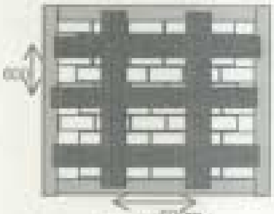


Figure 2.5.c

Q_{rupt}>145kN

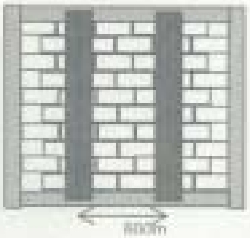


Figure 2.5.b

Q_{rupt}=70kN

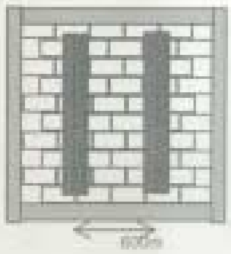


Figure 2.5.d

Q_{rupt}=55kN

The strengthening by TFC® bands anchored in a metallic frame improves the flexure capacity of the wall by at least 4 (for horizontal and vertical bands the improvement factor exceeds 7 – for test 2, the testing bench capacity was not sufficient to lead to wall failure). For bands directly anchored on the masonry blocks, the failure was reached for a load equal to Q_{rupt}=55kN=2.5x Q_{rupt}' - the flexure capacity of the tested wall was then improved by a factor 2.5

3.3.3.2 *Strengthening of steel vertical cylindrical tanks*

For this application the problem was the dynamic buckling failure mode during earthquake of tanks 16m in height considered as important for the safety of the concerned nuclear plant. Using the mechanical minimal characteristics of the Design rules recommended for the French nuclear plants in the RCC-M rules, calculations lead to the buckling of tanks at 55% of the EDF design spectrum. Using real characteristics of the tanks steel, the buckling appears at the level of the EDF design spectrum. This situation was considered as unacceptable by the French Nuclear Authority which demands EDF to strengthen the tanks to achieve seismic margin on these important structures. Calculations and tests on models to scale 1/3 were performed in parallel. The testing-bench is represented by the figures 2.6.a and 2.6.b. The experimental program consisted of 4 tests of models in order to qualify the type of strengthening to be used. The results of these tests and their comparison with predictive calculation are summed up in the following table:

Model	Strengthening		Moment at the base	
	layer	phase	Calculations	Tests
1	0	Empty	702	594
2	1	Empty	1080	933
3	1	Full of water	1026 *	867
4	2	Full of water	1286	1145

These calculations were performed without taking into account defaults of tanks and defaults of limit conditions during tests. After these corrections, calculations lead to conservative results and fit very well experimentation. These corrected calculations were used to design the real strengthening of the tanks which finally conduces to stick a decreasing number of layer with height: 7 layers from 1m to 4.5m – 4 layers from 4.5m to 7.5m - 2 layers from 7.5 to 9.8m. With this strengthening, using the mechanical minimal characteristics of the Design rules recommended for the

French nuclear plants in the RCC-M rules, calculations lead to the buckling of tanks at 1.31 times the level of EDF design spectrum. Using real characteristics of the tanks steel, the buckling appears at 1.62 times the level of EDF design spectrum.

Figure 2.6.a

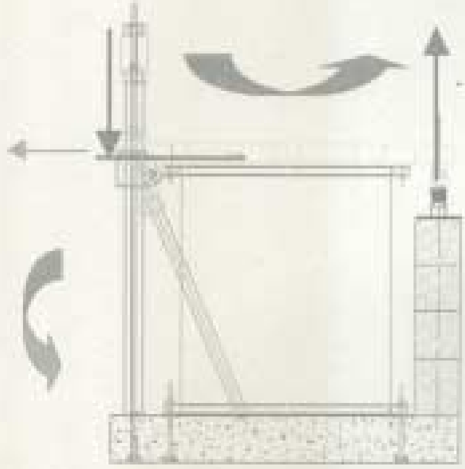


Figure 2.6.b

3.3.3.3. *Strengthening of cooling tower shell*

The concrete of some cooling towers present cracks at the top of the shell due, probably, to hydro_thermal and functional cycles. In order to prevent higher deformations, it is planned to put around the shell (near the top) some layers of TFC. This study is on the way and the reparation is foreseen for the next year.

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DETECTION OF REINFORCEMENT CORROSION AND ITS USE FOR SERVICE LIFE ASSESSMENT OF CONCRETE STRUCTURES

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1 INTRODUCTION

Corrosion of reinforcement is one of the main durability problems of concrete structures. The corrosion is induced by two main factors: the carbonation of the concrete cover and the penetration of chlorides providing from marine atmosphere or from chemicals in contact with concrete. Carbonation generally aims into uniform corrosion of the steel bar while chlorides mainly induce localised corrosion. Both types of corrosion are of electrochemical nature.

There is a third type of corrosion named stress corrosion cracking, SCC, whose mechanism is not entirely electrochemical, but the mechanical stress co-operates for its development. This last type will not be considered in present paper.

Reinforcement corrosion is not a common problem in nuclear power plants due to the limited life of these installations, except in case of cooling towers where frequent corrosion problems have been noticed. It can be however a key aspect to be taken into account when dealing with extension of power plant service life. It is as well a very relevant aspect in long term storage or repository installations where lives beyond 300 years are usually targeted.

In present paper, it is described first how to measure reinforcement corrosion in order to obtain the corrosion rate of the steel. Then, the effects of the evolution of corrosion are listed, in order to be considered as limit states and therefore indicators of repair criteria. Finally, a 3D model is presented on the possible release of chlorides being part of the low and medium radioactive wastes stored in drums. These chlorides may diffuse through the surrounding cement mortar and reach the reinforcement of the concrete containers used to encapsulate the drums. The model is part of a general one that will include not only the ionic diffusion, but also the corrosion of reinforcements and its evolution. This type of models, although theoretical and simple, will help to understand the long-term performance of concrete structures regarding the corrosion of reinforcements.

2 ON-SITE TECHNIQUES FOR CORROSION MEASUREMENT

2.1 Corrosion Potential and resistivity maps.

Up to the present the main techniques used on-site for appraising corrosion of reinforcements are of electrochemical nature due to that is the basis of the corrosion process.

Because of its simplicity, the measurement of Ecorr (rest or corrosion potential) is the method most frequently used in field determinations. From these measurements, potential maps are drawn which reveal those zones that are most likely to undergo corrosion in the active state[1]. However, such measurements have only a qualitative character, which may make data difficult to be interpreted[2]. This is due the potential only informs on the risk of corrosion and not in its actual activity. In addition, the developing of macrocells may as well mislead the deductions because corroding zones polarize the surrounding areas, which may seem corroding as well, when they are cathodic areas of the macrocell. In spite of which potential mapping still has a function to accomplish as a qualitative indication of the general performance and a complement of the other on-site techniques.

The same that said for the potential can be stated on Resistivity, ρ, measurements[3], which sometimes are used jointly with Ecorr mapping. The ρ values indicate the degree of moisture content of the concrete, which is related to the corrosion rate when the steel is actively corroding, but which may mislead the interpretation in passive conditions. On figure 1 is represented a risk map of a slab. The risk level has been calculated by a combination of these two parameters: Ecorr and ρ.

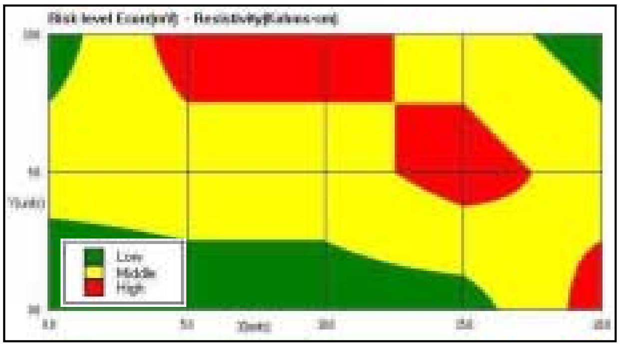


Figure 1: Corrosion risk map on a reinforcement slab calculated from the combination of Ecorr and ρ measurements.

2.2 Polarization Resistance

2.2.1 Laboratory measurements

The only electrochemical technique with quantitative ability regarding the corrosion rate is the so-called Polarization Resistance, R_p[4]. This technique has been extensively used in the laboratory. It is based on the application of a small electrical perturbation to the metal by means of a counter and a reference electrode. Providing the electrical signal is uniformly distributed throughout the reinforcement, the $\Delta E/\Delta I$ ratio defines R_p. The corrosion current, I_{corr}, is inversely proportional to R_p, $I_{corr}= B/R_p$ where B is a constant. R_p can be measured by means of D.C. or A.C. techniques[5], both of which have specific features in order to obtain a reliable corrosion current value in agreement with gravimetric losses.

2.2.2 On site measurements

Direct estimation of True R_p values from $\Delta E/\Delta I$ measurements is usually unfeasible in large real concrete structures. This is because the applied electric signal tends to vanish with distance from the counter electrode, CE rather than spread uniformly across the working electrode, WE. Therefore, the polarization by the electric signal is not uniform, and it reaches a certain distance that is named the critical length, Lcrit. Hence, $\Delta E/\Delta I$ measurements on large structures using a small counter electrode provides an apparent polarization resistance (R_p^{app}) that differs from the true R_p value depending on the experimental conditions[6]. Thus, if the metal is actively corroding, the current applied from a small CE located on the concrete surface is 'drained' very efficiently by the metal and it tends to confine itself on a small surface area. Conversely, if the metal is passive and R_p is high, the current applied tends to spread far away (e.g., around 50 cm) from the application point. Therefore, the apparent R_p approaches the true R_p for actively corroding reinforcement, but when the steel is passive, the large distance reached by the current needs a quantitative treatment.

Modulated confinement of the current (guard ring) method.

There are several ways of accounting for a True R_p value, among which the most extended one is the use of a guard ring[6], in order to confine the current in a particular rebar area, as Figure 2 depicts. The measurement is made by applying a galvanostatic step, lasting 30-100 seconds, from the central counter. Then, another counter current is applied from the external ring, and this external current is modulated by means of the two reference electrodes called “ring controllers” in order to equilibrate internal and external currents, which enables a correct confinement, and therefore, calculation of Rp. By means of this electrical delimitation to a small zone of the polarized area, any localised spot or pit can be first, localised, and second its measurement can be made by minimising the inherent error of Rp. Not all guarded techniques are efficient. Only that using a “Modulated Confinement” controlled by two small sensors for the guard ring control placed between the central auxiliary electrode and the ring, shown in figure 2, is able to efficiently confine the current within a predetermined area. The use of guard rings without this control leads into too high values of the I_{corr} for moderate and low values, and the error introduced in the case of very localised pits, is very high.

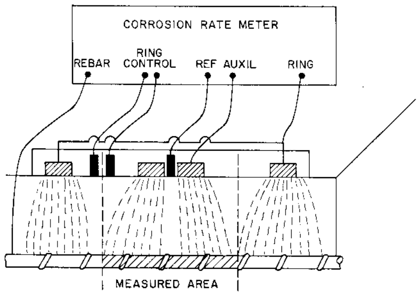


Figure 2: Modulated confinement of the current (guard ring) method

3 EMBEDDED SENSORS

The introduction of small sensors in the interior of the concrete, usually when placing it on-site is being one of the most promising developments in order to monitor the long-term behaviour of the structures. The most usual, as in the case of non-permanent on-site techniques, is to embed reference electrodes or resistivity electrodes. They can inform of the presence of moisture and on the evolution of corrosion potential. Others events that can be monitored are the advance of the carbonation or chloride fronts, the oxygen availability, temperature, concrete deformations and the corrosion rate.

A particular example of the use of embedded sensors is the case of storage facilities of low and medium radioactive wastes in El Cabril (Córdoba)[7]. There, a pilot container has been instrumented from 1995 by embedding 27 set of electrodes (Figure 3). The parameters controlled are: temperature, concrete deformation, corrosion potential, resistivity, oxygen availability and corrosion rate. The impact of temperature on several of the parameters is remarkable, and therefore, care has to be taken when interpreting on-site results.

Figure 3: Preparation of the embedded sensors in El Cabril

3 RANGES OF CORROSION RATE VALUES MEASURED ON-SITE

The experience on real structures[7] has confirmed the ranges of values previously recorded in laboratory experiments[4].

$\leq$0.2 μA/cm^2	**Negligible**
0.2 μA/cm^2 < Icorr < 0.5 μA/cm^2	**Low**
0.5 μA/cm^2 < Icorr < 1 μA/cm^2	**Moderate**
>1 μA/cm^2	**High**

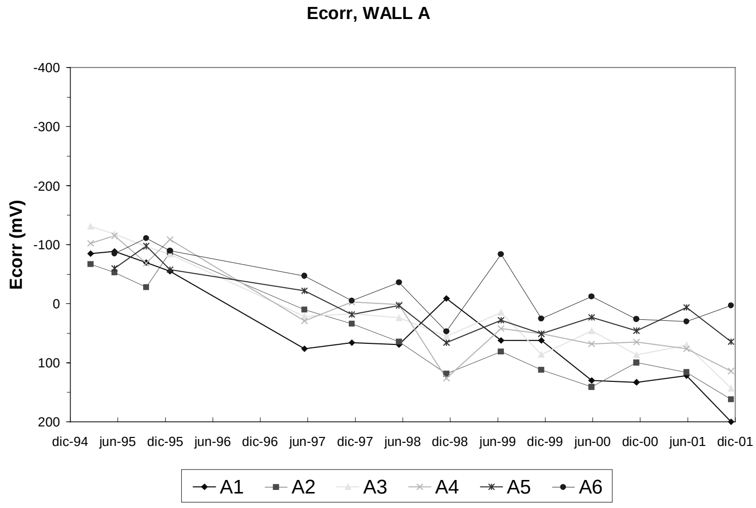
Table 1: Ranges of corrosion rate and risk levels.

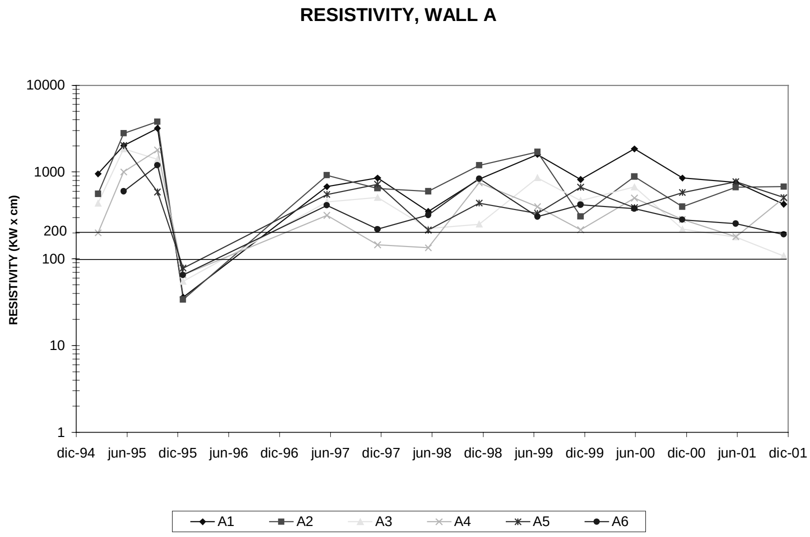
In general, values of corrosion rates higher than 1µA/cm^2 are seldom measured while values between 0.1-1µA/cm^2 are the most frequent. When the steel is passive very low values (smaller than 0.05-0.1µA/cm^2) are recorded.

A comparison of on-site I_{corr} values to electrical resistivity has allowed the authors to also rank the resistivity ones.

Practical measurements of on-site Icorr in El Cabril (Spain)

In order to control long term performance of concrete containers used for low and medium radioactive waste storage, Enresa is developing surveys of a set of parameters in the real structures. Corrosion rate of reinforcement is one among the parameters measured in the concrete cells in El Cabril – Cordoba – Spain. Figure 4 shows the results of Icorr measured during several years. The values indicate the passivity of reinforcement as expected.


Ecorr, WALL A
Ecorr (mV)
-400
-300
-200
-100
0
100
200
dic-94 jun-95 dic-95 jun-96 dic-96 jun-97 dic-97 jun-98 dic-98 jun-99 dic-99 jun-00 dic-00 jun-01 dic-01
A1 A2 A3 A4 A5 A6


RESISTIVITY, WALL A
RESISTIVITY (KW x cm)
10000
1000
200
100
10
1
dic-94 jun-95 dic-95 jun-96 dic-96 jun-97 dic-97 jun-98 dic-98 jun-99 dic-99 jun-00 dic-00 jun-01 dic-01
A1 A2 A3 A4 A5 A6

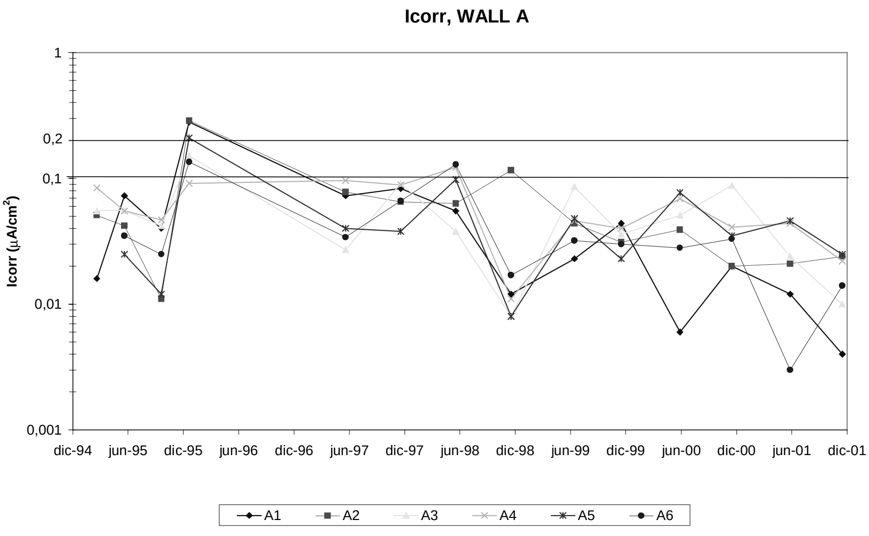

Icorr, WALL A
Icorr (µA/cm2)
1
0,2
0,1
0,01
0,001
dic-94 jun-95 dic-95 jun-96 dic-96 jun-97 dic-97 jun-98 dic-98 jun-99 dic-99 jun-00 dic-00 jun-01 dic-01
A1 A2 A3 A4 A5 A6

Figure 4: Results of Ecorr, resistivity and Icorr measured during several years over an internal wall of the container.

4 TRANSFORMATION OF I_{CORR} VALUES INTO CALCULATIONS OF LOSS IN BAR CROSS SECTION

Corrosion leads into four main structural consequences: 1)reduction of bar cross section, 2) reduction of steel ductility, 3) cracking of concrete cover and, 4) reduction of steel/concrete bond (composite effect). All these effects occurring in isolation, or simultaneously, will result in a loss in the load bearing capacity of the structure[8].

The primary information obtained from corrosion measurements is that concerning the loss in cross section of the bar. This parameter informs about all the other effects of the corrosion process. The attack penetration P_x is defined as the loss in diameter as is shows in Figure 4. It is obtained through the expression:

$$P_x = 0.0115 \cdot I_{corr}^{REP} \cdot t_p \tag{1}$$

Being t_p the time in years after corrosion started and 0.0115 a conversion factor of $\mu A/cm^2$ into mm/year (for the steel). This expression implies the need to know when the corrosion has started in order to account for $t_{p.}$

When the corrosion is localised (right part of figure 10), the maximum pit depth is calculated by multiplying expression (1) by a factor named α which usually takes a value of 10. Hence expression (1) above becomes,

$$P_{pit} = 0.0115 \cdot I_{corr}^{REP} \cdot t_P \cdot \alpha = 0.115 \cdot I_{corr}^{REP} \cdot t_p \tag{2}$$

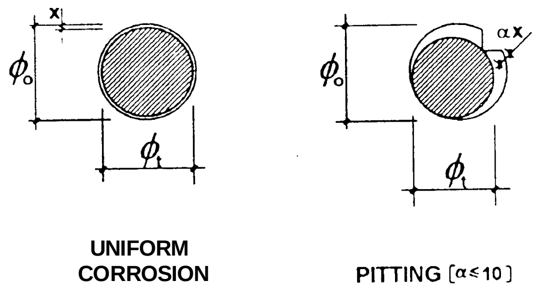


Figure 5: Residual steel section loss considered for the cases of uniform and localised corrosion.

5 MODEL OF SERVICE LIFE PREDICTION OF REINFORCEMENT CORROSION

In general, low and medium radioactivity waste is disposed in concrete containers. In the case of Spain, the primer container is a cube of 2x2x2 m with a wall thickness of 15 cm. Two layers of nine drums containing the radioactive waste mixed with cement (cement matrix) are placed inside this container. Once the deck of the container has been set, the space left is completely filled with low porosity mortar minimising the number of non-desirable air bulbs that could be formed.

In order to study the service life of these containers, several research programs are developed by Enresa (Spanish Agency for Nuclear waste management). That concerning the service life of cementitious materials in this type of disposal sites is made with the collaboration of the IETcc in Spain.

5.1 Physical model

Experimental data has shown that transport of ions, from a macroscale point of view, can be described through an apparent diffusion coefficient D_a as a fickian diffusion process. Therefore the flux can be expressed through the Fick´s First Law:

$$\mathbf{q} = -D_{ap}\nabla C(\mathbf{x}) \tag{3}$$

Where **q** denotes the flux vector of chloride, D_a is the apparent diffusion coefficient and C the concentration. Although two materials with different diffusion properties are modelled (mortar and concrete), each one is considered as a sub-domain in which D_a is constant. Besides, the conservation of the total amount of ions implies:

$$\frac{\partial C}{\partial t} = D_{ap}\mathrm{div}(\mathbf{q}) \tag{4}$$

From (3) and (4) we obtain, for a constant diffusion coefficient the governing equation can be written as:

$$\frac{\partial C}{\partial t} = D_{ap}\Delta\mathbf{C} = D_{ap}\left(\frac{\partial^2 C}{\partial x^2} + \frac{\partial^2 C}{\partial y^2} + \frac{\partial^2 C}{\partial z^2}\right) \tag{5}$$

5.2 Numerical model

The whole model has been meshed with linear hexahedral elements. In some scenarios some regions have been meshed with a coarse mesh and others with a denser one depending on the gradient of the chloride flux expected. The three main parts of the container (drums, mortar and container walls) have been meshed separately in order to assign different material properties. Thirteen scenarios have been modelled.

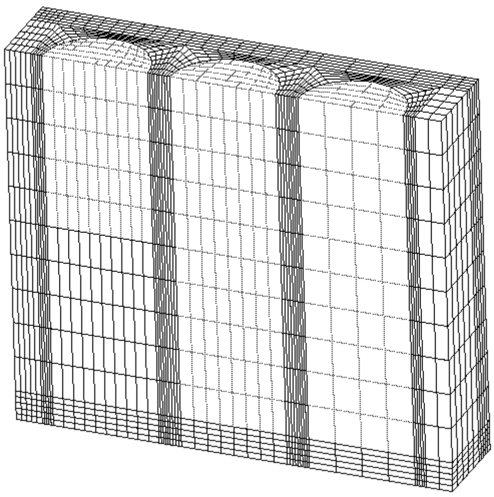


Figure 6: Section of the mesh used for the container

5.3 Predictions

Only the concentration history of some critical points has been plotted. A scheme of their location is shown in figure 7. Since the bottom face of the container has no mortar protection, a point B located on the surface that contains the bottom bars (BS) is selected. B is within all the points in BS the one that reaches the maximum concentration values. The same has been done for the point L but related to the lateral surface (LS). Curves in figure 7 show that for a constant surface concentration the level of chlorides is continuously increasing (scenario 1). On the contrary, preserving the total amount of chlorides that initially the active drums contain the curves show a maximum that in all the cases appears before the first 150 years. This maximum is approximately 30 % of the maximum attained with the first approach. However, this ratio is for the values at 300 years only 10-15 %.

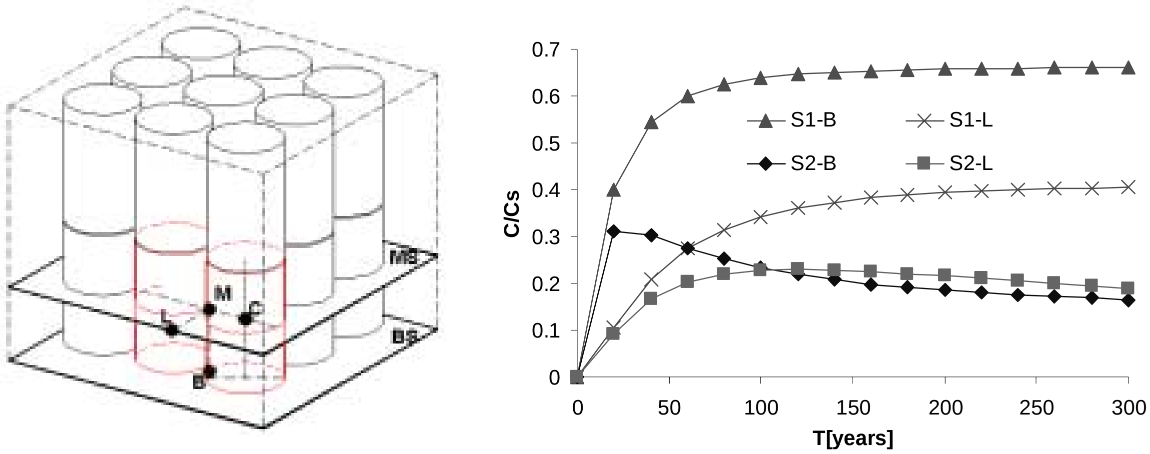


Figure 7: Points monitored in the analysis and concentration history

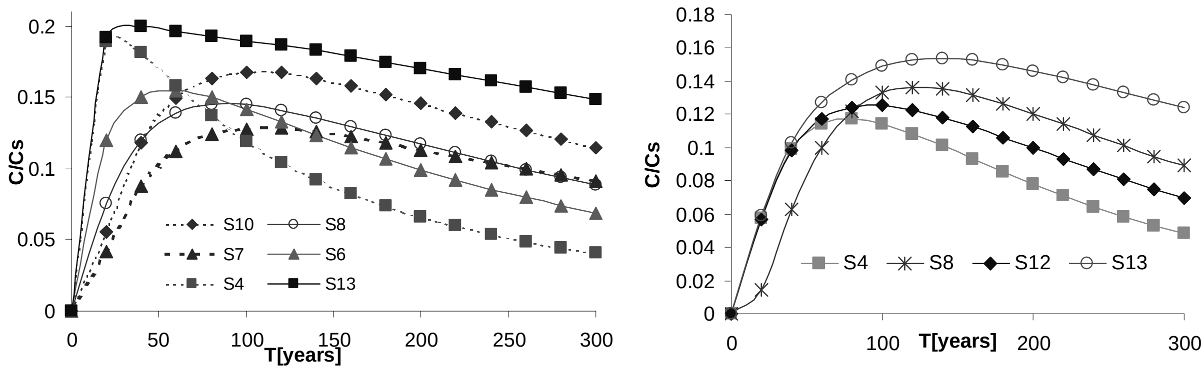


Figure 8: Bottom and lateral critical points concentration history

Both graphics in figure 8 show that the values of the point B are in general higher than those of the point L. Moreover, the decrease of the concentration values is more gradual in the second case. In scenario 13, the more critical one, the final concentration levels are quite similar for both points. However, the bars in the bottom wall of the container are subjected to more aggressive conditions during the whole history.

The start of the corrosion can be considered when the chloride concentration values reach the 0.4-0.7% of the cement weight. That is to say between 0.068 and 0.122 % of the total weight of the concrete for a content of 400 kg/m^3 and a density of 2350 kg/m^3. The real surface concentration is variable and difficult to determine, but it can attain 6000 ppm. However, the initial surface concentration that reaches the threshold level calculated for a final value of concentration of 0.20 (scenario 13) is:

$$C_s = 0.122\%/0.20 = 0.61\% = 6100\ ppm \qquad (6)$$

Results show that a constant surface concentration is a roughly approach that must be verified for the model under study. Furthermore, the concentration of chlorides decreases when more real boundary conditions are imposed. The deterioration process is therefore determined not only by the maximum concentration level attained, but also by the whole history of the concentration profiles. New experiments must be undertaken to show the behaviour of the bars under those conditions.

The coupled effects of corrosion in concrete and the mechanical behaviour of the reinforcement are now under study. Since the chemical environment is very dependent on the materials, the geometry and time, the mechanical deterioration must be modelled jointly with the transport phenomena.

6 FINAL COMMENTS

Corrosion of reinforcement can be approximately model and accurately measured on-site. The periodical corrosion rate measurements on its monitoring through embedded sensors seems very necessary to assess present conditions of concrete structures and is a very useful tool in the case of cooling towers of power plants. Techniques based in the measurement of Polarization Resistance have been implemented in portable corrosion rate meters to obtain corrosion rate values, and corrosion-data-loggers are now operative in pilot containers to monitor corrosion related parameters.

The in-situ techniques should be complemented by models that although still too oversimplified, may help to make predictions of the advance of aggressive fronts towards the reinforcements and to predict very long-term performance.

7 ACKNOWLEDGEMENTS

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Improved Detection of Tendon Ducts and Defects in Concrete Structures Using Ultrasonic Imaging

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Introduction

At the beginning of the 90s the general opinion was, that ultrasonic inspection methods using pulse-echo technique were not suitable for the inspection of concrete because of the inhomogeneity and the strong scattering behavior of the embedded aggregates. In the meantime the progress in the development of new equipment and inspection strategies in connection with ultrasonic imaging techniques turns the pulse-echo technique into a powerful tool to solve problems related to concrete materials. These imaging techniques – developed for the inspection of homogenous materials like steel or aluminum – could be adopted to the very low frequencies needed for concrete inspections.

Since 1994 the BAM and the IZFP cooperate in the field of concrete inspection. The BAM performs measurements using laser-vibrometers, probe-arrays, and pitch-and-catch arrangements with two probes. The ultrasonic echoes received are digitized and stored on a computer and evaluated at the IZFP using the Synthetic Aperture Focusing Technique (SAFT) for 3-dimensional imaging. The results are very encouraging with respect to detection and positioning of reinforcement structures inside the concrete like tendon ducts and reinforcement bars as well as defects like compaction defects and voids (honey-combing represented by styrodur balls) and to detect and even size notches and natural cracks oriented vertical to the surface. These inspections are applied to bridges, nonballasted tracks, and foundation slabs. They were performed in the laboratory as well as in the field at prestressed concrete bridges. In addition the Federal Highway Research Institute (Bundesanstalt für Straßenwesen, BAST) organized two round robin trials, one on test specimen containing artificial defects, the other on a motorway bridge, which had to be replaced. Examples of reconstructed ultrasonic images are presented. Further basic research work on ultrasonic imaging of concrete structures is carried out within a research group of the German Research Council (FOR 384 of Deutsche Forschungsgemeinschaft, DFG).

1. A round robin trial with a test block containing artificial defects

On the concrete support of railway tracks, vertical cracks have been observed. Those cracks allow water to penetrate which causes a corrosion of the tendon ducts and thus reduces the time of life of the supporting structure. In the literature different methods based on time-of-flight evaluation of the ultrasonic pulses are reported. Difficulties arise from particles or water which act as ultrasonic bridges between the crack borders; in those cases the true penetration depth of the cracks can be much larger than the actual measured depth extension. Modern research like /1/, /2/ and ~~/2/~~/3/ is aimed to improve the reliability of crack depth measurements.

During the propagation in concrete material with additives of different aggregate sizes, the ultrasonic pulses are scattered and change propagation direction into all possible directions. In the case of a crack between an ultrasonic transmitter and the receiver probe the time-of-flight of received scattered amplitudes moves towards larger values /4/, /5/.

Most of the time-of-flight methods are based on the surface wave and the diffraction of the longitudinal waves at the crack tip. The principle is shown in fig. 1. For each probe position of the receiver the time-of-flight will be measured. If a surface crack lies between transmitter and receiver the signal will disappear and come up again with a step in the time-of-flight curve. The time difference is a function of the longer sound path around the tip of the crack and proportional to its depth extension.

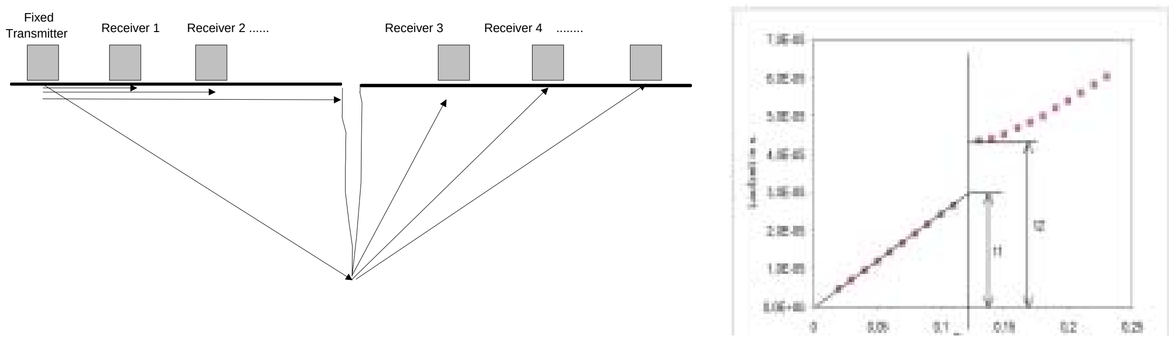
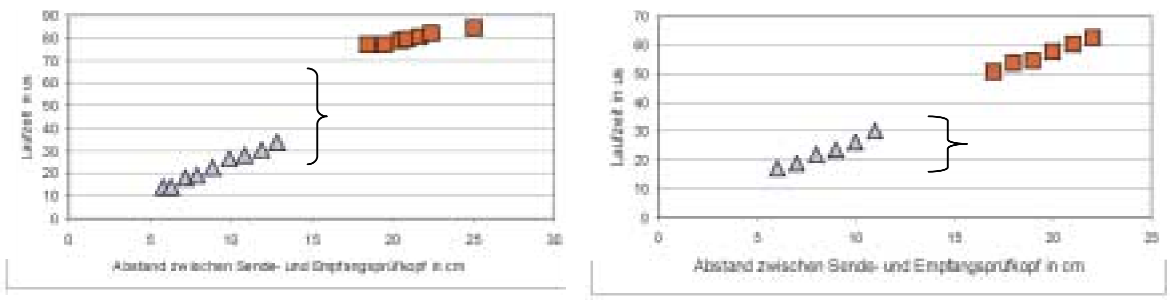


Fig. 1 Principle of crack depth measurement with acoustic surface and longitudinal waves

This principle has been used in an experiment at a 20 cm thick concrete test block which contained a 10 cm deep crack simulated by a notch. The center frequency of the ultrasonic probe was 100 kHz. If the crack is filled with water the crack gets penetrable to the ultrasonic pulses and one would expect that this method will fail. In fig 2 the time-of-flight methods are displayed for both cases



Air-filled crack; calculated depth: 91 mm   Water-filled crack; calculated depth: 22 mm

Fig. 2 Problems arise if the crack is filled with water

It is obvious that particles or water in a surface connected crack cause wrong ~~measurments~~measurements, because only the first arrival of the signal is used for evaluation. With regard to cracks in concrete elements reinforcement bars and uncracked aggregates can act as ultrasonic bridges. Therefore it is important to develop a more reliable procedure.

Ultrasonic Synthetic Aperture Focusing Technique (SAFT) has a great potential to image cracks in concrete material. Its algorithm superimposes data obtained by pulse echo at many positions which leads to the suppression of structural noise and to a more reliable positioning of indications.

Computer-based implementations of such a procedure like SAFT were developed by /6/ for the inspection of pressure vessels, a combination with ultrasonic holography for defect sizing and classification by /7/ and a three dimensional version implemented by /8/.

In conventional ultrasonic testing, a specimen is scanned with a narrow search beam to determine the position of an object. The situation in concrete testing is different. Given an average velocity of 4000 m/s in concrete and a transducer of 40 mm diameter a frequency of 200 kHz leads to a divergency beam of 15°, a 100 kHz transducer to 31°. Hence the ultrasonic beam is not small enough to find the lateral position of an object. The SAFT algorithm removes this disadvantage.

The movement of a relative small probe imitates a large transducer by sampling its area at many points. This can be done either by an array of transducers which is electronically scanned or by one or two transducer which are moved step by step. Such a general arrangement is applied to the problem of imaging a surface connected crack – fig. 3. In pulse echo the transmitter acts as a receiver too and one has to move a single probe across the whole surface. If the receiver is separated from the transmitter, it is possible to keep the transmitter probe fixed and to move only the receiver to different probe positions or to change the position of the transmitter simultaneously. In the following a scanning laser Doppler vibrometer is used as a scanning ultrasonic receiver /8/. In fig. 4 several transmitter positions have been selected and at each transmitter position a two dimensional aperture on the surface opposite to the transmitter position has been scanned by a laser vibrometer. The data have been superimposed to achieve best quality in the image.

Using these data the reconstruction calculation by means of 3D-SAFT is performed and results in a three dimensional image of reflected and scattered objects from the inside of the specimen

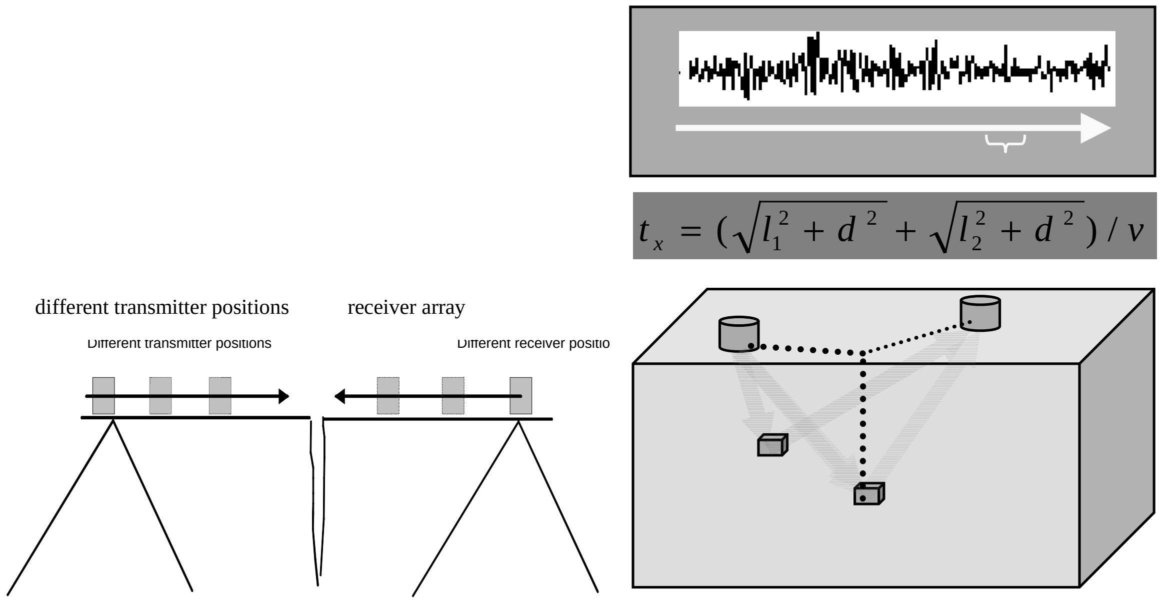


Fig. 3 Scanning arrangements for acoustic imaging with SAFT

The SAFT algorithm focuses the received signals to any point of the reconstructed image by coherent superposition; that amplitude of each A-scan which may originate from a given voxel due to ist time of flight value is calculated and averaged into the voxel. Scattered signals are statistical, true reflections not and therefore reflectors can be imaged with a higher signal-to-noise ratio. Scatterer or indications in concrete are localized at their geometric positions because they are not projected into a B-scan image with the nominal insonification angle of the probe; the beam opening angle has been introduced into the algorithm and takes care of all different angles within the divergent sound field.

In fig. 4 an example of a specimen containing a notch is presented. From a 150 mm deep notch vertical slices of the reconstructed image are shown from x- and y-directions.

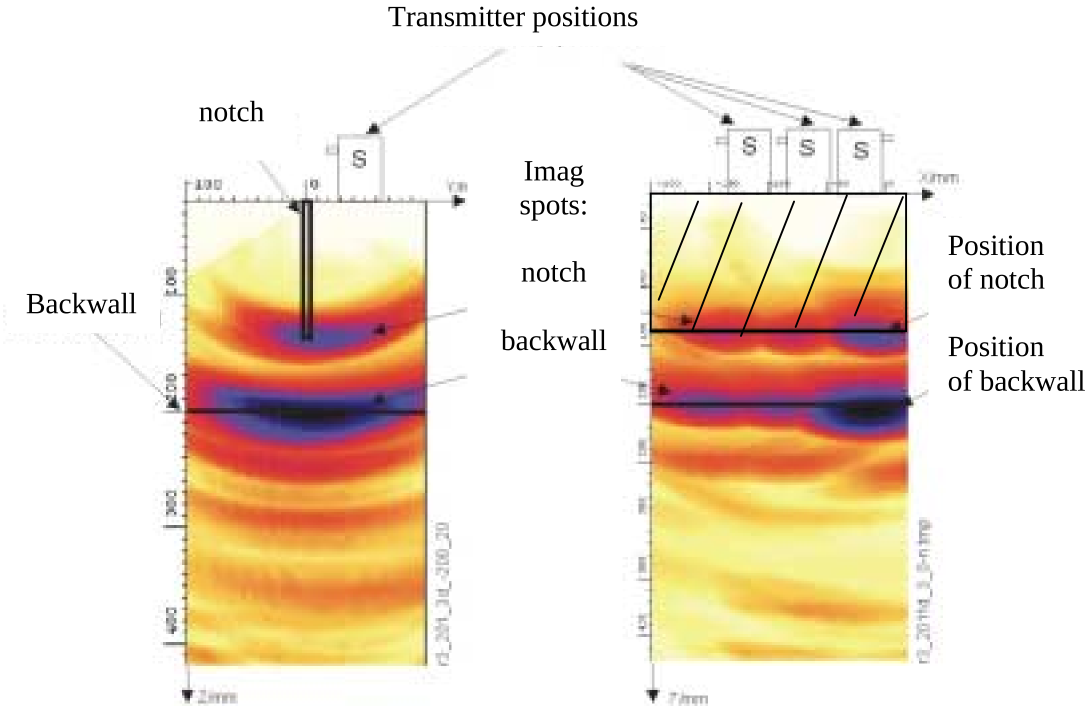


Fig. 4 Side View SAFT-images from a 150 mm deep empty notch

What advantages can we take from this procedure?
If the crack is partially filled by particles,. they act as scattering center and are imaged in addition to the other reflecting surfaces of the crack. This case was simulated in a specimen where a notch containt an aggregate which acted as a bridge from one side to the other side of the crack. The result of the SAFT image for this specimen is presented in fig. 5. It shows the B-scans from the 3D-SAFT reconstruction in two directions. The notch tip and the back-wall could still correctly imaged if one compares with the method explained in fig. 1 and fig. 2. The signal from the aggregate is imaged too. In this special case the image of the aggregate is not correctly located due to another effect. This is a mode conversion and knowing this effect one can locate the bridge into the correct depth. The result shows, that measurement of a crack depth is possible under site conditions using the imaging system described. This has been verified in addition by experiments at real cracks.

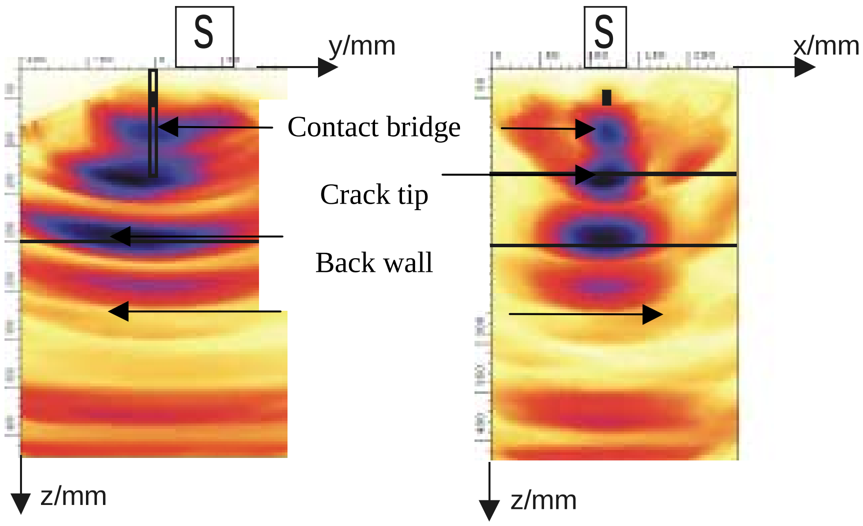


Fig. 5 Correct depth sizing despite a contact spot in the notch due to SAFT-application

2. Practical application of ultrasonic testing on a motorway bridge

In the following the application of the ultrasonic array method including 3D-SAFT reconstruction is demonstrated for the localization of honey combing and compaction faults of a base slab with a thickness of 300 mm. The specimen has been produced using concrete with a maximum aggregate size of 32 mm and contains twofold layers of reinforcement bars on the top and at the back side. The diameter are 25 mm and the mesh wide 125 mm. The faults were integrated as grains with one constant diameter and styrodur balls.

Fig. 6 Investigation of a 150 long bridge with a base plate and reinforcement

The transducer template was moved over the surface with a step width of 20 mm. The reconstruction in fig. 7 shows a vertical slice from the 3D-SAFT reconstruction. The upper and lower reinforcement layer perpendicular to the moving direction of the transducer template and the back wall echo are clearly detected. At x = 880 mm and z = 160 mm the reflection from honeycombing is revealed. This is confirmed by the shading of the back wall. A second compaction fault is seen from the shading of the back wall at the position of y = 120 mm without direct echo. The localization of the defects matches with the construction plane with an uncertainty of several cm only. In addition it was possible to image the second layer of the reinforcement bars. They could not be detected using the impulse radar technology.

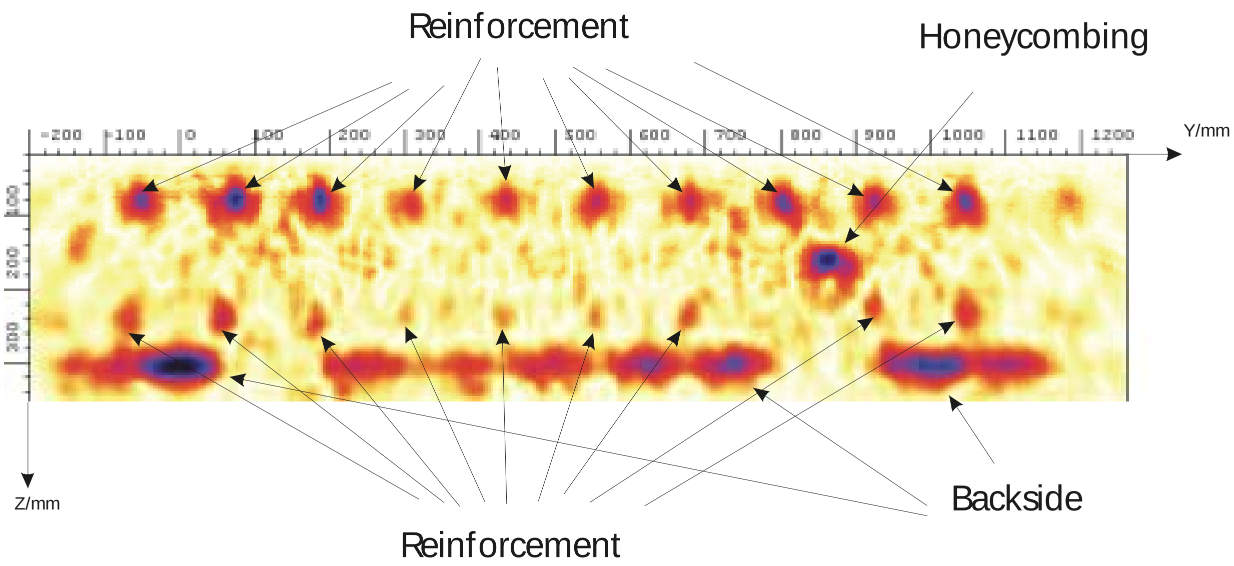


Fig. 7 3D-SAFT: B-Scan of base plate with reinforcement and two artificial flaws 88 mm x 34 mm

In a further application ultrasonic measurements on a post-tensioned concrete bridge deck have been performed. Fig. 8a depicts a 1 m long section through the duct; duct and back wall are visible on the left side. A vertical crack – at the position of x = 150 mm –is the reason that neither the back wall nor the duct could be imaged. In fig. 18 b and fig. 18 c other sections of the concrete bridge are shown where an overlap occurs.

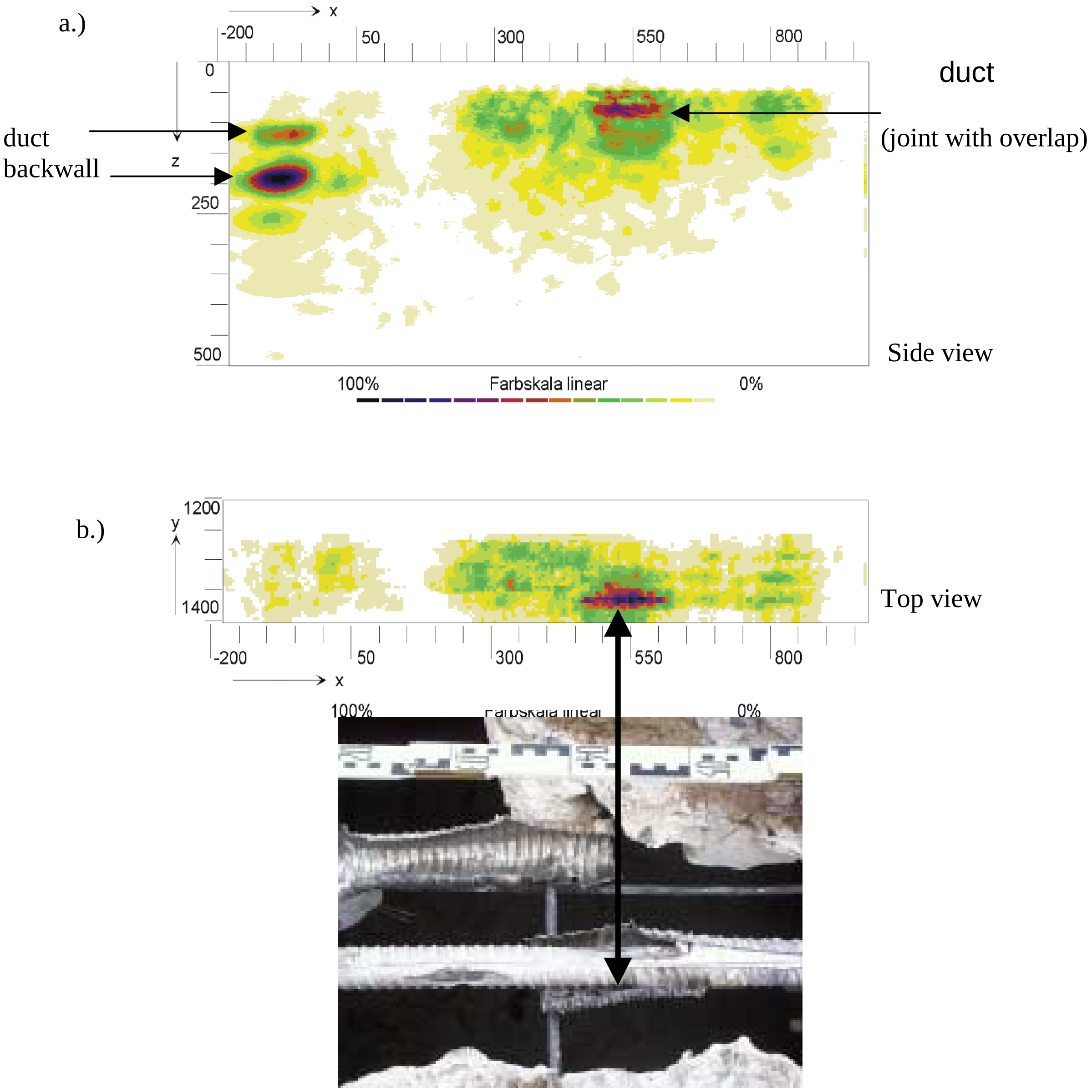


Fig. 8 Application of ultrasonic inspection on concrete motor way bridges

The results of a detailed analysis can be summarized:

- The acquisition of high frequency ultrasonic data with signal processing by an imaging scheme like SAFT allows to present an image where the direction and the concrete coverness thickness can be interpreted.
- It has been demonstrated that the array system together with 3D-SAFT reconstruction calculation can be used for the examination of transversal prestressed ducts having a concrete cover of about 100 mm.
- The system has already been successfully used on site.

3. Acknowledgments

The 3D-SAFT development was supported by the BMWi (Bundesministerium für Wirtschaft) in the frame work of reactor safety program. Part of the work was developed in the frame of research initiative „NDE of Concrete Structures using Acoustic and Electromagnetic Echo Methods“ by research funding of the Deutsche Forschungsgemeinschaft (German Science Foundation), of the Bundesanstalt für Straßenwesen (Federal Highway Research Instiutute) and by Deutsche Bahn AG (German Railway Corporation). These supports are gratefully acknowledged.

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Structural Integrity Evaluation of Kori-1 NPP Steel Containment for the Replacement of Steam Generator

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ABSTRACT

In order to replace the steam generator at Kori unit 1 NPP, in 1998, a comprehensive program for the structural safety evaluation of steel containment was performed. Based on the replacement process, the check list for inspection was made to do the replacement work efficiently and prevent any mistakes. Overall replacement process was appropriately formulated based on regulatory requirements related to nuclear design and repair. As a result of inspection according to the check list, several problems were found such as adhesive defect at interface between old and fresh concrete and appropriately corrected.

FEM analysis in order to determine the ultimate load of cylindrical steel shell with opening was performed using ABAQUS. Stress concentration and second order deformation due to crane load was investigated through FEM analysis considering inelastic large deformation. It was verified that the current approximation analysis using combined elastic buckling criteria gives conservative results. The analytical result has shown that the structure follows elastic load-deflection behavior under the given crane load condition with safety factor of 10.3. The result of this study will give useful information to the replacement of the steam generator of a nuclear power plant.

1. INTRODUCTION

In the field of electric power generation, an importance of nuclear power has been increased because of its large portion of electric facilities in Korea. Recently, extension of lifetime in the field of maintenance of nuclear power plants has been main concern from economical point of view. Among equipment in plant, the steam generator is one of the most important component that affect the lifetime of nuclear power plant.

KEPCO carried out the replacement of steam generator at Kori-1 nuclear power plant in 1998. In order to upgrade the steam generator to be safer and more stable, KEPCO planned the Kori-1 SGR(Steam Generator Replacement) project[1,3]. The containment vessel consists of a 32m in diameter and 44.5m in height cylindrical shell with thickness of 36.5mm and a spherical cap with thickness of 19mm.

In order to accommodate the steam generator replacement, an about 7m×7m opening hole was made temporarily as shown in Fig. 1 and filled back after replacement. A built-in polar crane was to be used for lifting and transporting the steam generators for the replacement.

To ensure the structural safety of the containment vessel, a comprehensive program for the structural safety evaluation of the Kori-1 Nuclear Power Plant was started in 1998 and was closed successfully. In the program, the main processes of inspection were defined and detailed check list was developed. There are many concerns in the constructing of safe structure. Specially, the stability of steel containment vessel against crane loading was considered as an important factor to guarantee the safety of the whole structure. The non-linear analysis of the steel containment vessel was performed in order to determine the ultimate load of steel vessel with opening.

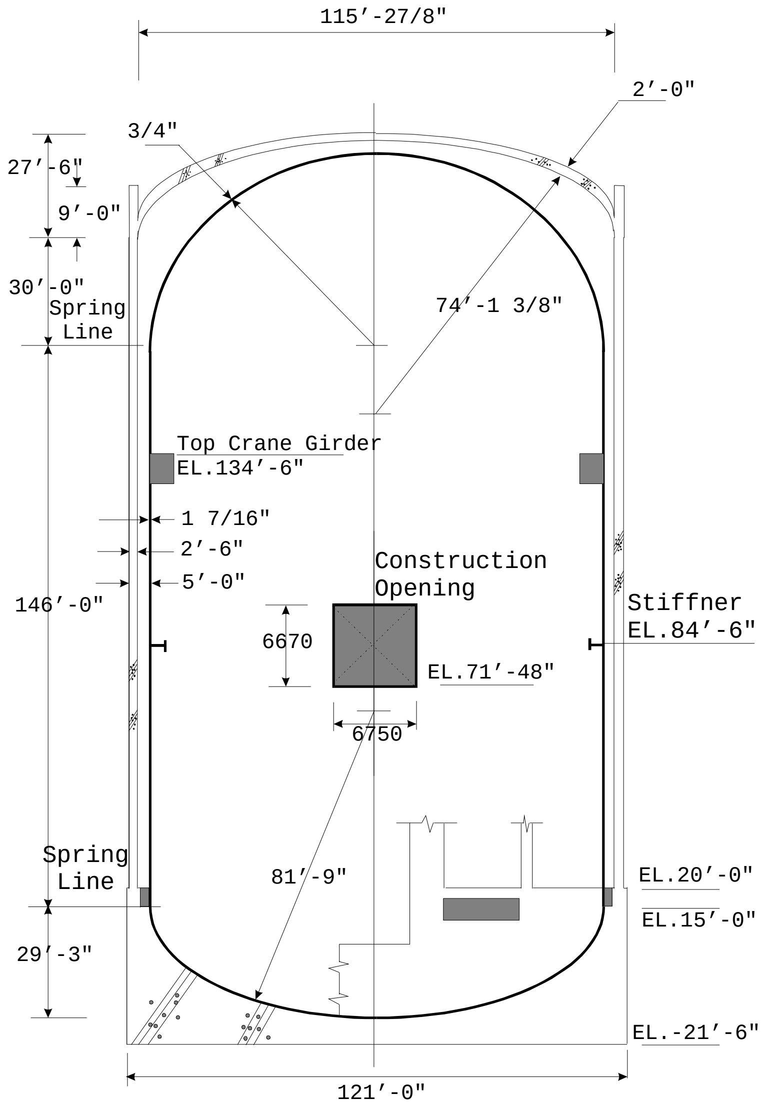

Fig. 1 Containment Vessel Cross Section for Kori-1 SGR Project

2. MAIN PROCESS AND CHECK LIST FOR THE INSPECTION

The inspection to containment vessel should be performed in accordance with nuclear regulatory requirements, such as regulatory guide, and ACI, ASME, and ASTM code, and so on. In order to ensure structural soundness and functionality of containment vessel, main process for the inspection in this project was defined as the following.

1. The cutting process of concrete shield building and rigging process of concrete block.
2. The cutting process of steel containment vessel and rigging process of steel plate.
3. Damage of containment facilities such as polar crane bracket during rigging process of steam generator.
4. The welding process of steel containment vessel.
5. Defect identification of welding part and repair process.
6. The process of concrete mix and production.

7. Pressure test of steel containment vessel.
8. Defect identification of opening concrete containment and repair process.

Detailed check list for the inspection was developed to do the replacement work efficiently and prevent any mistake and attached in appendix A.

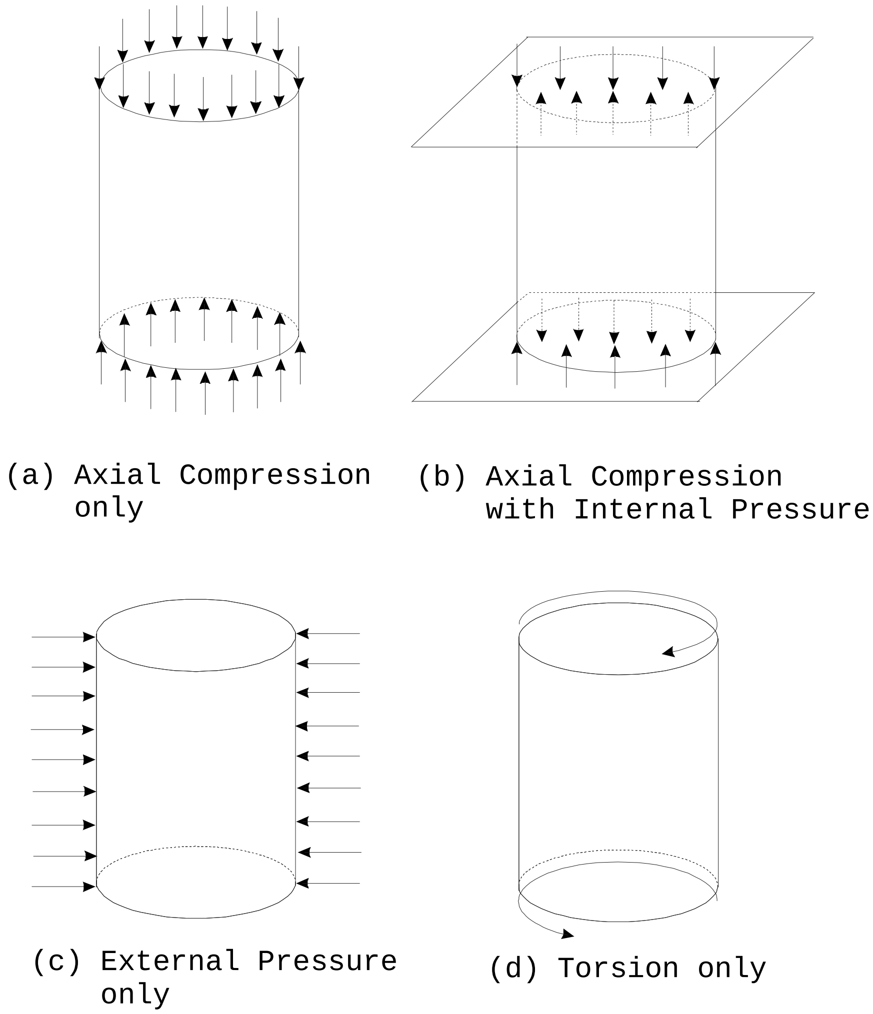

Fig. 2 Basic Loading Components of Buckling Stresses for Cylindrical Containment Vessel

3. STABILITY OF CONTAINMENT VESSEL

The primary concern in the Kori-1 SGR project was to investigate the stress distribution around the opening hole resulting from the crane operation during the period of replacement of steam generators. The stress resultants from the SAP90 finite element analysis were put into the "buckling criteria" prepared originally for the design of Kori-1 nuclear power plant in the previous work[4].

Fig. 2 shows basic loading components of buckling stresses for the cylindrical containment vessel under various loading ; (a) the axial compression only, (b) the axial compression with internal pressure, (c) the external pressure only, (d) the torsion only.

Equations for the basic components of buckling stresses were derived by using the theory of mechanics and supplemented by experimental observations. In application of the basic buckling stress components to the cylindrical containment vessel subjected to the various types of loading, they were combined by two or three. The analysis revealed that the hole at the steel vessel did not decrease the capacity of the vessel.

However, when a circular cylindrical containment vessel is under the various loading, shell elements around the opening hole may be subjected to more severe stress concentration than the other parts. Since the probable failure mode of the shell elements in such a condition has not been clearly defined, the buckling criteria may not be directly applicable to this case, in which an opening hole exists, as was to the containment vessel of Kori-1 SGR, in which an opening hole is not existing. It should be noted that the buckling criteria were originally derived for the containment vessel in service in which there may be some internal pressure existing due to the operation of steam generators. However, in Kori-1 SGR project a temporary opening hole is existing in the containment vessel that means any internal pressure cannot be generated during the period of replacement. Accordingly, it should be recognized that the loading condition over the containment vessel with and without an opening hole is very much different. The element around the opening hole may be subjected to rather yielding than buckling.

4. ULTIMATE STRESS ANALYSIS OF STEEL CONTAINMENT VESSEL

Because the element around the opening hole of a containment vessel may be subjected to rather yielding than buckling, nonlinear analysis considering plastic deformation was required. So, it is performed ultimate stress analysis, considering geometric and material non-linearity, of cylindrical containment vessel using finite element program ABAQUS in order to estimate the stability of containment vessel with a 7m×7m opening subjected to crane loading at both edges of polar crane. To evaluate the effect of stiffener around the opening, the cylindrical containment vessel with or without stiffener was analyzed.

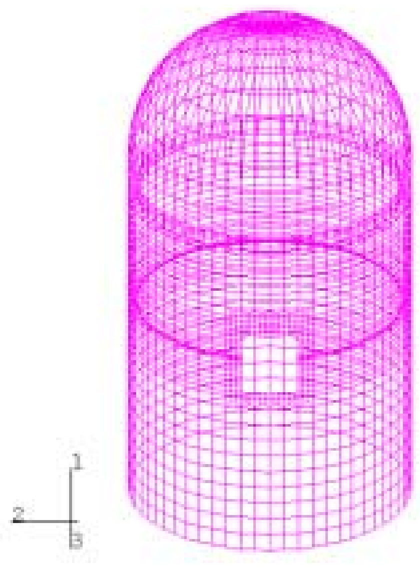

Fig. 3 3-D Finite Element Model

Fig. 3 shows the 3-D analytical model for investigating the ultimate structural behavior of containment vessel. The load applied to finite element model was increased and the stress and displacement was calculated. Fig. 4 shows Von Mises stress distribution around the cylindrical containment vessel subjected to ultimate loading without stiffener and Fig. 5 shows Von Mises stress distribution around the cylindrical containment vessel subjected to ultimate loading with stiffener.

Fig. 4 depicts the relation of crane loading vs. displacement at node 2431 (which is located the polar crane in Fig. 3). X axis in Fig. 4 shows the displacement at node 2431 and Y axis is the magnification factor of crane loading.

In the Fig. 5 and Fig. 6, it is shown that an installing stiffener around opening is effective to prevent deformation and stress concentration of the opening. In the Fig. 7, when the value of magnification factor of crane loading is within 1, cylindrical containment vessel behaves linear elastic. The maximum value of Von Mises stress is 2.47×103 t/m2 at area around opening with stiffener and is 9% value of Von Mises's yield criteria 2.67×104 t/m2

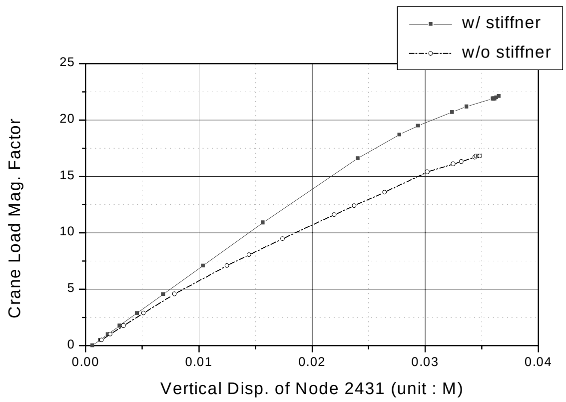

Fig. 4 The Relation of Crane Loading vs. Displacement at Node 2431

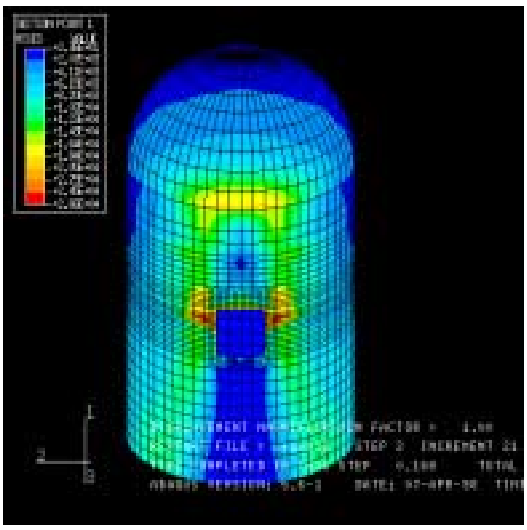

Fig. 5 Von Mises Stress Distribution subjected to Ultimate Loading without

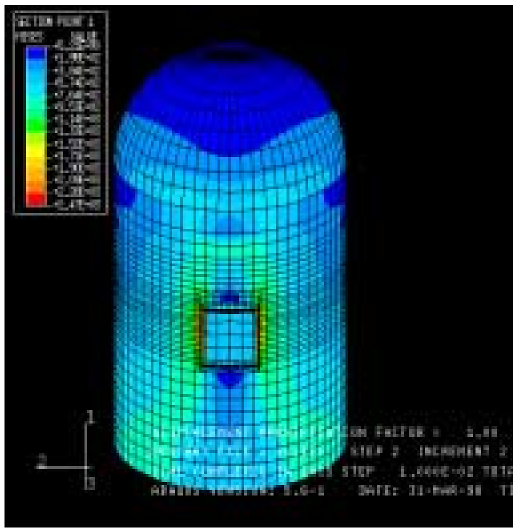

Fig. 6 Von Mises Stress Distribution subjected to Ultimate Loading with

Stiffener Stiffener

5. CONCLUSION

In order to replace the steam generator at Kori unit 1 NPP, in 1998, a comprehensive program for the structural safety evaluation of steel containment was performed. Based on the replacement process, the check list for inspection was made to do the replacement work efficiently and prevent any mistakes. Overall replacement process was appropriately formulated based on regulatory requirements related to nuclear design and repair. As a result of inspection according to the check list, several problems were found such as adhesive defect at interface between old and fresh concrete and appropriately corrected.

It was recognized that the direct application of the buckling criteria[2] for the Kori-1 SGR seems to be inappropriate. Therefore, in order to estimate the ultimate capacity of containment vessel with an opening, ultimate stress analysis was performed in consideration of geometric and material non-linearity of cylindrical containment vessel against crane loading at both edges of polar crane. As a result, it is found that when the cylindrical containment vessel is in service loading state, the vessel showed linear elastic behavior and service load is about 9% of ultimate load capacity. Therefore, it is concluded that strengthening design due to opening is not needed.

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APPENDIX A

Check List for the Inspection in the Replacement of Kori-1 Steam Generator

Object	Check Points
installation of temporary opening on the wall of steel containment vessel	1. corresponding work procedure and design drawings 2. cutting process and condition of temporary opening after cutting
installation of temporary opening on the wall of the concrete shield building	1. whether the cutting work of concrete block is performed under the appropriate operating mode or not. 2. whether rigging of concrete block damages the steel containment vessel and concrete shield building or not.
recovery of temporary opening on the wall of the steel containment vessel	1. whether the surface treatment and cleaning of welding part are performed appropriately or not. 2. welding procedure and qualification of welding personnel and welding material 3. defect identification of welding part and appropriateness of repair work 4. structural integrity evaluation of steel containment (pressure test)
recovery of temporary opening on the wall of the concrete shield building(rebar work)	1. corresponding work procedure and design drawings 2. cutting process and condition of temporary opening after cutting 3. appropriateness of mechanical rebar splice method 4. appropriateness of rebar welding method 5. whether rebars are installed appropriately or not
recovery of temporary opening on the wall of the concrete shield building (concrete mix and production)	1. whether the physical and chemical characteristics of cement and admixture are in compliance with the corresponding technical guidelines or not 2. whether the certification and confirmation tests of fine and coarse aggregate are performed appropriately and the those results are acceptable or not 3. certification of testing personnel and appropriateness of testing equipment 4. appropriateness of administrating test 5. appropriateness of concrete mix design 6. certification of batch plant and inspection and correction of measuring devices 7. whether the row material and temperature control of hot weather concrete are appropriate or not
recovery of temporary opening on the wall of the concrete shield building(concrete work and curing)	1. appropriateness of chipping work, cleaning, wetting in the contact surface between old and fresh concrete 2. appropriateness of form work design and installation

Object	Check Points
	3. whether the mortar thickness at the contact surface between old and fresh concrete is conformed to the requirement of technical guidelines or not 4. whether the concrete pouring work is performed in compliance with the temperature requirement of hot whether concrete or not 5. whether the concrete quality assurance is performed in compliance with technical guidelines and that result is appropriate or not. 6. appropriateness of the pouring and vibration of fresh concrete 7. occurrence of cold joint in hardened concrete 8. appropriateness of form removal based on the technical guidelines 9. appropriateness of curing method 10. occurrence of defects such as honeycombing and voids etc. 11. appropriateness of concrete repair material and method 12. appropriateness of disposition method for the non conformance things and those results
structural integrity of containment internal structures	1. whether rigging of steam generator damages the operating floor and steel containment and concrete shield building or not 2. whether rigging of steam generator damages the polar crane bracket or not

New Methods for the Reconstruction of Safety Compartments of Nuclear Power Plants

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Safety Compartments – Challenges and Construction Specialities

Safety compartments are essential structures for the operation of nuclear power plants. Due to their importance these concrete structures have to be carefully observed. Certain special requirements must be fulfilled in the construction phase of the structures. When these requirements are not fulfilled the chance of damages occurring is great. Simultaneously these structures demonstrate a series of distinctive constructional features:

Picture 1: Total view of a safety compartment whilst reconstruction

A safety compartment is a symmetrical structure with a cylindrical base and crowned with a half-dome (Pic. 1). The wall thickness varies up to 2 meters in width. Furthermore the diameter of the reinforcement is very large. Due to constructional changes, the concrete cover can vary and in some instances very small. Simultaneously the high number of and large diameter of stirrups present near the concrete surface are also a cause of the damages.

Found mainly in older structures, the distribution of reinforcement for the reduction in surface cracking is missing, and therefore cracking does occur.

Damages to be restored

From the inspections of the concrete surfaces, the following was discovered:

The carbonization depth was up to 15 mm and in some areas the concrete showed detoriation. (Pic. 2)
Due to the weather conditions, rain water containing CO_2 and the influence of wind, the concrete surface became washed out. Additional, found mainly on the top of the safety compartments, microorganism were observed. These organisms have started to produce acids, which added to the wash out of the surface.

Picture 2: Concrete surface of a safety compartment with damages and detoriations

Of greater importance are the surface cracks present. These cracks have reached deep into the concrete, some of which have reached the reinforcement. These cracks have occurred either due to temperature changes that were not taken into account during the construction design, or even from shrinkage cracking occurring shortly after the completion of construction.

Demands for upgrade

At none of the inspected safety compartments were reconstruction measurements necessary. All the works carried were to minimize future repairs and to keep the state of art. There was at no time any doubt for the safety of the concrete structure or the power plant. Furthermore, to reassure the public, nuclear power plants should present a perfect optical appearance.

The main objective of the works at the surface of the safety compartments was to stop any further carbonization. Simultaneously the further development of cracking had to be stopped and existing cracks had to be closed.

Concepts for restoration

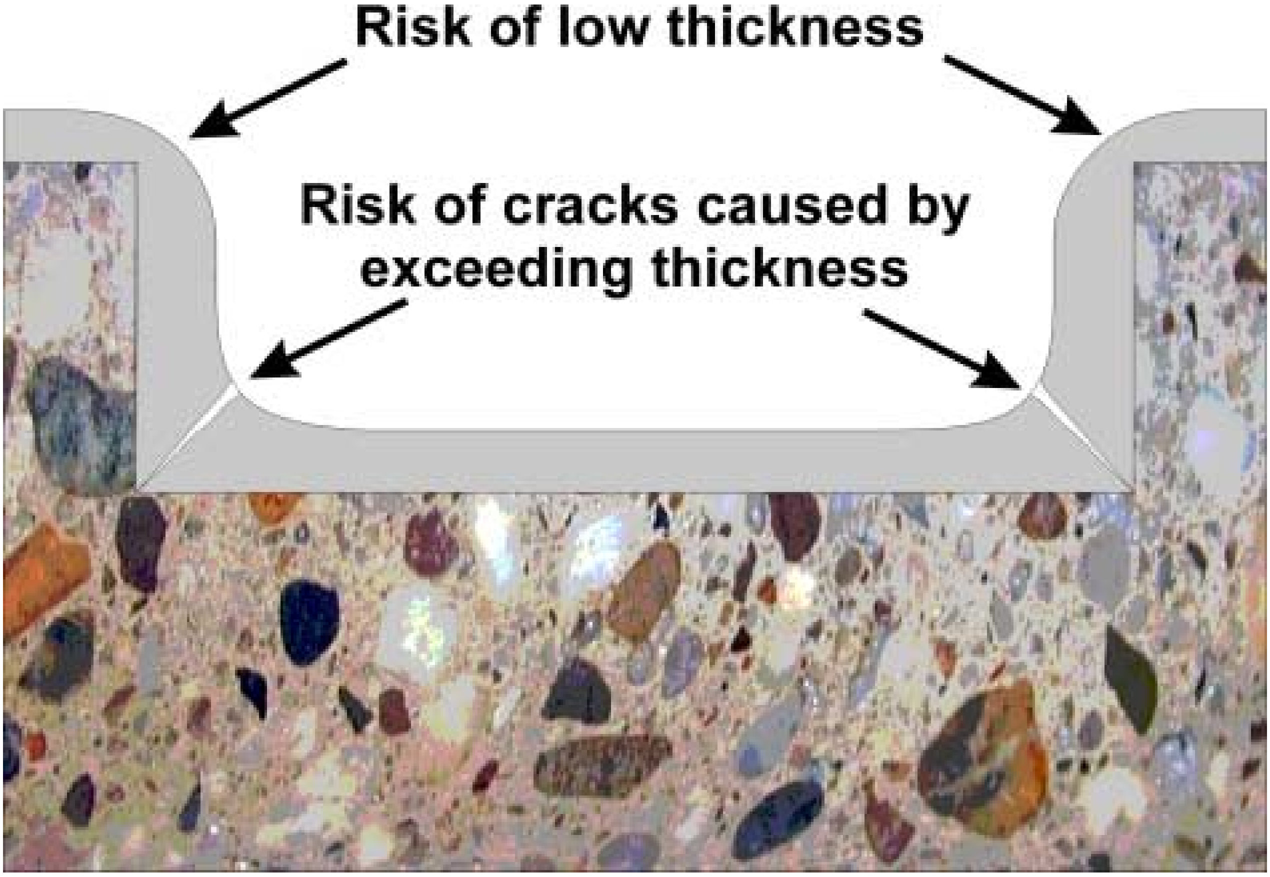
The development of the restoration concept was based on the data and experiences gained from the repair of other concrete surfaces such as cooling towers, stacks, foundation walls, bridges, etc. Their evaluation as well as the first tests on the safety compartments showed that stiff repair systems even with flexible coatings would not achieve a satisfactory solution. Therefore the aspect of crack bridging and weather protection was addressed in two steps, simultaneously looking for an architectural satisfactory solution. This concept with its optimal results was only possible due to the new developments of repair and coating materials achieved in the recent years. Table 1 shows the different measures taken in the last twelve years on German concrete safety departments.

Table 1 **:** Development of repair systems for concrete surfaces in the last twelve years

The following concept resulted from these experiences:

- Preparation of the concrete by detecting and removing all loose areas by hand and with the use of a hammer
- Blasting of the complete concrete structure
- Preparation of all corroded reinforcement and application of a protective coating.
- Injection of cracks when possible, i.e. the wide cracks
- Reprofiling of the damaged and deteriorated areas with a PCC Mortar
- Closing of pores with a stiff mortar
- Application of a mineral based crack-bridging coating
- Final application with an elastic, pigmented top coat based of pure acrylic resin

An area of concern was at the edges of the hollow areas, most of which had occurred due to architectural reasons during the construction. As shown in Picture 3 – at the edge there is the danger that the coverage may become too thin while on the inside cracks could occur due to too much material being placed



Picture 3**:** Cross – section of a fluting, showing minimal and excess material thickness

In former times with the materials available, it was not possible to solve these problems and so hollows were completely filled with a PCC mortar before applying a crack-bridging mortar. At the “minimum restored areas” new materials were used. These new materials were able to take care of the problems. None of the different techniques used showed a lack in the optical appearance.

Picture 4: Dome with fluting structures

For all the materials that were developed and tested severely, it was of great importance that the materials remain flexible even at temperatures of minus 20^0 C, as well as keeping at least 50% of its flexibility even after 20 years. These demanding requirements were tested on existing safety compartments. The results showed crack movements only due to temperature changes.

Scaffoldings

An important device for undertaking the repair work is the use scaffolding. In the past a scaffolding was constructed around the whole compartment. This was not only expensive and ineffective, but also did not comply to the existing safety standards within a German nuclear power plant. These scaffoldings had to be guarded to ensure that only restricted personnel were allowed to step on the scaffolding. Another problem of this scaffolding construction was the many anchor points into the concrete surface. These anchor points had to worked on after the scaffold was removed.

Therefore a movable scaffolding/platform was developed, which is fixed to one huge anchor bolt on the very top of the dome. This scaffolding/platform is circulating on wheels around the dome. This system worked well with the old protection systems coating the concrete. But when the flexible protective systems were used, the forces of the wheels started to damage the applied mortar, as shown in picture 5.

Picture 5: Damages on the repair system caused by high load on scaffolding wheels

Therefore a new light weight cradle-type scaffolding was designed, as shown in picture 6. This lighter scaffolding was used without any problems on the last safety compartments. The light weight scaffolding could move in all directions more easily, and like the former model was also fixed to an anchor bolt on the top.

Picture 6: **New designed light weight scaffolding/platform**

Concrete protection with crack – bridging systems

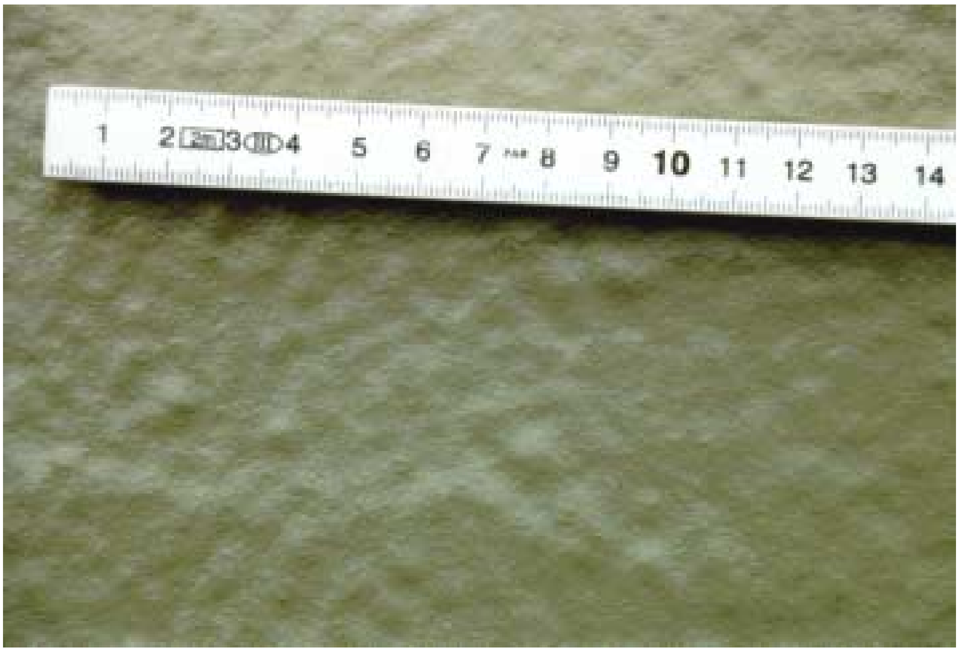
The repair of the concrete surface is started, as is the norm with all other concrete structures, by blasting and cleaning the concrete and the all open reinforcement. The reinforcement has then a protective coating applied, and then the concrete is re-profiled. What is unique about the reported system here is the use of a crack–bridging system for the first time. Due to the costs and time it was agreed to use a fine mortar to close all the pores and small caverns in the concrete surface before applying the flexible system.

Through experience it was observed that leaving out the scratch mortar would result in damaging the bridging layers. This is due to the lack of thickness or small holes, as shown in picture 7.

Picture 7**:** Surface of the crack – overbridging material with pores and hollows

The optimal application method for crack–bridging, is to wet spray apply it onto the surface. By optimizing every application step it is possible to achieve the required thickness of the layer and to achieve a surface which can accepted the final coating.

An important area of concern during the application is to pay attention to the change of climate, especially the change in humidity. If these conditions are not regarded carefully, “Calcium-hydroxide / Calcium-carbonate” may develop, which will influences the color and physical properties.



Picture 8**:** Perfect surface of the crack overbridging surface protection system

Quality Control

Most important, beside the use of proper materials and correct techniques is the installation of a quality control system from the very beginning. Before the start up of works a quality handbook was created by the contractor, the owner of the plant and the supervising engineer. This quality handbook contains all the quality checks and the production documentation. Its contents also include the various times and specifications that the contractor has to control during the repair.

It was also agreed upon how often the supervising engineer would make his quality control checks. And that before the next stage of repair commenced the supervising engineer had given his approval. The reason for this is that the preparation priming coat can not be checked after the coating later.

One of the most important measurements is the determination of the film thickness, for example with the use of a penetrometer, but other methods without destroying the coating are better. The main goal for the repeated controls, for which the costs were calculated in advance and paid separately, was to reach a constant and optimal thickness of the crack–bridging and the final layer of coating. If these layers are too thick possible problems may occur in the future**.**

Picture 9: **Partial View of a restored concrete surface of a safety compartment**

Future Aspects

There are no problems with the applied coatings at present. The materials and techniques used on the safety compartments can be used of course on other concrete structures such as cooling towers and stacks. One problem in Germany has still not been solved. As shown in picture 10, the pigeons are attracted to these domes, and since their droppings are quite aggressive. Therefore the coating in the pole area has to be checked carefully in the future.

Picture 10: **Completely restored surface, biological attack caused by birds residues**

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