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From Structural Assessment to Appropriate Preservation Strategies

De l'inspection structurale aux stratégies appropriées de conservation

Von der Zustandsaufnahme zu angemessenen Erhaltungsstrategien

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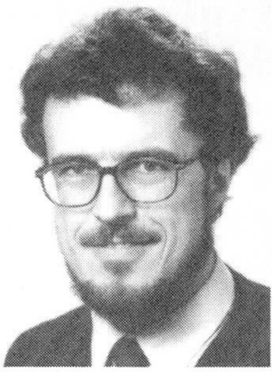

Thomas Vogel, born 1955, graduated from ETH Zurich in 1980. He gained his first five years of experience in bridge and structural design in Chur, Switzerland. For another six years he worked at first as project engineer and later as managing director of a Zurich-based consulting firm. In 1982 he joined ETH as a professor of structural engineering.

SUMMARY

The way from structural assessment activities via the evaluation of feasible options to recommendations for preservation measures is explained by showing on which level considerations are made and decisions are taken. Although the information expected from structural assessment could lead to an extensive and expensive investigation, in a first step those assessments are justified that allow a sound evaluation of feasible options. The illustrating examples concern an exposed concrete façade of a post office building and three bridges of different types.

RÉSUMÉ

L'article décrit le cheminement de l'inspection structurale à l'appréciation de ses résultats et à la recommandation de mesures de conservation. Il montre à quel niveau les considérations sont faites et les décisions prises. Quoique les informations attendues de l'inspection puissent conduire à une investigation étendue et coûteuse, dans une première phase seules sont justifiées les études qui permettent un jugement consciencieux des variantes de conservation possibles. Les exemples présentés concernent la façade en béton de parement d'un bâtiment postal et trois ponts de types différents.

ZUSAMMENFASSUNG

Der Weg, der von Zustandsuntersuchungen und deren Beurteilung bis zur Empfehlung von Erhaltungsmassnahmen führt, wird erläutert, indem gezeigt wird, auf welchem Niveau die Ueberlegungen angestellt und die Entscheide getroffen werden. Obwohl die Aussagen, die von der Zustandsuntersuchung erwartet werden, zu einer umfangreichen und kostspieligen Kampagne führen könnten, sind in einem ersten Schritt nur diejenigen Untersuchungen gerechtfertigt, die eine zuverlässige Beurteilung der möglichen Erhaltungsvarianten erlauben. Die illustrierenden Beispiele streifen eine Sichtbetonfassade eines Postgebäudes und drei Brücken mit verschiedenen Tragsystemen.

1. INTRODUCTION

1.1. Sphere of experience

A working group of the Swiss structural concrete committee SIA 162 is producing a directive on the preservation of concrete structures. Simultaneously, a working group of the European Committee for Standardization (CEN TC 102/SC 8/WG 7) is elaborating general principles for the use of products and systems for the protection and repair of concrete construction works.
Both groups started their tasks with more general considerations, even valid beyond the field of concrete structures, regarding the optimal process of evaluating existing structures and of finding reasonable solutions for their preservation.
The author is member of both groups and wishes to report on some ideas that lie behind the normative rules, always keeping his individual point of view, however.

1.2 Nature of the task

The evaluation of existing structures consists of a **structural assessment**, the **evaluation** of its results and a **recommendation** for actions.
By the way, this is a typical task for an engineer, because
- all desirable or even necessary information is never available
- nevertheless decisions have to be taken (not to decide is the worst option of all)
- the consequences of decisions decrease while the amount of information available increases during the process

In such situations a stepwise procedure is appropriate; lack of knowledge is overcome by plausible assumptions which have to be checked later.
As formulations of questions can change during the process, a stepwise procedure is justified from the economical point of view as well.

1.3. The levels of considerations and decisions

To follow how considerations are made and decisions are taken effectively it is useful to define four levels:
(i) The **totality of structures** managed by an owner or belonging to a certain region (district, county, country). On this level the final decisions are made because risks, benefits and funds are related to the owner and legal or codification requirements normally are applicable following the territorial principle.
(ii) A **group of similar structures** regarding structural system (beam, arch, frame, etc. for bridges; frame-structures, wall-structures for buildings), building material (steel, reinforced concrete, prestressed concrete, masonry, wood, composites etc.) construction method and construction period. Within a group of similar structures often similar problems occur so that is the level where the exchange of knowledge and know-how has to take place.
(iii) The **individual structure**, normally subject to a structural assessment, carried out by an engineer appointed by the owner.
(iv) The **structural element**, regarded as a part of a structure with a well-defined structural purpose (e.g. a column, a diaphragm wall, a footing or a flat slab). On this level only clear and definite statements can be made about bearing capacity, deterioration processes and their future developments.

On which level the procedures described hereafter are situated, is shown by Fig. 1.

2. STRUCTURAL ASSESSMENT

2.1. Reasons for a structural assessment

The reasons for a structural assessment can have their origin on any level:
The favorite case is the owner having a preservation plan that includes a periodical assessment of all his construction works.

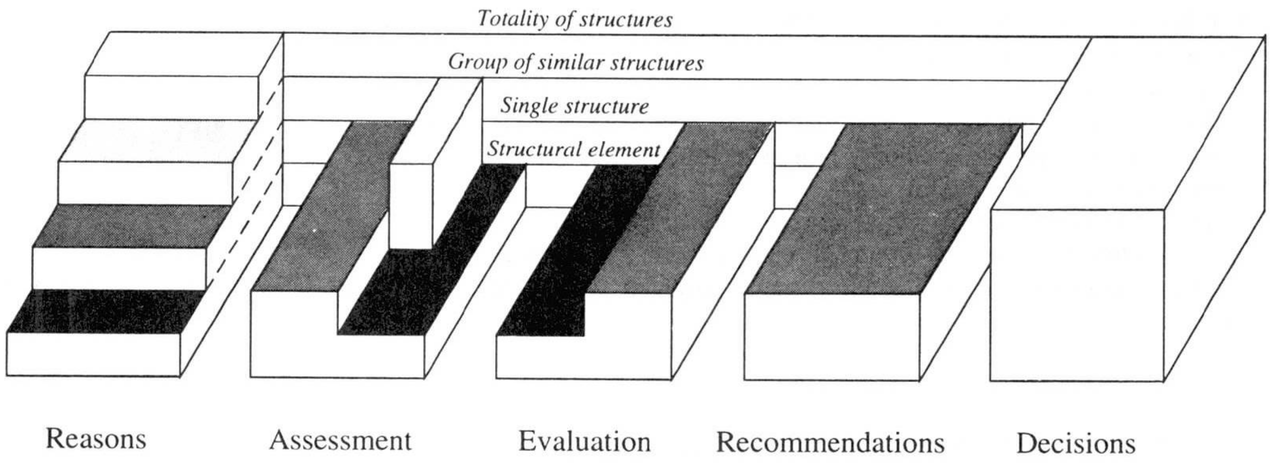

Fig. 1 Procedure with levels of considerations and decisions

Severe damage or even failures somewhere in the world lead to questions about similar structures that have to be answered. Depending on the quality of the records, assessments are inevitable.
Individual structures having been subject to hazards (overload, fire, accidents, impact, earthquake etc.) or having to be altered for other reasons (change of use, restorations, etc.) should be assessed as well.
Finally, structures or parts of them that do not behave satisfactorily or show apparent damage, defects or deterioration will be subject to an assessment sooner or later.

2.2 Purpose of structural assessment

Once agreed upon a structural assessment is not complete in itself. It has to lead to a recommendation of actions to be taken without prejudicing them. Extent and methods shall be oriented on the problems faced and to be solved, the results expected and the risks to be taken or to be avoided.

2.3. Sequence of procedures

2.3.1. Preliminary assessment

Every assessment should start with general methods to judge the extent of the expected and to detect unexpected phenomena.
Normally, a carefully prepared and executed visual inspection under the best conditions available will meet these requirements. Besides, the records of the construction work should be checked regarding completeness, previous preservation actions and special notes.

2.3.2. Further investigations

Further investigations at this stage of a project will be undertaken only in cases, where the need of further knowledge justifies them and where their results are necessary for the evaluation.
For examinations at the structure non-destructive tests are preferrable.
The benefits of destructive tests or sampling must be compared with the damage produced by core drilling etc. The validity of these tests is normally very local and cannot be extended to a whole structure unless the basic processes are understood.

2.4. Objective of assessment

The above-mentioned restrictions lead to the question, what assessment should be aimed at.
Although the general view should not be neglected, the investigations will concentrate on structural elements, that have been recognized as damaged before, that are crucial for the overall behavior or that have

been found to be designed inadequately from the records. Experience of other similar structures and the problems related to them should flow in at this stage.
The process of assessment should take into account
- the present condition of the existing construction work
- the environment and its influence on the construction work
- the conditions during construction
- the history of the construction work
- the conditions of the present and the requirements for the future use

Regarding damage and deterioration the assessment will identify their nature, causes and approximate extent.

3. EVALUATION AND RECOMMENDATIONS

The delimitation between the assessment of a structure and the evaluation of the results thereof is flexible; let us consider it there where the reflections in the time-domain commence.

3.1. Objective of evaluation

The evaluation shall describe
- the present state regarding structural safety, serviceability and durability
- the likely rate of increase of deterioration and subsequently decrease of the criteria mentioned above, including the approximate time to reach a crucial state.

As mentioned above these statements refer to structural elements in a first step. The extension to the whole structure needs more structural analysis considerations.

3.2. Required immediate actions

In case the structural safety or other requirements concerning health and safety are not fulfilled, immediate actions have to be proposed by the engineer.

3.3. Recommendations for long-term actions

For the long term the engineer should consider the following options:
(i) Do nothing for a certain time, possibly supplemented by monitoring, downgrading of the function or other measures (i.e. accept the present condition).
(ii) Prevent or reduce further deterioration with or without improvement of the construction (i.e. correct the present condition).
(iii) Change the construction work by adaption, reconstruction or enlargement.
(iv) Demolish the construction work, possibly after having made use of a residual service life.

The recommendations to the owner cover the reasonable ones of the alternatives listed above. They will contain a statement concerning the achievable residual service life for every alternative. Subsequently, the decision of the owner on how to proceed is requested.

4. FURTHER PROCEDURES

4.1. Decision of the owner

The owner will decide also taking into account factors, that are not related directly to the object under discussion like priorities regarding to the function in a production or traffic system, mutual dependencies of repair sites, funds for rehabilitation existing or able to be raised and political considerations.

4.2. Preliminary design

In case alternative (ii) or (iii) are chosen, a preliminary design of the preservation work has to be carried out. Further investigations may be necessary to decide what principles and methods of protection or repair are suitable. These are related to the causes of damage and deterioration.

5. EXAMPLES

5.1. Sihlpost Zurich

The main post office of Zurich built in 1928/29 with a structure designed by Robert Maillart was enlarged from 1986-92. Before offices of other postal and railway services are incorporated, the building including the exposed concrete façade (Fig. 2) is being presently restored.
Assessment showed the following situation:

Structural element	Concrete cover	Carbonation depth	Comment
Piers between window openings	27 - 55 mm	35 - 57 mm	increased carbonation risk
Lintels of recessed NW-façade	0 - 65 mm	28 - 56 mm	concrete spalling, first reinforcement layer corroded at any grade

Since the building is regarded as a historical monument by the local authorities, an unchanged concrete surface with formwork texture was required. According to the different stages of deterioration and structural function, the following repair systems are used:
The piers between window openings were repaired locally concentrating on corroding areas of reinforcement by a polymer-cement-concrete-(PCC) mortar. An elastic coating on the whole surface will complete the rehabilitation work.
The recessed north-western façade was treated as follows:

- Removing of carbonated concrete following a-corrosion criteria by high-water pressure.
- Replacing with spraying concrete.
- Levelling out of the surface by a PCC-mortar.
- Coating, able to bridge future cracks.

Fig. 2 SE-façade of the Sihlpost building

5.2. Dünnernbrücke Oensingen (Object Z53A)

At Oensingen (SO) an access road to the Swiss national highway N1 crosses the Dünnern creek in a skew angle on a grillage, spanning 24 m with curved girders (Fig. 3) built in 1963/64.
Structural assessment showed a deteriorated slab due to de-icing salt ingress and lack of waterproofing, an insufficient capacity for shear forces and a minimal capacity for bending moments.
As options a strengthening and a total reconstruction were studied. The will to preserve a example of elegant and material-economizing structures of the sixties overrided economical considerations. So only the slab was removed by high-water pressure and reconstructed.
The beams were strengthened by steel plates glued on the underside in mid-span and diagonal prestressed bars on either side close to the abutments.

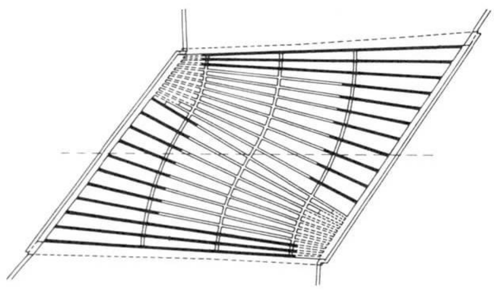

Fig. 3 Plan view of Dünnernbrücke

5.3. Arch bridges Nanin and Cascella, Mesocco

The Swiss national highway N13, traversing the Alps at the San Bernardino pass, crosses at its southern ramp the Moesa creek twice on elegant arch bridges designed by Christian Menn and built in 1966-68. In 1986/87 first preservation activities like removal of the interior formwork of the box girders , improving the drainage system and replacing the bearings took place.
Since 1985 occasional heavy-load transports were used to measure deflections of the girder due to traffic loads. The subsequent structural assessment covered a structural analysis, detailed examinations on site and long-term measurements of deflections, sampling of deck slab concrete and screening by electrical potential measurement.
In spite of severe deterioration of the steeper parts of the arches due to dropping water of the road draining system, the bridges still behave perfectly elastically and fulfil structural safety. In the transverse direction, however, the deck slab has inadequate load bearing capacity.

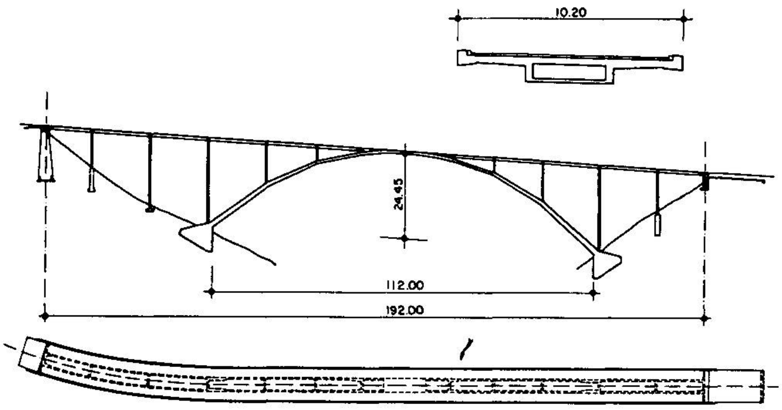

Fig. 4 Section, elevation and plan of Nanin Bridge

The actual rehabilitation started with the Cascella bridge in 1994 and will end up with the Nanin Bridge in 1996, including replacement of parapets and asphalt layer, strengthening of the deck slab by an additional concrete surfacing with additional reinforcement, fixing the downhill bearings to the abutments and enclosing the steeper parts of the arches with shotcrete on the lower and special cast concrete on the upper side.

6. CONCLUSIONS

As a result of the work in progress and illustrated by the examples, it can be stated, that
- assessment and evaluation are demanding engineering tasks,
- mostly, at least two options are conceivable and have to be considered carefully,
- the final decision on how to proceed, however, often depends on other non-technical requirements.

That is why a stepwise procedure with milestones for decisions is reasonable.

7. ACKNOWLEDGMENTS

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REFERENCES

1. ACI COMMITTEE 364, Guide to Evaluation of Concrete Structures Prior to Rehabilitation. ACI Materials Journal, Sept.-Oct. 1993 pp.479-498.
2. CEN TC104/SC8/WG7, General Principles for the Use of Products and Systems. Draft 11, July 1994. Unpublished.
3. SIA 162/5. Erhaltung von Betontragwerken. Various drafts. Unpublished.

Structural Evaluation of the Leaning Bell Tower of S.Stefano in Venice

Analyse structurale du clocher penché de Saint Stéphane, Venise

Konstruktionsanalyse des Schiefen Glockenturms der St. Stephankirche in Venedig

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SUMMARY

The San Stefano bell tower, Venice, approximately 60 m high, was constructed in two separated phases between the end of the 15th and the first half of the 16th century. It suffered severe structural deterioration due to a continuously increasing inclination caued by differential foundation settlements. The situation is not yet stabilised even after the extensive strengthening interventions executed at the beginning of the 20th century. The paper presents the main results of the experimental and theoretical investigations recently carried out in order to assess the actual safety conditions of the tower and, if needed, to suggest new interventions.

RÉSUMÉ

Le clocher, de 60 mètres de hauteur environ, a été construit en deux phases à la fin du 15e siècle et dans la première moitié du 16e siècle. Il a été soumis à des dégradations structurales très importantes résultant des tassements différentiels de sa fondation. Le phénomène n'est pas encore arrête malgré les restaurations effectuées au début du 20e siècle. L'article présente les résultats principaux de recherches expérimentales et théoriques qui ont été conduites récemment pour établir les réelles conditions de sécurité de la tour et définir des interventions supplémentaires.

ZUSAMMENFASSUNG

Der Glockenturm ist ungefähr 60 Meter hoch und wurde in zwei verschiedenen Zeitabschnitten zwischen dem Ende des 15. Jahrhunderts und der ersten Hälfte des 16. Jahrhunderts gebaut. Infolge einer ständig wachsenden Neigung bedingt durch das Nachgeben und die Senkung der Grundmauern, erlitt der Glockenturm schwerwiegende strukturelle Verschlechterungen. Dieser Zustand hat sich bisher noch nicht stabilisiert, obwohl zu Beginn des Jahrhunderts Eingriffe zur Verstärkung und Festigung vorgenommen wurden. In den Unterlagen werden die wichtigsten Ergebnisse der experimentellen und theoretischen Untersuchung dargelegt, die kürzlich zum Nachweis der derzeitigen Sicherheitsbedingungen des Glockenturms und gegebenenfalls zur Einführung neuer Eingriffe durchgeführt wurde.

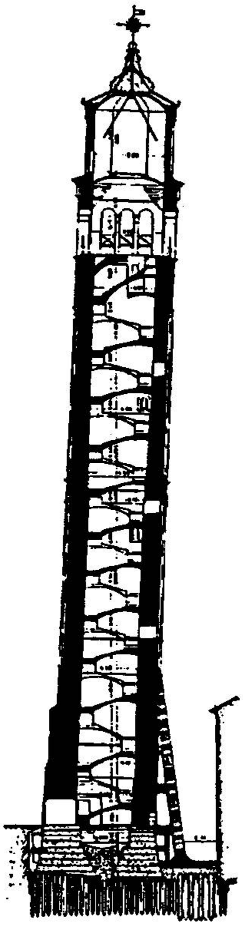


Fig.1 Section of the bell tower (1905)

1. INTRODUCTION

The S. Stefano bell tower, standing in the very center of Venice, suffered very similar structural deterioration phenomena as many historical tall masonry constructions in Italy. The tower has been built in two phases, the first in the second half of the XV century and the second in the middle of the XVI century. Its history is characterized by a continuously increasing inclination and sever damages, the most important due to lightning and fires.
What makes the case very peculiar are the very important strengthening interventions executed at the beginning of the 20th century. At the end of the XIX century the inclination was considered in fact to be increasing at a rate of the order of 7 mm per year, and several inspections of the cracks and measures of the inclination were being made. The collapse of the S.Marco tower made of course the situation even more dramatic, and after long discussions the project was approved to widen the foundation system and to construct inclined buttresses, in addition to execute several local strengthening interventions on the existing masonry walls (1903-1905).
In the following period (1903-1933), the inclination was still increasing, even if at a reduced rate of the order of 1.5 mm per year and new deterioration phenomena in the mean time appeared, regarding above all the steel ties installed at the beginning of the century.
For such reasons the tower is currently subjected to an extensive program of experimental and theoretical investigations with the aim of assessing its current structural behavior and safety level and deciding any possible interventions.

2. PRINCIPAL CHARACTERISTICS OF THE TOWER

The structure is an isolated tower, without any structural connection with the surrounding large buildings, among which is the S.Stefano church. Its height is approximately 60 m and the main dimensions of the horizontal plan section are 7.3 and 7.3 m. The thickness of the walls varies from 180 cm at the base to 110 cm at the belfry.
The original foundation rests on short timber piling and the east side of the building is bordered by a narrow channel, the "Rivo Menuo". At the height of 44 m above the sea level the tower show an out-of-plumb in the East direction of about 2m and in the South direction of 0.3 m, corresponding to an inclination of 2.6°.
The following intervention, executed during the period 1903-1905, are worthnoting from the structural point of view.

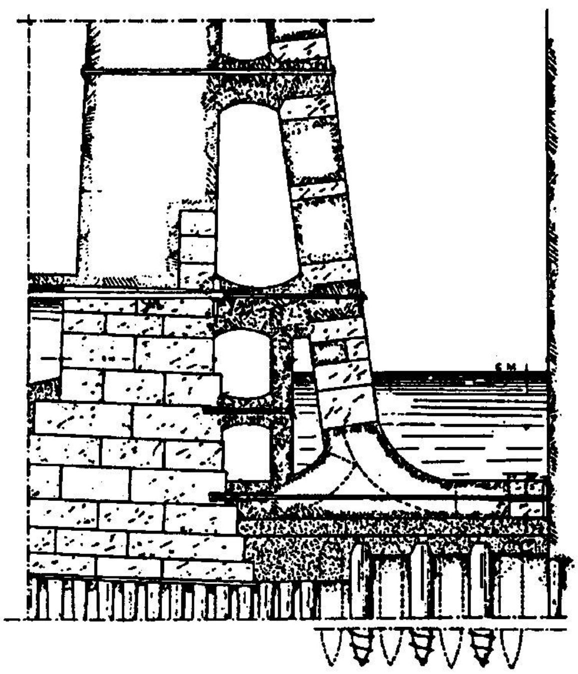


Fig. 2 Detail of the strengthening works executed in 1903-1905

Several steel ties were installed at different levels, starting from the foundation. The masonry walls were extensively repaired and strengthened by means of local reconstructions and injections of cement-water admixtures. The original foundations were first strengthened and then widened over the channel-bed constructing a 50 cm thick concrete slab over a concrete piling. New strong masonry buttresses were built with cement mortar, inclined with respect to the vertical direction as indicated in figure 1. They were securely connected to the tower walls and to the foundations, in the second case through masonry arches (Fig. 2).
The structural behavior of the tower was in this way substantially modified. The variation of the inclination was since then efficiently counteracted , even if not eliminated as expected by the designer, by the new structures as demonstrated by the sudden reduction of the increasing rate of the inclination. A mechanical pendulum was in that occasion installed to control the effectiveness of the interventions.

3. EXPERIMENTAL AND NUMERICAL INVESTIGATIONS

The first studies, which are here summarized, regarded the masonry structure in order to assess the actual safety conditions and to adopt, if necessary, adequate and immediate protection measures, while the investigations on soil-foundation system have been scheduled for a following phase.
At first a geometric survey was made together with an accurate detection of cracks and deterioration phenomena using mountain-climber's techniques to examine the external side.
Besides the deterioration phenomena which were already described at the beginning of the century, in particular the cracks pattern, new phenomena resulted to be clearly connected to the recent interventions. In particular the oxidation processes of all type metallic components resulted to be very severe, in some cases jeopardizing the efficiency of the important ties.

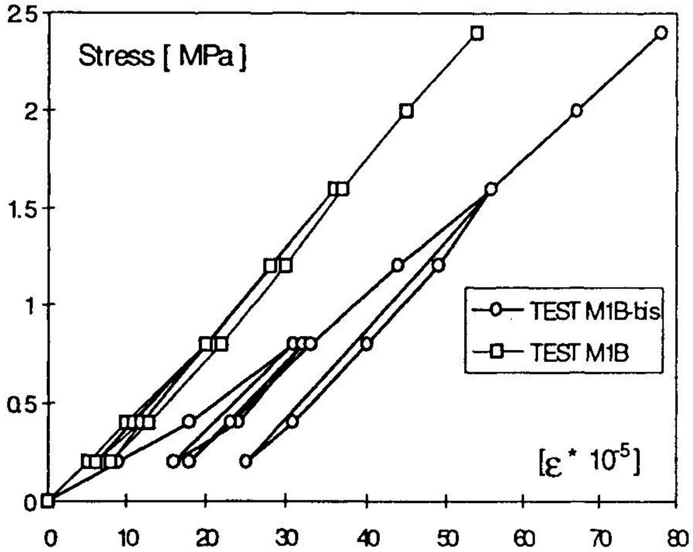


Fig. 3 Stress-strain diagram measured on the external leaf (M1B) and in the internal leaf (M1B-bis)

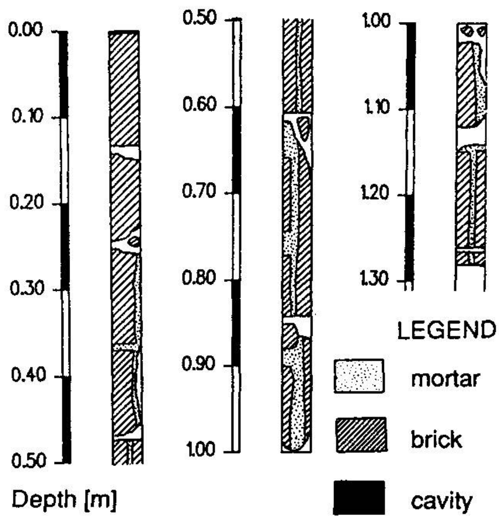


Fig.4 Typical "through-the-thickness" masonry composition

The masonry composition was then investigated by means of cores boring and bore-hole video surveys while the mechanical properties and the state of stress have been measured by flat-jacks tests.
A typical example of masonry composition is shown in Figure 4, where the inner core of the wall appears similar in composition to the outer leaf and almost without voids and cavities.

The masonry constructed in the XV century exhibits a mean modulus of elasticity in the vertical direction of about 4000 MPa (in the 0.4-0.8 MPa range of applied stresses) while the upper part of the tower, built in the middle of the XVI century, show an higher deformability with a mean value of the modulus of 2800 Mpa.
One of the tests was performed both in the external leaf and in the inner core of the XV century masonry. The comparison of the stress-strain curves (Fig. 3) clearly show the difference in the deformability, the inner core modulus is however greater than 2800 MPa and the behaviour is approximately linear-elastic until the maximum imposed stress level of 2.4 MPa.
The modulus of elasticity of the buttresses has been estimated (by means of the results of the single flat jack tests) to be of the order of about three time higher than the modulus of the original masonry.
Several single flat-jack tests were performed at different height to assess the distribution of vertical stresses with a special attention to the problem of the efficiency of buttresses and their effects on the overall structural behaviour of the tower.
Such results were compared with those obtained numerically by means of a FE model (Fig. 5) constructed assuming the mechanical properties of the materials obtained from the above tests.
The FE model has been subsequently calibrated also by comparing dynamic parameters experimentally obtained by recording and elaborating environmental and forced vibrations with the theoretical ones.
On the model thus calibrated static analyses were performed in both linear and non-linear ranges of material behavior mainly to indagate the influence of the buttresses on the distribution of the internal stresses (Fig.7).
The comparative analyses demonstrated that, as there are not no-tension zones, the influence of the material non-linearity is negligible.

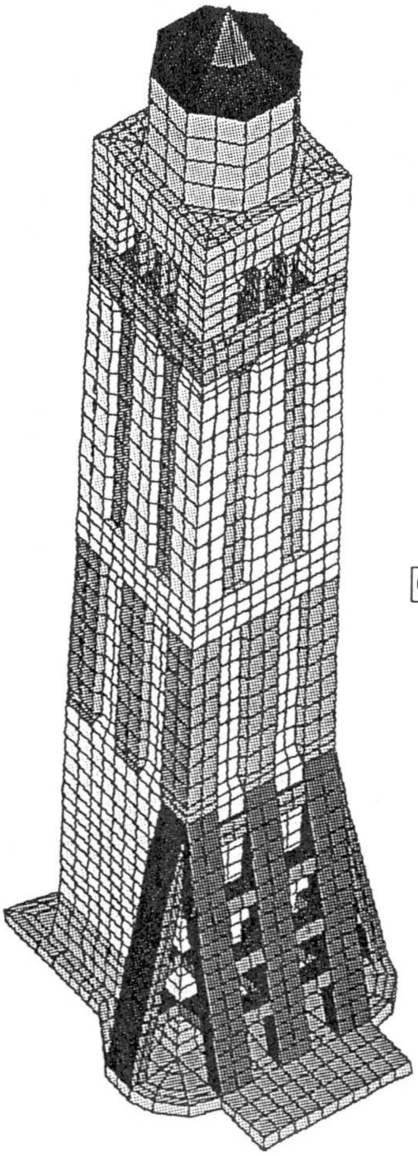


Fig. 5 Mesh of the numerical model

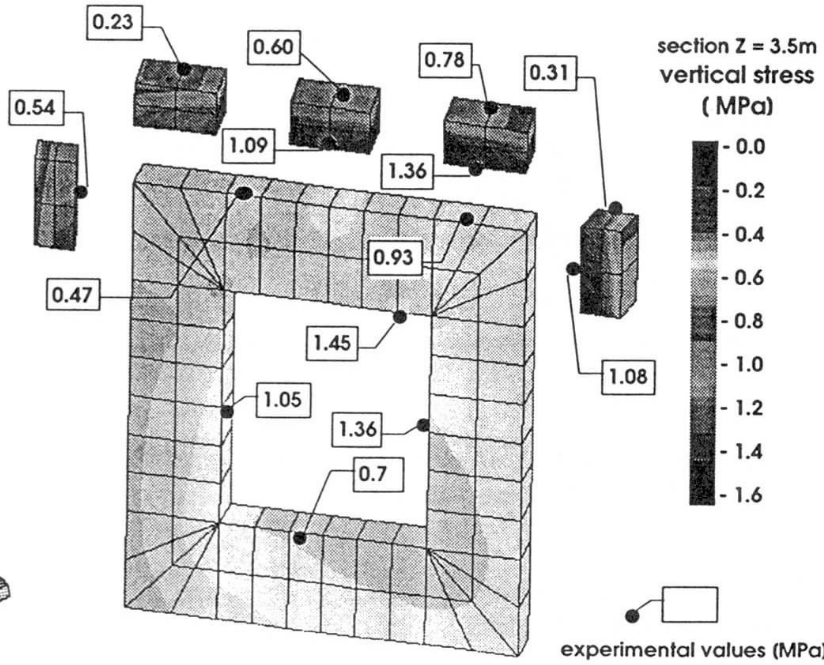


Fig. 6 Experimental and numerical stresses (horizontal section)

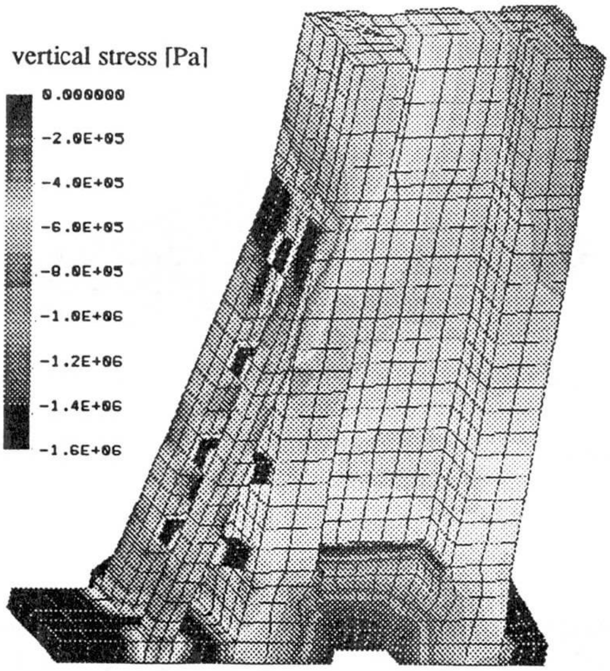


Fig.7 Vertical stresses at the base of the numerical model (vertical section)

Figure 6 shows one example of comparison between theoretical and experimental distribution of normal stresses due to gravitational load. As it can be seen, a fairly good agreement exists between the experimental and theoretical results, except for local inconsistencies which can be explained taking into account the material inhomogeneity, experimentally demonstrated but not yet included in the model. The state of stress is in general compatible, with acceptable safety margins, with the compressive strength of the masonry.

The analysis of the model reproducing the configuration before 1903 (i.e. without buttresses) with the present inclination, shows that the compression stresses would be very close to the masonry strength.

Dynamic investigations have been also performed and helped in both model identification procedures (as discussed above) and in practical evaluations, especially as far as the excitations induced by the bells are concerned.

The measurements were made by means of seismic piezoelectric accelerometers fixed to the structure at different height on the two lateral directions. On those tests two types of dynamic excitation were analysed: vibrations from wind and vibrations from bell movements.

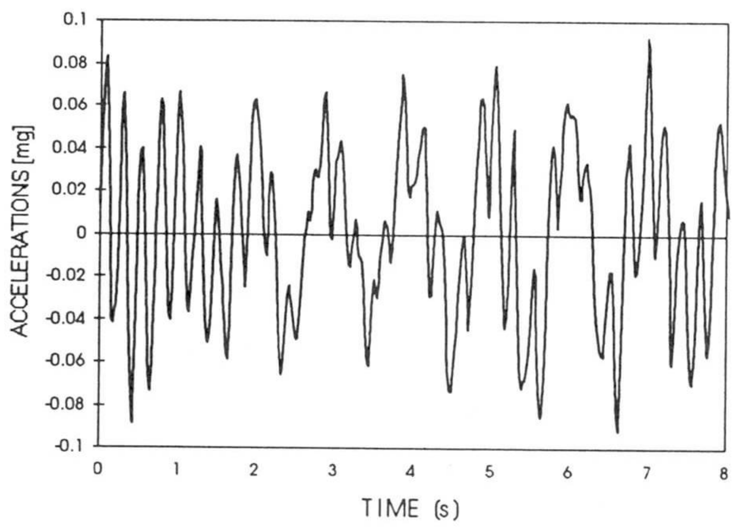


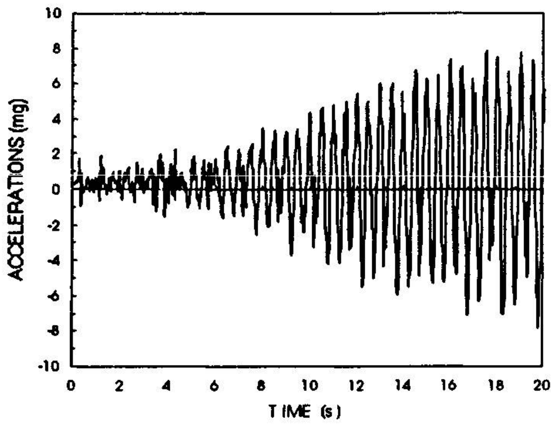
Fig. 8 Wind induced vibrations

From the analysis of wind induced vibrations the first five vibration modes of the structure (two flexural modes in both North-South and East-West direction and the first torsional mode) were determined [2].

The analysis of horizontal forces associated with bells (strike rate of 40 impacts/min. and swing angle of about 90°) showed that the third harmonic component of the force predominates and it is very close to the frequency of the first flexural mode of vibration of the structure causing a dangerous resonance phenomena. The effects measured on the structure were vibrations clearly perceptible to the persons [1] with a displacement amplitude of over 1 mm.

In Figure 9 and 10 a typical record of an acceleration time-history and the correspondent evolutionary power spectra are displayed, from which the above consideration were derived.

Finally, the mechanical pendulum was restored allowing for a new measure of the inclination, which demonstrated that since 1933 the inclination increased at a rate of 1.3 mm per year.



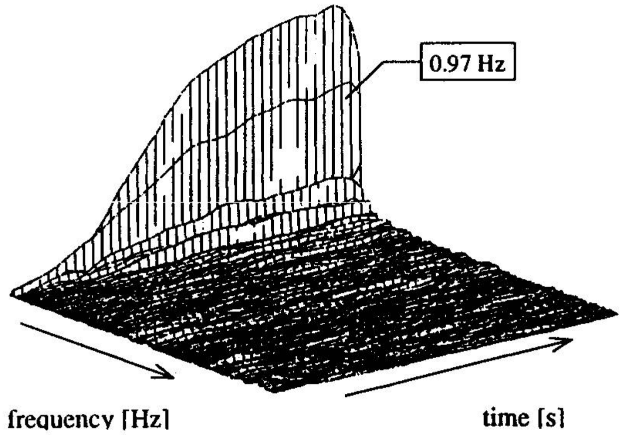


Fig. 9 Vibrations induced by bell movements
Acceleration Vs Time diagram

Fig. 10 Vibrations induced by bell movements:
Short Time Fourier Transform

4. CONCLUSIONS

The investigations demonstrated that, in the average, the tower is presently in acceptable safety conditions. The state of stress is in fact compatible with the masonry strength and the inclination will attain critical values for the global stability [4], with the current increase rate, not before 60 years.
They also demonstrate the crucial role played by the strengthening intervention of the beginning of the century in ensuring the current safety condition of the structure.
The problem of the stability of the foundations has not been however definitively solved, and adequate investigation and intervention must be carried out within the above indicated time limit. Moreover. the bell system must be changed in case it will be used again in order to avoid the previously mentioned resonance problems.

REFERENCES

1. BACHMANN H. AMMANN W., Vibrations in structures. Structural Engineering Documents IABSE, July 1987.
2. EWINS D.J., Modal Testing: Theory and Practice, Research studies press, February 1991.
3. MODENA C., Repair and upgrading techniques of unreinforced masonry structures utilized after the Friuli and Campania/Basilicata earthquakes, Earthquake Spectra, February 1994.
4. J. HEYMAN J. , Leaning towers, Meccanica vol. 27, 1992.
5 CASTOLDI A., CHIARUGI A., GIUSEPPETTI G.,FANELLI M., In-situ dynamic tests on ancient monuments, IABSE Colloquium, Bergamo 1987
6. ROSSI P.P., Non destructive evaluation of the mechanical characteristics of masonry structures, Proc. of the Conference Non Destructive Evaluation of Civil Structures and Materials, University of Colorado, October 1990.

Evaluating and Strengthening Industrial Buildings Damaged by Fires

Réparation de structures de bâtiments industriels endommagées par un incendie

Brandschäden und nachfolgende Reparaturen von Industriebauten

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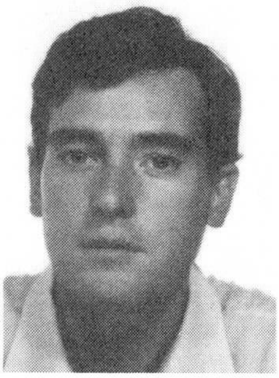

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SUMMARY

The lifespan of an industrial building may be dramatically reduced by fire, even if all safety precautions have been taken. The restoration of a fire-damaged reinforced concrete frame requires, first of all, the evaluation of the damage suffered by the members. In this respect non-destructive testing methods appear suitable for a quantitative definition of the damage; this can lead to the selection of a proper restoration technique. The proposed procedures are illustrated with an example.

RÉSUMÉ

La vie d'un bâtiment industriel peut être réduite de façon dramatique à la suite d'un incendie, même si tous les systèmes de sécurité étaient installés. Le renforcement d'une ossature en béton armé nécessite, au préalable, l'évaluation du dommage aux éléments struc-turaux. Les essais non destructifs sont indiqués pour apprécier la sévérité des dommages pour choisir la technique de réparation, et son intensité. L'utilisation des procédées pré-sentées est illustrée par un exemple.

ZUSAMMENFASSUNG

Die Lebensdauer eines Industriebaues kann durch Brand erheblich reduziert werden. Die Verstärkung eines durch Feuer zerstörten Stahlbetonrahmens bedarf zuerst einer Erhebung der zerstörten Elemente. Zu diesem Zweck scheinen zerstörungsfreie Testmethoden für die quantitative Beurteilung des Schadens günstig. Die vorgestellten Abläufe werden an einem Beispiel illustriert.

1. INTRODUCTION

One of the major events that can dramatically reduce the life span of a structure is fire spread; as a concern, industrial buildings, due to the inherent danger of machinery and plants, show a very high proneness to fire. Moreover, the need of a free plan leads to a generalised use of large span, thin walled, precast prestressed structures which are very sensitive to fire.
Despite the large effort of outstanding Technical Committees in order to produce classification schemes and strengthening guidelines [5,6], a successful run of a fire damaged building restoration requires the crossing out of some critical decisions, and the collection of a large base of quantitative data which are always puzzling their correlation to a general description of the phenomenon.
However, although experience can help to set the correct redesign philosophy, it does not possess any value in establishing the pointwise distribution of strengthening in a large span damaged area.
So, it appears clearly that one of the earlier critical decisions is the selection of appropriate testing and evaluation techniques, as well as the mesh at which to do the measurements. In fact, the difficulty to obtain a reliable base of quantitative data, can sometimes lead to a choice of total demolition.
In the following, we firstly present a summary of critical decisions that can help in the preparation of an operative program to be detailed for a particular site under investigation. After, we reassess the use of combined experimental techniques of both destructive and non destructive type, in order to quantify the damage suffered by R/C structures during the event of a large fire spread. With relation to a far field investigation carried out in the past [2], some practical hints regarding the consistency of data acquisition are presented.
Finally, with reference to a particular case history, we discuss the established repair techniques and their grading on the basis of the damage level assessed; in fact, once the structural strengthening has been detailed, we are able to forecast the life span extension produced by it [10].

2. PLANNING THE OPERATION SCHEDULE AND DATA ACQUISITION

In general, the consulting work begins few days later the fire accident, when some urgent remedies are to be planned immediately. After, we have time for field and laboratory measurements, and finally the building can be restored to its original functionality. These three design steps involve different knowledge levels; so it is important to activate only such operations that agree with the level of understanding gained. As a concern we can cite the following points:

a) *initial operations*
- locking of the chemical pollutant resulting from combustion,
- demolition of unsafe or near collapse structures,
- evaluation with the company management of the conditions needed for the restart of productive lines.

b) *mean time operations*
- collection of experimental data and interpretation,
- restoration of undamaged parts of the building,
- restart of power and fluid distribution plants all along the safe or undamaged zones.

c) *final operations*
- generalised rehabilitation of structures and technological systems in the damaged parts of the building,
- support to the company management in setting the reports relevant for the submission of the insurance refund request.

The definition of the activities involved in a given time schedule detailing the previous list, is a complex task that cannot be discussed in a general way; so, we shall reconsider only the strategies available for the damage evaluation.

2.1. In Situ Strength Determination of Concrete

Several combined non destructive (NDT) and destructive (DT) testing techniques have been prepared and studied in the past [4,7]; however the complex nature of interacting phenomena underlying the fire withstands their rational use.
More precisely the temperature gradient due to fire induces non homogeneous property distributions which are not likely to be detected by NDT sensing surface properties. So we recognise that Schmidt Rebound Hammer, Windsor probe and pull out methods give information only on an external layer, and cannot be combined with volume averaging probes such as ultrasonic or x-ray scanners.
In the industrial site of Merloni Company at Nocera Umbra, Italy, which is a deeply studied interesting example [1,2], we used both DT and NDT procedures:
- ultrasonic pulse velocity (UPV) measurements of both direct and indirect type;
- Schmidt rebound index measurements (SRH);
- drilled core sampling and analysis (DC, as described later),
- load testing of simply supported elements up to the serviceability level (LT).

With relation to the statistical robustness of the measures [8], we adopted the following procedure (see fig.s 2.1÷2.3):

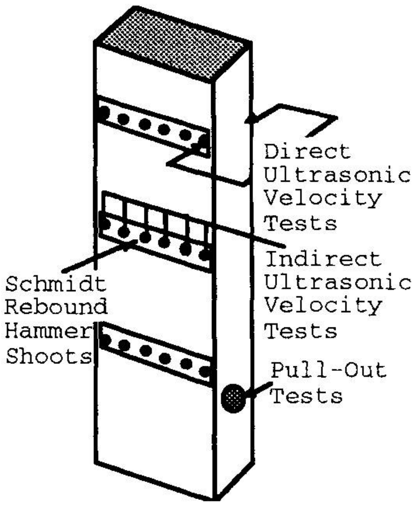

Fig. 2.1: *Test positioning arrangement for a column*

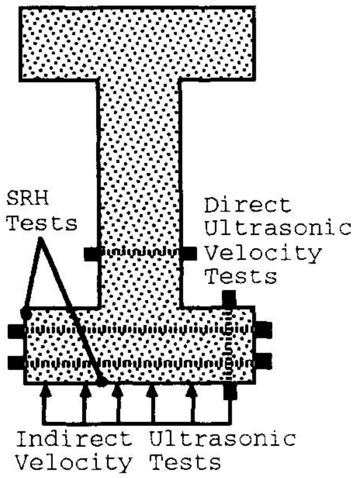

Fig.2.2: *Test positioning arrangement for a beam*

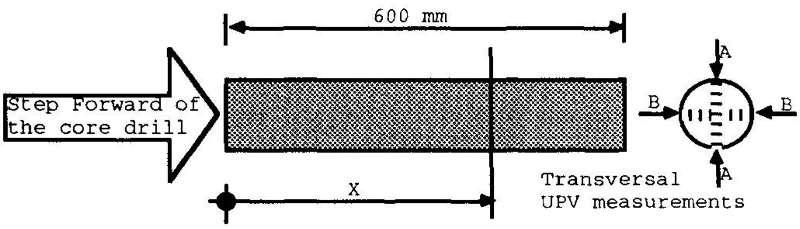

Fig. 2.3: *Test procedure for the assessment of transversal UPV in cores*

a) *data collection*
   - definition of three representative sites in each investigated element,
   - execution of ten SRH index evaluation for each site,
   - execution of three UPV direct transmission evaluation for each site,
   - execution of indirect transmission UPV test on the more damaged site,

b) *calibration tests*
   - extraction of CD samples for the whole length of a member side,
   - UPV transversal measurements along the cylindrical core at a fine mesh,
   - preparation of standard height cylinders cutting the core,
   - execution of axial UPV measurements and strength tests on the cylinders.

The correctness of the calibration tests can be improved doubling the sampling in some typical elements and charging for tests two different Laboratories; this care can be used also for the in situ measurements charging a second team for the repetition of some investigations. With this technique systematic errors or incorrect workmanship execution can be detected and compensated.

2.2. Damage evaluation

Several different damage pattern are present in a fired building and the strongly non homogeneous distribution of the residual strength can withstand the selection of a representative damage measure. On the other hand we must remember that scientific knowledge does not shadow engineering interpretation which is the main tool for redesign.
As a matter of fact we can try to construct residual strength distribution inside damaged elements, but the only necessary information is the mean residual strength of the core and the depth of the superficial deteriorated layer.
In the Literature there are several proposal for the damage evaluation of concrete structures [6], but the lack of specific analytical methods let us resort to phenomenological models. In detail we have:

- strength vs. ultrasonic velocity calibration curves,
- damage vs. ultrasonic velocity decay experimental relationships,
- indirect analytical and numeric methods.

In this last set up we make use of one or more transition variables, such as expressing the max. temperature with relation to the ultrasonic velocity, and the damage as a function of the temperature [1,3].
In the evaluation made in the past we found that SRH has low sensitivity in predicting the average strength, so we decided to use this method only for the detection of debonded layers. In fig. 2.4 the experimental correlation of SRH is illustrated for the cited example.

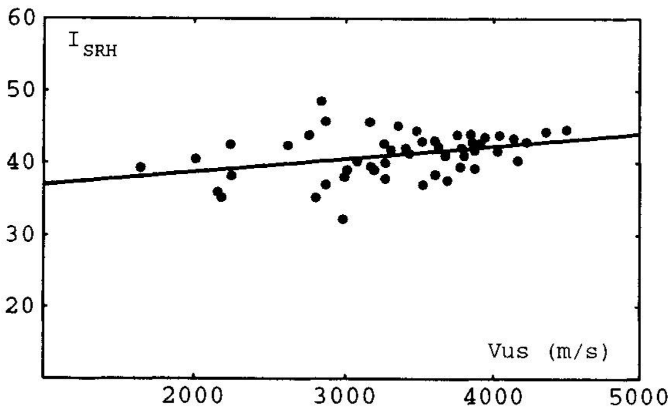

Fig. 2.4: *SRH Index as a function of the direct UPV measure*

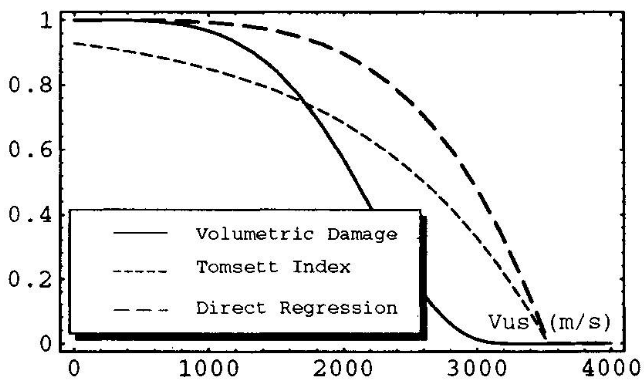

Fig. 2.5: *Damage evolution resulting from the three models*

As a matter of exposition we present the following relationships (the index "T" denotes a quantity reduced by temperature rise):

a) *direct regression of calibration core data:*

$$f_{cT} = k\, V_{usT}^4 \; ; \tag{1}$$

the coefficient k, can be obtained from a statistical analysis of core tests.

b) *Direct damage index evaluation (Tomsett [12]):*

$$\mathrm{Ln}\left(\frac{f_{cT}}{f_c}\right) = \mathrm{Ln}\,(\rho_{cT}) = -\frac{f_c}{f_{ref}}\frac{V_{us}}{V_{ref}}\left(1 - \frac{V_{usT}}{V_{us}}\right), \tag{2}$$

where the reference strength and ultrasonic velocity correspond approximately to the lower allowable concrete class ($f_{ck} \approx 15$ MPa);

c) *Indirect set up [2]:*

we postulate the functional relations:

$$\frac{V_{usT}}{V_{us}} = \Lambda(x_\alpha) \ , \qquad \frac{f_{cT}}{f_c} = \Phi(x_\alpha) \ , \qquad \alpha=1,n \ , \tag{3}$$

where the internal variables x_α must be connected by n-1 relationships; we can then construct the following damage ratio formula:

$$D_{cT} = 1 - \frac{f_{cT}}{f_c} = 1 - \rho_{cT} = 1 - \Phi\left[\Lambda^{-1}\left(\frac{V_{usT}}{V_{us}}\right)\right] . \tag{4}$$

A more involved problem is the evaluation of the depth of the external deteriorated layer (for a deeper discussion see[2]); however, the demolition works of the concrete cover corresponds to a generalised consistency test, which allows to leave out more involved evaluation tests.

3. A SUMMARY OF REHABILITATION TECHNIQUES

With relation to the cited example (which will be discussed later), we set up different restoration strategies for the element classes composing the building:

a) prestressed precast this sections: total substitution,
b) massive R/C beams and columns: graded levels of restoration.

It should be pointed out that massivity (high volume over surface ratio), is a determinant feature toward fire resistence of structures.

3.1 Column Restoration

Under the proposed restoration techniques [1,6] lies the notion that transversal confinement can improve by far the allowable axial load. The work schedule is:

a) Demolition of the external cover up to steel reinforcement net and locally, of the still remaining low compactness concrete,
b) execution of the holes for the connecting steel ties,
c) clamping of steel ties in the R/C core by means of epoxy resin mortars,
d) construction of a new steel reinforcing net linked to ties,
e) erection of the wooden case and cast of the new external layer using super fluid additivated high strength micro concrete (gravel size < 8 mm).

3.2. Beam Restoration

In this case we had a very small damaged layer on the lower flange but care was to take of the anchorage zones; we provided a sequence similar to that of the columns, with the only difference that the concrete was cast in place by means of grouting of a tixotropic no shrinkage mix and hand levelling.

4. AN EXAMPLE: THE MERLONI INDUSTRIAL BUILDING

We conclude presenting some interesting features of a restoration work completed few years ago; in fig. 4.1 the appearance of the building after the fire event is outlined. One of the preliminary requests of the company management was to allow for a production restart as soon as possible.
So we decided to test the survived thin roof shell prestressed elements in order to define the extent of roof owed to substitution. In fig. 4.3 is presented the final dismount line as defined making use of direct load tests on all the tiles placed up to the line itself. The complexity of a decision problem of this type can be suggested by the number of parts included in the damaged zone: we dealt with three modules comprising each 77 columns, 66 beams and 260 roof shells.
The wide experimental investigation carried out onto the beams and columns of the two highly damaged modules, allowed for the construction of a chart describing average residual concrete strength ratio ρ_{cT} shown by columns (fig. 4.4), and to apply rehabilitation techniques graded to the damage distribution.
We concluded also that the beams, due to their high massivity, were only deteriorated in the thin superficial layer acted on by fire. A reason for this can be inferred from the relatively low (500°-700°C) temperatures suffered by the R/C frames [3], as a consequence of the premature collapse of the roof.

Fig. 4.1.: *A.Merloni industrial Site after the great 1993 fire*

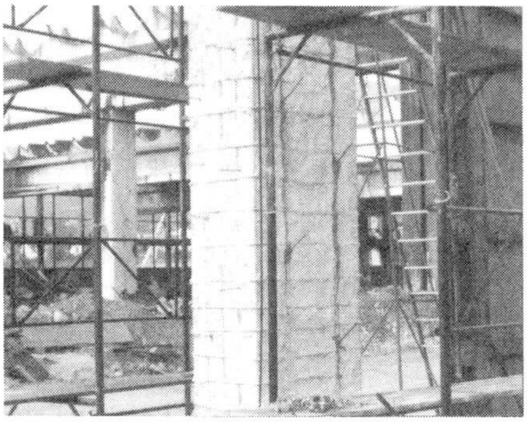

Fig. 4.2: *A column core during the rehabilitation works*

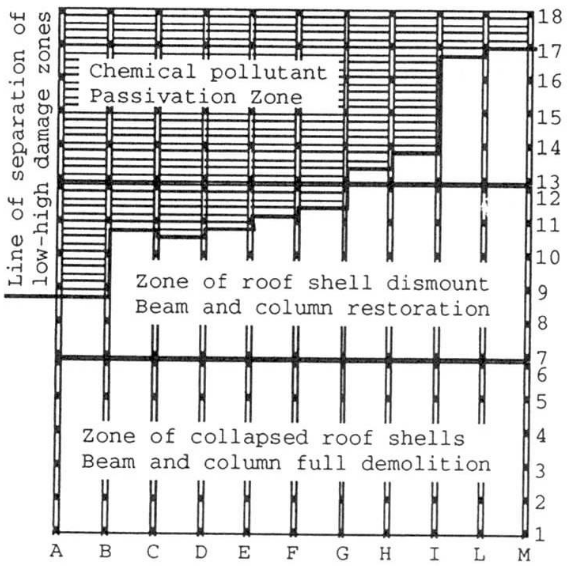

Fig. 4.3.: *After tests definition of restoration zone*

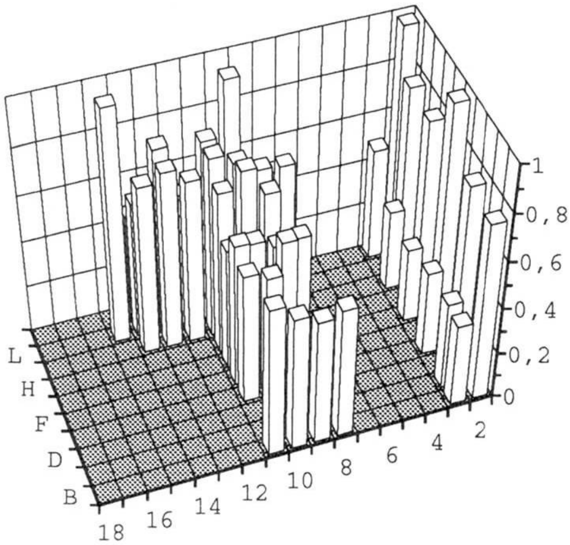

Fig. 4.4: *Residual strength plot of investigated columns*

REFERENCES

1. A. BENEDETTI, M. BARBONI, (1993), "Quantificazione del danneggiamento da incendio in strutture di c.a.: lo stabilimento A. Merloni di Nocera Umbra", 7° *Congresso Nazionale A.I.P.nD.*, Ferrara, Italy.
2. BENEDETTI A., (1995), "On the Ultrasonic Pulse Propagation into fire damaged concrete", submitted to *Mat. & Struct.*
3. LOGOTHETIS L., ECONOMOU C., (1981), "The influence of high temperatures on calibration of non destructive testing of concrete", *Mat. & Struct.*, **79**, 39-43.
4. RILEM TC 43, (1993), "Draft recomendation for *in situ* concrete strength determination by combined non-destructive methods", *Mat. & Struct.*, **26**, 43-49.
5. RILEM TC 104, (1994), "Draft recomendation for damage classification of concrete structures", *Mat. & Struct.*, **27**, 362-369.
6. SCHNEIDER U. Ed., (1990), "Repairability of Fire Damaged Structures. CIB W14 Report", *Fire Safety Journal,* **16**, 251-336.
7. TOMSETT H.N., (1980), "The practical use of ultrasonic pulse velocity measurements in the assessment of concrete quality", *Mag. of Concr. Res.*, **32**, 7-16.

Structural Assessment of Corrosion Damaged Steelwork

Évaluation structurale des dégâts dûs à la corrosion des ouvrages en acier

Tragfähigkeitsbeurteilung korrosionsgeschädigter Stahlbauteile

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SUMMARY

Four severely corroded universal beams were removed from a structural frame, that was undergoing refurbishment, and were load tested to failure in the laboratory. The failure loads were compared with several analytical models including one simulating their as-new condition. They were found to possess surprisingly high residual capacity, prompting a proposal to quantify current visual inspection procedures.

RÉSUMÉ

Quatre poutres sévèrement corrodées extraites d'une structure en cours de rénovation ont été testées jusqu'à la ruine en laboratoire. Les charges ultimes ont été comparées avec plusieurs modèles numériques, dont un simulant leurs nouvelles conditions. Les essais ont montré que les poutres possédaient une capacité résiduelle étonnamment élevée, conduisant à une proposition pour quantifier les procédures d'inspections visuelles courantes.

ZUSAMMENFASSUNG

Von einem Rahmentragwerk, das gerade überholt wurde, wurden vier stark korrodierte, allgemein gebräuchliche Träger entfernt und im Labor bis zum Bruch getestet. Die Bruchlast wurde mit mehreren Berechnungsmodellen verglichen, darunter einem, das den Neuzustand darstellte. Die Tests ergeben eine überraschend hohe Resttragfähigkeit, aufgrund derer ein Vorschlag zur Quantifizierung derzeit verwendeter Inspektionsverfahren ausgearbeitet wurde.

1. INTRODUCTION

In the UK there are literally thousands of exposed steel structures supporting chemical manufacturing plant and refineries. The aggressive atmosphere and exposed location of such plant can result in severe and rapid corrosion. Moreover, in the UK about 25% of steel structures in the chemical industry are more than 50 years old resulting in the need for substantial maintenance and repair which is difficult and costly because of the need to sustain continuous production.

Current practice in the management of inspection and repair of exposed structural steelwork has been described by Gallon [1]. He found that the current inspection and assessment methods, while being safe, were significantly conservative in some instances indicating the need for more realistic appraisal of the capacity of degraded structures.

Similar concerns exist for steel girder bridges. Kayser and Nowak [2] developed a theoretical corrosion damage model for typical short span steel highway bridges. They showed that the governing criteria, which depend on the relative thicknesses of the web and flange, change with time as corrosion progresses. However, the web is generally the most vulnerable element in the long term.

Regular inspection of steel structures in the chemical industry is usually based on visual examination and classification into categories which identify the need for appropriate action [3]. The most severe visual category refers to the presence of serious structural defects and the consequent need for full structural assessment and repair. However, as yet there is no clear relationship between the magnitude of structural defects and the corresponding reduction in capacity. There is an urgent need for this information to avoid the financial penalty of plant closure when the capacity of the supporting steelwork may be adequate.

The objectives of this paper are to compare a corrosion damage model with tests to failure on some severely corroded steel I-beams, and to use the model to help quantify visual inspection.

2. LOAD TESTS ON CORRODED BEAMS

2.1 Specimen beams

Four identical universal beams were recovered from the site of a plant undergoing demolition. The beams formed corner supports for a tank as shown in Fig 1 and were all in severely corroded condition. Detailed measurements of the thicknesses of the webs and flanges were made and are summarised in Table 1.

Table 1 Average measured thicknesses of corroded I-beams (mm)

Element	**As new**	**Beam 1**	**Beam 2**	**Beam 3**	**Beam 4**
Flanges					
Top	10.20	7.25	7.55	6.55	7.94
Bottom	10.20	4.95	5.70	4.52	6.62
Average thickness	10.20	6.10	6.63	5.54	7.28
Thickness loss	nil	4.10	3.57	4.66	2.92
Web					
Average thickness	6.10	4.98	5.38	4.93	5.56
Thickness loss	nil	1.12	0.72	1.17	0.54

It will be noted in Table 1 that the loss of thickness on average was more significant in the flanges than in the webs. However, holes were found in the webs and flanges of two of the beams, one of which is shown in Fig 2.

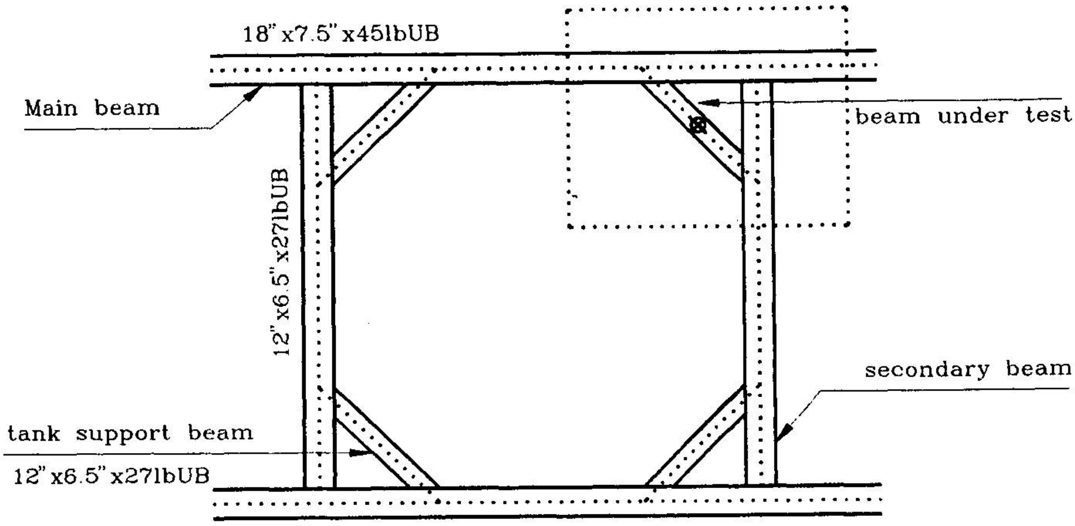


Fig 1 General arrangement of tank support beams

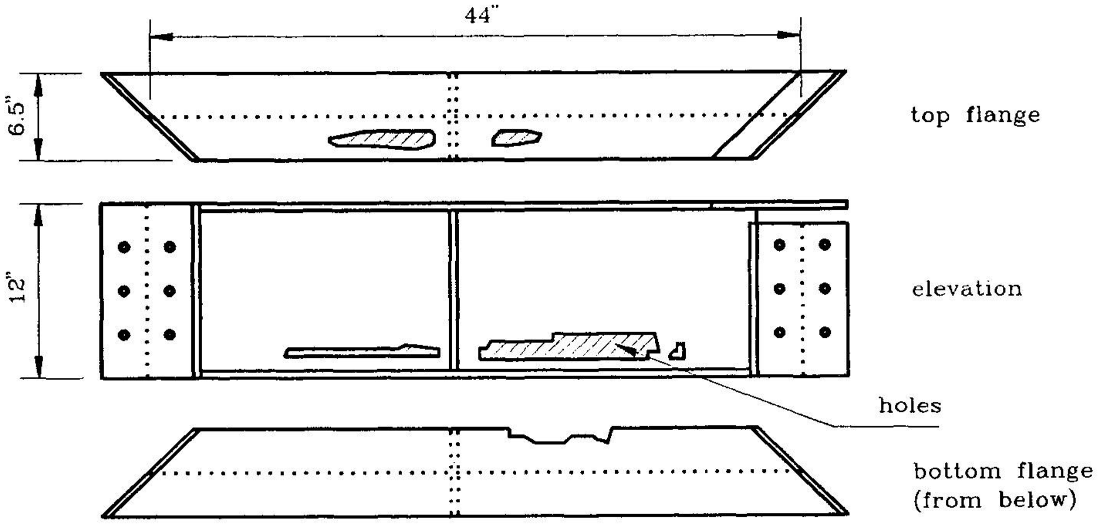


Fig 2 Details of holes in beam No 1

2.2 Load tests to failure

Each beam, when tested, was bolted at its ends to the webs of short lengths of I-beam designed to simulate the support conditions for the steel tank in service (see Fig 1). The assembly was installed in an hydraulic testing machine and the load was applied vertically to the test beam directly over the central stiffener. The load was applied through a ball and slider seating to permit rotation of the cross section of the test beam, as would be the case in service. The vertical deflection of the loaded point was recorded, together with lateral deflections of top and bottom flanges at the same section. This provided information on the onset of lateral torsional buckling and is shown in Fig 3.

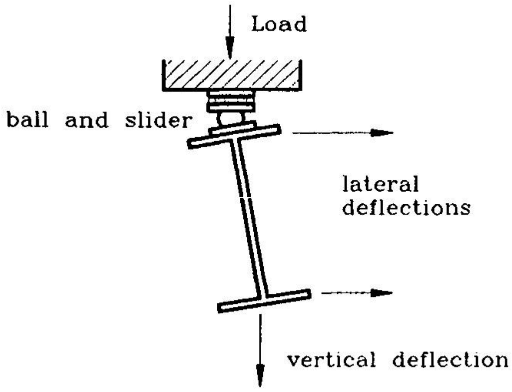


Fig 3 Loading and deflection measurements at central section

The ultimate loads obtained in the tests are given in Table 2. It may be seen that there is a correlation between the condition of each beam and its failure load. The failure loads are compared with the capacity of a new beam calculated using the formulae in the UK code for structural steelwork [4]. Partial safety factors were not used for this purpose. The predominant mode of failure was by lateral torsional buckling originating from the cut-out portion of the top flange.

Table 2 Ultimate loads in the tests compared with calculated as-new capacity

Beam No	**Ultimate Load/kN**	**General condition of beam**
1	277.0	Severe loss of material; holes in flanges and lower web
2	318.0	Loss of material all over but no holes
3	287.0	Severe loss of material; holes in flanges and lower web
4	440.0	Loss of material all over but no holes; fairly good condition
all	522.7	Calculated capacity of as-new beam

3. STUDY OF FAILURE MODES OF CORRODED BEAMS

A steel I-beam subjected to bending can fail in the following modes:

a) exceeding moment capacity as determined by the yield strength;
b) lateral torsional buckling;
c) failure of the web in shear;
d) bearing failure of the web under point loads or at the supports.

BS5950 [4] was used for the assessment of the moment capacity, which is mainly dependent on the yield strength of the steel and the flange area of the beam. Lateral torsional buckling was the most important failure mechanism for these beams, but this was largely because of the cut-out in the flange at one end. This is called a "coped beam" and the lateral end restraint is considerably reduced because rotation of the flange in plan is not resisted at the coped end. The method of assessment proposed by Cheng et al [5] was used in this case.

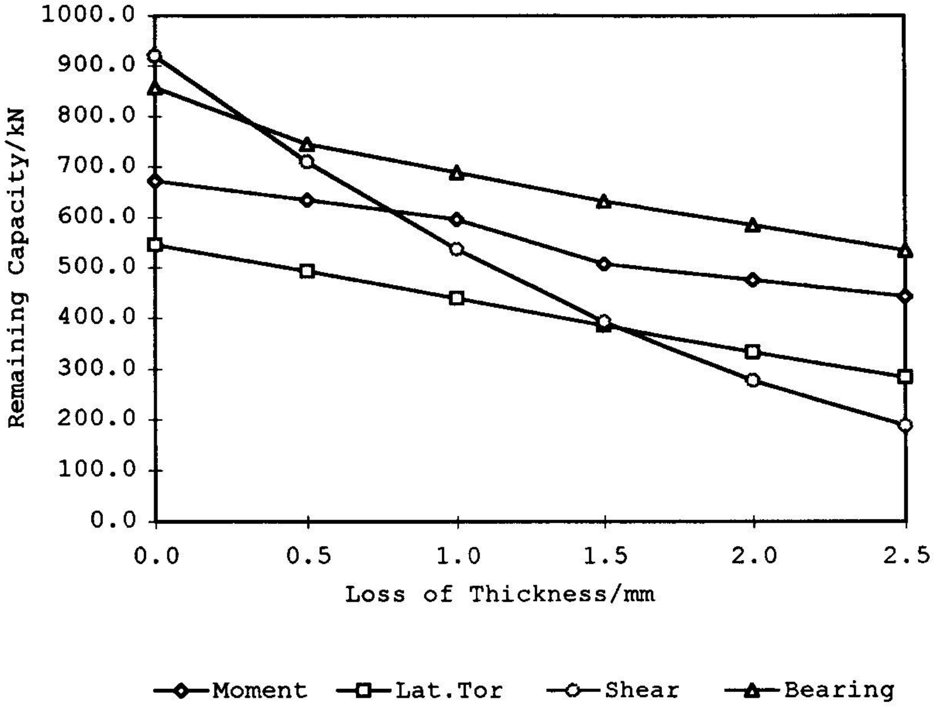
Failure of the web due to shear buckling was the next most significant failure mechanism. The effect of varying web thickness (VWT) was calculated using the formulae in BS5950. The effect of web holes (WH) was assessed by the method of Wang et al [6]. This assumes that plastic deformation occurs near each of the corners of the opening (rectangular openings). A moment-shear interaction curve is identified from which the capacity of a particular load case may be obtained.

While loss of thickness generally reduces the capacity of a loaded beam, it can also change the mode of failure from one mechanism to another depending on the relative thickness loss in the various parts. Three corrosion damage models were therefore analysed, as follows:

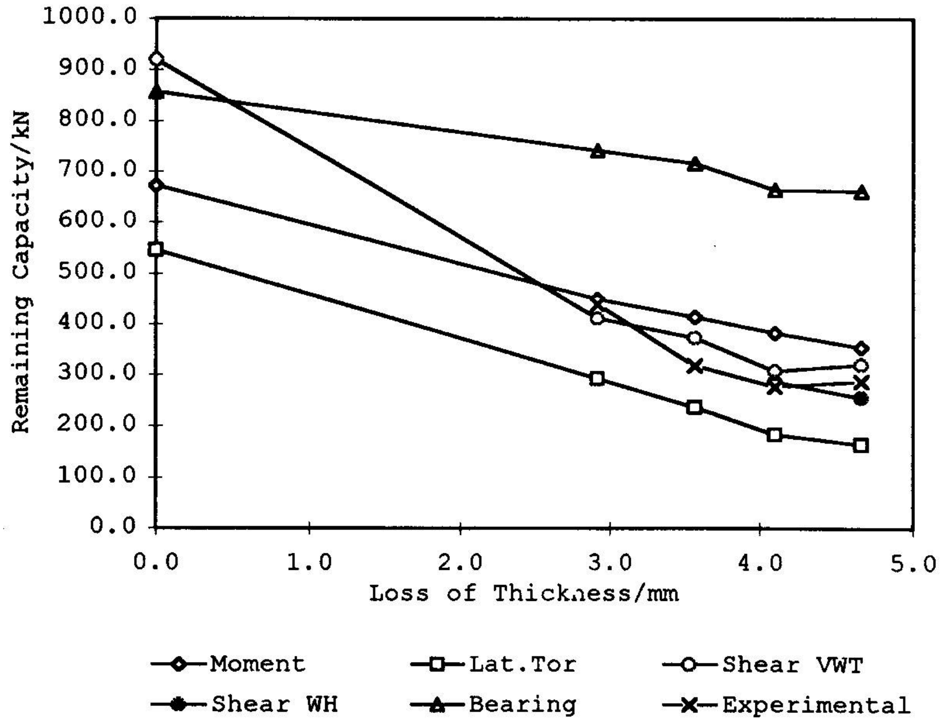
i) the I-beam in its as-new condition;
ii) uniform thickness loss in both flanges and web;

iii) variable thickness loss in flanges and web (the ratio of thickness loss in flange to web was the same as that of the beams tested)

The results are shown in Fig 4.



a) Uniform thickness loss



b) Variable thickness loss

Fig 4 Remaining capacity of corrosion damaged I-beams

4. APPLICATION TO VISUAL INSPECTION

Formal procedures for inspection of structural steelwork already exist in the petro-chemical industry [3]. This involves initial appraisal, preliminary inspection, detailed examination if necessary, and reporting. The preliminary inspection is basically a visual assessment and requires structural elements to be described using defined condition categories, for which subsequent actions are prescribed. Brief descriptions of typical visual categories are as follows:

i) minimal deterioration; no need for immediate action
ii) paint system generally defective, but loss of section estimated visually to be less than about 15%; action required to arrest deterioration
iii) loss of section estimated to be greater than 15%, holes in members, distortion, damaged connections; requirement for further investigation, repair, replacement or removal.

These procedures, properly carried out, provide invaluable assurance that structures for production plant are safe. However, it would be most helpful if visual inspection could be related quantitatively to remaining capacity. It is proposed that the results of the study of failure mechanisms be used as a starting point to achieve this objective. It is evident from Fig 4 that lateral torsional buckling and web shear were the most critical mechanisms. In these cases it was found that if the loss of flange thickness was less than 15% then the remaining capacity was about 75% of the as-new condition. However, more work is required to define a generally acceptable lower bound for a wider range of load cases and configurations. Loss of moment capacity is almost proportional to loss of flange area, so this mechanism is less critical.

The existence of holes in a member is likely to reduce the capacity significantly, although this will depend on the size and location of the holes. At the present time no definitive guidance may be given. However, it should be noted that in this investigation it was found that even the most severely corroded beam possessed more than 50% of its calculated as-new strength.

ACKNOWLEDGEMENTS

The authors are grateful to ICI Engineering Technology for their assistance, and to the Engineering and Physical Sciences Research Council for their support.

REFERENCES

1. GALLON M J, Managing Structural Corrosion in Chemical Plants. New Steel Construction, p14, February 1993.

2. KAYSER J R and NOWAK A S, Capacity Loss Due to Corrosion in Steel-Girder Bridges. J of Struct Engng, ASCE, Vol 115, No 6, 1525-1537, June 1989.

3. ICI ENGINEERING, Requirements for Inspection of Plant Structures and Pipebridges. ICI Engineering Procedure, EDP.CIV.52.40A, June 1990.

4. BRITISH STANDARDS, Structural Use of Steelwork in Building. BS5950: Part 1, British Standards Institution, London, 1985.

5. CHENG J J R, YURA J A and JOHNSON C P, Lateral Buckling of Coped Steel Beams. J of Struct Engng, ASCE, Vol 114, No 1, 1-15, January 1988.

6. WANG T M, SNELL R R and COOPER P B, Strength of Beams with Eccentric Reinforced Holes. J Struct Div, ASCE, Vol 101, No ST9, 1783-1800, 1975.

Evaluation, Repair, and Maintenance of a Mobile Conveyor

Évaluation, réparation et entretien d'un transporteur mobile

Überprüfung, Instandsetzung und Wartung einer fahrbaren Bandanlage

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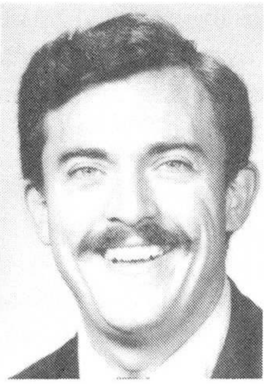

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SUMMARY

The paper describes the inspection and analysis procedures used to evaluate and strengthen a space truss mounted on crawler treads. The structure had experienced distress related to its static strength and its fatigue resistance. Care was taken to calibrate the analysis to actual performance. Evaluation criteria were tailored to the specific con-ditions of this structure by modifying a modern load and resistance factor design stan-dard. A program of systematic inspection for and repair of fatigue cracks was instituted.

RÉSUMÉ

L'article décrit les méthodes d'inspection et d'analyse employées pour le renforcement d'une charpente métallique montée sur chenilles. La charpente avait donné des signes de faiblesse tant aux sollicitations statiques qu'à la fatigue. L'analyse a été soigneusement comparée au comportement réel. Les critères d'évaluation ont été adaptés aux condi-tions spécifiques de cette charpente en modifiant un règlement moderne aux états limites. Un programme systématique de contrôle et de réparation des fissures a été mis en place.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die Prüf- und Berechnungsverfahren, die bei der Verstärkung eines räumlichen Fachwerkträgers, der auf einem Raupenfahrwerk montiert ist, eingesetzt wurden. Die Konstruktion wies Schäden bezüglich ihrer statischen Tragfähigkeit und ihres Ermüdungswiderstandes auf. Das Rechenverfahren wurde sorgfältig am gegenwärtigen Tragverhalten kalibriert. Die Ueberprüfungskriterien wurden für den speziellen Fall aus einer modernen Norm mit Teilsicherheitsbeiwerten abgeleitet. Es wurde ein Programm für die systematische Inspektion und Reparatur von Ermüdungsrissen aufgestellt.

1. INTRODUCTION

The Round Mountain Stacker is a mobile crawler-mounted stacker that builds a gold ore pile for cyanide process leaching at the Round Mountain Gold Mine. Within three years of construction, the mine personnel noticed some cracking and excessive deflections. A preliminary evaluation revealed that fatigue and overstress were of concern.

Due to the importance of the stacker in mine operations, a much more detailed level of inspection and analysis was undertaken. Inspections were intended to identify potential problems, classify structural details for fatigue resistance, accurately quantify the magnitude and cyclic load history of important loads, and obtain information which could be used to calibrate the analytical model.

The analysis efforts focused on quantifying the static stress levels and stress ranges experienced by the structure, establishing appropriate strength and fatigue evaluation criteria, and using these results to identify structural members and/or details which would require repair or more frequent inspection. Based on the inspection and analytic findings, repair and operational recommendations were made.

2. DESCRIPTION OF STRUCTURE

The stacker consists of a 90 m long bridge that is mounted on crawlers and a tripper that rides rails on the bridge (See Figure 1). The stacker is a 3-dimensional space truss composed primarily of wide flange and angle rolled shapes, made of A36 steel. Some of the members have additional cover or doubler plates welded to them. Nearly all of the connections are fully welded. There is a luffing boom at the tail (feed) end of the stacker that is composed of structural tees and angles.

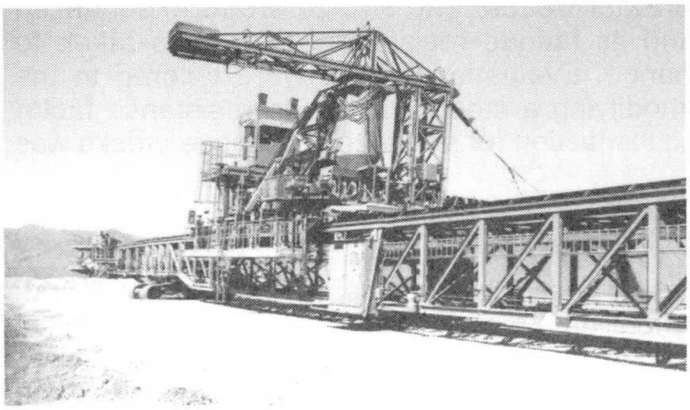

Figure 1 - Stacker and Tripper Car

Because the structure is a space truss, the bending moments are generally relatively low. However, the rails on which the tripper rides are directly attached to the top chord. Therefore, when the tripper wheels are between truss panel points, fairly large moments are induced in the top chord.

The fatigue resistance of connection details is generally inversely proportional to the severity of stress concentration. Many of the details used in this structure cause fairly large stress concentrations and have a correspondingly small allowable stress range for fatigue. In several of these areas, cracking consistent with fatigue was already evident. Most of the fatigue cracks were at the connections of secondary members. However, there were cracks approaching 40 mm long in the flange of the bottom chord members at midspan; rapid propagation of these cracks would have led to total failure.

3. INSPECTION

Our preliminary inspection of the stacker and tripper led us to believe that the tripper's weight was not equally distributed to its wheels. The complexity of the tripper framing and large amount of additional plate steel made accurately calculating tripper weight unfeasible. Since the tripper is the primary load on the bridge, and by far the most important contribution to fluctuating member stresses, an accurate determination of its overall weight and the distribution of this weight to its wheels was necessary. A field weighing of the tripper was carried out by jacking between the rail and the tripper undercarriage near each wheel using a calibrated hydraulic ram. To assure valid results, measurements were taken with the tripper boom slewed at three different angles, and with the tripper located over a support and at midspan of the stacker. The overall tripper weight was about 565 kN, which is considerably more than our preliminary estimates. The results were incorporated in our final analysis of the stacker.

In order to calibrate and confirm our analytical model, a deflection survey was undertaken by mine personnel. The survey results in the form of north, east, and elevation coordinates for several points along the bridge with the tripper in various positions were reported to us.

The analysis predicted that some localized yielding should have occurred in some members, particularly over the supports. Such yielding would cause a noticeable sag of the cantilevered ends of the stacker, which was noted during preliminary inspections and was measured in the deflection survey. The permanent deflection set in the structure is consistent with analytical predictions.

Since some fatigue cracks had already been found in primary structural members, a much more careful inspection of fatigue susceptible connections was made using the dry magnetic particle method. This careful inspection was also used as a vehicle by which mine personnel were taught how to perform periodic inspections for strength and fatigue problems.

4. ANALYSIS

4.1 Basic Model

A linear-elastic member analysis of the stacker was performed using a general purpose 3-D structural analysis program called SAP90. The member properties specified in the analysis match the properties of the members on the actual structure, which in many cases had been field modified by the addition of cover and/or doubler plates. Since the connections in this space truss were fully welded, the members were modelled using frame elements which can resist axial force, torsion about the longitudinal axis, and shears and moments along the major and minor section axes. In locations where the physical connections of the structure do not allow the transmission of certain of the frame forces, member degrees of freedom were appropriately released so as to prevent these forces from developing.

Dead loads are those loads which are known fairly accurately and are considered permanent, although some of these loads may potentially be moveable (as in the case of the tripper weight). By far, the most significant loads on the structure come from the weight of the tripper. In addition to tripper weight, the dead loads included member self-weight and the weight of drive motors and pulleys, electrical cabinets, rails, conveyor belts, idlers, drive chain and power cable, conduit, and ore spillage on the structure.

Live loads are those loads which are produced by the use of the structure and they are typically known with less certainty than dead loads. On this structure the live loads are limited to those loads produced by the ore payload and by belt tensioning. Wind loads were resolved into distributed loads on the top and bottom chords of the bridge for wind on the members of the bridge, and point loads at the tripper wheels for wind on the tripper. The wind on the tripper causes essentially equal loads at the wheels in the direction of the wind, and vertical forces that resist the overturning of the tripper.

In order to prevent spurious tension at the soil-structure interface, and to allow the redistribution of loads by differential support settlement, the supports were modelled using linear-elastic springs for translation degrees of freedom. Spring constants were adjusted during model calibration as noted below.

4.2 Calibration

Two bases were used to establish the stiffness of the vertical soil springs. The first basis was to use the stiffest spring that produced no net tension on a crawler for any load condition. The second basis was to compare that soil spring with geotechnical information for subgrade stiffness. The vertical soil spring stiffness used was consistent with a subgrade modulus of 13600 kN/m^3, which is reasonable for an uncompacted fill. Longitudinal and transverse soil springs were made just stiff enough that the computed reactions did not indicated slipping.

The considerable care taken in member, load, and support modelling, resulted in structural response which met the analysis objectives and was fairly consistent with the measured response.

4.3 Processing Results

An analysis post-processor, called faSAP, was developed for this project to assist in a determination of the stresses and stress ranges experienced by individual members of the structure.

This post-processor reads the SAP90 input file, which defines the structural model, including member connectivity, orientation, and properties. It then reads a user supplied file which specifies the fatigue load condition (number of cycles) and the allowable compressive and tensile stresses, associates an AISC fatigue category with each member, and optionally provides ratios by which the area, and section moduli may be adjusted for fatigue stress determination (such as at the end of a cover plate). Then a binary file containing the member forces for each combination of loads considered is read, and axial and bending stresses are calculated and combined. For each member, the stresses are calculated at each end and at three intermediate sections (quarter and midpoints). At each section, the stresses are calculated at the four extreme corners of the section. The minimum and maximum stresses which occur at any points in the member and the maximum stress range experienced by a given point of the member are calculated. The experienced stress range is compared to the stress range allowed for each member based on its fatigue category.

This post-processor produces three forms of output. The primary output is a text file which presents the fatigue stress analysis results for each member. For each member, this file indicates the minimum and maximum stresses and the load cases that produce them. The maximum stress range experienced at any point and the load cases which define the extremes of the range are output along with the section (of the five sections checked) and the location on that section where the maximum stress range occurs. Members which fail to satisfy the strength and/or fatigue evaluation criteria are flagged. The second form of output is a DXF (AutoCAD compatible) file which shows three plots of the structure, with its members color-coded to indicate the most severe stresses, maximum stress range, and comparison of calculated stress range to the allowable stress range for fatigue. The program also writes spreadsheet files which may be used for graphing the overall member force, stress, and stress range trends for the structure as a whole.

5. EVALUATION CRITERIA

5.1 Static Strength

Rather than making strict use of conventional standards in evaluating the structure, maximum stress criteria specifically suited to this structure were developed. The *Load and Resistance Factor Design Specification for Structural Steel Buildings* was used as a base reference [1]. This standard was chosen as a basis because it includes a clear exposition of the true capacity of structural steel members and systems, which does not vary by type of structure or engineering process, and because it provides safety levels that depend upon the degree of uncertainty in loads, structural material strength, member and system tolerances, and analytical and behavioral models. The conceptual advantages of strength-based design in evaluation of existing structures is well established.

The load factors used in the *LRFD* for dead, live, and wind loads are 1.2, 1.6, and 1.3, respectively. These factors are based upon statistical reliability analyses for ordinary building construction. For this project, the tripper weight is by far the most significant load causing stress, and that weight is known quite precisely. For the total mobile conveyor, the tripper dead weight is 35% of the total dead plus live, with the truss self weight being 31% and the distributed dead load being 17%. Live load, which is primarily the ore on the two belts, is only 17% of the total. When considering stress at any point, the tripper dead weight is an even larger fraction of the total, because only about half the truss weight and distributed dead load is really effective in increasing the stress in any single member. These considerations led to a reduction of the load factor for dead loads from 1.2 to 1.1 for this structure, because of the weighing done on the tripper. Similar considerations allowed a reduction of the load factor for live loads from 1.6 to 1.4 because there is much less uncertainty about the weight of ore on the belt than there is with conventional live loads on buildings.

The resistance factors used in the *LRFD* for axial compression, axial tension, and bending are 0.85, 0.90, and 0.90, respectively. There are many issues that enter into establishment of these factors. For this evaluation, each resistance factor was raised by 0.05 to account for the past inspections of the structure and the ability to continue easy inspection in the future due to the exposed nature of the structure. Another reason for the increase is the calibrated analysis, which is somewhat more precise than would be used in ordinary design.

The revised load and resistance factors were used to develop modified allowable stresses for flexure, tension, and compression, against which the computed stresses were compared. The fact that the load and resistance factors developed for this project allowed a smaller margin of error was taken into account when areas that required strengthening were being chosen.

5.2 Fatigue

The fatigue evaluation undertaken in this study was based on the stress-life, S-N, method which is widely used in high-cycle applications where the applied stress is primarily in the elastic range of the material. This method is based on empirical determination of appropriate constant and exponent in the relation $N{\cdot}S^k = constant$, for various structural details (where N is the number of cycles and S is the stress range).

This relation plots as a straight line on log-log paper. The constant, and to a degree the exponent, depend on the precise configuration of the structural element. There is a large range in resistance to fatigue, with "clean" elements (those with few or no details creating stress concentrations) being much more resistant than "complex" elements. For any single type of element there is considerable scatter in experimental data, and the allowable stresses cited in standards such as the *LRFD* are considerably below the mean of experimental values. For a given type of structural element, the *LRFD* gives allowable stress ranges for various ranges in the number of cycles of stress. The European standard, *Recommandations pour la vérification à la fatigue des structures en acier* specifies the linear relation on a log-log graph (effectively specifying an allowable value for the constant and the exponent) [2].

The detail classes specified in the *LRFD* were used, but the constant and exponent corresponding to the specified allowable values were calculated to arrive at an allowable stress for the number of cycles expected in the life of this structure. The *Recommandations* specifies an exponent of 3 for fewer than 2,000,000 cycles, and a plot of the *LRFD* values clearly shows that an exponent of 3 is a good fit. Based upon the operation of the structure prior to this evaluation and the recommendations for changes in this operation, 175,000 cycles was used as the desired life to compute the allowable stress ranges.

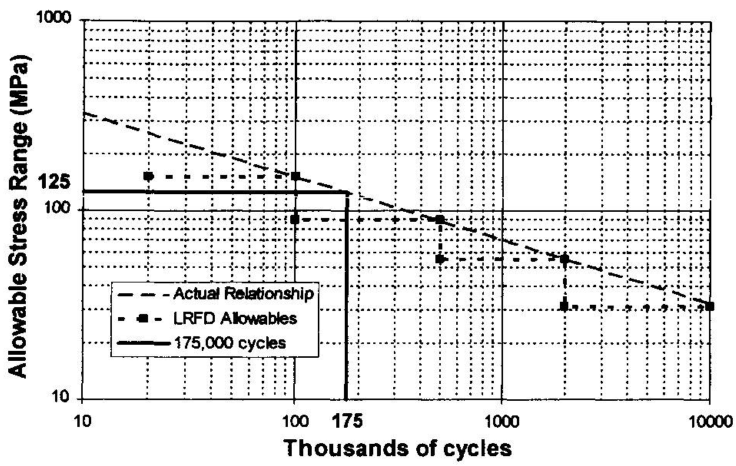

Figure 2 - S-N Diagram for LRFD *Category E*

Figure 2 shows the development of this criterion. This value strictly applies for stress ranges where the maximum stress is tensile and the minimum stress, if compressive, is not larger than the maximum stress. For those members in which the entire stress range is in compression, fatigue is not ordinarily a failure possibility. An exception to this exists in elements or details stressed so highly that yielding occurs; yielding over several cycles will shift the mean stress such that a location that originally experienced only compression will begin to experience tension. Where the compression stress is larger than the tension stress, an artificial stress range equal to twice the tension stress was used for comparison.

6. REPAIR AND OPERATIONAL RECOMMENDATIONS

The evaluation found that the structure had both safety and serviceability problems: some members were significantly overstressed, and many members experienced stress ranges that were already causing fatigue cracks. Repairs were designed and constructed to remedy the locations where static stresses were above the modified allowable value. The repair scheme took into account operational constraints and the isolated nature of the site. It is not feasible to improve the fatigue life categories of the various member and connection details. The concept followed was to schedule detailed inspection at fatigue-sensitive locations on a frequent enough basis so that cracks would be detected and repaired before they reached the critical size to initiate brittle fracture. Lack of detailed information forced this to be based on coarse approximations. An inspection program was developed, which included a training program to instruct mine personnel in visual and dry magnetic particle crack inspection procedures. Standard details were developed for repair of cracks to extend the useful life of the stacker in the face of potential fatigue problems.

REFERENCES

1. AISC, *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, 1986.

2. CECM, *Recommandations pour la vérification à la fatigue des structures en acier*, Convention Européenne de la Construction Métallique, 1985.

Reconstruction of Reinforced Concrete Walls of Cylindrical Silos

Reconstruction de parois en béton armé de silos cylindriques

Rekonstruktion der zylindrischen Stahlbetonsilowänden

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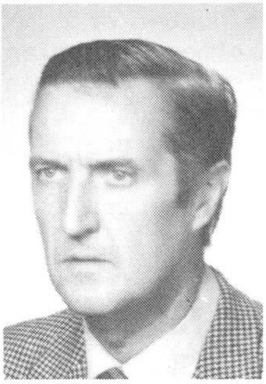

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SUMMARY

Walls of reinforced concrete silos long in service can become damaged due to various external and/or internal influences. Repairs of such walls require exact appraisals and individual methods. Two examples of large cylindrical silos are presented.

RÉSUMÉ

Les parois des silos en béton armé en exploitation depuis longtemps, sont souvent endommagés par différentes influences extérieures et intérieures. La réparation des parois exige une évaluation précise et un choix de méthodes individuelles. Deux exemples de réparation de grands silos cylindriques sont présentés.

ZUSAMMENFASSUNG

Bei lange benutzten Stahlbetonsilowänden erfolgt die grösste Beschädigung infolge verschiedener äusserer oder innerer Einwirkungen. Die Reparatur dieser Wände erfordert eine genaue Bewertung und individuelle Methodenauslese. Es werden zwei Beispiele von Reparaturen grosser zylindrischer Silowände vorgestellt.

1. INTRODUCTION

In the past decades the development of reinforced-concrete shell structures was a reason of designers' proud in many countries. The Sydney Opera and many other projects, particularly domes, were the symbols of structural concrete possibilities. Similarly in industrial structures, like containers of many kinds, high chimneys of large diameter, silos and cooling towers, the use of concrete was very common.

Looking for the current economical efficiency designers used very thin shells with minimum cover. Sometimes, the proper execution of those structures was really difficult or even impossible.

Nowadays, after twenty or thirty years of continuous exploitation, the serious problems with safety of the structures have appeared.

For instance, in Poland we experienced a lot of troubles with structures which have been built in sixties or seventies. One 100m high cooling tower collapsed, and few similar had to be demolished. Several high chimneys had to be replaced by new projects, tens of reservoirs were demolished and the large number of containers and silos should be reconstructed. Presently, like in other countries the problems of durability of structures and the necessity of extending lifespan of many of them are treated as more important than erection of new structures. The reasons of damages are very typical sometimes, while in other cases are of quite specific nature.

Two examples of destruction and reconstruction of large silos are presented to illustrate such different situations.

2. EXTERNAL DAMAGE OF SILOS AND REPAIR PROCEDURE

The group of six silos has been built in 1969 to store grain, mainly wheat and rye. The storage station was located at the eastern border of industrial zone and the buildings were attacked by polluted air, mainly by acid rains. The great majority of winds in that area were from west, so the surfaces of concrete in silos walls have been badly damaged, particularly from the west side (Fig.1).

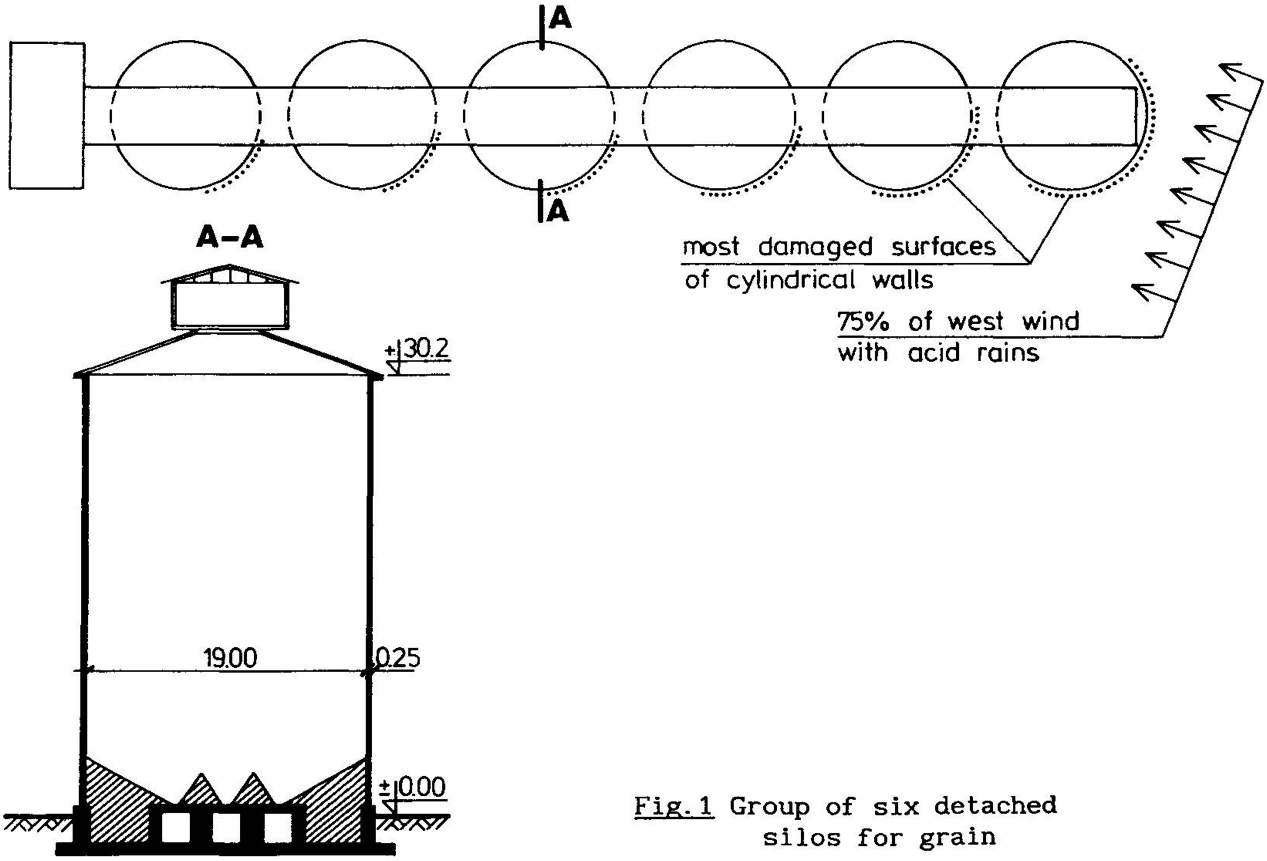
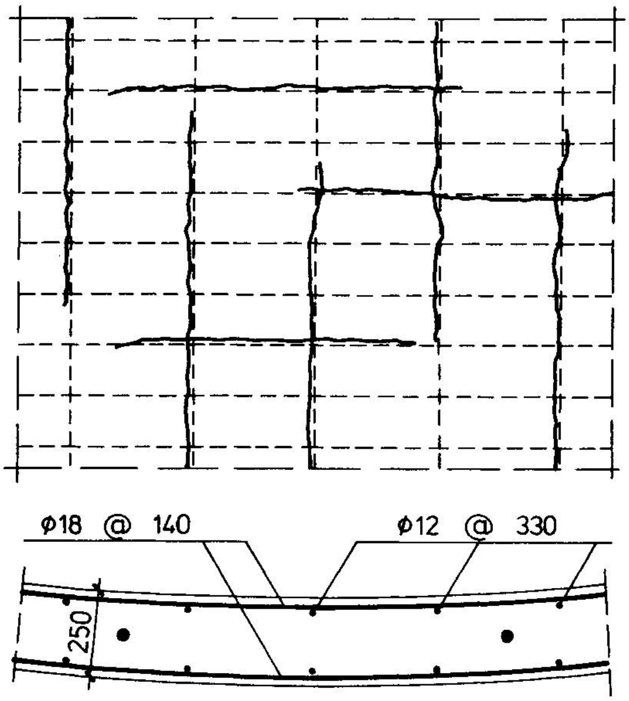


Fig.1 Group of six detached silos for grain



The corrosive environment in combination with porous concrete surfaces and poor cover caused on more than 1/3 of the external surface area advanced damages with corrosion of reinforcement and serious cracking of concrete along the bars (Fig.2).
Some parts of reinforcement were uncovered at all.
The tests of concrete samples (drilled cores throughout walls) indicated that the strength of concrete was quite sufficient but advanced processes of carbonation were measured to the depth from 60 to 100mm (Fig.3a).

Fig.2 Cross-section of silo wall and view of external surface with cracks width of 0.3 to 0.5mm along the bars of reinforcement

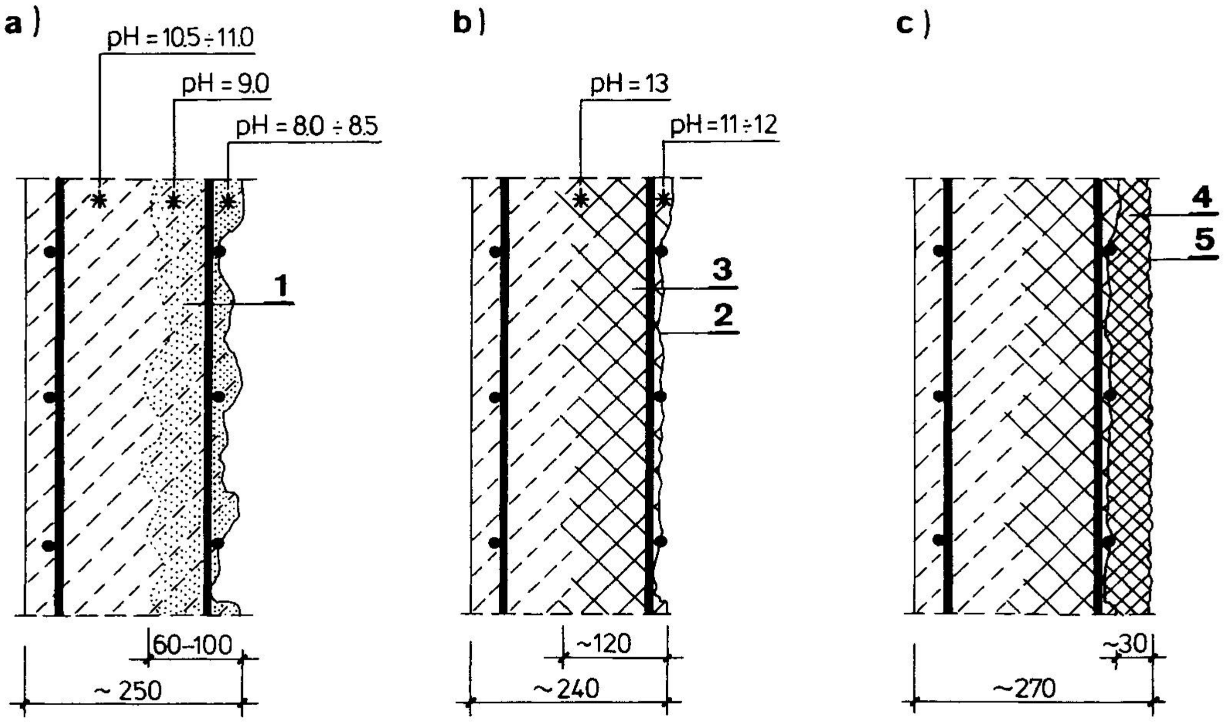


Fig.3 The steps of reconstruction: a) - damaged wall, b) - after sand-blast cleaning and alkalization, c) - after full reconstruction;
1 - zone of carbonized concrete (pH ≤ 9), 2 - sand-blast cleaned concrete surface, 3 - alcalized concrete (pH ≥ 11) and filled cracks, 4 - shotcrete with microsilica admixture, 5 - protection paint with possible air penetration

The reconstruction work has been designed and executed as follows (Fig.3b,c):
- cleaning of concrete surfaces by sand blasting,
- re-alcalization by double or triple impregnation from outside (Consecrate CC)
- local repairs with highly adhesive mix (Sika admixtures),
- single layer of shot crete ~30mm thick, placed by dry method (microsilica additions),
- external protection with air-penetrable painting (Sarsil).

As an interesting idea introduced during reconstruction was the use of silo overloading (by filling over the nominal level with humid, heavy grain) at the impregnation of cracked concrete and at shotcrete placing and hardening. Double benefits were obtained in this way - the increased humidity of concrete, which was important for the effects of both processes and quasi-prestressing of shotcrete layer after unloading.
The tests of samples taken from the walls after re-alcalization showed the average depth of penetration about 120mm, with effective alkali reaction (pH ≥ 11) - Fig.3b. Final tests of the cores taken out from the repaired walls concerned the bond between old and new concrete. The results were satisfactory - more than half of samples have been broken not in the contact section but through the old concrete.

3. DESTRUCTION AND RECONSTRUCTION FROM INSIDE

The second example of silos reconstruction refers to quite different case of damage of structure. The three groups of four cylindrical silos each have been exploited continuously for more than 15 years in the cement plant. The silos were necessary to store cement clinker before grinding (Fig.4).

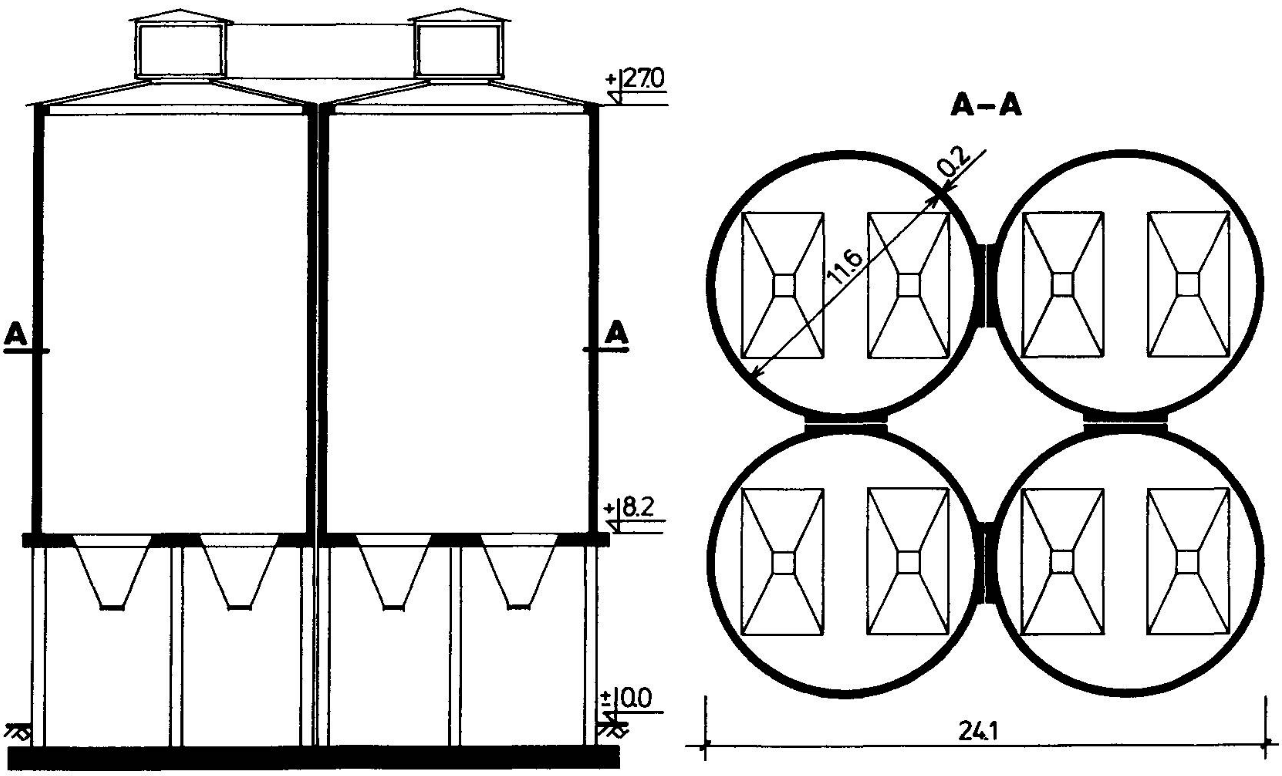


Fig.4 Group of silos in cement plant

The thermal influences were taken into account by designer and over the common bottom-slab the cells were quite independent. During the proper production process the temperature of clinker at loading should not be more than 70°C. Unfortunately, very often the material of 200°C and sometimes even almost 300°C was loaded into the silos, as a result of irregularity of production.
The random thermal actions were not noticed for long time. From outside almost no signals of damage could be observed, particularly in very dusty atmosphere of the cement plant. The local cracks of width up to 0.3mm were not visible without special cleaning of surface.
By accident, during the replacement of steel hoppers in one silo the part of concrete cover from internal surface fell down and uncovered the surface of seriously cracked concrete. The investigations were udertaken and completely destroyed layer of concrete up to the depth of 60 - 100mm from inside was discovered (Fig.5a). In many parts the solids of concrete (like cubes up to 100mm) could be taken away by hand, because, apart from cracking, the bond between concrete and reinforcement was entirely destroyed.

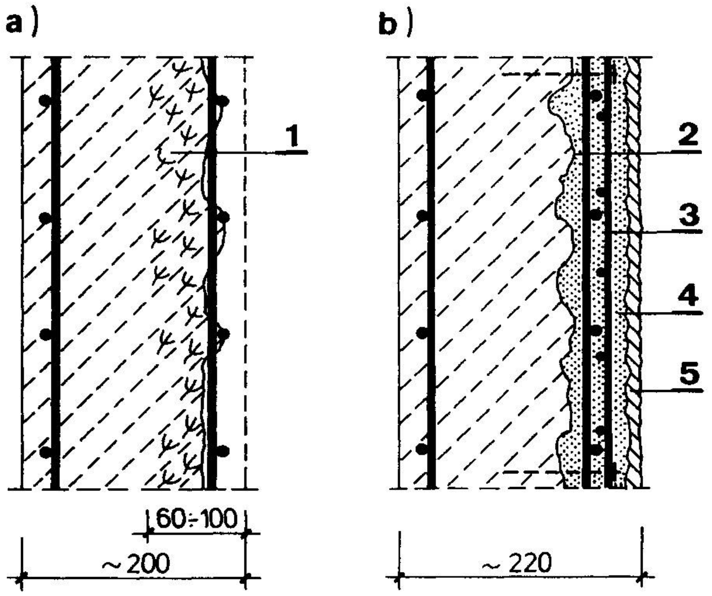


Fig.5 The wall of cement clinker silo:
a) damaged by multiple thermal actions,
b) reconstructed from inside;
1 - zone of highly cracked/crushed concrete,
2 - cleaned surface of slightly cracked concrete and uncovered reinforcement,
3 - additional steel fabrics ø8@100/100mm,
4 - shotcrete with basalt aggregate,
5 - anti-abrasive layer with fibers

Due to dry environment inside the steel bars were relatively in good stage, almost without signs of corrosion, but in many parts the bars were bent by mechanical action of sliding down solids of clinker.
The only problem, but very significant was the long time of process of dehydration of concrete connected with destruction due to local overheating of the wall.
The immediate general repair was necessary with replacing part of existing reinforcement and addition of reinforced shot crete supplementary layer. The shot crete was placed in three steps, 25 - 30mm each. The final layer of shotcrete was of particular recipe, with anti-abrasive components (basalt aggregate, reinforcing fibers and Sikacrete admixture) - Fig.5b.
Obviously, the correct process of production has been ensured parallely by the technological changes.

4. CONCLUSIONS

The critical stage of reinforced-concrete or prestressed-concrete structures is usually the result of three groups of reasons:
- design incorrectness, particularly lack of designer's imagination about exposure to severe external environment or influences of internal conditions,
- execution errors, most often connected with quality and casting of concrete or cover size,
- utilization faults, especially lack of periodical control and repair, and deviations from the proper exploitation.
Nowadays, testing methods for diagnosis of structures as well as various methods

for repair, with respect to the required lifespan of structures are commonly available.
Sometimes the specific situations may occur: a building originally planned for 20-25 years has to be used for the next period, e.g. 20 years. The analysis of necessary protection and/or reconstruction should be then especially careful.
Taking no account of symptoms of destruction and delay of actions for protection and repair are the approaches which usually lead to either cutting down the structure lifespan or to the costly reconstruction process necessary to extend the safe use of building. Unfortunately, there is the third possibility - the disaster of part or entire structure.

Numerical Analyses and Strengthening of Reinforced Concrete Structures

Analyses numériques et réparation des constructions en béton armé

Die Rolle numerischer Analysen bei der Sanierung von Stahlbetonbauwerken

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SUMMARY

The role of nonlinear finite element analyses in the design process of the repair of damaged reinforced concrete structures is illustrated in the context of the strengthening of a damaged cooling tower shell. The effectiveness of the repair by attaching stiffening rings to the shell is demonstrated by means of numerical simulations using realistic material modelling based on the state of damage of the structure.

RÉSUMÉ

Le rôle des méthodes non-linéaires par éléments finis en liaison avec le projet et la réparation des constructions en béton armé est décrit, en particulier dans le cas d'une tour de refroidissement fissurée. L'efficacité de la réparation avec des anneaux renforcés est démontrée par des simulations numériques utilisant des modèles constitutifs réalistes, tenant compte de la détérioration de la structure.

ZUSAMMENFASSUNG

Die Rolle nichtlinearer Finite Elemente Analysen bei der Sanierung von Stahlbetonbauwerken wird anhand der Verstärkung einer durch Risse geschädigten Kühlturmschale aufgezeigt. Die Wirksamkeit einer Sanierung durch Versteifungsringe wird mit Hilfe numerischer Simulationen unter Verwendung wirklichkeitsnaher Materialmodelle und unter Berücksichtigung der Vorschädigung demonstriert.

1. MOTIVATION

Reinforced concrete (RC) structures, designed and constructed in the 1950s and 60s, frequently show signs of damage, such as cracks, which, in general, are caused by stresses that were not considered adequately by the design provisons of that time. Typical examples are reinforced concrete shells, showing meridional cracks due to bending stresses, induced by temperature gradients [1], [2]. If such a state of damage is observed, the question arises, whether a repair is technically and economically feasible, and, in case a repair is taken into consideration, how to design suitable provisions for the strengthening of the RC structure. Here, the term "optimality" is defined as the most economic design which guarantees sufficient structural safety within the anticipated lifetime. For many complex structures the answer to this question can only be obtained by means of modern analysis tools such as the Finite Element Method (FEM). This paper adresses the role of the FEM in the context of the design of the repair of a cracked cooling tower constructed in 1964. The sequence typically followed in the design procedure involving realistic numerical simulations is discussed and illustrated by means of this specific example.

The dimensions of the cooling tower are illustrated in Fig.2. The nominal shell thickness is 10 cm. Its present state is characterized by an approximately uniform distribution of long, meridional cracks caused by thermally induced stresses (Fig.1). Details of the numerical analyses of the restoration of this cooling tower are published elsewhere [2], [5].

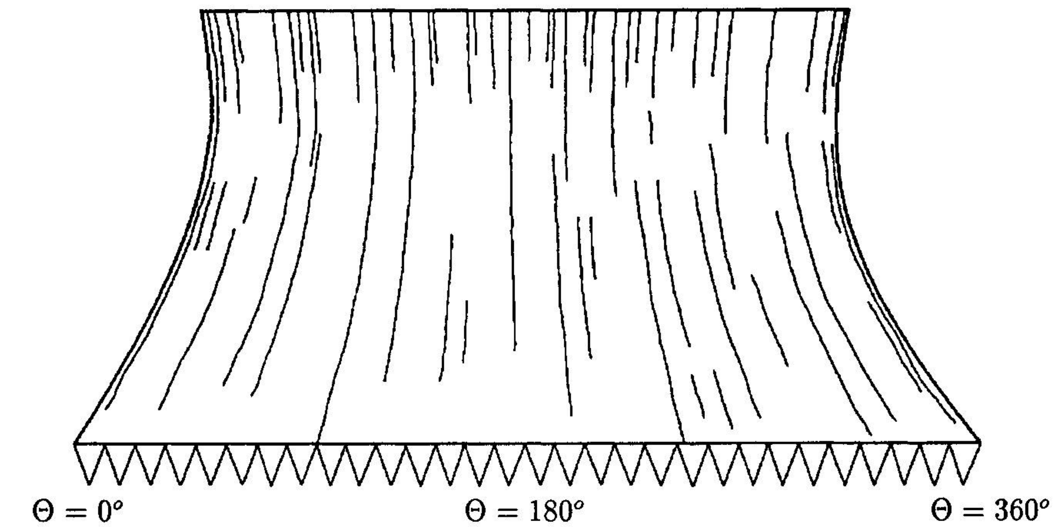

Figure 1: Distribution of Cracks at the Outside Surface of the Cooling Tower Shell

2. ASSESSMENT OF STATE OF DAMAGE

As a preliminary step in the design process of a damaged RC-structure, a thorough assessment of its present state by means of a survey and of laboratory investigations is necessary. In the reported investigation, the spatial distribution has been obtained from a detailed inspection of the shell and the opening of cracks (crack width ranging between 0.1 to 2.7 mm), the state of corrosion of the reinforcement located at cracks or in their vicinity (reduction of diameter ranging from 11 % to 30 %) and the actual thicknes of the cooling tower shell (thickness varying between 7.6 cm and 10.2 cm) were obtained from specimens taken from different locations of the shell.

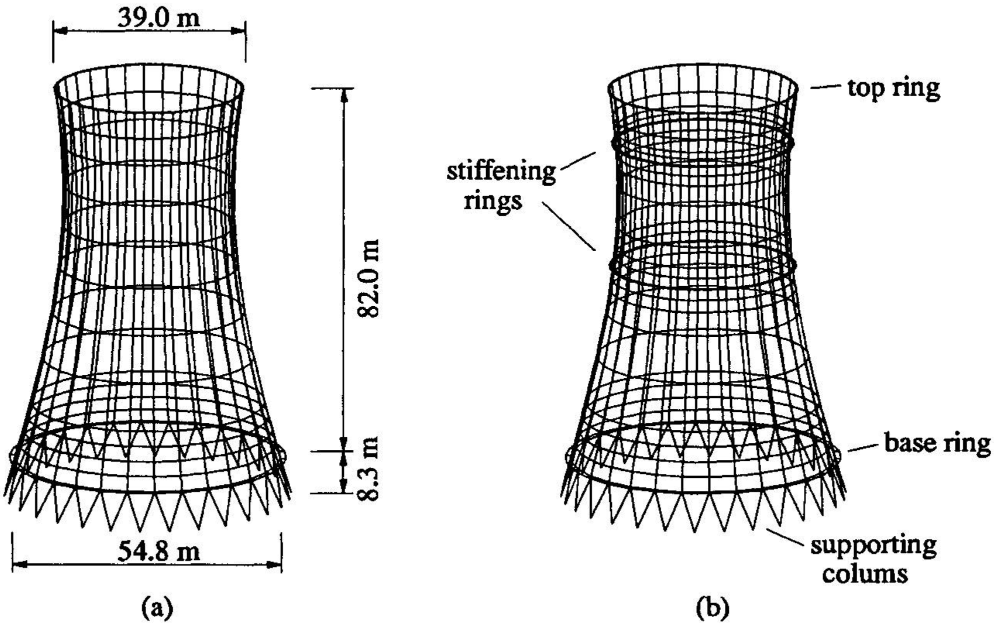

Figure 2: FE–Meshes: (a) Coarse, (b) Fine

3. SELECTION OF COMPUTATIONAL MODEL

The selection of an appropriate computational model depends on the degree of sophistication necessary for a realistic representation of the damaged (and restored) state of the structure. In this context, the element type (truss or beam elements, plate or shell elements, 3D elements), the numerical procedure (geometrical and physical nonlinearity, displacement or non-standard methods, etc.), the material model (elasticity, elasto (visco-) plasticity, damage theory etc.) needs to be specified.

In the presented example, an updated-Lagrangian FE-approach based on isoparametric thick-shell elements subdivided into 13 layers is employed. Fig. 2 contains two FE meshes used at different stages of the numerical investigations. The spatial distribution of the initial cracks is unsymmetric. Hence, specification of symmetry conditions would have been inadmissible. The numerical representation of cracked concrete is based on the "fixed crack" concept in the context of the "smeared-crack" approach. Cracks will begin to open normal to the direction of the maximum principal stress, if this stress reaches the tensile strength. Secondary cracks are restricted to the direction perpendicular to the primary cracks [5]. The ductile behavior of concrete under compression is accounted for by an elastoplastic strain-hardening Drucker-Prager material model. In the numerical investigations the reinforcement (meridonal and circumferential bars) is represented by mechanically equivalent, thin layers of steel with only an axial stiffness in the respective direction. A linearly elastic, ideally plastic constitutive law is assumed [5].

4. NUMERICAL REPRESENTATION OF DAMAGE

The numerical representation of the damaged state of the investigated structure may either be accomplished through modification of the respective geometrical data and model parameters or by re-analysis of the process which presumably has caused the observed state of damage. For the purpose of determination of the safety coefficient of the structure before and after its repair, the expected residual lifetime of the structure needs to be defined. Moreover, prognoses of the state of damage at the end of the expected lifetime should be made. Generally, this results in several scenarios for the present and the future state of damage of the structure, involving "worst case" and "mild" assumptions for the expected damage evolution.

In the considered example, the meridonal cracks are accounted for in the FE-model by a reduction of the concrete tensile stress at the integration points in the vicinity of these cracks. The opening of the cracks is then triggered by the application of a thermal load history prior to applying the standard wind load [4]. This wind load is multiplied with a dimensionless factor λ which is increased incrementally [2]. The temperature load induces a nonuniform crack pattern which corresponds to the crack distribution obtained from the survey. The investigation of several damage-scenarios results in a present state and an anticipated state of damage at the end of the residual lifetime of the structure in the year 2018. This includes "mild" and "worst case" assumptions for the state of cracking (crack depth, crack width) as well as for the corrosion of the reinforcement.

5. DETERMINATION OF THE SAFETY COEFFICIENT OF THE UNRESTORED STRUCTURE

Before deciding on a repair of the structure, the coefficient of safety of the unrestored structure has to be evaluated. Here, the term "safety" is defined according to the type and the utilization of the building and to respective regulations [3], [4]. In particular, different safety factors with regards to the limit of serviceability and the ultimate load have to be considered. For realistic numerical simulations of damaged RC structures, the actual safety margin required within the residual lifetime depends on the reliability of the experimental data and of the underlying assumptions for the different scenarios of the present and future state of damage. As far as a critical interpretation of the numerical results is concerned, the influence of the chosen discretization has to be taken into account.

The assessment of the structural safety of the cooling tower shell is based on three dimensionless factors λ_c, λ_y^S and λ_u. Herein λ denotes the wind loading according to BTR [4]. λ_c refers to the level of the crack plateau, λ_y^S to the beginning of yielding of the reinforcement of the shell, which may be regarded as a sufficiently conservative limit of serviceability of the structure, and λ_u to the ultimate load-carrying capacity (Fig. 3). Taking into consideration that the serviceability of the cooling tower is not severely influenced by moderately large deformations, the decision for a repair was based upon λ_u. In a preliminary investigation the relevant load case including the most critical direction of wind loading was determined. Among the different aspects of damage, the corrosion of the reinforcing bars was found to be the determining factor for the structural safety within the anticipated lifetime. From the average value of $\bar{\lambda}_u = 1.035$, obtained as the coefficient of safety against structural collapse in the year 1993, and the corresponding average value of $\bar{\lambda}_u = 0.951$ representing the coefficient of safety against structural collapse in the year 2018, it was concluded that the cooling tower shell will not be suffiently safe against structural collapse for the remaining lifetime of 25 years unless provisions for a repair are taken.

6. SPECIFICATION OF REPAIR PROVISIONS

A suitable strategy for the strengthening of the structure must be developed on the basis of engineering judgement and of the safety levels obtained from the simulations of the unrepaired structure. Very often, the access of the building or the maintenance of production required during the construction play a crucial role in the decision for a specific repair procedure. As will be shown below, the effectiveness of the repair may be optimized in the sense of minimum

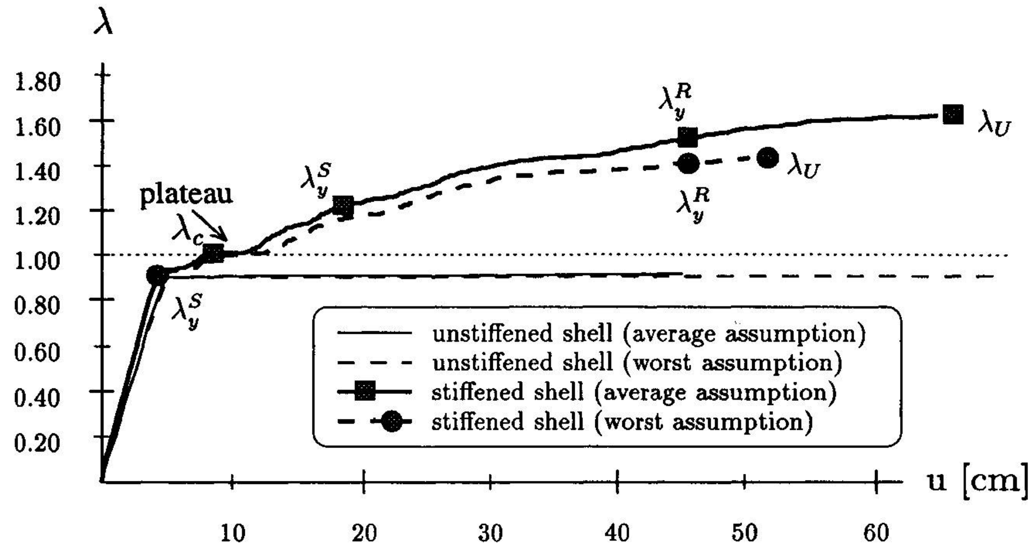

Figure 3: Load-Displacement Curves for Load Case Wind

costs while ensuring suffient structural safety by means of comparative numerical simulations. In general, the costs for advanced simulation techniques are by far compensated by savings from a FEM-supported design of the repair.

For the cooling tower shell, a repair by attaching reinforced cast-in-situ concrete stiffening rings was considered as the only feasible means of repair. In an extensive numerical study the influence of the number and the location of the stiffening rings upon the structural safety of the restored cooling tower was analyzed. The optimum position of one single ring was determined first. The location of a ring 52.0 m above the base has turned out to be most effective, resulting in an increase of the value λ_u for the unstiffened shell of 33.5 %. Next, combinations of two and three stiffening rings were investigated. For two stiffening rings located at levels of 70.0 m and 43.0 m above the base of the concrete shell, the increase of the collapse load is 73.1 %. Three stiffening rings (at levels 70.0 m, 52.0 m and 32.0 m above the base of the shell) cause an increase of λ_u by 107.0 %. As a conclusion from the comparative investigation, the use of two stiffening rings was found to be the best choice for the repair of the cooling tower shell. It will result in an increase of the maximum sustainable gradient wind load from $\bar{v}_G = 135.0$ km/h to $\bar{v}_G = 167.4$ km/h.

7. DESIGN OF STRENGTHENING ELEMENTS

Following the numerical optimization of the repair of the structure, the strengthening elements need to be designed and subsequently modified according to an iterative dimensioning procedure, involving re-analyses of the structure which, in general, is modelled by a refined FE–mesh.

For the repair of the cracked cooling tower shell, the final design of the cast-in-situ concrete rings (Fig. 4) was obtained from an iterative dimensioning process based on the refined mesh shown in Fig. 2b. The rings are bolted to the shell by "automatic undercutting bolts" (Figure 4). For the transfer of the forces from the shell to the ring it was assumed that shear stresses are transmitted by friction and that the axial forces are transferred by the bolts, connected with stirrup bars by means of threads, respectively. This assumption requires that the contact surface between the shell and the stiffening rings is adequately prepared in order to enhance friction in this surface.

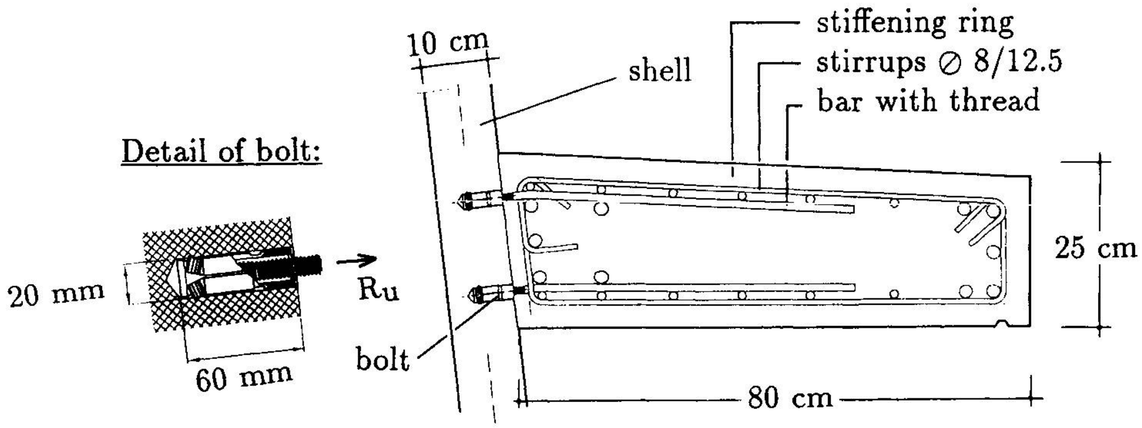

Figure 4: Typical Cross-Section Through Stiffening Ring

8. CONCLUSIONS

The role of sophisticated numerical modelling in the design of repair of RC structures was investigated in general. In particular, this role was illustrated in the context of the strengthening of a cracked cooling tower shell. It was shown that advanced simulation techniques based on realistic material models and on an adequate representation of the observed state of damage may be used for a relatively precise assessment of the residual structural safety. It is further concluded that advanced numerical analyses may be successfully employed to obtain a very economic means of repair. In case of the investigated cooling tower, the economic efficiency is characterized by the small number of stiffening rings required to guarantee a sufficient level of safety.

REFERENCES

[1] BOSMAN, P.B.
"Repairs of two severely cracked cooling tower shells using stiffening rings." Proc. Third Int. Symp. on Natural Draught Cooling Towers, Paris, France, Alsthom Etablissement SCAM, Paris, 381-399, 1989.

[2] MESCHKE, G., MANG, H.A. and KOSZA P.
Finite element analyses of cracked cooling tower shell, J.Struct.Eng., ASCE, 117(9), 2620-2639, 1991.

[3] EUROPEAN COMMITTEE FOR STANDARDIZATION
Eurocode No. 2: Design of Concrete Structures, Part 1: General Rules and Rules for Buildings, 1991.

[4] VGB-Rules for Design and Construction (BTR).
"Bautechnische Richtlinien" Bautechnik bei Kühltürmen, Part 2, VGB-Kraftwerkstechnik GmbH, Essen, Germany, 1990.

[5] MESCHKE, G., HUEMER, TH., KROPIK, CHR. and MANG, H.A.
'Design of the Strengthening of a Cracked Cooling Tower Shell on the Basis of Nonlinear FE-Analysis', Proceedings of the EURO-C Conference, Innsbruck, Austria, 689-702, 1994.

Extending the Lifespan of Nuclear Power Plant Structures

Prolongement de la durée de vie des structures de centrales nucléaires

Verlängerung der Lebensdauer von Kernkraftwerksbauten

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SUMMARY

Reinforced concrete structures play a vital role in the safe operation of nuclear power plants. Isolated incidences of degradation indicate that there is a need for improved quantitative evaluation methods and surveillance. inspection/testing, and maintenance activities to ensure continued safe operation. The Structural Ageing Program has addressed these issues through development of an ageing management methodology that encompasses a materials property database, inspection and repair technologies, and reliability-based techniques to indicate the current and future condition of these structures. Although aimed at nuclear power plant concrete structures, program results are equally applicable to the general civil engineering infrastructure.

RÉSUMÉ

Les ouvrages en béton armé jouent un rôle vital dans la fiabilité du fonctionnement des centrales nucléaires. Des cas isolés de dégradation partielles montrent l'importance de l'amélioration des méthodes d'évaluation, de surveillance, d'inspection, d'essais et de maintenance, afin d'assurer le fonctionnement et la sécurité fonctionnelle de tels ouvrages. Une procédure systématique a été mise en place pour la gestion du vieillissement, comportant une banque de données sur les propriétés des matériaux, des techniques d'inspection et de réparation, toutes basées sur la théorie de fiabilité, qui permet de visualiser les états actuels et futurs de ces structures. Bien que prévue pour des centrales nucléaires, il est possible d'appliquer cette méthode à des ouvrages de génie civil.

ZUSAMMENFASSUNG

Stahlbetonbauten spielen eine lebenswichtige Rolle für den sicheren Betrieb von Kernkraftwerken. Vereinzelte Verfallserscheinungen deuten darauf hin, dass verbesserte Erfassungsmethoden, Ueberwachungs-, Inspektions- und Testverfahren benötigt werden, um einen anhaltend sicheren Betrieb zu gewährleisten. Im "Structural Ageing Program" wurde daraufhin eine Methodik zum Alterungsmanagement entwickelt, die eine Datenbank mit Werkstoffeigenschaften, Inspektions- und Reparaturtechniken und Verfahren auf Grundlage der Zuverlässigkeitstheorie umfasst, um den gegenwärtigen und zukünftigen Zustand dieser Tragwerke anzuzeigen. Obwohl auf Kernkraftwerksbauten ausgerichtet, sind die Ergebnisse des Programms auch auf allgemeine Bauten anwendbar.

1. INTRODUCTION

By the end of this decade, 63 of the 111 commercial nuclear power plants in the United States will be more than 20 years old, with some nearing the end of their 40-year operating license term. Faced with the prospect of having to replace lost generating capacity from other sources and substantial shutdown and decommissioning costs, many utilities are expected to apply to continue the service of their plants past the initial licensing period. In support of such applications, evidence should be provided that the capacity of the safety-related systems and structures to mitigate potential extreme events has not deteriorated unacceptably due to either aging or environmental stressor effects during the previous service history.

2. REINFORCED CONCRETE STRUCTURES EXPERIENCE

Reinforced concrete structures are important to the overall safety of nuclear power plants in that they provide foundation, containment, support and shielding functions. Aging of these structures occurs with the passage of time and has the potential to degrade strength and increase the risk to public health and safety. Degradation mechanisms that can impact the performance of these structures include corrosion of the steel reinforcing systems, chemical attack, alkali-aggregate reactions, sulfate attack, frost attack, leaching, salt crystallization, and microbiological attack. When structural degradation has occurred, it primarily has done so early in life and has been corrected. Causes were related to either improper material selection, construction/design deficiencies, or environmental effects. Examples of some of the more serious instances include voids under vertical tendon bearing plates resulting from improper concrete placement; cracking of post-tensioning tendon anchorheads due to stress corrosion or embrittlement; containment dome delaminations due to low quality coarse aggregate material and absence of radial reinforcement or unbalanced prestressing forces; reinforcing steel corrosion in water intake structures; leaching of concrete in tendon galleries; and low prestressing forces [1–2].

3. CONCRETE STRUCTURAL AGING PROGRAM

Experiences summarized above indicate the possibility that degradation effects may reduce the margin that concrete structures have to accommodate loadings beyond the design basis. There is a need for improved quantitative evaluation methods and surveillance, inspection/testing, and maintenance to ensure continued safe operation of nuclear power plants. Guidelines and criteria for use in evaluating the remaining structural margins (residual life) have been developed under the Structural Aging (SAG) Program [3]. These activities were conducted under three major technical task areas — materials property data base, structural component assessment/repair technology, and quantitative methodology for continued service determinations.

3.1 Materials Property Data Base

A reference source containing data and information on the time variation of material properties under the influence of pertinent environmental stressors or aging factors has been developed. The data base, in conjunction with service life models, has application in the prediction of potential long-term deterioration of reinforced concrete structural components and in establishing limits on hostile environmental exposure for these structures. The results also have application to establishment of maintenance and remedial measures programs that will assist in either prolonging component service life or improving the probability of the component surviving an extreme event such as an earthquake. The data base has been developed in two formats — a handbook and an electronic data base.

The *Structural Materials Handbook* is an expandable, four volume, hard-copy reference document containing complete sets of data and information for each material. Volume 1 contains performance and analysis information (i.e., mechanical, physical, and other properties) useful for structural assessments and safety margins evaluations. Volume 2 provides the data used to develop the performance information in Volume 1. Volume 3 contains material data sheets (e.g., constituent materials, general information, and material composition). Volume 4 contains appendices describing the handbook organization and revision procedures. The *Structural Materials Electronic*

Data Base is an electronically-accessible version of the handbook that has been developed on an IBM-compatible personal computer. It provides an efficient means for searching the data files.

Two approaches have been utilized to obtain the data and information contained in the data base — open-literature information sources and testing of prototypical samples. A total of 143 material data bases have been developed addressing concrete and steel reinforcing materials. Examples of concrete material property data and information files currently available include compressive strength, modulus of elasticity and flexural strength versus time for several concrete materials cured under a variety of conditions (i.e., air drying, moist, or outdoor exposure) for periods up to 50 years; ultimate compressive strength and modulus of elasticity versus temperature at exposures up to 600°C for durations up to four months; and compressive strength versus time for concrete materials obtained from prototypical nuclear power facilities. Metallic reinforcement (ASTM A 615 and A 15) performance curves are available for fatigue, and ambient and temperature-dependent (A 615 material only) engineering stress versus strain. Temperature-dependent engineering stress versus strain, and tensile yield strength, ultimate tensile strength, and ultimate elongation versus temperature performance curves are available for both prestressing tendon (ASTM A 421, Type BA) and structural steel (ASTM A 36) materials. A more detailed description of the data base and the files it contains is provided elsewhere [4].

Also under this activity, methods for predicting the service life or performance of reinforced concrete have been assessed [5]. Models for each of the environmental degradation processes noted earlier were established and evaluated. A major conclusion of this study was that theoretical models need to be developed, rather than relying solely on empirical models. Predictions from theoretical models are more reliable, far less data are needed, and they have wider application. Purely stochastic models have limited application because of the lack of adequate data to determine statistical parameters. The best approach to providing realistic predictions of the service life of an engineering material is to combine deterministic and stochastic models.

3.2 Structural Component Assessment/Repair Technology

A methodology has been developed that provides a logical basis for identifying the critical concrete structural elements in a nuclear power plant and the degradation factors that can potentially impact their performance [6]. Numerical ranking systems were established to indicate the relative importance of a structure's subelements, the safety significance of each structure, and the potential influence of the particular environment to which it is exposed. Results of this activity can be utilized as part of an aging management program to prioritize in-service inspection activities.

Direct and indirect techniques used to detect degradation of reinforced concrete structures have been reviewed [7]. Capabilities, accuracies, and limitations of candidate techniques were established (e.g., audio, electrical, infrared thermography, magnetic, stress wave reflection/refraction, radioactive/nuclear, rebound hammer, and ultrasonic). Information was assembled on destructive (e.g., coring, probe penetration, and pull-out) and emerging (e.g., leakage flux, nuclear magnetic resonance, and capacitance-based) techniques. Recommendations were developed on application of testing methods to identify and assess damage resulting from typical factors that can degrade reinforced concrete. Also, statistical data were developed for nondestructive testing techniques commonly used to indicate concrete compressive strength (i.e., break-off, pull-out, rebound hammer, ultrasonic pulse velocity, and probe penetration) [8]. This information is required where destructive and nondestructive tests cannot be conducted in tandem at noncritical locations to develop a regression relation between the technique parameter measured and the structure parameter of interest. The methods developed can be used to estimate variance in strength or to yield information about distribution of strength population that is required to calculate the characteristic strength for use in structural integrity assessments .

As corrosion resulting from either carbonation or the presence of chlorides is the dominant type of distress that impacts reinforced concrete structures, corrosion mechanisms and types (e.g., uniform, pitting, bimetallic, crevice, etc.) were identified as well as conditions that affect the corrosion rate (e.g., oxygen, electrolyte conductivity, ion concentration, temperature, etc.). Methods available to detect corrosion occurrence include visual observations, half-cell potential measurements, delamination detection, electrolyte chemistry, corrosion monitors, acoustic emission, radiography, ultrasonics, magnetic perturbation, metallurgical properties, and electrical resistance. Remedial measures include damage repair, cathodic protection, inhibitors, chloride removal, membrane sealers, stray current shielding, dielectric isolation, coatings, and environmental modifications. Stray electrical current resulting from any of a number of sources (e.g., cathodic protection systems, high voltage direct current systems, and welding operations) could also lead to corrosion. Techniques to detect stray current include half-cell potential versus time measurements, half-cell potential versus distance measurements, and cooperative (interference) testing. Mitigation measures for stray current include prevention or elimination of the current source, installation of cathodic protection, draining the current from the source, and shielding the structure from the source. Use of sacrificial or impressed current cathodic protection systems as both a rehabilitation technique for corroding structures and a corrosion prevention technique for steel that may lose its inherent passivity at a later date was investigated. Design considerations, advantages and disadvantages, and commentary on when cathodic protection should and should not be used were also addressed [9].

Damage repair practices commonly used for reinforced concrete structures in both Europe and North America have been reviewed [10-11]. Basic repair solutions for corrosion of steel reinforcement in concrete include: (1) realkalization by either direct replacement of contaminated concrete with new concrete, use of a cementitious material overlay, or application of electrochemical means to accelerate diffusion of alkalis into carbonated concrete; (2) limiting the corrosion rate by changing the environment (e.g., drying) to reduce the electrolytic conductivity; (3) steel reinforcement coating (e.g., epoxy); (4) chloride extraction by passing an electric current (DC) from an anode attached to the concrete surface through the concrete to the reinforcement (chloride ions migrate to anode); and (5) cathodic protection. Repair strategies and procedures were developed in the form of flow diagrams. Information specifically addressing inspection, degradation, and repair of reinforced concrete structures in light-water reactor plants was assembled through a questionnaire sent to U.S. utilities. Responses provided by 29 sites representing 42 units indicate that the majority of the plants perform inspections of concrete structures only in compliance with integrated-leak-rate test requirements (visual inspections), and surveillances of the post-tensioning systems of prestressed concrete containments. The most common deterioration causes were drying shrinkage, acid/chemical attack, thermal movement, freeze-thaw cycles, and sea water exposure. Most of the repair activities were associated with problems during initial construction (cracks, spalls, and delaminations), with the repairs performed on an as-needed basis. When the performance of a repair was evaluated, visual inspection was used.

3.3 Quantitative Methodology for Continued Service Determinations

Structural loads, engineering material properties, and strength degradation mechanisms are random in nature. The strength, R(t), of a structure and the applied loads, S(t), both are random (or stochastic) functions of time. At any time, t, the margin of safety, M(t), is

$$M(t) = R(t) - S(t). \tag{1}$$

Making the customary assumption that R and S are statistically independent random variables, the (instantaneous) probability of failure is,

$$P_f(t) = P[M(t) < 0] = \int_0^\infty F_R(x)\, f_S(x)\, dx. \tag{2}$$

in which $F_R(x)$ and $f_S(x)$ are the probability distribution function of R and density function of S. Equation 2 provides one quantitative measure of structural reliability and performance, provided that P_f can be estimated and validated.

For service life prediction and reliability assessment, the probability of satisfactory performance over some period of time, say (0, t), is more important than the reliability of the structure at the particular time provided by Eq. (2). The probability that a structure survives during interval of time

(0, t) is defined by a reliability function, L(0, t). If n discrete loads $S_1, S_2, \ldots, S_n$ occur at times $t_1, t_2, \ldots, t_n$ during (0, t), the reliability function becomes,

$$L(0,t) = P[R(t_1) > S_1, \ldots, R(t_n) > S_n]. \quad (3)$$

If the load process is continuous rather than discrete, this expression is more complex.

The conditional probability of failure within time interval (t, t+dt), given that the component has survived during (0, t), is defined by the hazard function:

$$h(t) = -\,d[\ln L(0,t)]\,/\,dt \quad (4)$$

which is especially useful in analyzing structural failures due to aging or deterioration. For example, the probability that time to structural failure, T_f, occurs prior to a future maintenance operation at $t+\Delta t$, given that the structure has survived to t, can be evaluated as,

$$P[T_f \le t + \Delta t \mid T_f > t) = 1 - \exp\left[-\int_t^{t+\Delta t} h(x)dx\right]. \quad (5)$$

The hazard function for pure chance failures is constant. When structural aging occurs and strength deteriorates, h(t) characteristically increases with time.

Intervals of inspection and maintenance that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis. When a structure is inspected and/or repaired, something is learned about its in-service condition that enables the density function of strength, $f_R(r)$, to be replaced by the (conditional) density $f_R(r|B)$, in which B is an event dependent on what is learned from the in-service inspection. The updated density of R following the inspection is,

$$f_R(r|B) = P[r < R \le r+dr, B]/P[B] = c\,K(r)\,f_R(r) \quad (6)$$

in which K(r) is denoted the likelihood function and c is a normalizing constant. The time-dependent reliability analysis then is re-initialized using the updated $f_R(r|B)$ in place of $f_R(r)$. The updating causes the hazard function to be discontinuous in time and lowers the failure probability in Eq. (5).

Uncertainties in methods of in-service inspection/repair affect the density $f_R(r|B)$. A combination of methods is usually more effective from a reliability point of view than using one method. When there are limited resources, it is most effective to select a few safety-critical elements and concentrate on them [5,12]. Optimal intervals of inspection and repair for maintaining a desired level of reliability can be determined based on minimum life cycle expected cost considerations. Preliminary investigations of such policies have found that they are sensitive to relative costs of inspection, maintenance and failure. If the costs of failure are an order (or more) of magnitude larger than inspection and maintenance costs, the optimal policy is to inspect at nearly uniform intervals of time. Additional information on application of the methodology to investigate inspection/repair strategies for reinforced concrete elements in flexure and shear is presented elsewhere [13–14].

4. REFERENCES

1. NAUS, D. J., *Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants*, NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l Lab., Oak Ridge, TN, September 1986.

2. ASHAR, H., NAUS, D. J., and TAN, C. P., "Prestressed Concrete in U.S. Nuclear Power Plants (Part 1)," pp. 30–34 in *Concrete International,* 16(5), American Concrete Institute, Detroit, MI, May 1994.

3. NAUS, D. J. and OLAND, C.B., *Structural Aging Program Technical Progress Report for the Period January 1, 1993, to June 30, 1994,* ORNL/NRC/LTR-94/21, Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l Lab., Oak Ridge, TN, November 1994.

4. OLAND, C. B. and NAUS, D. J., *Summary of Materials Contained in the Structural Materials Information Center*, ORNL/NRC/LTR-94/22, Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l Lab., Oak Ridge, TN, November 1994.

5. CLIFTON, J. R., *Predicting the Remaining Service Life of In-Service Concrete,* NISTIR 4712, National Institute of Standards and Technology, U.S. Department of Commerce, Gaithersburg, MD, November 1991.

6. HOOKHAM, C. J., *Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-90/17, Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l Lab., Oak Ridge, TN, March 1991.

7. REFAI, T. M. and LIM, M. K. *Inservice Inspection and Structural Integrity Assessment Methods for Nuclear Power Plant Concrete Structures*, ORNL/NRC/LTR-90/29, Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l Lab., Oak Ridge, Tn., September 1991.

8. SNYDER, K. A., CLIFTON, J. R. and CARINO, N. J. , *Nondestructive Evaluation of the In-Place Compressive Strength of Concrete Based Upon Limited Destructive Testing*, NISTIR 4874, National Institute of Standards and Technology, U.S. Department of Commerce, Gaithersburg, MD, July 1992.

9. SWIAT, W. J., YOUNG, W., PAJAK, J., FUNAHASHI, J., M. , BURKE, D. and WAGNER, J., *State-of-the-Art Report — Corrosion of Steel in Concrete*, ORNL/NRC/LTR-93/2, Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l. Lab., Oak Ridge, TN, May 1993.

10. PRICE, W. F., BAMFORTH, P. B. and GLASS, G. K., *Review of European Repair Practice for Corrosion Damaged Reinforced Concrete*, Report No. 1303/91/5823, Taywood Engineering Ltd., London, England, April 1993.

11. KRAUSS, P. D., *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-93/28, Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l Lab., Oak Ridge, TN, March 1994.

12. ELLINGWOOD, B. and MORI, Y., "Probabilistic Methods for Condition Assessment and Life Prediction of Concrete Structures in Nuclear Plants," pp. 155–166 in *Nucl. Eng. and Design,* 142, 1993.

13. MORI, Y. and ELLINGWOOD, B., *Methodology for Reliability Based Condition Assessment,* NUREG/CR-6052, The Johns Hopkins University, Baltimore MD, August 1993.

14. MORI, Y. and ELLINGWOOD, B., *Reliability Assessment of Degrading Concrete Shear Walls,* ORNL/NRC/LTR-94/6, Martin Marietta Energy Systems, Inc., Oak Ridge Nat'l. Lab., Oak Ridge, TN, April 1994.

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Nuclear Power Plant Structural Monitoring Program

Programme de contrôle structural dans une centrale nucléaire

Bauliches Überwachungsprogramm in einem Kernkraftwerk

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SUMMARY

This paper discusses the key features of a structural monitoring program in place at one American nuclear power plant located on the Pacific coast. The program has several purposes including defining the condition of the civil structures, identifying problems at an early stage that require corrective or preventive maintenance, and developing "baseline" reference information on the structural conditions for future reference. The monitoring program is expected to yield benefits to the utility in extending the useful life of the plant, reducing repair and maintenance costs, planning and budgeting capital and operations/ maintenance expenditures, and addressing license renewal issues.

RÉSUMÉ

Cet article souligne les points essentiels du programme de surveillance de la structure porteuse d'une centrale nucléaire nord-américaine. Les objectifs sont d'esquisser le développement de l'état structural de l'ouvrage, d'ordonner en temps voulu des mesures de réparation et d'entretien et d'établir un état de référence de la structure permettant des évaluations futures. Les avantages d'un tel programme devraient se traduire par une prolongation de la durée d'utilisation de l'installation, une réduction des coûts de réparation et d'entretien, une amélioration de planification et de budgétisation des dépenses ainsi qu'un renouvellement de l'autorisation d'exploitation.

ZUSAMMENFASSUNG

Der Beitrag erörtert die Hauptteile eines Ueberwachungsprogrammes des Tragwerks in einem Kernkraftwerk an der US-Pazifikküste. Ziele sind die Aufzeichnung der Entwicklung des baulichen Zustands, eine frühzeitige Veranlassung von Reparatur- und Unterhaltsmassnahmen und die Erstellung eines Referenzzustandes des Tragwerks für zukünftigen Beurteilungen. Als Vorteile verspricht man sich eine Verlängerung der Nutzungsdauer der Anlage, eine Reduktion der Reparatur- und Unterhaltskosten, verbesserte Planung und Budgetierung der betreffenden Aufwendungen sowie die Erneuerung der Betriebserlaubnis.

1. BACKGROUND

Existing nuclear power plants in the United States were designed and built, and are operated under rigorous regulatory scrutiny of the U.S. Nuclear Regulatory Commission and its predecessor the U.S. Atomic Energy Commission. The principal goal of this regulatory scrutiny was, and still is, to assure the performance of safety functions of these plants under extreme, and often highly unlikely conditions such as earthquake, tsunami, tornado and high-energy pipe breaks, in order to protect the public from an uncontrolled release of radioactive material.

Most U.S. nuclear power plants were issued Construction Permits (regulatory approval to begin construction) during the 1960s and 1970s. The civil structures of these plants were, thus, mostly built during the period 1965 to 1975 and are from twenty to thirty years old. The design life for these plants was intended to be forty years of operating life. As the plants approach or exceed the half way point in their design lives, regulatory and engineering attention have shifted from design and construction to operation and maintenance. Plants sited on both the east and west coasts are exposed to a marine environment and the increased likelihood of corrosion of steel structures and reinforcing steel in concrete structures. New regulatory guidelines are being prepared and issued regarding maintenance of nuclear power plants for example, the U.S. NRC's Maintenance Rule (10 CFR 50.65) specifies that all nuclear power plant owners must develop a maintenance program for plant systems, structures and components determined to be sensitive to aging. Concerns for the environment and the financial risks associated with building large, new generating plants mean that many utilities will want to use the full design life and in some cases consider life extension of their nuclear power plants.

Much of the focus by regulators and operators has been on the active systems (electro-mechanical systems) which must operate and perform safety functions during or following an extreme event. While this is appropriate, attention must also begin to be given to the passive civil engineering features. Most plant operating organizations are made up of electrical and mechanical engineering staff and few have civil/structural engineering expertise on staff. These skills are generally obtained from in-house central engineering groups or from outside consultants. Many operating organizations are looking to add or have added civil engineering expertise into the operating group or have provided an on-site civil engineering presence.

Many utility power facilities such as dams and their power waterways are required by regulatory agencies, such as the Federal Energy Regulatory Commission, to be inspected on a regular basis. Other power plant facilities such as chimneys and chimney liners are also subject to regular inspection although not a regulatory requirement. Although safety is a primary consideration, concern for economic loss associated with outages and massive repair costs are also part of the decision to inspect and monitor.

Utilities have recognized aging management as an activity vital to the long-term safety, reliability, and economic operation of a nuclear power plant. Because of the extreme expense associated with testing of civil structures, a structural monitoring program is an attractive and relatively inexpensive way of assuring performance.

2. OBJECTIVES OF THE STRUCTURAL MONITORING PROGRAM (SMP)

The overarching goal of the Structural Monitoring Program (SMP) is to provide additional assurance that the passive functions of the civil structures are performed to the relevant specifications. There are a number of supporting objectives outlined below:

- To provide for systematic and periodic inspection of the structures and features and ensure that licensing and design basis requirements are satisfied with acceptable margins;
- To provide a baseline of the current condition of important structures and features and a regular comparison of these conditions with the baseline conditions;
- To provide for the inclusion and prioritization of the appropriate civil structures and structural features into the program;
- To provide a mechanism by which operating experience and lessons learned can be shared and applied;
- To integrate the above information into the plant maintenance program so that preventive and corrective maintenance can be planned and carried out as needed to prolong useful life.

3. SCOPE

The SMP focuses mainly on plant civil structures that are safety related, important to nuclear safety, and/or involve a large risk of economic loss. At one nuclear plant located on the Pacific Coast, the structures currently included in the program are the containments (exterior reinforced concrete shell with steel lining, interior concrete and steel structures), auxiliary and fuel handling buildings (reinforced concrete and structural steel) turbine building and turbine pedestal (reinforced concrete and structural steel), intake structure (reinforced concrete structure and circulating water tunnels), and outdoor water storage tanks (steel tanks, some with concrete exterior).

These structures include both reinforced concrete (structural walls and floor slabs, columns, beams, and other special elements) and steel structures (floor beams, columns, bracing, trusses, and connection elements). Since most of these structures are constructed of reinforced concrete, they receive the most attention. The marine environment, through direct seawater contact or coastal fog, provides two of the three ingredients needed for corrosion activity (moisture, oxygen, and salts). Degradation of reinforced concrete structures due to chloride intrusion and reinforcing steel corrosion also may not be evident until significant damage has already occurred. Typical symptoms of problems include surface discoloration, spalling, cracking, and in advanced stages, delamination. Such areas often exhibit a hollow sound when struck by a hammer. Slowly developing damage in the early stages is typically followed by rapid deterioration, increasing the importance of early detection.

Reinforced concrete applications in nuclear power plant structures also differ somewhat from other industrial and commercial structures. Because of the massive proportions of nuclear power plant equipment, and unique requirements such as shielding and severe design conditions (e.g., pipe break, seismic, tornado wind and wind-driven missile), reinforced concrete structures tend to have thick, heavily reinforced walls and slabs, massive basemats, and unusual configurations. Strengthening modifications such as thickening of walls after the original construction was completed also create a condition where two layers of concrete with different ages may be placed in contact. High thermal loads due to operating characteristics of equipment and systems may also occur in structures. These characteristics may cause more significant cracking than in ordinary concrete structures. While such cracking usually has no effect on structural integrity, it does increase the potential for reinforcing steel corrosion.

Degradation or damage to steel structures is generally more evident and noticeable than in concrete structures. Evidence of a problem such as corrosion, damaged coatings, and deformation is also more apparent because steel structures are more often exposed and accessible to inspectors.

In addition to important structural elements noted above, secondary features or elements such as architectural siding and equipment supports are also given a cursory review by the program. Inappropriate modifications and changes to configuration might also be discovered through the walk down. However, this coverage is not the primary emphasis of the walk down or inspection team.

4. IMPLEMENTATION OF THE PROGRAM

The key program steps include preparatory work, walk down implementation, and follow-up actions. Preparatory work includes data gathering and planning for the walk downs. In addition to gathering information on the configuration of each of the structures, design criteria, design and as-built drawings, and material and construction specifications are collected and reviewed. Lists of system problems and current projects or modifications are also compiled and placed in a central location.

The purposes of walk down planning are to identify the important elements to be covered in walk downs, develop an efficient walk down path or sequence, checklists, and other practical tools. In order to provide the most cost-effective walk downs, the key structural elements or features are selected during the planning phase. These elements or features are primarily chosen based on their importance to the structural system of the building or plant safety systems. In addition, structures subjected to harsh environmental or operating conditions, for example, exposure to seawater or differential displacements caused by pressurization during integrated leak rate testing (ILRT) of the containments are included in the walk downs.

Limitations on access to the identified structural elements also deserve consideration. Physical barriers, hazardous conditions, operating temperatures, radioactive contamination or attempts to keep radiation dose rates to walk down team members "as low as reasonably achievable" may impede access to the structures requiring monitoring. In addition, the timing for walk downs should be considered. In some cases, parts of structures are accessible only during a plant outage (e.g., inside containments). However, the scheduling of walk downs during refueling outages may also hinder access in other cases. Materials staging and disassembled equipment may prevent inspection of parts of the structure. Walk down planning, of course, should be a living process in which improvements and lessons learned are continuously incorporated into the plan for subsequent walk downs. A written monitoring plan also provides a road map for future users, and provides guidance in the event that the designated personnel are unavailable.

Walk downs are generally performed by two civil engineers, one serving as a back-up or alternate for the other. In addition, engineers from other disciplines, or maintenance or operations personnel may be included. The need for additional personnel is considered case-by-case. Data collection forms which have been developed in advance are usually used on the walk downs. Besides personnel safety equipment, other tools brought along on the walk down may include general area drawings, flashlight, tape rule, pens and markers, crack measuring device such as a thickness gage, camera, and hammer for sounding concrete. In areas where potential for radioactive contamination is significant, tools, papers, and the like are kept to a minimum.

The collection of baseline data for various structures or elements is also included. In this program, baseline information is defined as the data collected on the physical condition of the structure or element being inspected, in the initial walk down under the SMP. This information is then used for tracking and trending to monitor changes. For example, for structural concrete this information may take the form of crack maps, records of widths and length or areas that are spalled or delaminated, or some measures of settlement or alignment. In order to select the appropriate features for monitoring and to gather and maintain this data in a cost-effective manner, consideration is given to factors such as importance of the structural elements, redundancy in the structural system, seismic margin possessed by the particular element, exposure to harsh environments, and structural configuration. The set of baseline features is not intended to be static. Instead, if important changes or events occur, additional information or additional elements would be added to the data set.

Following the completion of the walk down, a concise report of observations and findings is prepared. This report summarizes the events of the walk down, discusses follow-up or corrective actions, and identifies items that need to be done at the next walk down. Problems or corrective actions are handled according to an established procedure for dealing with plant problems. Follow-up actions may include engineering evaluation to ensure qualification or adequacy of margin, and additional testing or investigation by specialists or experts (e.g., corrosion rate testing, obtaining cores from damaged concrete, pulse velocity or other in-situ tests).

Repairs to the damaged area may be considered. Factors related to repairs include costs of repairs, consequences of deferral, corrective actions to the root cause of the degradation or damage, and the practicality and effectiveness of design changes and repair methods. Design changes or repairs might include increased concrete cover, different concrete mix designs for repairs or use of grouting for patch work, protective coatings to both concrete surfaces and rebars, and the addition of sacrificial zinc strips to reinforcing steel. The importance and function of damaged structural elements should also be considered (e.g., the effect that spalled or delaminated concrete may have on the shear capacity or compression zone of a reinforced concrete wall or beam). In a power plant, the accessibility of the structural elements requiring repair is often hindered by attachments such as electrical raceways, piping, and equipment. The temporary support, removal, or de-activation of such attachments is often necessary to facilitate the repairs. In some cases, it may be more economical to replace rather than refurbish commodities such as electrical raceways and heating and ventilation ducts. These factors will also influence the budget planning process, where repairs will compete with other projects for scarce funds in today's operating environment.

5. CONCLUSIONS

The SMP, properly implemented, provides additional assurance that civil engineering features at a nuclear power plant will meet performance objectives. In addition, there are benefits of an economic nature that accrue to the owner/operator by implementing an SMP. These are summarized below.

- Through regular inspection, significant degradation of structures important to safety or continued generation will likely be discovered in a timely manner. For many structures, degradation that is discovered only after it has progressed to a certain point can lead to an extended outage or be expensive to repair. The SMP also allows for planned, smaller, incremental investments in maintenance instead of a potentially large, and unplanned investment.
- The baseline which is established by the SMP, allows comparison of effects after an event, such as an earthquake, against the baseline. This can be useful in convincing regulators to allow production to begin more quickly, or, perhaps, at all. The baseline can also assist in validation of insurance and or warranty claims.
- The SMP allows for optimization of expenditures for maintenance and rehabilitation by providing information for more effective maintenance planning. Trend analysis allows evaluation of the need for expenditures and for acceleration or delay of expenditures.

- The SMP provides an effective tool in dealing with regulatory bodies in that it puts dealings on a more factual basis. This can prevent delay in receiving approvals and avoid problems that might lead to loss of production and revenue.
- The SMP provides an aid to investment decisions about life extension. Trending information, gathered over a relatively long period, is felt to be more useful than a one time study that cannot provide information as to whether effects are accelerating, stable or diminishing with time. The SMP can provide for a more intelligent decision.
- By integrating the SMP into the plant maintenance program, communication between the plant maintenance staff and the civil engineers will be enhanced. This communication should lead to improved understanding by both groups to the benefit of plant safety and economy. The team-building environment fosters networking (within the plant organization, other plants and coastal installations), development of knowledge about new products and techniques, and current research by universities and professional groups.

In conclusion, the SMP is believed to be a positive contribution to both plant safety and to the efficient operation of the plant for economic benefit.

6. REFERENCES

1. ASHAR, HANS and DAVID JENG, Regulatory Perspective on Performance of Structures, Proceedings of the Fifth Symposium on Current Issues Related to Nuclear Power Plant Structures, Equipment and Piping, 1994.
2. GERWICK, BEN C., et al, Corrosion Damage. Repair, and Protection of Cooling Water Intake/Circulating Water Systems, (to be published in 1995).
3. GREGOR, F.E., and C.J. HOOKAM, Remnant Life Preservation of LWR Plant Structures, Proceedings of the 12th International Conference on Structural Mechanics in Reactor Technology, 1993.
4. NAUS, D.J., Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants, NUREG/CR-4652, U.S. Nuclear Regulatory Commission, 1986.
5. ZWICKY, P., and D. KLUGE, Aging Management for Safety Related Concrete Structures in Switzerland, Proceedings of the 12th International Conference on Structural Mechanics in Reactor Technology, 1993.