Session 2: Inspection and monitoring

Objekttyp: **Group**

Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

Band (Jahr): **46 (1983)**

PDF erstellt am: **23.07.2024**

SESSION 2

Inspection and Monitoring

Inspection et enregistrement des mesures

Überwachung und Bestandesaufnahme

Leere Seite
Blank page
Page vide

Appraisal · a Cyclical Process of Inspection and Calculation

Evaluation · un procédé alternatif d'inspection et de calcul

Bewertung · ein zyklischer Prozess der Inspektion und Berechnung

Paul BECKMANN
Technical Director
Ove Arup & Partners
London, England

Paul Beckmann, born 1924, graduated from the Danish Technical University. Responsible for many restoration projects. Awarded the I.C.E. Telford Gold Medal for paper «York Minster Structural Restoration». Lecturer at ICCROM-Rome, Member ISE committee on «Appraisal of existing structures».

Edmund HAPPOLD
Professor
University of Bath
Bath, England

Edmund Happold, born 1930, Engineer for many restoration projects including St. Katherine's Dock and Theatre Royal, Bath. Awarded Oscar Faber medals of I.Struct.E for papers on working on Centre Pompidou and the Riyadh Conference Centre. Member of Technical Committee and Chairman WC V, Chairman ISE committee on «Appraisal of existing structures».

SUMMARY
This paper describes the process of appraisal of existing structures with special emphasis on the refining of calculations by a conscious cyclical process of inspection and calculation.

RESUME
L'article décrit un procédé pour l'évaluation de structures existantes, mettant l'accent sur l'exactitude des calculs suite à un procédé alternatif conscieux d'inspection et de calcul.

ZUSAMMENFASSUNG
Der Artikel beschreibt das Vorgehen der Bewertung von bestehenden Bauten unter spezieller Betonung der Verfeinerung der Berechnungsmethoden durch einen bewussten zyklischen Prozess der Inspektion und Berechnung.

In 1742 Pope Benedict XlV, concerned with the state of the dome of St. Peters, requested three men, Le Seur, Jacquier and Boscowich to carry out a structural survey to determine the causes of distress and to devise remedial measures. The report, published the following year, was prefaced by an apology that said they had assessed it with theoretical mathematical reflection only because the building was so unique. Then followed a detailed survey of the dimensions and a discussion on possible explanations for the damage and named the yielding of the tie rings at the circumference as the cause. But the interesting part of this report was the second part because an attempt was made to calculate the horizontal thrust and to prove that the two tie rings built in at the time of erection were no longer able to carry this thrust.

The report caused a forore. One comment at the time stated: 'If it were possible to design and build St. Peter's dome without mathematics and especially without the new fangled mathematics of our time, it will also be possible to restore it without the aid of mathematicians and mathematics ... Michelangelo knew no mathematics and yet was able to build the dome ... Heaven forbid that the calculation is correct. For, in that case, not a minute would have passed before the entire structure would have collapsed.' Certainly the analysis contained some errors. But in spite of disagreements as to the causes of the damage most people were agreed on the measures to be taken, and in 1743 five additional rings were built in the cupola.

The importance of this event was that, contrary to tradition, the stability of a structure had not been based on empirical rules and opinion but on a detailed survey and mathematical analysis.

Today we are even more interested in developing the art of structural appraisal. We have a large stock of structures and buildings representing successive deposits of human imagination, which we are reluctant to discard for emotional or hard economic reasons. Urban renewal is a rapidly expanding exercise.

The art of appraisal of structures is different from design. In design the forces follow the choice of form and the analysis follows that. In appraisal the engineer is left face to face with an existing structure of definable qualities and must determine its condition and suitability of use. This is not an easy task.

The reasons for appraisal to assess the present condition may arise from change of ownership, change of use, deterioration in service, defects in the structure, future safety, accidental damage etc.

This requires consideration of the levels of safety appropriate to the further use of the construction, the assessment of loading, the evolution of methods for determining the strength of the structures, their components and constituent materials, and the derivation of suitable methods for calculating their composite behaviour. Requirements for remedial measures, restriction of use and monitoring performance may also form part of an appraisal procedure.

With this in mind the Institution of Structural Engineers in 1976 formed a working committee of experienced engineers to produce a guide to the appraisal of existing structures. This committee produced its report in 1980 and the authors of this paper are, in effect, representing the committee since one author was the Chairman and the other author one of the other two writers.

THE PROCESS

The process of appraisal is cyclical as shown in the flow charts (see Figs. 1, 2, 3 and 4). Information is collected and assessed. If the result shows that the structure is adequate the process can stop there. If inconclusive more information can be collected, assessed more thoroughly and so on. The action required should be taken in stages, each stage depending on the findings of the previous one.

Like all engineering activities, structural assessments are usually subject to cost and time limitations. The time spent on calculations should therefore be used as effectively as possible: There is no merit in an elaborate elastic analysis of a truss, if the strengths and stiffnesses of the joints are only imperfectly known. It would be far better to spend time studying the behaviour of the joints, using member forces from an approximate calculation.

STAGE 1

Gathering of Information

Fig. 1 illustrates the initial gathering of information. The first loop emphasises the need to obtain as much documentary evidence as possible: some effort at this stage may save a considerable amount of calculations and physical testing later. The other important operations are the site inspections: a) to make sure that the paper information is relevant to the actual building (and has not been superseded by a subsequent design alteration, lost in the meantime); b) to give the appraising engineer a first hand visual impression on how the structure performs in its present condition; this can be a very useful check on the validity of later calculations.

The Initial Assessment

Fig. 2 shows the processes which may be involved in ensuring that the structure is a stable configuration and not liable to progressive collapse in case of relatively minor accidental damage. It is important here, as elsewhere in appraising, to choose a mathematical model which not only is easily understood, in terms of stability, but also takes advantage of such physical features, which in ordinary design might be ignored but which, in the actual building, contribute significantly to stability (eg. infill brickwork panels in a framed structure).

The instructions following the question on collapse are reminders to the engineer not to leave an inherently dangerous structure just because it happens still to be standing, nor to be too easily satisfied with his first answer to the question of why it fell down.

Fig. 3 indicates the steps in the initial assessment of the strength of the structure.

The assessments of loads, forces and strengths of materials will at this stage usually be based on the available documents combined with the information from the in-situ survey. Tests on the actual materials will rarely be appropriate before the "simple check calculation"; they may however be called for in the course of the re-cycling loop under: "Re-assess Strengths ...".

"Simple check calculation" refers to the absence of assumptions and/or procedures beyond what is normally used in initial design. The "frame analysis" may at first be no more than reasonable estimates of support moments, but when "recycling" a proper analysis may be necessary. "Check satisfied" means that the calculation indicates (possibly by inference) that the recommendations of the relevant code of practice could be shown to be observed.

Attention is drawn to the repeated instruction: "Re-inspect Structure in-situ": This is a most essential, perhaps the most essential, step in the process, and without this cross-reference to reality, the entire appraisal can become invalid.

"Drastic Deficiency" may be assumed to be the case if the calculated overall load factor is 1.1, or less, on dead load alone.

If the results of the assessment at the end of this stage are unequivocal, one way or the other, there remains only to report the conclusions. If, however, the structure has been observed to carry most of its load with little or no sign of distress, but the calculations indicate an overall factor of safety greater than 1, but less than what is normally accepted, then it may be profitable to improve the basis for the calculation.

STAGE 2

Improving the Assumptions

The calculations, so far, have been based on conventional design assumptions. It is therefore worth examining the mathematical model for simplifications which may have led to over-conservative results of the calculations.

Another field for re-examination is the values used for loads and materials' properties: if they can be ascertained with less uncertainty than is the case for conventional pre-construction design calculations, then the same real factor of safety can be achieved with a lower calculated factor.

This may be most easily understood by considering the basic design equation in the partial factor format:

$$\gamma_s \times \text{load effects} = \frac{\text{structural resistance}}{\gamma_m \times \gamma_c}$$

where γ_s is a factor compensating for the uncertainties in predicting the effects of the loads, γ_m is a factor compensating for the uncertainties in predicting the resistance of the structure and γ_c is a modification factor compensating for differences in failure sequences and failure consequences. According to ISO 2394, each of these γ factors is made up of two or more sub-factors and their relation to appraisal of existing structures is discussed below.

I.S.O. 2394 DEFINITION	COMMENTS
γ_{s1} *takes account of the possibility of unfavourable deviation of the loads from the characteristic external loads, thus allowing for abnormal or unforseen actions*	The inherent variability of the live loads is clearly independent of whether the structure is existing or only at design stage. There is therefore usually no justification for reducing the γ_{s1} for live loads. Dead loads can often be ascertained with less uncertainty in an existing situation: thicknesses and densities of partitions and floor finishes can be measured, and so can actual structural dimensions. γ_{s1} can therefore be reduced for dead loads, provided adequate measurements and sampling are carried out.
γ_{s2} *takes account of the reduced probability that various loadings acting together will all be s simultaneously at their characteristic value.*	The probability of simultaneous occurence of loads of different origin should not change significantly from 'design stage' to 'as existing'. There is therefore usually no justification for varying γ_{s2}.

γ_{s3} is intended to allow for possible adverse modification of the loading effects due to incorrect design assumptions (introduction of simplified support conditions, hinges, neglect of thermal and other effects which are difficult to assess), constructional discrepancies such as dimensions of cross-section, deviation of columns from vertical and accidental eccentricities.

It is usually possible, albeit to a varying degree, to reduce the amount of approximation in the assumptions, when one is analysing a particular structure or element. 'Constructional discrepancies' can also sometimes be measured and included in the calculations of an existing structure.

Subject to the verisimilitude of the analytical model and adequate measurements of structural dimensions, γ_{s3} can therefore be reduced.

γ_{m1} is intended to cover the possible reductions in the strength of the materials in the structure as a whole as compared with the characteristic value deduced from the control test specimens

If the strength of the material in the actual structure is adequately tested, then the reason for introducing γ_{m1} has been eliminated. Usually, however, the testing regime, which complete elimination of γ_{m1} would require, is too onerous, but a reasonable amount of testing should nevertheless justify a worthwhile reduction of γ_{m1}.

γ_{m2} is intended to cover possible weakness of the structure arising from any cause other than the reduction in the strength of the materials allowed for in γ_{m1}, including manufacturing tolerances

This covers, among other things, the local variations, within the structure, of the strength of the material. When measurements include this, eg. when concrete core samples from the top and from the bottom of columns are tested, a reduction of γ_{m2} is justified.
This applies equally to design and to appraisal. Many design codes do not appear to vary their safety factors to take sufficient account of this but it should be possible to do so when assessing existing structures.

γ_{c1} is intended to take account of the nature of the structure and its behaviour: for example structures or parts of structures in which

In the case of brittle structures such as cast-iron columns and over-reinforced concrete beams, an increase in γ_{c1} will be called for.

partial or complete collapse can occur without warning, where redistribution of internal forces is not possible, or where failure of a single element can lead to overall collapse

γ_{c2} *is intended to take account of the seriousness of attaining a limit state from other points of view, for example economic consequences, danger to community etc.*

Here again, some existing design codes do not show any graduation, ie. the stipulated safety factors are the same for a 1m lintol as for a 15m beam over an assembly hall.

When appraising an existing structure one should distinguish between secondary members, failures of which will not cause progressive collapse, and primary members supporting other parts of the structure or secondary members which, if collapsing, might cause loss of life and limb.

The values for imposed loads are usually defined by Standards and Codes of Practice. All other values can, in the case of appraisal of an existing structure, be defined by observation and measurement. Defining at an appropriate level the values used for materials' properties is extremely important and the Institution of Structural Engineers' report provides a statement of the state of the art including a section on load testing.

Fig. 4 illustrates the improvement of the assumptions and reconsideration of partial safety factors.

It must however not be overlooked that extensive measurements, sampling and testing are time consuming and expensive and in some historical buildings they are nearly impossible to carry out without causing unacceptable damage to finishes.

The engineer should beware of initiating a surveying and testing exercise if he has doubts that they will lead to significant savings on strengthening works, because the end result could be that his client has to pay for both survey and remedial works.

The process of appraisal is cyclical because refinement of calculations is only justifiable if they are based on equally accurate factual information and facts may be expensive to collect.

On the other hand, there is no excuse for the intellectual laziness which condemns an old, good, building on the grounds that a conventional design calculation indicates non-compliance with a present-day code of practice.

Fig. 1

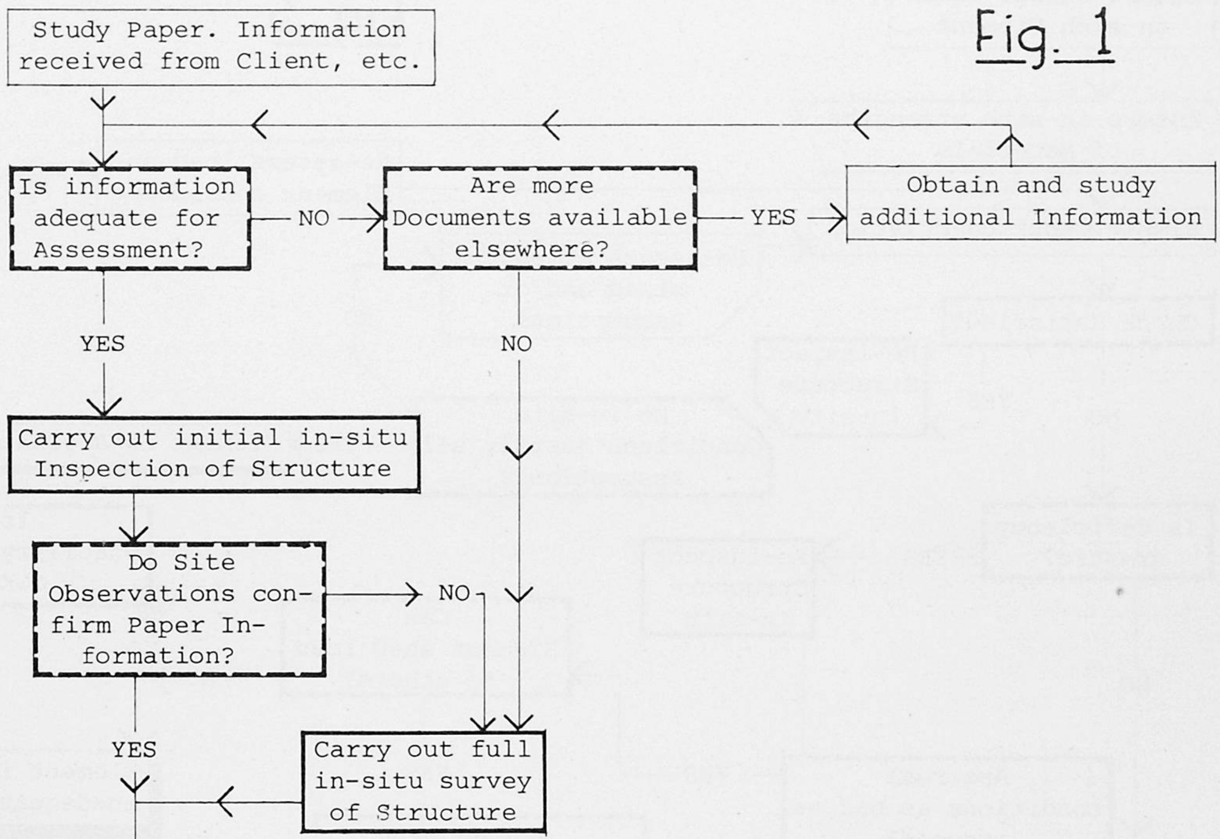
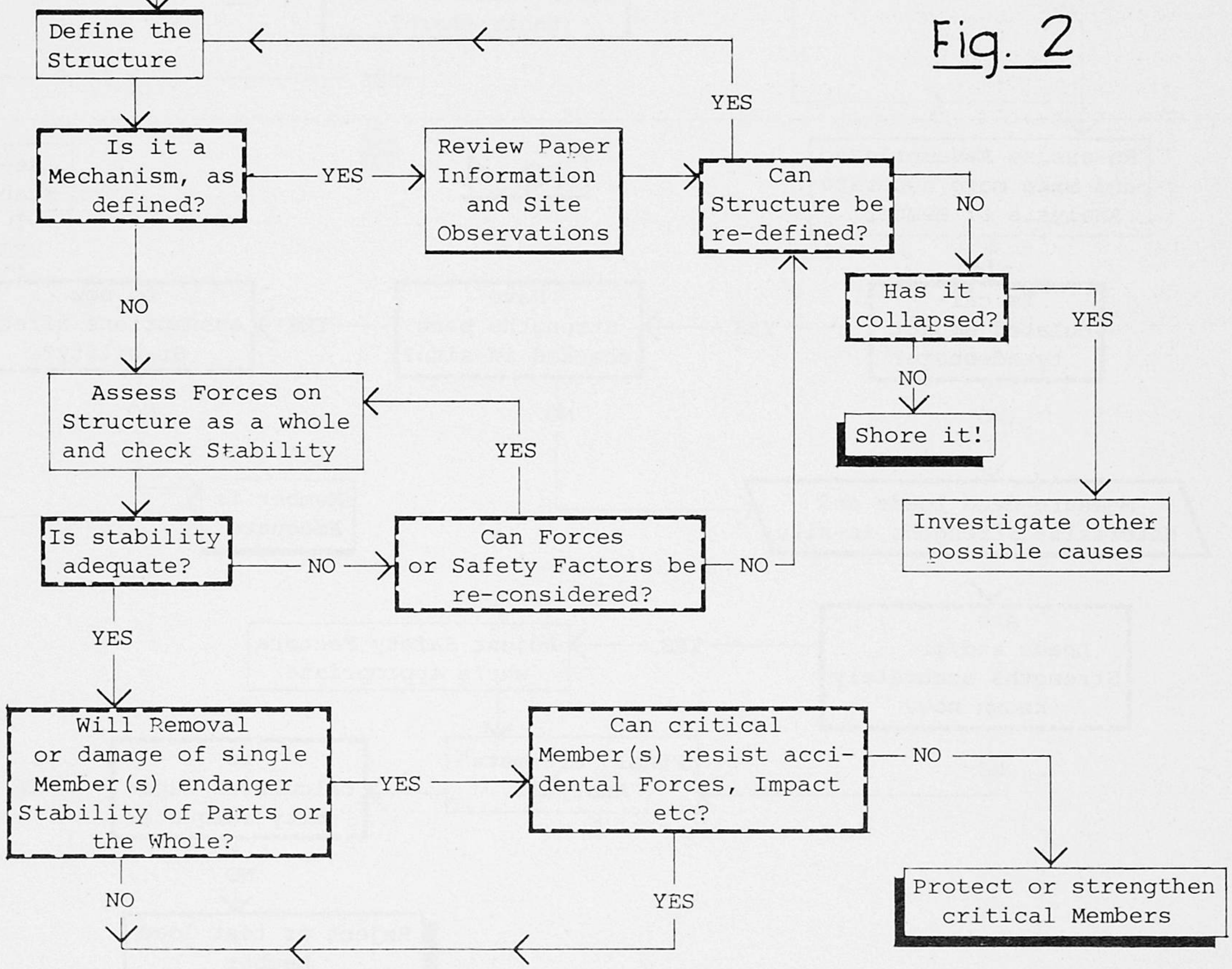

Study Paper. Information received from Client, etc.
Is information adequate for Assessment?
NO
Are more Documents available elsewhere?
YES
Obtain and study additional Information
YES
NO
Carry out initial in-situ Inspection of Structure
Do Site Observations confirm Paper Information?
NO
YES
Carry out full in-situ survey of Structure

Fig. 2


Define the Structure
YES
Is it a Mechanism, as defined?
YES
Review Paper Information and Site Observations
Can Structure be re-defined?
NO
Has it collapsed?
YES
NO
Shore it!
NO
Assess Forces on Structure as a whole and check Stability
YES
Investigate other possible causes
Is stability adequate?
NO
Can Forces or Safety Factors be re-considered?
NO
YES
Will Removal or damage of single Member(s) endanger Stability of Parts or the Whole?
YES
Can critical Member(s) resist accidental Forces, Impact etc?
NO
Protect or strengthen critical Members
NO
YES

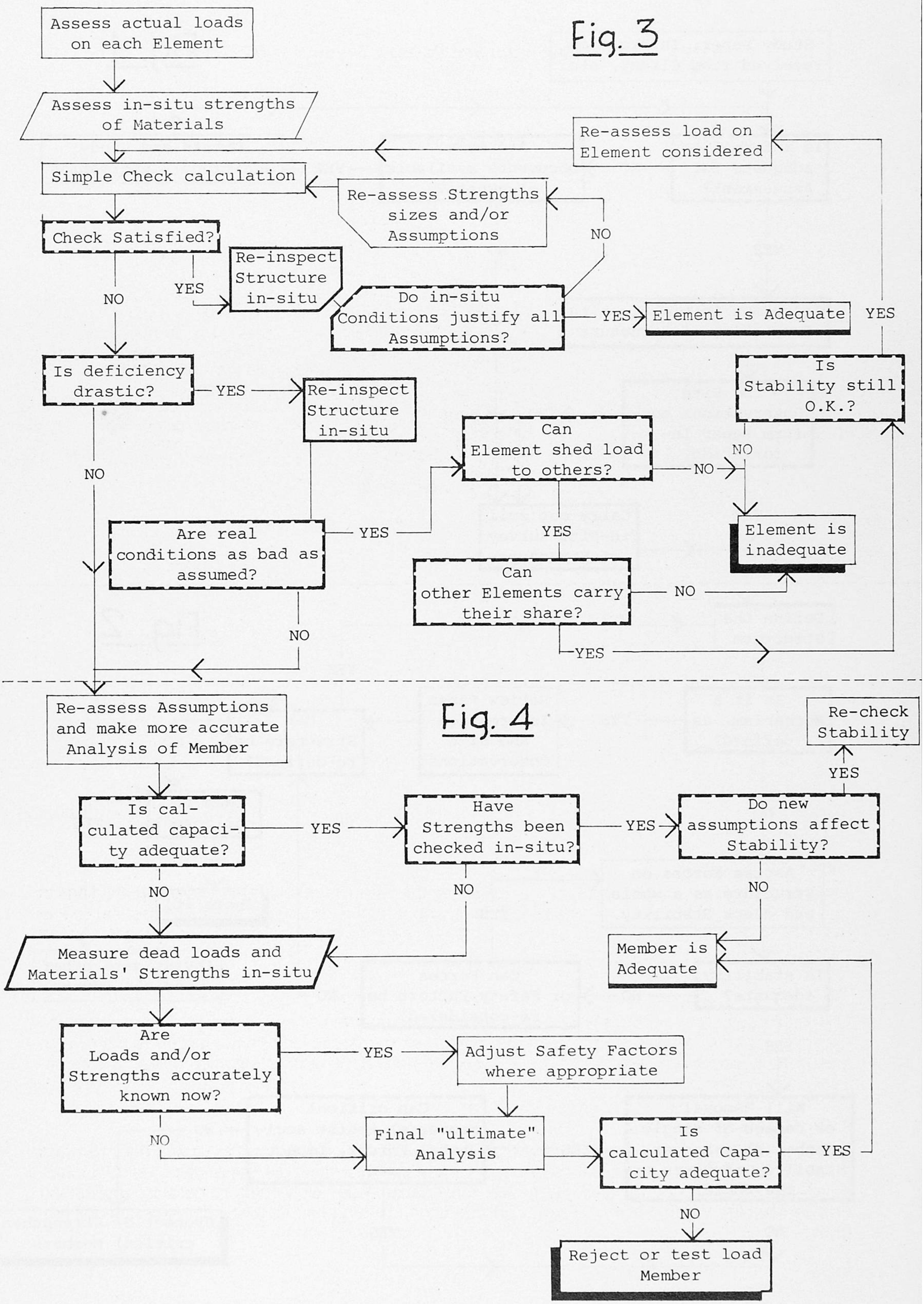

Assess actual loads on each Element
Assess in-situ strengths of Materials
Simple Check calculation
Check Satisfied?
YES
NO
Re-inspect Structure in-situ
Re-assess Strengths sizes and/or Assumptions
Re-assess load on Element considered
Do in-situ Conditions justify all Assumptions?
NO
YES
Element is Adequate
YES
Is deficiency drastic?
YES
Re-inspect Structure in-situ
NO
Is Stability still O.K.?
Can Element shed load to others?
NO
NO
Element is inadequate
Are real conditions as bad as assumed?
YES
YES
Can other Elements carry their share?
NO
NO
YES

Fig. 3

Fig. 4

Re-assess Assumptions and make more accurate Analysis of Member
Re-check Stability
YES
Is calculated capacity adequate?
YES
Have Strengths been checked in-situ?
YES
Do new assumptions affect Stability?
NO
NO
NO
Measure dead loads and Materials' Strengths in-situ
Member is Adequate
Are Loads and/or Strengths accurately known now?
YES
Adjust Safety Factors where appropriate
NO
Final "ultimate" Analysis
Is calculated Capacity adequate?
YES
NO
Reject or test load Member

Safety of the «Tabularium and Palazzo Senatorio» Monuments

Sécurité des monuments «Tabularium et Palazzo Senatorio»

Die Sicherheit der Monumente «Tabularium» und des «Palazzo Senatorio»

Giorgio CROCI
Professor
University of Rome
Rome, Italy

Giorgio Croci, born in 1936, received his civil engineering degree and is now professor of Tecnica delle Costruzioni with the Faculty of Engineering at the University of Rome. He has carried out a large number of projects for strengthening monumental works, and has organized courses and conferences on the subject.

SUMMARY

This paper discusses a methodology for examining the safety of monuments and of old buildings in general. In fact, the way uncertainties are dealt with in these structures plays an essential role in the evaluation of their safety as they are, and thus in defining the necessary investigations and the criteria for the restoration operation.

RESUME

Une méthodologie est proposée pour l'étude des monuments et des constructions anciennes en général. Dans ces ouvrages, la manière de traiter les incertitudes joue un rôle essentiel dans l'évaluation de la sécurité à l'état actuel et, en conséquence, dans la détermination des enquêtes nécessaires et des critères d'intervention.

ZUSAMMENFASSUNG

In diesem Beitrag wird eine Methodologie zur Untersuchung der Baudenkmäler und der Altbauten im allgemeinen beschrieben. Bei diesen Bauwerken spielt nämlich die Behandlung der Unsicherheitsfaktoren eine bedeutende Rolle für die Einschätzung der Sicherheit im derzeitigen Zustand und demzufolge für die Feststellung der erforderlichen Untersuchungen und der Kriterien des Eingriffs.

1. INTRODUCTION

Evaluating the safety of a building is tied to the investigations that have been made and to the operations planned on to be made. The evaluation must first be made taking things as they are, using as support all the data that is immediately available.

Should this evaluation reveal inadeguate safety margins, two things may be done, either alternatively or together: extend and deepen the investigations so as to reduce uncertainties and better clarify the facts, or plan operations for the reinforcement and adaptation of the structure.

The choice of the level of knowledge at which to stop the investigation and go on to the planning of any operations depends on a number of aspects. Among these are: the economic (comparison of the cost of further investigation with the cost of operations, taking into account the possibility that further studies may not reduce operations); the risk-factor (to be evaluated is the probability that further damage may develop, or that the building may even collapse, during the longer time taken for investigation); and the building's artistic value (any changes made by investigations or operations must be kept limited).

2. RELIABILITY AND CREDIBILITY

2.1 Two things condition a safety evaluation:
- the uncertainties in the magnitudes, laws, models, hypotheses etc. involved in the study;
- how far the phenomena or actual situations may be adequately represented mathematically.

2.2. Reliability

The dealing with uncertainties, that is, with the random nature of the quantities and magnitudes involved, in the object of probabilistic and semi-probabilistic analyses.

In applyng this method to existing constructions, and in particular to those that are damaged or weak two kinds of difficulties present themselves: the definition of the characteristic parameters, and the problem of how to deal with uncertainties in structural behaviour.

Regarding the characteristic parameters (material's strengths, indirect forces induced by ground settlement, etc.), it is in fact hard to follow standardized procedures, without introducing subjective content. The result is that the partial safety coefficients γ mean very little in themselves, and thus only the ensamble of γ and characteristic value has real meaning: the more precautionary figure the former is set at, the smaller may be the value of the latter, and conversely.

Regarding structural behaviour, the most reasonable appearing path to follow is to use several models for reference, it being in general quite difficult to establish which of them will be the "worst case" a priori.

Furthermore, and quite otherwise from the case of a building that is a yet to be built, in studying an already-existing structure it is worthwhile taking even magnitudes or situations having slight or no reliability at all into account, should it come about that their actual existence will mean a less favorable structure behaviour. Doing so, investigations may be so oriented as to find that the probability of these situations coming about has increased, or even become a certainty.

From these remarks it appears obvious that if it is desired to keep a rational approach to the application of the semiprobabilistic method, without recurring to higher-level probability analyses (a thicket of difficulties), the uncertainties in magnitudes, laws, hypotheses and models adopted must be brought out in some way; the "reliability index" in fact answes to this purpose, and lets account be taken of the precautions taken in formulating an opinion on the building's safety (see Section 3).

2.3. Credibility

The limits of scientific knowledge and the incompleteness of the available data (this relative too to how far the investigations have been taken) do not often permit of adequately knowing and representing the true phenomena and behaviours. Typical examples of this problem can be found in the soil-structure interaction phenomena, in the laws according to which the development of certain phenomena can be predicted, in the models of structure behaviour after cracking, crushing, deterioration, induced stress states, etc.

These aspects can find no place in a semi-probabilistic analysis, nor indeed even in a higher-level probability analysis. Even the characteristic crushing strength becomes a quantity that is essentially defined on subjective bases, thick with wrtitrary assumptions, when only a few unhomogeneous tests are available (sclerometric tests, ultrasonic tests, core-borings, etc.), which themselves are difficulty correlated on the basis of standard rules.

But these problems have anyway one aspect in common: if high reliability is wished, then severely conservative magnitudes, laws, models and hypotheses, representing limit situations that are often quite far frome reality, must be adopted. And at times this can be difficult to do, since it is not always possible to foresee what direction the approximations must take so as to have the highest reliability.

In conclusion, the designer may find himself in the fix of having to make "reliable" evaluations of safety only by veering far away from "reality" or from the "average values" and this can lead to excessively conservative opinions, making structural operations appear necessary that in fact are not.

So as to bring out these situations, as the study phases go ahead it is necessary to develop too, together with an indication of their "reliability", a "judgement" on the "credibility" of the values chosen, ad is summarily indicated in the description of the study phases (Section 3) and in the attached table.

This kind of judgement is something new relative to traditional, whether deterministic or probabilistic, analyses, since it lets the subjective and the objective, the empirical and the theoretical, all find their places within a single methodology, throwing into sharp relief the scant significance that theoretical analyses can have when not even further investigation can clarify the facts.

2.4. Safety, investigations and interventions.

The "theoretical verification" of a building's safety in its as-is state is fully meaningful, and lets a favorable or definitive opinion of it be formed, only when the corresponding indices of reliability and of credibility are high. But this is not always the case.

As already mentioned, it is worthwhile taking situations of low reliability into account, the purpose of this being to have an indication of more favourable and economic operational criteria. In fact, by modying strengths, constraints, etc., these operations will make reliable situations that originally were not so.

However, it is often indispensable to take situations of low credibility into account, and not only because some complex phenomena may never be fully understood, but especially because it is expedient to begin the study with the few data initially available and go on, taking advantage of the information deriving from the investigations to improve credibility, thus rendering situations reliable and hence less serious than initially assumed.

The logic is similar to that followed in the branch of probability theory making use of the Bayesian criterion.

This, starting from a priori distribuion functions based on a small amount of original data and on subjective assessments, changes the "a priori distributions" little by little as new objective date becomes available, into "a posteriori distributions". For equal reliabilities, this lets the partial coefficient γ be corrected.

In a limit-state analysis -- which is operationally a deterministic analysis -- a deeper knowledge, that is, an improved credibility on the relevant mecanical quantities and on structural behaviour allows on to modify the characteristic values and structural behaviour models. For equal reliability, the partial coefficients γ may remain unaltered.

When credibility remains very low, despite the carrying out of the investigations that appeared suitable within a technic-economic context, the judgement may be deemed acceptable only if the safety is "theoretically verified"; if it is not, this doesnt necessarily imply that the construction does not possess adequate safety margins in reality. In this case, the safety assessment must rely on a proper balance between theoretical verification and judgements of empirical and intuitive nature; their relative weight depends on the difference between the degrees of reliability and of credibility.

3. THE ORGANIZATION OF THE STUDY

The study of the Palazzo Senatorio and Tabularium was repeated several times, account being taken first of only the immediately available data, wider-ranging investigation programs then being gradually defined, within the spirit of what has just been said, ad the checking out of the safety in the as-it-is state manifested inadequate margins.

The study was organized into the five phases to be set forth below; in the penultimate column (as shown in the attached table) is indicated the "credibility" of the mathematical representations, the interpretations, the assumptions, the hypotheses, the schemes, the values, etc., that is, the correspondence between the choices made and the reality of the phenomena and thus the credibility of the results. In the last column is indicated the "reliability" of those same choices listed above, that is, the prudence and precaution taken regarding safety when the reliability can be a priori defined.

I. DESCRIPTION OF THE WORKS

This is summarized in six items (see the table); each of these, is the result of the investigations made and is characterized by a greater or lesser "credibility". The "reliability", instead, is introduced only in the later phases (figs. 1 and 2).

II. THE MODELLING OF THE FORCES, OF THE MATERIALS CHARACTERISTICS AND OF THE STRUCTURAL BEHAVIOUR

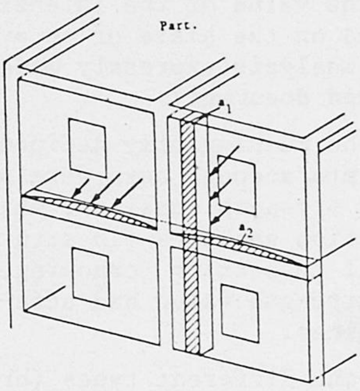
With reference to the facade facing on the Forum, the salient aspects the model necessary for making the computations are defined.

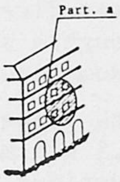
Rearding the forces and materials, their characteristic values are defined and the corresponding partial coefficients are assigned (see phase IV). Three modes of collapse were considered (fig. 3) for the structural behaviour:

a) "local" behaviour of the individual facade portions lying between floor structures and cross walls;

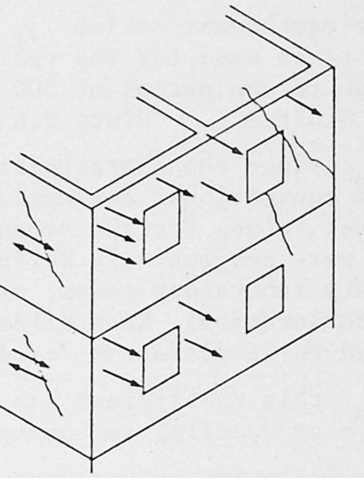
Fig. 1 - Facade looking on the Forum

Fig. 2 - Facade looking on the P.zza del Campidoglio.





b) Facade connections with walls and floor structures behind



a) Local behaviour of individual facade porzion.
a_1 Combined compression and bending in the vertical strips tied to the floor structures.
a_2 Arch effect in the horizontal strips tied to the cross walls.

c) Overall behaviour depending on the shear strength of the cross walls.

Fig. 3 - Collapse possibilities for the Facade looking on the Forum.

b) the breaking-off of the Facade as a whole from the floor-structures and cross walls behind it;

c) a behaviour as "a whole" that involves the shear breakage of the cross walls.

III. COMPUTATIONAL PROCEDURES

The computation procedures refer back to the theory of elasticity, local plasticization being allowed in individual cross-sections.

The material was considered homogeneous, isotropic, and continuous.

The forces and moments, N, M, T are found, and then the stresses, σ_v (vertical) and τ_o (horizontal).

In analyzing the bearing capacity of the walls due to the arch effect, just as in analyzing the shear-walls, their ultimate capacity is first computed; in this way the maximum value of seismic action they can take is determined.

IV. THEORETICAL SAFETY VERIFICATION

The theoretical safety evaluation is done by combining, according to the code the stresses calculated in Phase III and checking that the following inequality is met:

(1) $S_K \cdot \gamma_f \leqslant \gamma_n \quad R_K / \gamma_m \longrightarrow S \leqslant R$

Since compound stresses are being dealt with, a check is made that the point representing the force or stress-state S (M, N, ..., or σ_v, τ_o ...) falls within the corresponding domain R (M, N, ... σ_v, τ_o).

The choice of values to be assigned to the coefficients is closely tied to the schematizations made and to the calculation procedures; in this particular case the following criterion was adopted:

γ_f: because of its reference lifetime - in a monument this must be measured in centuries - and because of the unknown redistribution of the forces to craks diffusion, successive alterations, soil settlements, etc. ..., owing γ_f was taken as 1.7 for the permanent and live loads.

For earthquake action γ_f was instead taken as 1 since the value of the intensity to be used for the verifications was directly obtained on the basis of an avrage return period of 500 years, through a seismic risk analysis expressly made by Giuffrè A. - Pinto P.E., for the Rome area (unpublished document).

γ_m: since the characteristic strength values could not be meaningfully defined, the conventional assumption of γ_m = 1.5 was made; taken into account here were several values for the strengths (and various forms of the strength domains), which were subjectively defined on the basis of the information gathered (in situ tests laboratory tests, ultrasonic testing) and of visual inspection (cracking, deterioration). Each different kind of domain and each strength value had attached to it different "credibility" and "reliability" indices.

γ_n: this coefficient has been used here to account for the different types (brittle or ductile) and consequences of collapse.

The following values were assigned, for the facade giving onto the Forum:

γ_n: 0.8, for facade breakaway (II, b)

γ_n: 0.9, for local failure of the facade (II, a)

γ_n: 1, for shear-breakage of the cross walls (II, c)

The various structural models considered and calculation procedures were also affected by the "credibility" and "reliability" indices.

V. THE SAFETY EVALUATION

When looked at from the point of view just expressed, it turns out that the theoretical verification of safety, as traditionally understood, has in general no significance, since the same structural element will have a range of different values of S and R tied to it - some of which will satifsfy equation (1), and others will not.

This is pointed up graphically by different strength curves and different stress points: some fall within all the domains, others fall outside them all, while still others fall within some and outside the rest (fig. 4).
It follows that an evaluation of safety cannot but derive by a proper consideration of the ensamble of all stress points and domains, indices of credibility and

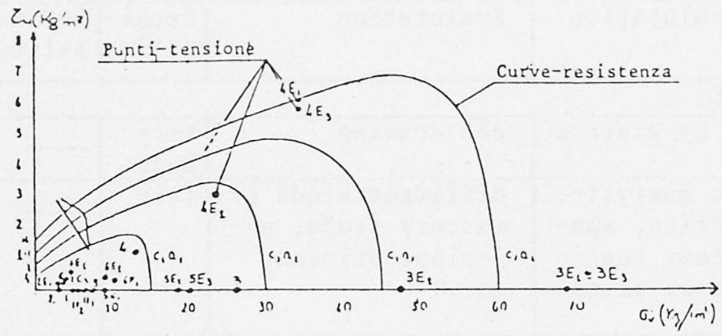


Fig. 4 - Strength domain

of reliability being tied to them. These indices themselves being defined by the linking in a subjective way of the same indices as establisched for the individual elementary operations. Only more advanced probabilistic analyses will be able to reduce or eliminate these subjective contributions - and efforts are being made in this direction.

Stress- points beyond the strength curves in general correspond to high realibility indices, but to rather low credibility indices, so that such a situation does not always mean true danger, or anyway no immediate danger.

Therefore, the following evaluations ar actually subjective, and further investigation is needed to define any intervention operations, whether provisional or definitive:

a) the local behaviours of the facade looking on the Forum display fair safety margins, so that urgent operations are not needed. But the situation is different for several columns of the Gallery, which are working very hard relative to their present deteriorated state.

b) the overall connection of the Facade to the structures behind it is very precarious, so that even a weak earthquake could cause the Facade to break away and collapse. Therefore, urgent operations along the lines of those indicated by previous checks are needed.

The installation of chains causes the credibility of most of the stress-points falling outside the breakage domains to vanish, and thus eliminates them from consideration.

c) overall behaviour, which involves the shear strength of the cross walls, is fair; though the problem of their reinforcement must be faced, no need is seen for emergency operations.

4. INTERVENTION CRITERIA

A this point in the study, and in light of the safety evaluations that have been made earlier, a number of priority or emergency intervention operations have been singled out for the Facade giving on the Forum. These concern:

- the shoring of several columns and their strengthening by cement-mortar grouting;
- the restoration of at least a minimum level of connection between the Facade and the structures behind it by consolidating strips of it to which pretensioned cross-chaincs may be anchored (fig. 5).

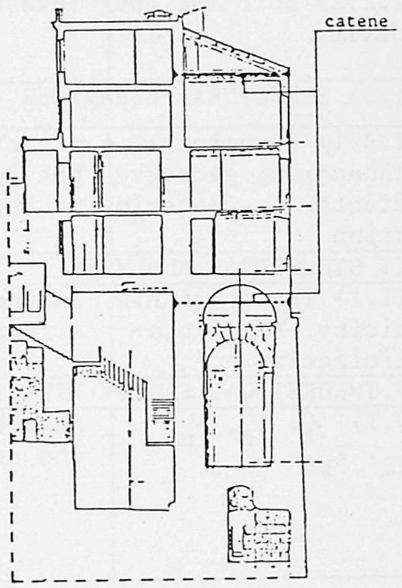


Fig. 5 - The emergency interventions.

Factors determining safety	Source of evalutation	Evalutation	Credibility	Reliability
I DESCRIPTION OF THE PRESENT STATE				
I.1) Geometrical survey	carried out by experts	see drawing	very good	———
I.2) Identification of the material	petrographic analysis, sight inspection, samples laboratory testing, in situ test ultrasonic tests	different kinds of masonry (tufa, peperino, brincks, etc.)	fair	———
I.3) Interpretation of the local failures, degradation	sight inspection, observations	crushing, spreading deterioration, out of perpedicularity, ...	fairly good	———
I.4) Identification of foundations and soil	sight inspection, samples laboratory testing	lacustrine alluvional sediments, tuff, ...	good	———
I.5) Strenghtenings and pre-exstent changes, structural connection	sight inspection, samples	inefficient strenghtnings, different tamperings	fairly good	———
I.6) Developing trend of phenomena	levelling, investigations	slight movement deteriorating	fairly good	———
II MODELLING				
II.1) Actions $\rightarrow \gamma_f$				
II.1.1) Direct action: forces	geometrical survey and Normativa	caratteristic values, γ_f	good	very good
II.1.2) Indirect action: settlements, etc.	investigations	negligible	good	average
II.1.3) Indirect action: earthquake	analysis of a seismic risk	response spectrum	good	good
II.2) MATERIALS $\rightarrow \gamma_m$				
II.2.1) Strength domain	investigations	different carattefristic values γ_n adopted	varies according the adopted values	
II.2.2) Soil and foundation	investigations	deformability only on sismic actions	good	good
II.3) STRUCTURAL BEHAVIOUR $\rightarrow \gamma_n$				
II.3.1) Structural connection, geometry, effects, ... deterioration	structures samples sight inspection, surveys interpretation, etc	different assumptions (see 3.II, fig. 3)	varies according the adopted values	
III STRUCTURAL ANALYSIS				
III.1) Theory of elasticity. Local plasticization	on the basis of the strength of materials	stresses in the structure	awerage	good
IV THEORETICAL SAFETY VERIFICATION				
IV.1) γ_f, γ_n, γ_m S_{ki}, R_{ki}	structural analysis and strength domain	$S_k \ \gamma_f \lesseqgtr \gamma_n \ R_k/\gamma_m$ (see fig. 4)	varies according the adopted values	

Appraisal of Existing Ferrous Metal Structures

Examen de constructions en fonte et en fer

Begutachtung bestehender Eisenkonstruktionen

Michael BUSSELL
Associate
Ove Arup & Partners
London, England

Michael Bussell has been with Ove Arup & Partners since graduating in civil engineering at the University of Bristol in 1966. His involvement in structural aspects of building re-use is complemented by an interest in the history of structural engineering.

SUMMARY

The investigation of structures built since the late 18th century will often demand the appraisal of loadbearing members of cast iron, wrought iron, or steel. The paper considers the various aspects of such an appraisal, and in particular emphasises the need for an understanding of the original materials and their characteristics, construction methods, and design standards.

RESUME

L'examen des ouvrages construits depuis la deuxième moitié du XVIII^{e} siècle exige souvent l'évaluation des éléments de charpente en fonte, en fer forgé, et en acier. L'article traite les divers aspects d'une telle recherche, et particulièrement de l'importance de la connaissance des matériaux originaux et de leurs caractéristiques, méthodes de construction, et modes de calcul.

ZUSAMMENFASSUNG

Die Untersuchung von Gebäuden, die seit Ende des 18. Jahrhunderts gebaut wurden, verlangt oft eine Begutachtung tragender Elemente aus Gusseisen, Schmiedeeisen oder Stahl. Die verschiedenen Aspekte einer solchen Begutachtung werden diskutiert, und insbesondere wird die Notwendigkeit betont, die ursprünglichen Baumaterialien und ihre Eigenschaften sowie die Baumethoden und Entwurfsnormen zu verstehen.

1. INTRODUCTION

There is at present a growing tendency for old buildings to be renovated or adapted for re-use. In itself this is not a novel situation, but it is significant that many of these were constructed during the last two centuries, in the period which has seen the introduction and widespread use of first cast iron, then wrought iron, and ultimately steel as structural materials. Consequently the engineer is likely nowadays to be faced more often with the appraisal of structures containing one or more of these ferrous metals.

The aim of the appraisal will be to show whether the structure can be retained for the future intended use of the building, whether it needs to be strengthened, or whether total renewal is the only practical course of action. Factors other than purely structural considerations will of course also need to be taken into account: these include for example adequacy of storey heights, and legal requirements for preserving architecturally important facades.

It is worth stressing here the contrast between design and appraisal, which is particularly relevant in regard to ferrous metal structures. In a design, the engineer can prescribe through drawings and specifications the material properties, member sizes, and construction details to ensure that his (usually simplified) assumptions of strength and behaviour are achieved in the built structure. An appraisal however is concerned with a structure that has already been built: its characteristics exist but are initially unknown and must be defined by investigation of the structure itself, coupled with an awareness of the original materials, and design and construction practice of that time.

2. GENERAL APPRAISAL PROCEDURE

The appraisal process generally involves investigation and assessment in two stages, broadly analogous to the 'scheme' (or preliminary) and 'detailed' (or final) stages in the design process.

The aim of the preliminary appraisal is to establish in principle whether it is feasible to retain the existing structure for the future. (If the answer to this is 'no' there is clearly no point in making any further study.) This stage involves:

- search for available drawings and other documentary evidence
- identification of the metal(s)
- outline structural survey to establish existing construction thickness and spans, member sizes, and major defects
- preliminary assessment
- decision in principle on feasibility of retention

The timescale for this preliminary appraisal is often very short, being imposed by the building owner or client. This can paradoxically be of benefit in helping to focus attention on generalities rather than particulars of the structure, in encouraging the minimum and simplest of calculations, and in postponing the commissioning of slow and expensive detailed surveys and testing programmes. (In this stage it is suggested that any testing of materials is done only to confirm the identification of the metal(s).)

The final appraisal, once the feasibility of retention has been established in principle, will be a more thorough exercise involving:

- detailed structural survey of all construction that is required to remain (especially connection details)
- detailed definition of renovation and re-use needs as they affect the structure
- testing of materials
- comprehensive assessment of members and connections
- decisions in detail on strengthening and other alterations to structure

Following this, the necessary work can be specified.

It should not need saying that the early involvement of the building control authority is essential in any appraisal for alterations or re-use in which it can exercise statutory powers, for it - as well as the engineer involved - must be satisfied that the existing structure has been properly investigated and realistically assessed, and it would be foolish as well as time-wasting to develop a detailed scheme which is then rejected on submission to the authority because of fundamental disagreements on approach.

The rest of this paper concentrates on specific aspects of the appraisal of ferrous metal structures, namely: identification of the metal, preliminary assessment, testing, and final assessment. The general principles and approach to be applied in any structural appraisal have been described elsewhere (e.g. [1]), and are not discussed further here.

3. IDENTIFICATION OF THE METAL

3.1 Visual Aids to Identification

Distinctive features to aid in the recognition of cast iron members are:

- pitted or 'gritty' surface (from the sand or loam mould)
- thick or coarse cast sections
- 'flowing' sections and profiles (e.g. solid and hollow circular, and X-and H-shaped sections; 'classical' column heads with integral endplates; shaft entasis)
- bottom (tension) flange larger than top flange
- beams of inverted T- or V- section
- bottom flange of beams often curved on plan or elevation
- internal corners rounded (to deal with cooling shrinkage stresses)

Connections between cast iron sections were by simple bearings or by wrought iron threaded rods and nuts fixed through pre-formed holes. Hollow circular columns were often cast in two semi-circular pieces which were then brazed together.

Wrought iron resembles steel in being formed into structural sections by passing billets through rollers. The earliest beams were built up from plate and angles rivetted together. Subsequently rolled beams became available, often being strengthened by rivetted flange plates and web stiffeners. Its tensile superiority over cast iron led to its early use as chains, cables, and links for suspension bridges, and as tie-rods in buildings. The rods were frequently employed compositely with cast iron to form trussed beams and roof trusses.

Wrought iron can be distinguished visually from cast iron by its smoother rolled surface - assuming that not much corrosion has occurred. If more corroded, wrought iron tends to delaminate into thin sheets of nearly pure iron alternatively with slag which can be pulled away from the surface.

It is more difficult to distinguish sound wrought iron from steel as their production and structural forms are so similar. Unless there is conclusive evidence from documents or dating as to which metal is present, it is best to take small samples for identification.

3.2 Dating Evidence

The chronology of iron and steel use in structures is fairly well-defined, although dates vary between one country and another. In the UK, for example, cast iron was used between the 1790s and the early 20th century (columns only after about 1860); wrought iron from 1840 (built-up beams from plates and angles) and 1860 (rolled I-beams), being obsolete by 1914; while steel was introduced structurally in the late 1870s and subsequently was the only ferrous metal used in new construction after 1914.

Thus, if the building can be dated by documentary and/or stylistic evidence, it should be possible to distinguish between wrought iron and steel, except in the 'overlap' period when both were in use. Care in relying on dates is obviously needed when a structure has been altered since construction.

3.3 Sampling for Confirmation

It is necessary to take only a small sample of metal, for chemical and metallurgical identification by a specialist testing house. A 25mm square piece core-drilled from a lightly-stressed location will be adequate for this purpose. In the case of cast iron it is important that identification includes the particular type of cast iron as these have significantly varying properties.

4. PRELIMINARY ASSESSMENT

4.1 Approach

It should be recognised that material specifications, methods of quality control, and regulations covering design and loading, have only recently been developed into the rigorous and numerically-orientated instruments that they are today. It is therefore not appropriate to appraise a 19th century structure of cast or wrought iron by calculations based on a modern steelwork code of practice, even if suitably factored to recognise a different basic stress, not least because the characteristics of these materials as manufactured then are not consistent with those of today's steel.

Nevertheless some simple calculations must be made to establish an idea of member strength and hence the feasibility of re-use. If these are to be relevant, the engineer needs an understanding of material quality and contemporary practice at the time of construction.

4.2 Material Quality

Cast and wrought iron, and early steels, were seldom produced to a nationally defined standard as is steel today. Instead, each ironworks would offer a variety of grades suited more or less to the needs of its market. Extensive testing was carried out on these, and the results were published in commercial literature and textbooks. Quoted strengths were usually at ultimate (breaking) load.

A study of test results for any grade generally reveals considerable variability in strength, which was accommodated by correspondingly large factors of safety (between 3 and 10) for working use.

4.3 Original Design Practice

Before building legislation laid down allowable stresses in structural metals, design was based largely on experience and elementary structural theory. These are frequently to be found in the contemporary textbooks, which in many cases can be regarded as the equivalent of modern codes in recording good practice, as well as providing an invaluable reference source on construction details.

Where building legislation had laid down a design approach and quantified allowable stresses, and was in force at the time of construction, it is reasonable to assume that the structure would have been designed to comply with this, which may be used as a present-day standard for appraisal. (Some building control authorities will indeed require such an approach.) The quoted stresses in these are generally conservative, and may also be used for preliminary assessment of structures pre-dating such legislation.

4.4 Loadings

Nineteenth century textbooks show a wide variation in the allowance made for imposed loads: they were however generally higher than would be considered necessary today.

It is clear, however, that many builders did not adopt such onerous figures for domestic and commercial timber floors (which were sized by experience and/or rule of thumb), and this is probably true also of many building structures with metal beams and columns.

The fact that such structures were clearly always incapable of supporting the over-generous design loads quoted in the textbooks and yet today exhibit no signs of overloading, has led building control authorities to be increasingly reluctant to accept unquestioningly schemes for which current live loading requirements would appear to be less than the original 'assumed' loading. It will therefore usually be necessary to establish member sizes and show by calculation that the existing structure is adequate.

Wind loading on early building structures - rather than bridges - was not often considered.

5. TESTING

5.1 The Need for Testing

Once the preliminary assessment has shown re-use to be feasible, it will generally be necessary to obtain more comprehensive information on the existing structure before making the final, detailed, assessment. In particular, materials testing may be considered.

There is little point in making tests if an initial appraisal has shown that the structure is in an unsound state already, or that it is grossly overstressed in its new use. Conversely, a 'young', well-documented steel structure - and often older structures too - may need little or no testing if they are in sound condition and will be stressed only to modest levels in the future.

It is important that the building control authority requirements for testing are identified.

Some authorities are very dubious about the usefulness of testing as an indication of typical strengths in the actual structure: this may be understood by considering that manufacturing quality control in the 19th century was very much cruder than it is today, as was recognised by the generous factors of safety applied. There is thus no guarantee that sampling for testing, or even in situ load testing to failure (e.g. of elements typical of the building but unwanted in the proposed scheme), will give results that can be confidently regarded as 'average', still less as 'lower-bound', for the elements as a whole.

5.2 Criteria for Sampling

It is generally assumed that the strength of a group of similar ferrous metal elements will vary in accordance with a normal distribution: an approximation quite adequate for most circumstances. It is then possible to use statistics to give an estimated strength.

Usually this is a 95% confidence limit based on test results, i.e. a figure below which no more than 5% of the actual strengths should fall. To find this, it is necessary to calculate the mean value and the standard deviation of the test results. The 95% confidence limit will then be a number of standard deviations below the mean, that number being a function of the number of samples taken. Hence, if only two samples have been taken it will be necessary to use a value 6.3 standard deviations below the mean, whereas if six samples are taken this is reduced to 2 standard deviations. For an infinite number of samples, the figure is 1.65, so there is little to be gained by taking more than six samples.

Beams and columns may not necessarily be from the same source of supply or even of the same material. This should be established in the initial identification exercise, the number and location of samples being extended as appropriate.

5.3 Choice of Tests

The most useful information can be gained from a standard tensile test, i.e. yield stress, ultimate strength, Young's modulus, and elongation to fracture. This obviously involves destructive laboratory testing of samples cut from the structure. Ideally, samples 200 x 100mm should be cut, which will then be machined to the required shape by the testing laboratory. Samples may be removed using a hacksaw, interlocked drilled holes, or flamecutting (which should only be used with an additional 1-15mm allowance around each cut face to enable removal of the heat-affected zone). In taking the sample a prime requirement must be not to weaken the structure dangerously.

Where it is intended to weld to existing steelwork, a chemical analysis for weldability should be made. This can be done on part of a broken tensile specimen. It is in general not advisable to consider welding existing cast or wrought iron.

5.4 Interpretation

If a sufficient number of test results are obtained, the 95% confidence limit can be calculated as previously described. This should then be divided by a suitable factor of safety. For cast iron a factor of 3 is suggested in view of its brittle nature; similarly, in view of the greater variability of wrought iron, compared with steel, a factor of 3 on the 95% confidence limit of tensile strength seems appropriate.
Recent or better-quality older steel should exhibit a narrower variation in strength: the tests should also confirm whether the steel is of mild or high-tensile quality. The permissible stress adopted for appraisal should, it is suggested, be less than the minimum value of the elastic limit, and be not more than either 0.67 x 95% confidence limit of the yield stress or 0.375 the ultimate strength.

6. FINAL ASSESSMENT

The final assessment of the structure can follow one of two courses.

The first is to appraise the members and their connections using design rules and allowable stresses that were in force at the time of construction. This may be particularly appropriate for earlier structures where the in situ material strengths vary widely, and of course could also be applied in the appraisal of a more modern steel structure. The advantages of such an approach include speed and simplicity of calculation, and the high probability of acceptance by building control authorities, especially when the design rules used were prescribed by them or their predecessors.

The disadvantage of this method is that it may not be possible to justify the adequacy of all parts of the structure using the inevitably simple and conservative assumptions built into such rules. In this case it will be necessary to apply an assessment using theoretical first principles; this will inevitably be more time-consuming, and does not necessarily guarantee success in justifying the structure, but does give much more scope for the engineer to take account of the real conditions in which the particular structure will be serving, and to exercise his judgement in relating these to the analytical model he uses. This approach is particularly useful in assessing the strength of columns with varying degrees of end restraint and imperfections in line and straightness.

REFERENCES

1. INSTITUTION OF STRUCTURAL ENGINEERS, Appraisal of Existing Structures, 1980.

Strengthening of the «Rotonda» Monument in Salonica

Renforcement du monument «Rotonda», Salonique

Verstärkung des Monumentes «Rotonda» in Saloniki

George PENELIS
Professor
Univ. of Salonica
Salonica, Greece

George Penelis has been the director of the Lab. for Concrete Structures in the Univ. of Salonica since 1971. He has been engaged together with his collaborators who are co-authors of this paper with the strengthening of some of the most important Roman and Byzantine monuments of Salonica.

M. KARAVEZIROGLOU
Lecturer
Univ. of Salonica
Salonica, Greece

K. STYLIANIDIS
Scient. Assistant
Univ. of Salonica
Salonica, Greece

Dim. LEONTARIDIS
Civil Engineer
Min. of Public Works
Salonica, Greece

SUMMARY
An effort is made to outline briefly the in situ measurements and laboratory tests that were necessary together with the analytical work for the strengthening procedures of the Monument «Rotonda» in Salonica that was affected by earthquakes in 1978.

RESUME
Le rapport décrit la recherche expérimentale, sur place et au laboratoire, ainsi que les calculs analytiques nécessaires au choix des méthodes appropriées pour le renforcement du monument «Rotonda» à Salonique. Ce monument a été endommagé lors de tremblements de terre en 1978.

ZUSAMMENFASSUNG
In dieser Veröffentlichung sind die in-situ Messungen und die Laborversuche kurz präsentiert, die zusammen mit der analytischen Berechnung für die Verstärkung des durch das Erdbeben von 1978 beschädigten Monuments «Rotonda» in Saloniki notwendig waren.

1. INTRODUCTORY REMARKS

Rotonda of Salonica is one of the older and more splendid monuments of the City (Fig.1,Fig.2). Its core had been built as a part of the Roman Imperial palace layout about 300 A.D. Covered by a huge dome 24.5m in diameter it is an imposing circular building. Eight barrel-vaulted niches lighten the thickness of its 6.25m walls. About 400 A.D. Rotonda was turned into a church. A vaulted chancel and apse were added. So the middle and a part of the upper ring of the cylindrical wall was also cut for connection of the chancel with the main core, an intervention which has caused many structural problems to the monument.

During the course of the monuments' life, the christian intervention, the successive earthquakes and the time have caused extended damage to it as no serious attempt for strengthening had been made for many centuries. The earthquake of June 20, 1978 made the whole situation worse. The main crack due to shear at the

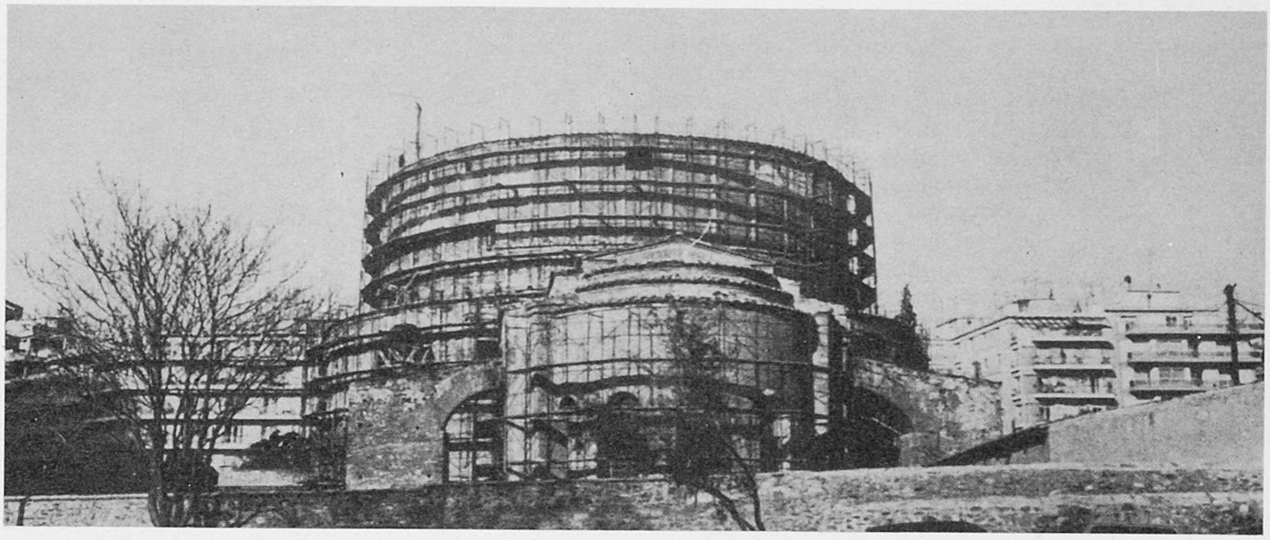


Fig.1 General view of Rotonda

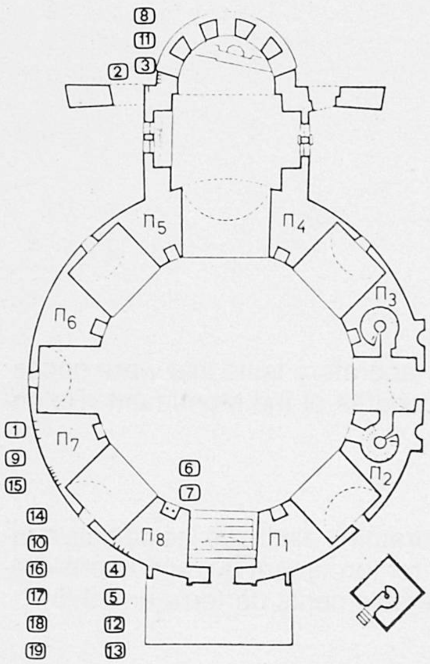


Fig.2 Locations of core-taking on the Monument

southern piers Π_2,Π_3 appeared a widening of 1mm per month indicating that the part of the dome bound by these piers had began slowly to slide.

The Working Group which was assigned the supervision of the restoration and strengthening of Rotonda after the earthquake of June 20,1978 was at first concerned with the immediate remedial measures that were necessary to prevent the collapse of the monument. At the same time with the works of the temporary shoring the study for the final repair and strengthening was started.

In order justified decisions to be made about an intervention in the monument, all the available means of research were used.This research covered three areas:

- In situ investigations
- Laboratory tests
- Analytical research with the aid of computers

In the present paper the in situ investigations and the lab.tests used as basis for the repair and strengthening of the Monument are briefly outlined.

2. IN SITU INVESTIGATIONS

The in situ investigations covered the following areas:

2.1 Geometrical and constructional surveying of the Monument

This included:
- The geometrical surveying of the superstructure carried out with standard as well as photogrammetric methods
- The geometrical and constructional surveying of the foundation carried out through four ground cuttings at the immediate neighborhood of the foundation
- The geometrical and constructional surveying of the cover and the dome carried out through ten cuttings at the cover and
- The constructional surveying of the piers carried out through inspections on their external surface and on the ruins of buildings built at the same period with the same external type of construction.

2.2 Surveying of the damage

The surveying of damages includes, first, a detailed surveying of the cracks, of their width, of the relative displacement of their edges both in the direction of the crack and vertically to it, as well as of their depth. It also includes a photographic surveying of the main cracks. The surveying of the cracking was done by means of geometric methods. The depth was determined using special rods that were forced into the cracks.

Together with the surveying of damage we should include the continuous recording of the variation in the crack widths by means of 11 velometers installed at the main cracks of the Monument. The recording of these movements serves as an additional indicator of the Monument's trend to collapse. It also serves as a basic indicator for the effectiveness of the final strengthening procedure under consideration. Actually, the cease of the crack propagation and the reversing of the relative movement in some of them, after the partial (measured with velometers) prestressing of the rings of the temporary shoring of the Monument, convinced that this type of strengthening has to be a positive one for the final restoration of the Monument (Fig.3). Finally, in the procedure of surveying the damage the measurement of the deviation of the piers from the vertical position should be included (Fig.4).

2.3 Core-taking and non destructive tests

In order to determine in the laboratory the mechanical constants of the Monument, an amount of 19 cores of 10cm diameter was taken from various locations on the Monument (Roman and Christian) (Fig.2). It is beyond any doubt that this limited number of specimens can not give credible values of the mechanical constants for the whole structure. On the other hand, an unlimited number of core could not be taken because of the risk to cause extended damage to the Monument due to core-taking. Therefore the cores were combined to N.D.T. i.e. wide range hammer tests (600) as well as ultrasonic measurements (300) which were extended on the whole surface of the Monument. The correlation of hammertesting to the corresponding compressive strengths found in the laboratory, through D.T. was found to be very satisfactory (correlation rate equal to 0.867). The correlation of ultrasonic measurements to the corresponding compressive strengths determined through D.T. at the locations of core-taking did not provide satisfactory results (correlation rate $r < 0.500$). Anyhow ultrasonic measurements have given very good correlation results, concerning the dynamic modulus of elasticity, which is a quantity indispensable to the analysis as well as to the choise of the type of the grout for the repair.

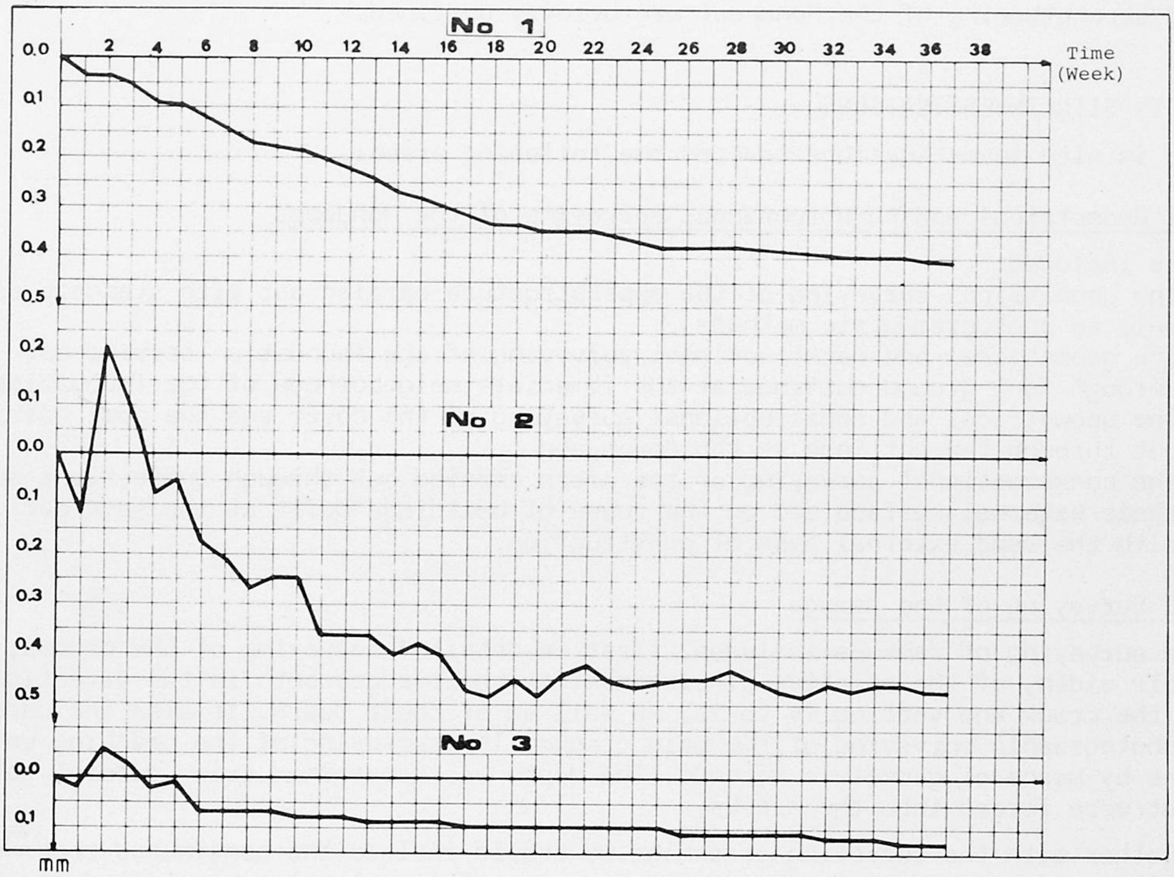


Fig.3 Variation of crack width with time

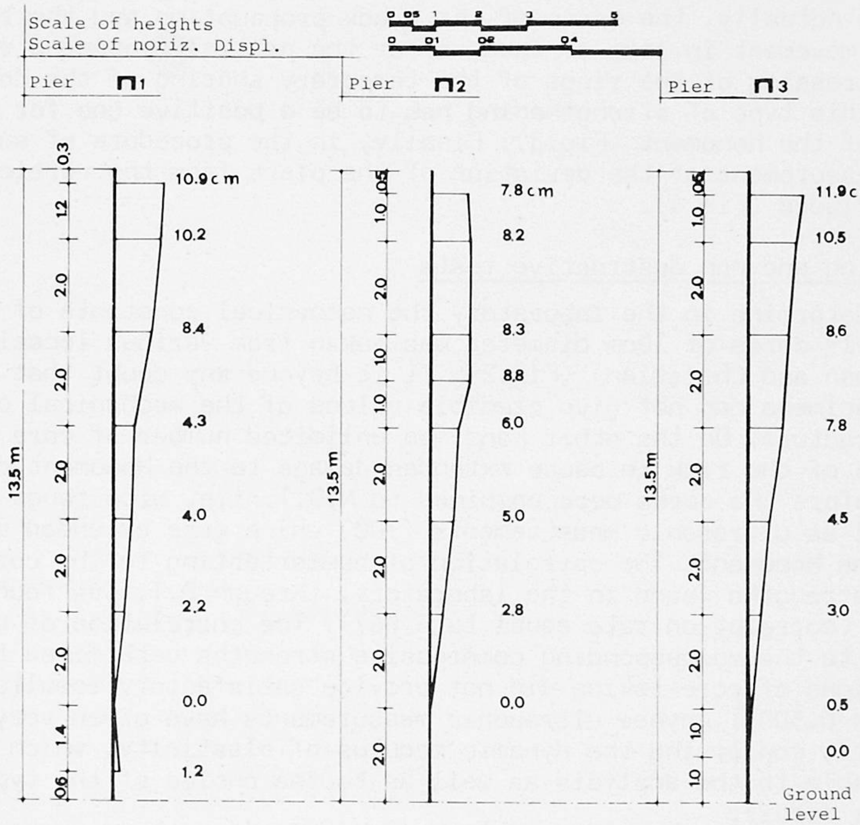


Fig.4 Horizontal displacements of the piers

The materials of the cores were also used in the chemical analyses for the determination of the composition of the structural materials of the Monument.

2.4 Natural period and degree of damping of the Monument's dynamic response

For the determination of the natural period of the Monument ambient vibration measurements were carried out in cooperation with the Laboratory of Engin.Seismology and Aseismic Structures, Nat. Tech. Univ. of Athens. Forced vibration measurements are also under way. The conclusions of these measurements are in a very good agreement to the analytical estimation of the natural period determined through the Rayleigh Method. So while the foundamental period according to ambient vibration measurements was found to be T_1=0.42sec., the analytical approach gave for T_1 prices between 0.53 and 0.33sec.

2.5 Soil borings

For the evaluation of the influence of the soil on the existing damage, four borings were carried out at a depth of approximately 25.0m. The results of the soil exploration were also used for the estimate of the soil conditions influence on the seismic loads (base shear) to be used in the strengthening procedure.

3. LABORATORY TESTS

The laboratory tests covered the following areas:

3.1 Chemical analyses

The chemical analyses had the following tasks:
- The determination of the proportions between the paste and the aggregates in the construction mortar.
- The determination of hydraulic elements into the mortar. So, for the Roman part it was concluded that the existing hydraulic materials were due to the brick powder while for the Christian part they were due to addition of pozzolanic ash together with the brick powder (Table 1).

Composition	First analysis with coarse ceramic material			Second Analysis without coarse ceramic mater		
	Determined values			Determined values		
	(gr)	%	%	(gr)	%	%
Burning loss	2.672	12.78		4.3045	21.52	
Insoluble compounds	13.852	66.30		8.989	44.94	
Soluble compounds	(4.372)	(20.92)	Composition of compounds	(6.71)	(33.54)	Compo - sition of compounds
SiO_2	0.540	2.58	12.35	0.640	3.20	9.53
R_2O_3	0.673	3.22	15.39	0.775	3.87	11.55
CaO	2.988	14.31	68.36	5.10	25.48	76.00
MgO	0.107	0.51	2.44	0.1175	0.59	1.75
SO_3	0.029	0.14	0.67	0.029	0.14	0.44
Na_2O	-	-	-	0.0060	0.03	0.09
K_2O	-	-	-	0.0400	0.20	0.59
Not determined	0.035	0.16	0.79	0.0025	0.03	0.05
Total	20.896	100.00	100.00	20.003	100.00	100.00

Table 1 Chemical components of the Roman mortar (mortar specimens, from core 10)

- The determination of the composition of the structural bricks and more specifically whether they were of the same composition with the mortar's brick powder and whether they have incorporated hydraulic materials. So, for the bricks of the Roman part it came out that they have hydraulic components, while for the bricks of the Christian part it came out that the hydraulic components are transformed to ceramic compounds because they were burned in high temperatures On the other hand, it was concluded that the mortars both in Roman and Christian part contain brick powder of the corresponding parts, both hydraulic active, a conclusion that was cross checked with the archaeological evidence.
- The chemical analysis of the materials that could be possibly used for the composition of grouts, i.e. hydrated lime, pozzolanic ash,cement and brick powder.

3.2 Mechanical properties of the structural materials of the Monument

Using the cores taken from the two parts of the Monument, the Roman and the Christian, mortar specimens were prepared in the Laboratory of Reinforced Concrete, Univ. of Thessaloniki, and their flexural strength was determined as well as the compressive strength, the modulus of elasticity of the mortar and the bond strength between the bricks and the mortar.

Brick specimens were also prepared and the bricks compressive strength as well as modulus of elasticity were determined. As we have already mentioned, the number of specimens was limited and was used for the calibration of the N.D.T. instruments by means of which the in situ investigation was extended all over the Monument (Table 2). As it is clear, from the above laboratory tests, completely reliable data were provided, concerning the structural materials-especially the mortar-which are necessary for the preparation of the grouts.

Constr. Phase	Location of Tests	Number of Tests	Mean value μ (kg/cm^2)	Standard deviation σ (kg/cm^2)	Lower 5% fractile μ-1,645σ (kg/cm^2)	Upper 5% fractile μ+1,645σ (kg/cm^2)	Minimum value (kg/cm^2)	Maximum value (kg/cm^2)
Roman	Π_1	54	28,33	4.61	20,75	35,91	17,80	39,40
	Π_2	54	29,96	6,91	18,59	41,33	19,19	44,94
	Π_3	54	26,04	5,54	16,92	35,16	15,16	42,93
	Π_4	54	32,36	5,89	22,67	42,05	15,16	42,93
	Π_5	54	29,67	6,64	18,75	40,59	15,16	49,08
	Π_6	54	28,23	6,35	17,78	38,68	17,13	39,99
	Π_7	54	28,13	6,44	17,54	38,72	13,90	39,04
	Π_8	54	27,34	7,57	14,89	39,79	11,51	48,03
	Top	144	32,44	6,34	22,01	42,87	15,16	49,08
	Midhight	144	28,27	5,04	19,98	36,56	17,80	39,04
	Bottom	144	25,56	6,13	15,48	35,64	11,51	42,93
	Inside	216	27,36	5,25	18,72	36,00	15,16	42,93
	Outside	216	30,16	7,29	18,16	42,16	11,51	49,08
	Total	432	28,76	6,50	18,07	39,45	11,51	49,08
Christian	Top	54	36,96	11,60	17,87	56,05	15,81	60,16
	Midhight	54	37,39	11,15	19,05	55,73	19,19	59,00
	Bottom	60	39,88	14,32	16,33	63,43	13,90	69,77
	Inside	72	33,91	11,25	15,40	52,42	13,90	59,00
	Outside	96	41,32	12,51	20,74	61,90	15,81	69,77
	Total	168	38,14	12,50	17,57	58,71	13,90	69,77

Table 2 Statistical evaluation of the values compressive strength of the mortar

These data also used for the indirect determination of the compressive strength and of the modulus of elasticity of the compound masonry,using semiempirical relations established by current Codes together with destructive measurements on masonry cores.

3.3 Mix design of the grouts and testing of their properties

Based on the chemical analyses and the tests for the determination on the mechanical properties of the Monument's mortars, the composition of grouts was attempted in the Laboratory of Reinforced Concrete, Univ. of Thessaloniki, using traditional materials, compatible to the mechanical properties of the construction mortar of the Monument. Meanwhile, the chemical analyses and the testing of the machanical properties for a whole series of Byzantine Monuments carried out in the same Laboratory, led to the thought that this research had to be carried out in a uniform way for all the Monuments, in order to avoid unnecessary repetitions. So, a whole series of mixes was tested and we chose those that fulfilled the current specifications for grouting injections, as far as the fluidity, the bleeding, the shrinkage and the sedimentation are concerned (Table 3); from these mixes the choise of the appropriate grout for every Monument was made, according to the strength of the mortar and to its modulus of elasticity.

Grouting		Mixing Proporsions (parts by weight)								Sinking
Category	Label	Pozzolanic Ash	Lime	Cement	Ceramic powder 900°	Ceramic powder 650°	Sand	Water	Tricosal	time (sec.)
I	E_1	10	5	-	-	-	-	15,50	-	33
	E_1	10	5	-	-	-	-	12,00	0,15	30
	E_2	10	8	-	-	-	-	12,50	0,18	32
	EΠ2	6	4	-	5	-	-	13,50	0,15	28
	$EΠ_4$	6	4	-	-	5	-	14,50	0,15	29
	K_{1-1}	10	5	-	-	-	15	17,50	0,15	28
	K_{1-2}	10	5	-	-	-	30	18,75	0,15	26
	K_{1-3}	10	5	-	-	-	45	20,50	0,15	27
II	ET_1	10	5	1	-	-	-	12,50	0,16	32
	ET_2	10	5	2	-	-	-	12,50	0,17	34
	ET_3	10	5	3	-	-	-	12,75	0,18	34
	ET_4	10	5	4	-	-	-	14,00	0,19	28
	ET_5	10	5	5	-	-	-	14,50	0,20	30
	ET_6	10	5	6	-	-	-	14,00	0,21	32
	ET_7	10	5	7	-	-	-	15,00	0,22	
	ET_8	10	5	8	-	-	-	15,50	0,23	33
	ET_9	10	5	9	-	-	-	16,00	0,24	31
	ET_{10}	10	5	10	-	-	-	16,50	0,25	
III	KT_{1-1}	10	5	2	-	-	17	15,75	0,17	32
	KT_{1-2}	10	5	2	-	-	34	20,00	0,17	29
	KT_{1-3}	10	5	2	-	-	51	22,50	0,17	30
	KT_{2-1}	10	5	4	-	-	19	18,00	0,19	23
	KT_{2-2}	10	5	4	-	-	38	21,00	0,19	
	KT_{2-3}	10	5	4	-	-	57	25,00	0,19	31
	KT_{3-1}	10	5	6	-	-	21	18,50	0,21	
	KT_{3-2}	10	5	6	-	-	42	23,00	0,21	30
	KT_{3-3}	10	5	6	-	-	63	25,50	0,21	32
	KT_{4-1}	10	5	8	-	-	23	19,50	0,23	33
	KT_{4-2}	10	5	8	-	-	46	23,00	0,23	30
	KT_{4-3}	10	5	8	-	-	69	29,50	0,23	28
	KT_{5-1}	10	5	10	-	-	25	19,00	0,25	31
	KT_{5-2}	10	5	10	-	-	50	23,50	0,25	30

Table 3 Mixing proportion of the grouts

The materials used are the following: Hydrated lime, pozzolanic ash, brick powder, cement and fine sand.

For each mix the water content was such that the time of sinking in the corresponding apparatus of the German specifications varied between 30″ and 32″,which ensures the entering of the injected mortar into the cracks.

In addition, the sample was tested to find whether it could be pumped in a hand-

-operated pump used for grouts.

For each mix, specimens were prepared according to current specifications and the volume reduction during coagulation was determined, as well as the shrinkage, the flexural strength, the compressive strength, the bond between the bricks and the grout and the modulus of elasticity. The strength tests for each composition covered a period up to nine months.

3.4 Soil tests

Based on the soil samples taken from the borings and the in situ collected data (soil profile, standard penetration tests, water table) lab. tests were carried out for the determination of the soil properties.

4. CONCLUSIONS

It is obvious from the above presentation that for a well justified repair and strengthening of a monument of world wide importance a very comprehensive investigation is necessary. The in situ investigations and the laboratory tests consist a very important part of the whole task necessary for the following steps in the intervention procedure.

5. ACKNOWLEDGEMENT

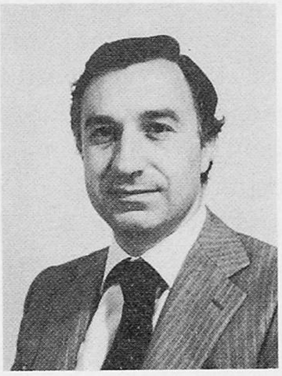
This paper is a brief presentation of a part of the research carried out for the strengthening of ROTONDA. The whole project was sponsored by the Ministry of Public Works whose support is gratefully acknowledged.

Dynamic Tests for Diagnosis of Damage Caused by an Earthquake

Essais dynamiques pour la détermination de dommages causés par un séisme

Dynamische Versuche für die Bestimmung von Schäden durch Erdbebenbeanspruchung

Aldo CASTOLDI
Co-Director
ISMES
Bergamo, Italy



Aldo Castoldi, born 1939, got the electronical engineering degree at the Polytechnic Institute of Milan. He joined ISMES in 1964, head of Department of Dynamics from 1971 to 1978, Co-director since 1978. Secretary of Italian subcommittee for seismic actions on dams of ICOLD, Chairman of the Technical Committee SLB-61 of RILEM.

SUMMARY

The results of dynamic tests carried out on two reinforced concrete piers of a bridge under construction in the area struck by the Irpinia earthquake of November 23, 1980, are illustrated. The experimental determination of the dynamic behaviour of the piers, carried out by excitation through mechanical vibrators, enabled the elastic and dissipative characteristics of the soil to be examined, as well as the soil-foundation interaction effects, and the type and degree of damage of the piers to be evaluated.

RESUME

Les résultats des essais dynamiques conduits sur deux piles en béton armé d'un pont en construction dans la zone frappée par le tremblement de terre d'Irpinia, le 23 Novembre 1980, sont présentés. La détermination expérimentale du comportement dynamique des piles, effectuée par excitation avec des vibrateurs mécaniques, a permis d'examiner soit les caractéristiques élastiques et de dissipation du sol, soit les effets d'interaction sol-fondations et d'évaluer le type et le degré de dommage des piles.

ZUSAMMENFASSUNG

Es werden die Ergebnisse aus dynamischen Versuchen an zwei sich im Bau befindlichen bewehrten Brückenpfeilern aus der Gegend, die am 23. November 1980 vom Irpinia- Erdbeben heimgesucht wurden, vorgestellt. Die experimentelle Untersuchung des dynamischen Verhaltens der Pfeiler, deren Erregung mittels mechanischer Vibratoren erfolgte, erlaubte auch die Ermittlung der elastischen und dissipativen Charakteristiken des Bodens. Ferner konnten auch Boden-Fundament-Interaktionseffekte sowie Typ und Grad des Schadens an den Pfeilern bestimmt werden.

1. INTRODUCTION

After the Irpinia earthquake of November 23, 1980, an inspection to the 12 piers of a highway bridge under construction on the River Calore, showed the presence of some fine horizontal cracks all around the piers, at a height ranging from 3.5 to 4 meters approx. over the foundation block. Following the discovery of these cracks, the problem arose of assessing their importance for the structural safety of the piers, and consequently of estimating their extent and penetration into the cross-section.

For the diagnosis of the damage and for the determination of the overall structural behaviour, an experimental research has been carried out with the aim of identifying the characteristics of the system through the evaluation of the foundation soil parameters and the response of the piers to the external loads. The procedure chosen has been the performing of dynamic tests, by applying at the top of two piers of different height (pier A: 17 m; pier B: 36 m) and foundation conditions (pier A: on piles; pier B: directly on soil) variable frequency sinusoidal forces. The general scheme of the pier is shown in Fig. 1; owing to the different dynamic characteristics of the piers under test two different mechanical vibration generators were used.

The response of the structures has been measured by means of 10 accelerometers (on the shaft of the piers), 6 seismometers (on the foundation block) and 8 strain and displacement meters (across the cracks): by means of these instruments the overall behaviour as well as the local one in points of special interest has been read out.

As output of the tests the time histories of the structure responses at the various different points have been obtained, as well as the transfer functions between these responses and the exciting force.

2. CRITERIA FOR INTERPRETATION OF TEST RESULTS

In order to obtain direct information on the behaviour of the various parts of the structural system, a reference scheme has been set up as shown in Fig. 2, in which the elements taken into account are: the top platform, the shaft, the foundation block and the soil. The response of the system depends on the following quantities:

- masses m_p and m_z, and inertia moments I_p and I_z of the top platform and the foundation block (rigid bodies)
- elastic (A), dissipative (B) and inertial (G) characteristics of the shaft, considered as a flexible element
- elastic (k_u, k_φ) and dissipative (b_u, b_φ) parameters of soil, with respect to displacement u and rocking φ of the foundation block.

The equations of motions of the structures under test can be expressed in terms of absolute displacements, relevant to the system as a whole; in this case the structural response is given - in the frequency domain - by the relationship:

$$Y_a(z,f) = H_c(z,f) \cdot C(f) \tag{1}$$

where $H_c(z, f)$ is the transfer function of the system, that is the ratio of the absolute response $Y_a(z, f)$ and the applied load $C(f)$. $H_c(z, f)$, in turn, depends on the modal characteristics of the system, its expression being:

$$H_c(z,f) = \sum_1^{\infty} {}_j \frac{r_c^{(j)} \cdot \psi_j(z)}{\left[1-\left(\frac{f}{f_j}\right)^2 + 2i\nu_j \frac{f}{f_j}\right]} \tag{2}$$

where: $f_j, \nu_j, \psi_j(z)$ and $r_c^{(j)}$ are respectively the frequency, damping coefficient, modal shape and participation factor of the j-th vibration mode of the pier.

A second way for the analysis of the system, more significant for the aims of the present study, is that of describing the motion (again in the frequency domain) by the following set of equations:

$$Y_r(z,f) = \bar{H}_c(z,f) \cdot C(f) - \bar{H}_u(z,f) \cdot \ddot{U}_o(f) - \bar{H}(z,f) \cdot \ddot{\phi}_o(f) \tag{3}$$

$$\begin{aligned} F_s(f) &= k_u \cdot U_s(f) + b_u \cdot \dot{U}_s(f) \\ M_s(f) &= k_\varphi \cdot \phi_s(f) + b_\varphi \cdot \dot{\phi}_s(f) \end{aligned} \tag{4}$$

In equation (3) the transfer functions $\bar{H}$ contain the modal parameters of the shaft with the top platform, however considered as "clamped" at its base. Equations (4) are relevant to the soil, and express the relationships between the external actions (horizontal forces F_s and overturning moments M_s) and the horizontal motion (U_s and $\dot{U}_s$) and rocking (ϕ_s and $\dot{\phi}_s$) of the foundation.

3. ANALYSIS OF THE RESULTS OF THE TESTS

3.1. Overall behaviour

The response of the two piers tested, as shown by the measures obtained from the accelerometers and the strain meters, is markedly non-linear in both directions of excitation. As an example, Fig. 3 a illustrates the response curves for pier A, whereas Fig. 3 b shows the time history of the top acceleration recorded on pier B, for tests with increasing and decreasing frequency. Both patterns are typical of a "softening" system.

In order to find out the causes of such non-linearity, the further processing of the data has been carried out with the aim of separating the behaviour of the pier from that of the foundation soil.

3.2. Elastic and dissipative characteristics of soil

As to the soil behaviour, its elastic and dissipative characteristics have been obtained from equations (4). Their expressions are the following:

$$k_u = RP\left[F_s(f)/U_s(f)\right] \qquad k_\varphi = RP\left[M_s(f)/\phi_s(f)\right]$$
$$b_u = RP\left[F_s(f)/\dot{U}_s(f)\right] \qquad b_\varphi = RP\left[M_s(f)/\dot{\phi}_s(f)\right] \tag{5}$$

that is the real parts (RP) of the ratios within the brackets.

The quantities U_s, ϕ_s, $\dot{U}_s$ and $\dot{\phi}_s$ are known since they have been measured experimentally by the seismometers placed on the foundation block; in turn, the values of F_s and M_s have been processed on the basis of the response accelerations, the test characteristics and the inertia and geometry of the piers.

The values of k and b obtained from the experimental results are plotted in Fig. 4: they show that the soil behaviour - at least within the limits, rather broad however, reached by the external loads during the tests - is practically linear. It follows that soil cannot be responsible for the highly non-linear behaviour of the systems.

3.3. Soil-structure interaction effects

The evaluation of the soil-structure interaction effects has been obtained directly from the experimental data, by a comparison between the response curve $\overline{H}_c(z, f)$ and the response $H_c(z, f)$ actually recorded. $H_c(z, f)$ is obtained straightforwardly from equation (1):

$$H_c(z, f) = Y_a(z, f)/C(f) \tag{6}$$

whereas $\overline{H}_c(z, f)$ is worked out from equation (3):

$$\overline{H}_c(z, f) = \frac{Y_r(z,f)}{\left[C_f - \frac{\overline{H}_u}{\overline{H}_c}\cdot\ddot{U}_o - \frac{\overline{H}_\varphi}{\overline{H}_c}\cdot\ddot{\phi}_o\right]} \simeq \frac{Y_a(z,f) - U_o(f) - z\cdot\phi_o(f)}{\left[C_f - \frac{r_u^{(1)}}{r_c^{(1)}}\cdot\ddot{U}_o - \frac{r_\varphi^{(1)}}{r_c^{(1)}}\cdot\ddot{\phi}_o\right]} \tag{7}$$

$r_j^{(1)}$ being the participation coefficients for the first mode, relevant to the external force (r_c) and to the foundation displacement (r_u) and rocking (r_φ). The approximation introduced in eq. (7) can be easily accepted since in case of a structure such as a cantilever, the first vibration mode contributes almost totally to the global response in a frequency range close to the first resonance.

An example of the results of the analysis described above is shown in Fig. 5. The effects of soil interaction on the pier frequency is remarkable, especially for the pier founded on piles: for pier A the natural frequency of the actual pier is 2.48 Hz, whereas for "clamped" conditions rises to 2.82 Hz; for pier B is approx. 1.35 Hz and 1.46 Hz respectively. The shape of the curve (Fig. 5 a) shows that the pattern of the response of the system at the frequency of the "clamped" pier is close to that of a linear system. The very low value of the exciting forces at that frequency (Fig. 5 b), much lower than the force acting at the actual resonance frequency, explains such linear behaviour.

The knowledge of the natural frequencies of the "clamped" piers has also allowed an average value of the Young moduli for concrete to be computed: these resulted in $E_A \simeq 23000$ MPa for pier A and $E_B \simeq 28000$ MPa for pier B.

3.4. Piers behaviour

Since soil characteristics and effects have been found not responsible for the highly non-linear behaviour of the piers, attention has been then devoted to the response of the shaft, with particular reference to the strain and displacement meters installed across the cracks. The plots of the readings of such instruments versus time (see Fig. 6) generally show highly distorted patterns. Actually, when the acting moment makes the cracks open, the measured strains reach large values, whereas, when the moment reverses and the cracks close, the diagram suddenly flattens, indicating contact between the two edges of the crack.

The measured values of the width of the cracks shows that they cannot be attributed to local effects or surface shrinkage, so that they go deep into the cross-section of the shaft. A more significant representation of these results is given in Fig. 7, in which the crack opening is plotted versus the overturning moment: again the non-linear behaviour is clearly shown. In other cases, the diagram is almost linear (Fig. 8): in these cases however, the strains are always larger than those corresponding to a solid cross-section. This means that the reinforcement is mobilized to resist external forces even in compression, thus indicating that reinforcement has yielded, at least partially, or underwent some lack of adherence.

Of interest is also the diagram of Fig. 9, which illustrates a similar correlation, referring however to a strain meter placed in correspondence of the principal axis of the cross section perpendicular to the excitation directions: irrespective of the sense of the acting moment, the crack always opens, thus showing that the position of the neutral axis changes.

Further confirmation of the importance of the cracks is supplied by the plots of the maximum strains versus the overturning moment (Fig. 10): they again show that the strains are larger than those relevant to a solid cross-section (line b), although they are smaller than those corresponding to the mobilization of the reinforcement alone (line a).

4. CONCLUSIONS

The results of the experimental study illustrated above show that by a proper project of the tests, from the point of view both of the theoretical background for the interpretation of the results and data processing, and of the operating techniques, a number of information could be obtained which allowed the various aspects of the problem to be enlightened.

Soil-foundation effects have been determined, and have been found more important for the pier founded on piles; from the determination of the elastic and dissipative parameters of soil it has been concluded that its behaviour is linear. Consequently, the remarkable non linearity of the systems, which is not consistent with that of a monolithic and intact structure, has been attributed to the shaft.

The analysis of the shaft behaviour has confirmed such conclusion, with the determination of the importance and extent of the cracks.

A final remark is relevant to the intensity of the forces acting on the structures during the tests: the strains due to these forces at the base of the shaft were equal or greater to the dead load strains, and the dynamic loads on the soil ranged between 25% and 50% of the static ones.

Tests acceleration reached from 12% to 20% of the seismic accelerations produced by the EW component of the Irpinia earthquake recorded at Bagnoli, a few kilometers from the site of the bridge.

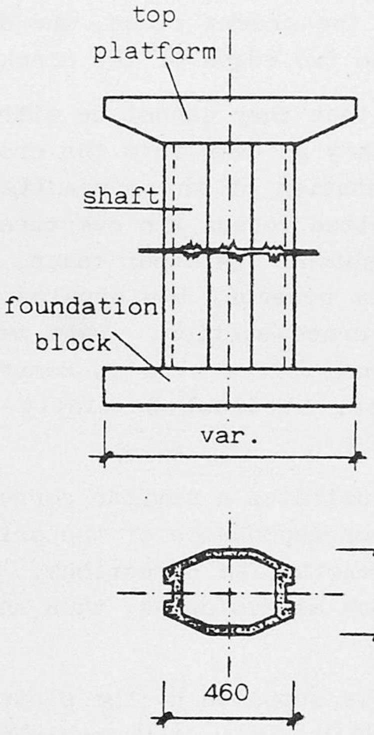


Fig. 1 Schematic representation of the pier geometry.

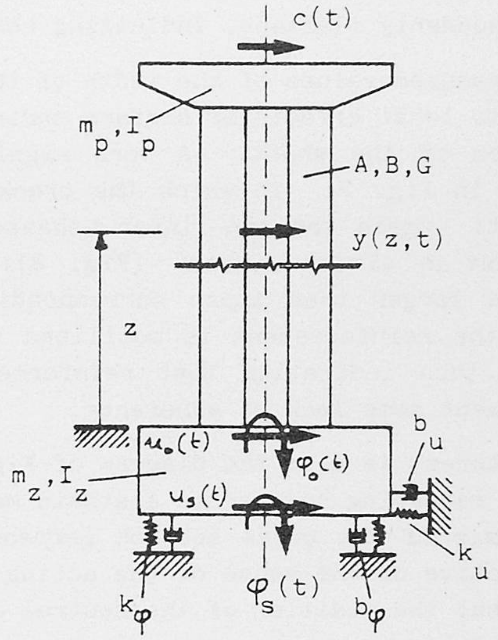


Fig. 2 Reference scheme for interpretation of soil behaviour.

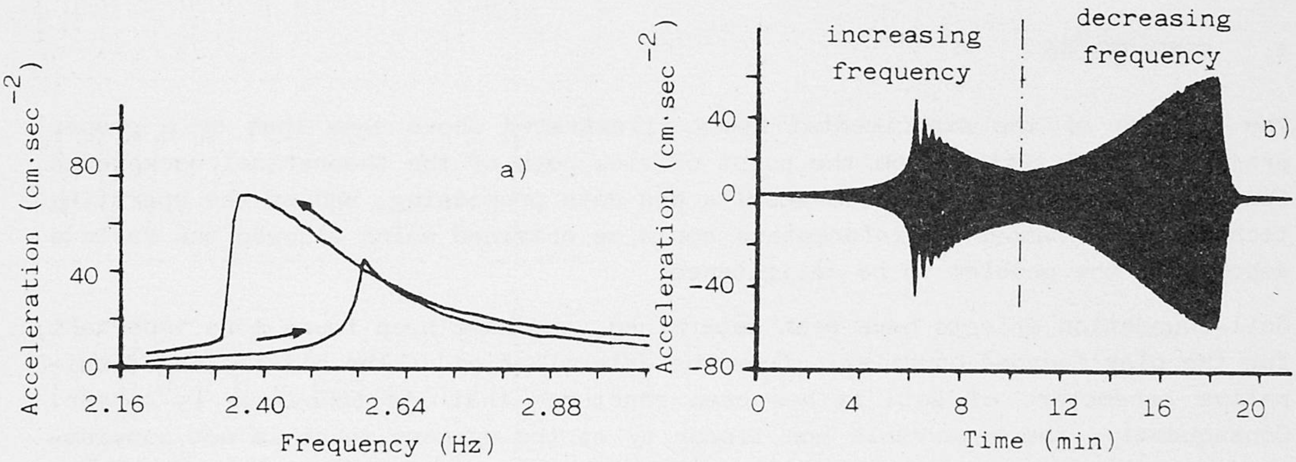


Fig. 3 Acceleration values at top of the piers. a) Response curves for pier A; b) Time history for pier B. Tests with increasing and decreasing frequency.

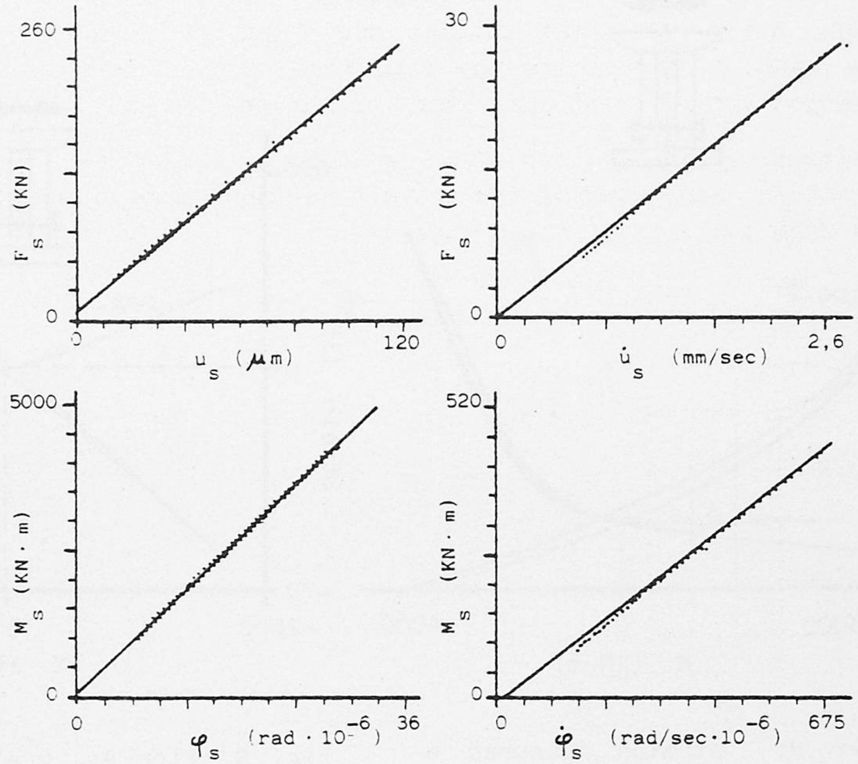


Fig. 4 Experimental correlation between applied loadings and foundation motions.

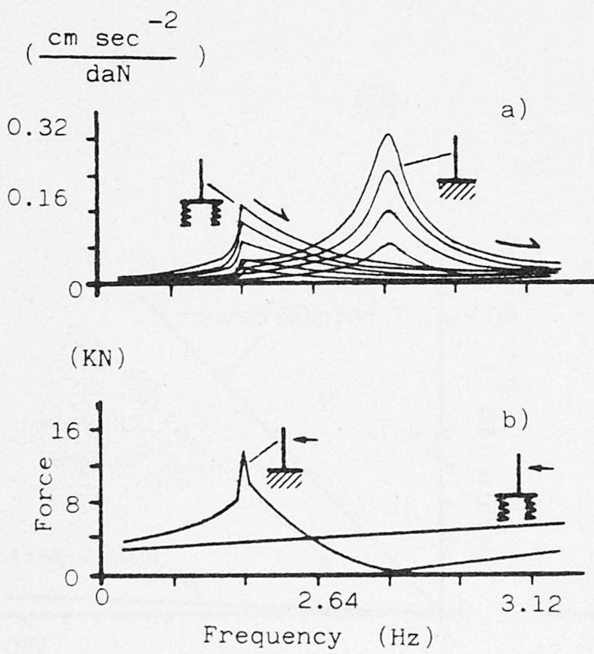


Fig. 5 Pier A: transfer functions (a) and exciting forces (b) for actual and "clamped" pier.

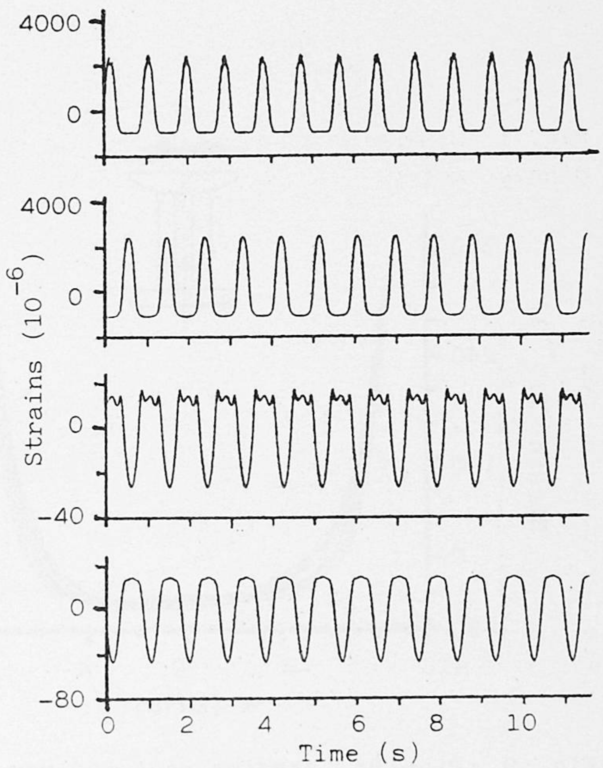


Fig. 6 Pier B: examples of strain behaviour showing contact of crack edges with shocks.

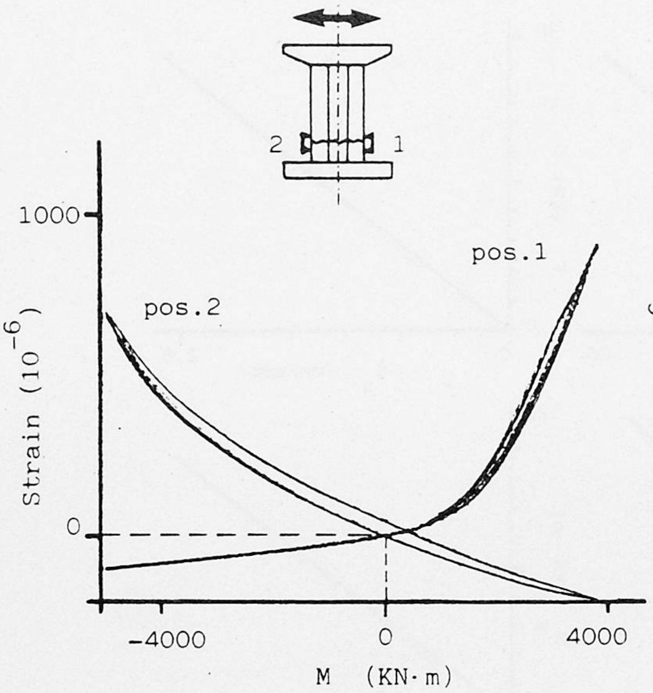


Fig. 7 Pier B: strains measured across cracks versus overturning moment. The non symmetric behaviour of the crack openings is cleary shown.

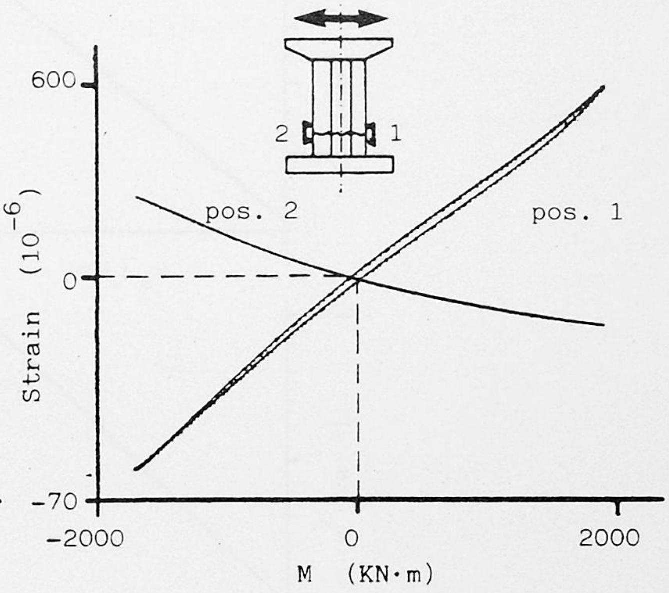


Fig. 8 Pier A: crack opening versus overturning moment. Position 1 shows mobilization of reinforcement in compression.

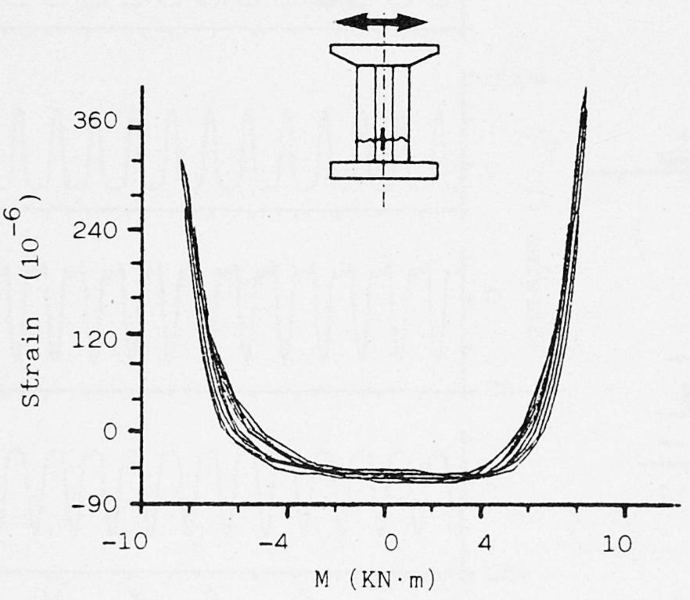


Fig. 9 Pier B: strains measured versus overturning moment across a crack on the "neutral axis" of the shaft.

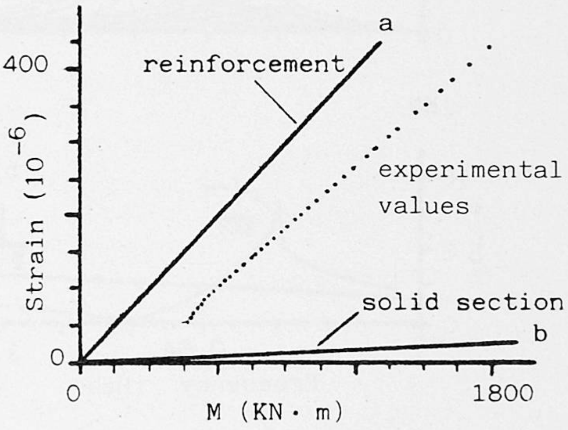


Fig. 10 Maximum strains versus overturning moment.

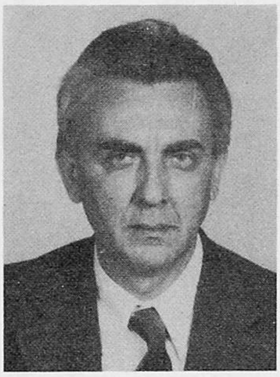
Probebelastung einer historischen Stahlbrücke in Budapest

Test Loading of an Historical Steel Bridge in Budapest

Essai de charge d'un pont historique à Budapest

Antal SZITTNER
Dr.-Ing.
Technische Universität
Budapest, Ungarn

Otto HALÁSZ
Dr.-Ing.
Technische Universität
Budapest, Ungarn



Antal Szittner, geboren 1926, promovierte als Bauingenieur an der TU Budapest. Er arbeitet seit 1959 am Lehrstuhl für Stahlbau der TU Budapest, als wiss. Hauptmitarbeiter. Sein Hauptarbeitsgebiet: Experimentelle Spannungsanalyse, Modelluntersuchungen, Ueberprüfung der Brückenkonstruktionen mittels Probebelastung auf dem Gebiet der Stahlkonstruktionen.

Otto Halász, geboren 1927, promovierte als Bauingenieur an der TU Budapest. Professor und Inhaber des Lehrstuhls für Stahlbau der TU Budapest, Dekan der Fakultät für Bauwesen. Forschungsgebiet: Stabilitätstheorie und Traglastverfahren von Stahlkonstruktionen. Korrespondierendes Mitglied der Ungarischen Akademie der Wissenschaften.

ZUSAMMENFASSUNG

Aufgrund der an einer Seitenöffnung der Margareten-Brücke durchgeführten Messungen wird eine Prüfmethode zur Erfassung der Tragfähigkeit von Brücken vorgestellt, bei denen das Kräftespiel wegen des komplizierten Trägeraufbaus auf rechnerischem Wege nicht genügend gut nachgewiesen werden kann und die sichere Weiterführung des Verkehrs aus dem Gesichtspunkt des Denkmalschutzes erwünscht ist.

SUMMARY

Based on the loading test carried out on a side-span of the Margaret Bridge in Budapest, a method for checking the carrying capacity of bridges is presented. It can be applied when the complicated set up of the structure does not allow a reliable computational analysis and the bridge, as a historical monument, should carry increased traffic.

RESUME

Sur la base d'un essai de charge d'une travée latérale du pont Marguerite à Budapest, une méthode est présentée pour la détermination de la capacité portante des ponts dont la structure est très compliquée et dont l'analyse statique ne donne pas de résultats satisfaisants. L'augmentation du trafic doit être pris en compte sur un pont qui a un caractère de monument historique.

EINLEITUNG

Alle Donau-Brücken in Budapest wurden während des zweiten Weltkrieges zerstört, doch einige Brückenteile wurden bei dem Wiederaufbau eingebaut und auch die nicht zerstörten Öffnungen mit der originalen Konstruktion erneuert.

Im Laufe der seitdem vergangenen 30-35 Jahre ist der Verkehr und die Belastung der Brücken bedeutend gewachsen, und so hat sich die Überprüfung der Brücken als notwendig erwiesen, was besonders bei den alten Baudenkmal-Konstruktionen viele Probleme hervorgerufen hat.

1. ÜBERPRÜFUNG DER BAUDENKMAL-BRÜCKEN

Die Feststellung des Kräftespieles der älteren Brückenkonstruktionen ist mit den gewöhnlichen Rechenmethoden im allgemeinen nicht, oder nur mit grober Näherung möglich, weil

- die genaue Materialqualität und Zustand nicht bekannt,
- das mathematische Modell wegen der verwickelten Trägerkonstruktionen problematisch,
- die Mitwirkung zwischen den Hauptträgern (Querverteilung), und die Mitwirkung zwischen den Hauptträgern im Fahrbahn nicht genau feststellbar ist.

Die Auflösung dieser Ungewissheiten ist nur mit den Methoden der experimentellen Spannungsanalyse mit den auf der tatsächlichen Konstruktion durchgeführten Messungen (Probebelastung) möglich.

2. ÜBERPRÜFUNG DER MARGARETEN-BRÜCKE VON BUDAPEST

Die entwickelte Methode, die auch bei der Überprüfung anderer älteren und neueren Brücken angewandt wurde, ist durch das Beispiel der an der Margareten-Brücke in Budapest durchgeführten Messungen vorgestellt.

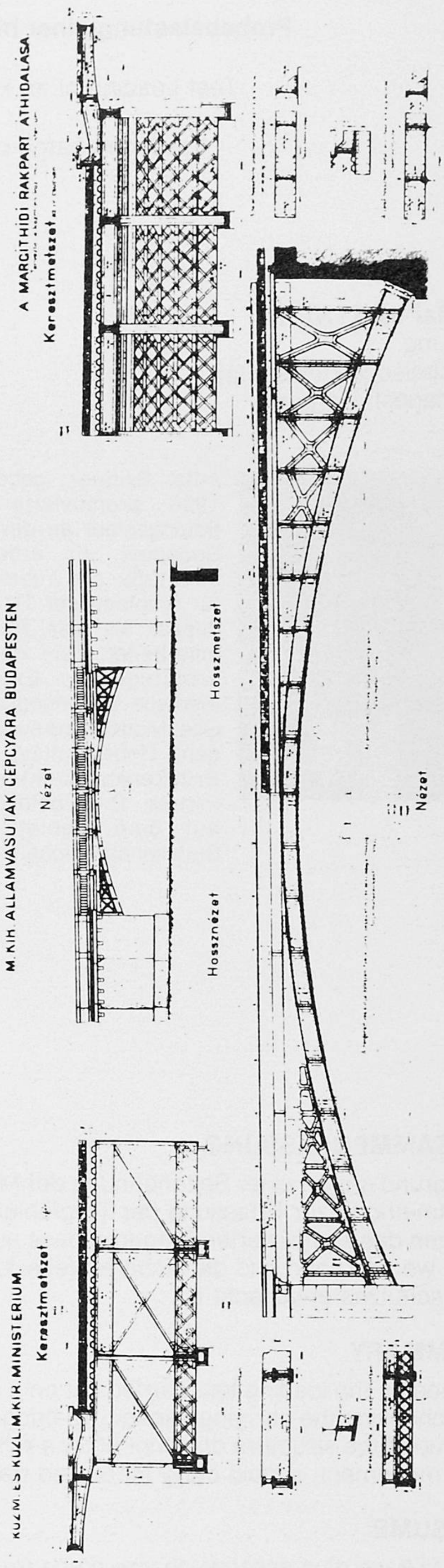


Abb. 1. Seitenöffnung der Margareten-Brücke in Budapest

2.1. Die Konstruktion

Die originale Konstruktion der Margareten-Brücke wurde nach den Plänen von Gouin und Komp. aus Paris im Jahre 1872-76 erbaut. Die sechs Hauptöffnungen über der Donau waren mit zwei Nebenöffnungen von 20,66 m Spannweite zusammengebaut, die schon in Ungarn hergestellt waren (Abb. 1). Die Nebenöffnungen wurden als Zweigelenkbogen, die Hauptöffnungen als eingespannte Bogen gebaut.

Die Hauptöffnungen wurden im zweiten Weltkrieg gesprengt, während die Seitenöffnungen unbeschädigt geblieben sind. Nach dem Wiederaufbau der Brücke wurden die Seitenöffnungen etwas verbreitert, sonst aber in originaler Form in Betrieb genommen.

2.2. Rechnerische Überprüfung

Die Brücke wurde aus Puddeleisen gebaut. Die Fahrbahnplatte mit Steinwürfel-Belag stützt sich auf die Längsträger mit der Vermittlung von Zores-Eisen und Querträgern II. Ordnung. Die Längsträger sind in der Öffnungsmitte mit dem Bogen vereinigt und so sind die Bögen erhöht. Bei der Umgebung der Kämpfer sind die Bögen und Längsträger mit Kreutzstreben-Fachwerk verbunden. Wegen der umsteigender Fahrbahnplatte ist die Konstruktion unsymmetrisch. So ist die Hauptträger-Konstruktion statisch mehrfach unbestimmt, und ausserdem ist die statische Unbestimmtheit nicht eindeutig definierbar. Dementsprechend musste man die statische Berechnung auch bei der Rekonstruktion mit bedeutender Vernachlässigung durchführen.

Die wichtigsten Vernachlässigungen waren:
- die Last-Querverteilung zwischen den Hauptträgern wurde mittels Zweistützbalken gerechnet,
- die Kreuzstreben-Gitterung, und
- die Mitwirkung der Fahrbahnplatte wurde nicht in Betracht genommen.

2.3. Feststellung der Brücken-Tragfähigkeit mittels Probebelastung

Wegen der bedeutenden Vernachlässigungen konnte die notwendige Tragfähigkeit nicht bewiesen werden, darum sollte das wirkliche Kräftespiel und die wirkliche Tragfähigkeit der Brücke mit den Methoden der experimentellen Spannungsanalyse, mit Probebelastung festgestellt werden. Im Rahmen der mit Dehnungs- und Durchbiegungsmessungen durchgeführten Probebelastungen wurde
- die Querverteilung zwischen Hauptträgern,
- das wirkliche Kräftespiel des Hauptträgers,
- der Einfluss der Mitwirkung der Hauptträger und Fahrbahn,
- das dynamische Verhalten der Brücke

beobachtet.

2.3.1. Querverteilung

Die Untersuchung wurde mit in den Bogenvierteln auf den unteren Gurt aufgeklebten Dehnungsmess-Streifen durchgeführt. Die Position der in der Brückenhälfte stehenden LKW wurde in Querrichtung verändert. Dehnungen, bzw. Spannungen an den Hauptträgern deren Proportion die Erfassung der tatsächlichen Querverteilungsumständen ermöglichte, wurden inzwischen gemessen.

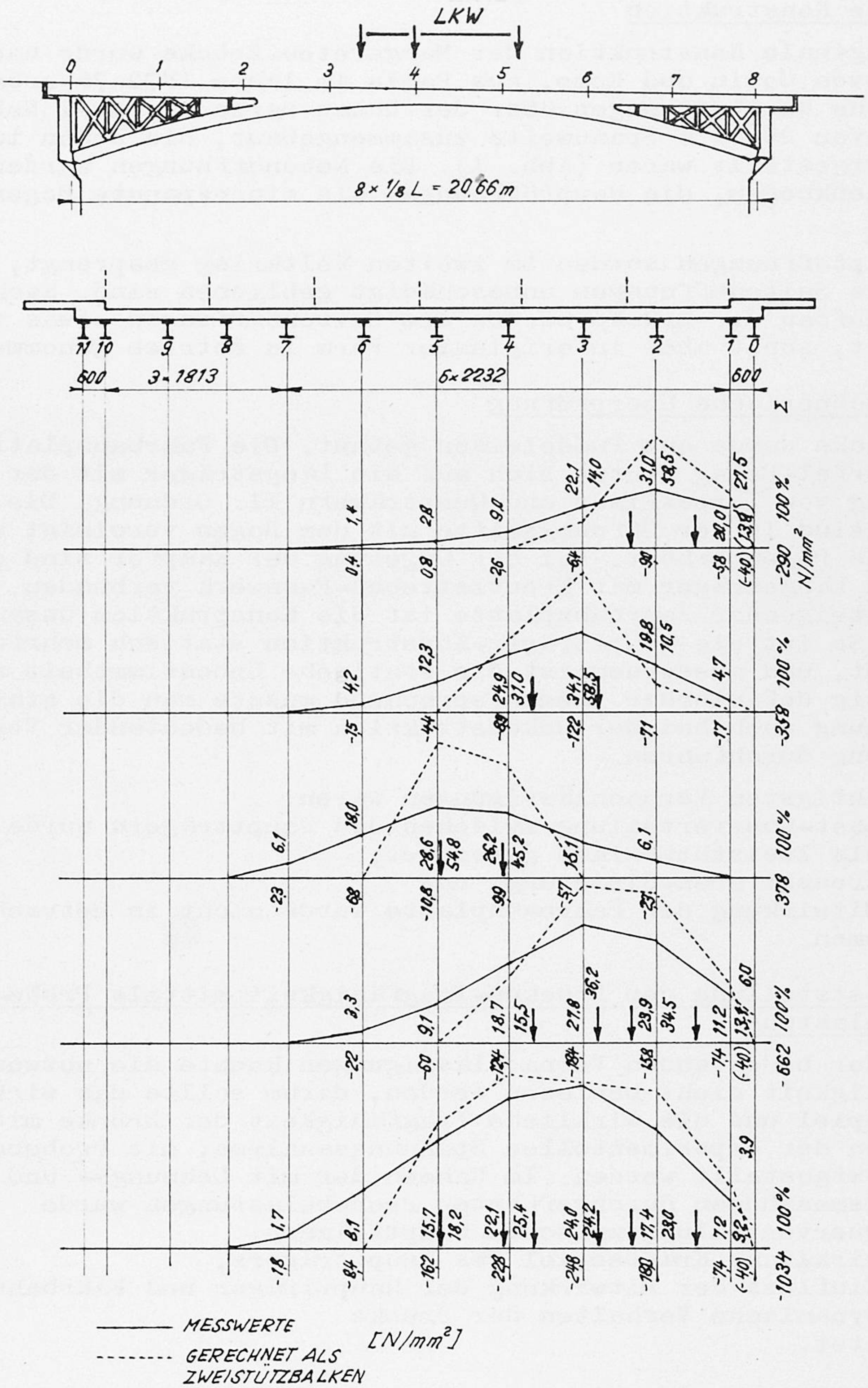


Abb. 2. Spannungen auf dem unteren Gurtblech der Bogenträgern in dem Viertelquerschnitt

Die Resultate der Spannungsmessungen sind in Abb.2 wiedergegeben, wodurch die starke Zusammenwirkung der Hauptträger bewiesen werden konnte. So konnte es auch festgestellt werden, dass die Vernachlässigung der wirklichen Querverteilung auch im Falle mehrerer Belastungswagen zu hohe, nicht reale Beanspruchungen mitbringt. Ähnliche Untersuchungen wurden auch im Mittelquerschnitt durchgeführt, sowohl mit Spannungsmessungen, als auch mit Durchbiegungsmessungen.

2.3.2. Lastenzug-Einflussfläche

Die Bemessung der Brückenkonstruktionen ist im allgemeinen mit Hilfe von Einflusslinien vollzubringen. Die theoretische Einflusslinie ist auch experimentell sehr schwierig herzustellen. Dies zu ersetzen haben wir bei dieser Brücke, ähnlich, wie früher bei anderen, die Feststellung der sogenannten Lastenzugeinflusslinien bzw. Lastenzugeinflussfläche gewählt.

Während der Ermittlung der Einflussfläche bewegte sich ein genau gewogener LKW auf der 6-spurigen Fahrbahnplatte in jeder Spur in gleicher Richtung und blieb mit seiner Vorderachse immer im Achtel der Brückenspannweite stehen. Bei diesen Laststellen wurden die Spannungen und die Durchbiegungen in mehreren Querschnitten bzw. in mehreren Punkten des Querschnittes gemessen. Die Messergebnisse eines Messgebers als Funktion der verschiedenen Laststellen aufgezeichnet liefern die Lastenzug-Einflussflächen. Die Einflussflächen demonstrieren die Wirkung der Laststellen in Längs- und Querrichtung sehr gut, und zeigen auch die wirkliche Beanspruchung bzw. Spannung in den gemessenen Punkten.

Als Beispiel ist im Abb.3 die Spannungs-Einflussfläche im Viertelquerschnitt des Untergurtes dargestellt.

2.3.3. Lastenzug-Einflusslinie

Die gemessenen Einflusslinien-Ordinaten mit den gerechneten Werten zu vergleichen, wurden einige Laststellen ausgewählt. Das linke Rad des LKW bewegte sich über dem Hauptträger No.4, und blieb in jedem Achtel-Querschnitt stehen. Die rechnerischen Spannungen sind mit Hilfe der Moment-Einflusslinie auf Punkt 29 bzw. 31 eines Zwischenbogens festgestellt. Die ausgerechneten Momente wurden mit den Querverteilungs-Koeffizienten nach Abb.2 reduziert und die Randspannungen mit verschiedenen Widerstandsmomenten errechnet. Die Widerstandsmoment-Varianten waren:

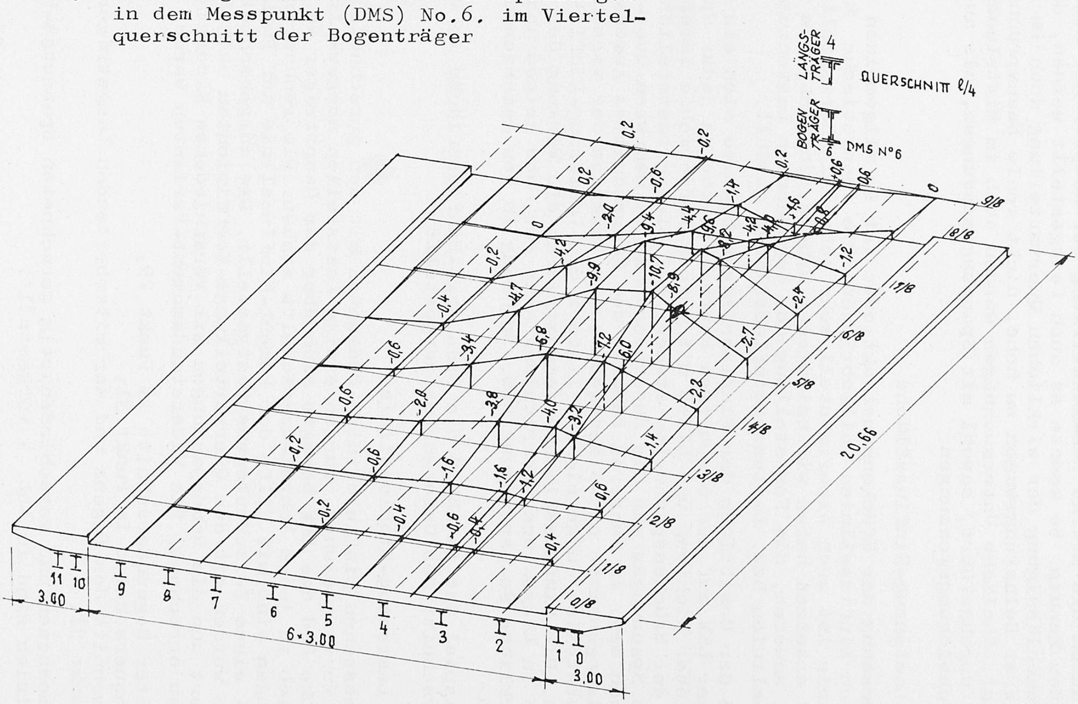
- erhöhter Bogenquerschnitt im Punkt 29,
- Bogenquerschnitt im Punkt 31,
- Querschnitt des Bogens und der mitarbeitenden Längsträger im Punkt 31.

Die gemessenen und verschiedenartig gerechneten Spannungs-Einflusslinien sind in Abb. 4 vorgestellt.

Ebenso wurde die gerechnete und gemessene Spannungs-Einflusslinie für den Mittelquerschnitt aufgezeichnet.

Aus den Abbildungen ist es zu sehen, dass eine bedeutende Abweichung auf die sichere Seite zwischen den gerechneten und gemessenen Randfaser-Spannungen bei beiden Querschnitten sich befindet. Die Abweichung ist bei dem Mittelquerschnitt nur quantita-

Abb. 3. Lastenzug-Einflussfläche der Spannungen in dem Messpunkt (DMS) No.6. im Viertelquerschnitt der Bogenträger



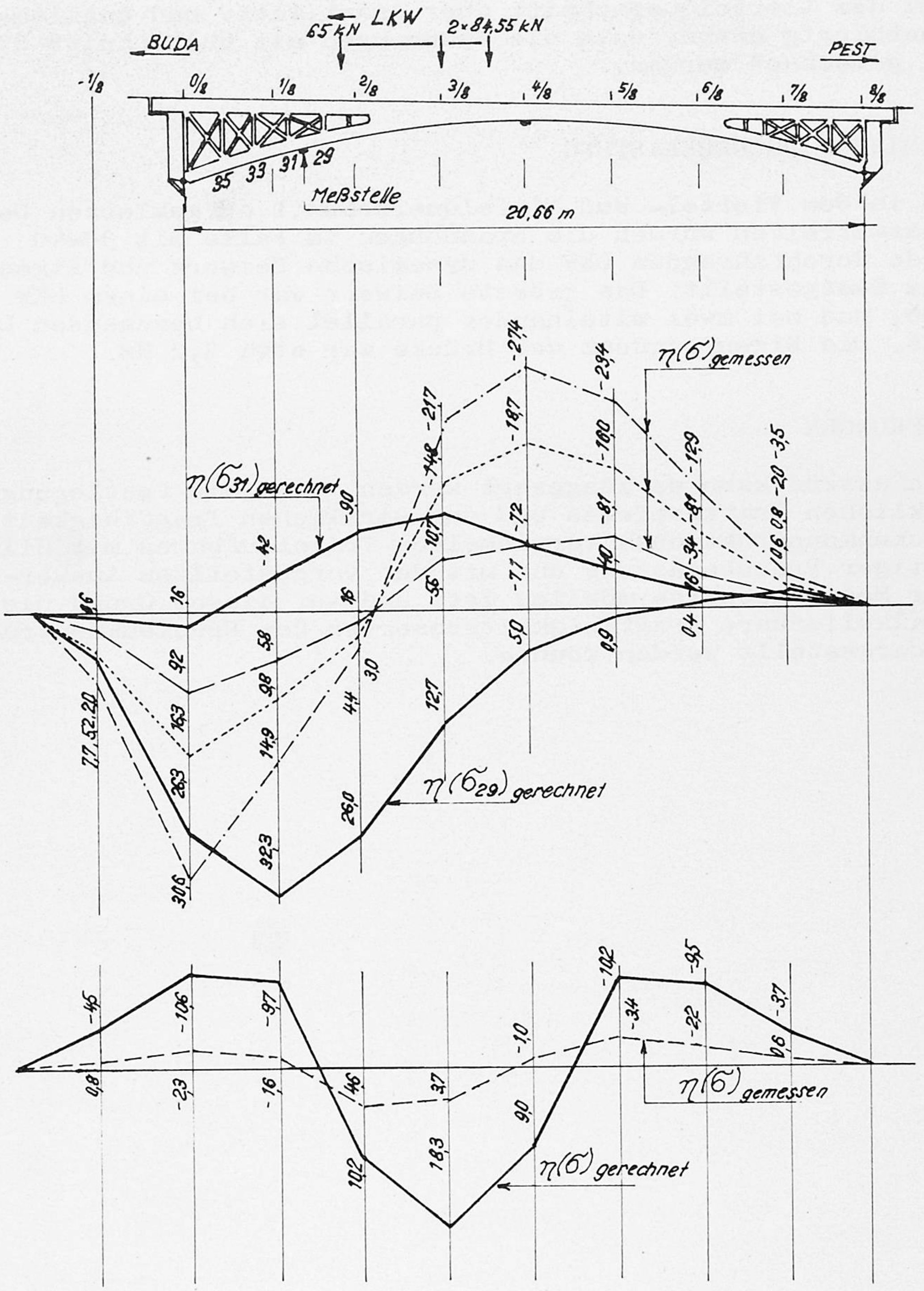


Abb. 4. Gerechnete und gemessene Lastenzug-Einflusslinie der Randfaser-Spannungen (N/mm²) in dem Viertel- und Mittelquerschnitt der Bogenbrücke

tiv, bei dem Viertelquerschnitt aber quantitativ und qualitativ, unabhängig davon, dass die Spannungen mit Querschnitt 29 oder 31 gerechnet wurden.

3. DYNAMISCHE PROBEBELASTUNG

Mit den in dem Viertel- und Mittelquerschnitt aufgeklebten Dehnungsmess-Streifen wurden die Spannungen im Falle mit 30-40 km/Stunde durchfahrenden LKW das dynamische Beiwert und Eigenfrequenz festgestellt. Das grösste Beiwert war bei einem LKW $\varphi = 1{,}06$, und bei zwei miteinander parallel sich bewegenden LKW $\varphi = 1{,}16$. Die Eigenfrequenz der Brücke war etwa 3,2 Hz.

4. FOLGERUNGEN

Zusammenfassend kann es ausgesagt werden, dass die Festlegung des wirklichen Kräftespieles und der wirklichen Tragfähigkeit der Brückenkonstruktionen verwickelten Trägeraufbaues mit Hilfe sorgfältiger Probebelastung und mit der vorgestellten Auswertung der Messergebnisse möglich ist, und auf diesem Grund die nicht kalkulierbare Tragfähigkeitsreserven der Brückenkonstruktionen dargestellt werden können.

Trag- und Verformungsverhalten von Bauwerken aufgrund von Probebelastungen

Strength Evaluation of Structures with Load Tests

Essais de charge et comportement des structures

Horst SCHMIDT
o.Prof. Dr.-Ing.
Technische Universität
Dresden, DDR

Horst Schmidt, geboren 1927, promovierte 1960 an der Technischen Universität Dresden. Er war als leitender Ingenieur im Industriebau an der Errichtung vieler Vorhaben und speziell an der Entwicklung des Stahl- und Spannbeton-Fertigteilbaues beteiligt. Seit 1976 ist er ordentlicher Professor für Festigkeitslehre und experimentelle Baumechanik.

ZUSAMMENFASSUNG

Im ersten Teil des Beitrages werden allgemeine Probleme einer Probebelastung wie zweckmäßiges Belastungsdiagramm, Bestimmung der Höchstlast, geeignete Meßdaten und eine Auswertungsmethode behandelt. Im zweiten Teil wird die durchgeführte Probebelastung an einer alten Stahlbetondeckenkonstruktion in einem Industriebetrieb beschrieben.

SUMMARY

The first part of this paper describes in general the problems of a load test such as an appropriate load diagram, determination of the maximum load, suitable measuring data as well as an evaluation method. The second part describes a test loading which was carried out on an old reinforced concrete floor in an industrial plant.

RESUME

Le rapport traite des problèmes généraux d'un essai de charge, comme le diagramme de charge, la détermination de la charge maximale, des données mesurées et une méthode d'interprétation des mesures. Il décrit un essai de charge réalisé dans une entreprise industrielle sur une ancienne construction avec des dalles en béton armé.

1. ALLGEMEINES ZUR PROBEBELASTUNG

Die genaue Kenntnis über das Trag-und Verformungsverhalten eines existierenden Bauwerkes ist stets dann ein dringendes Erfordernis, wenn sich durch objektive langzeitige Einwirkungen, durch subjektiv bedingte Fehler oder durch außergewöhnliche Ereignisse (Feuer,Explosion, Erdbeben) bedeutende Abweichungen von den der Berechnung und Bemessung bei der Projektierung zugrunde gelegten Annahmen eingestellt haben. Ein weiterer Grund für dieses Erfordernis können vorgesehene wesentliche Veränderungen der Nutzungsbedingungen eines vorhandenen Bauwerkes sein. In Ländern mit einer ralativ umfangreichen alten Bausubstanz tritt das Bedürfnis zur Bestimmung des Trag-und Verformungsverhaltens bestehender Bauwerke in zunehmender Breite auf,um die vorhandenen Baukonstruktionen im Zuge der allgemeinen entwicklungsbedingten Rekonstruktion und Modernisierung der baulichen Anlagen weiterhin rationell nutzen zu können.

Vielfach kann eine analytische Untersuchung mit den Mitteln der theoretischen Baumechanik an einem Gedankenmodell mit Berücksichtigung der eingetretenen Veränderungen zu einem befriedigenden Ergebnis führen, wenn die real vorhandenen geometrischen Maße der Konstruktion, Baustoffeigenschaften, Baugrundverhältnisse und Einwirkungen ausreichend genau bekannt sind. Praktisch lassen sich aber die genannten Parameter in manchen Fällen nur näherungsweise bestimmen, wie z.B. die genaue Lage der Bewehrung im Beton, das in Wirklichkeit wirkende statische System, die Druckfestigkeit des Betons im gesamten Bauwerk, die Verformungseigenschaften der Erdstoffe u.a.;so daß dann oft eine experimentelle Erprobung der realen Baukonstruktion zur Einschätzung der weiteren Trag- und Nutzungsfähigkeit durchgeführt wird.

Bei einer experimentellen Erprobung, bei welcher die vorhandene Baukonstruktion nicht zerstört oder geschädigt werden darf,tritt als Problem die Festlegung beurteilungsfähiger Kriterien für das Ende einer solchen Probebelastung auf. Bekanntlich findet vor oder mit dem Erreichen der Traglast, die einen definierten Erschöpfungszustand einleitet, stets eine wesentliche plastische Verformung an den meist-beanspruchten Querschnitten statt oder es kommt in Ausnahmefällen zu einem Sprödbruch. Beides muß bei einer Probebelastung eines Bauwerkes unbedingt vermieden werden. Somit stellt sich die komplizierte Frage: "Bis zu welcher Größe kann die Probebelastung gesteigert werden, ohne die künftigen Trageigenschaften des bestehenden Bauwerkes zu beeinträchtigen aber gleichzeitig deren reale Traglast sicher einschätzen zu können, um alle Reserven der Tragfähigkeit für die künftige Nutzung zu erschließen"?

In der DIN 1045 war bis zum Jahre 1971 ein Abschnitt zur Probebelastung enthalten 1 ,der eine teilweise Antwort auf diese Frage geben sollte. Als höchste Probelast sollte die 1,5fache Verkehrslast aufgebracht werden. Die fixierte Bedingung, daß die bleibende Durchbiegung höchstens 1/4 der gemessenen Gesamtdurchbiegung betragen darf, konnte nicht aufrechterhalten werden, da Untersuchungen 2 gezeigt hatten, daß mit diesem Kriterium "allein keinesfalls Schlüsse auf eine ausreichende Sicherheit der Konstruktion begründet werden können".

Für Probebelastungen an biegebeanspruchten Bauteilen werden in den Amerikanischen Stahlbetonbestimmungen [3] u.a.folgende Forderungen erhoben. Die höchste Probelast soll äquivalent der 1,19 fachen Eigengewichtslast plus der 1,45fachen Verkehrslast entsprechen. Als allgemeines Kriterium gilt, daß unter der Probelast keine sichtbaren Beweise für das Versagen der Konstruktion auftreten. Als solche Beweise werden z.B. unzulässige Risse und Durchbiegungen angesehen, die die Sicherheit der Konstruktion nicht mehr gewährleisten. Falls derartige "visible evidence of failure" nicht auftreten, soll die größte Durchbiegung den Wert von $l^2/20\,000 \cdot t$ (l=Stützweite, t=Dicke des Bauteiles) möglichst nicht überschreiten. Gemessen wird die maximale Durchbiegung nachdem die höchste Probelast 24 Stunden aufgelegt ist.

Probebelastungen ermöglichen eine Gegenüberstellung gerechneter und gemessener Werte, woraus Einschätzungen über bestimmte elastische oder plastische Eigenschaften, über die Steifigkeit einzelner Bauteile u.a. abgeleitet werden können.Die Zuverlässigkeit derartiger Formänderungsvergleiche hängt stark von der Beschaffenheit der geprüften Konstruktion ab. Bei Stahlkonstruktionen sind die Verhältnisse in der Regel klarer, so daß nachfolgend besonders auf Stahlbetonkonstruktionen eingegangen wird. Grundsätzlich sollte davon ausgegangen werden, daß die Beurteilung der Tragfähigkeit aufgrund einer experimentellen Erprobung auf dem gleichen Sicherheitskonzept fußt, welches einer theoretischen Berechnung nach den gültigen Bauvorschriften zugrunde liegt. Im folgenden werden deshalb entsprechend der Berechnungsmethode nach Grenzzuständen die Einflußparameter getrennt betrachtet.

Bei der Probebelastung eines bestehenden Bauwerkes oder Bauwerkteiles kann davon ausgegangen werden, daß die wirkliche Größe der Eigenlasten mitwirkt, daß das real existierende statische System die Aufnahme und Weiterleitung der Einwirkungen gewährleistet und daß die vorhandenen Baustoffe mit ihren konkreten Eigenschaften beteiligt sind. Somit muß die während der experimentellen Erprobung ermittelte Einwirkung, die von dem geprüften Bauwerk oder Bauteil ohne bleibende Schädigung aufgenommen wurde, zur Bestimmung künftig zulässiger Einwirkungen nicht durch Teilsicherheitsbeiwerte, die die Schwankungen im Eigengewicht, die vereinfachten Annahmen für das statische System und die stochastischen Eigenschaften der Baustoffe berücksichtigen, reduziert werden.

In der Regel werden auch die künftigen Einwirkungen stochastische Größen bleiben, so daß der zutreffende mittlere Lastfaktor n_{tot} bei der Bestimmung der zulässigen Verkehrsnormlast adm F zu berücksichtigen ist. Mit der bei der Probebelastung bestimmten maximalen Lastgröße obs F (ohne Eigenlast), bei der noch keine bleibende Schädigung auftrat, ergibt sich die dem Bauwerk oder Bauwerksteil mit Sicherheit zumutbare Last, die als "im Experiment ermittelte Normlast" bezeichnet werden soll zu

$$\text{adm } F \leqslant \frac{\text{obs } F}{n_{tot}} \qquad (1)$$

Die Eintragung der Probebelastung erfolgt in Laststufen. Erfahrungsgemäß empfiehlt sich von der vorgesehenen Normlast ser F einen Anteil von 20 % bis zu 0,8 ser F und darüber einen Anteil von 10 % oder kleiner als Laststufengröße entsprechend Bild 1 zu benutzen.

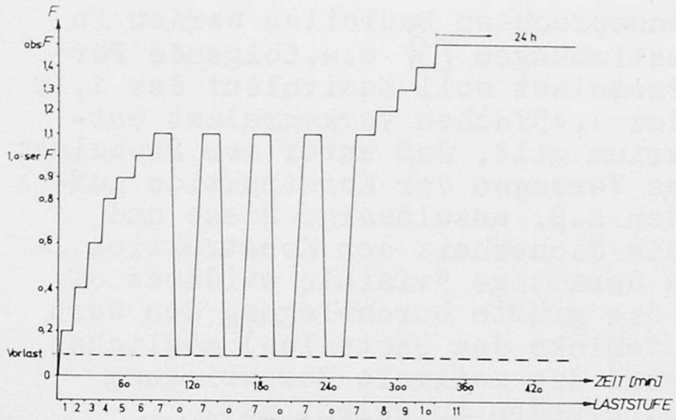


Bild 1 Zweckmäßige Belastungsfolge einer Probebelastung

Die viermalige Ent-und Belastung zwischen der versuchsbedingten Vorlast und der 1,1fachen Normlast ser F, qualifiziert den Kurzzeitversuch, weil die dabei gemessenen zunehmenden oder gleichbleibenden Verformungen wertvolle Meßdaten zur Auswertung beitragen. Hierzu kann auf die Tatsache verwiesen werden, daß das Verhältnis der nach Entlastung bleibenden Verformung f_{bl} zur Gesamtverformung f_{ges} unter Belastung $\varphi = f_{bl}/f_{ges}$ für die jungfräuliche Belastung wesentlich größer als für die folgenden Belastungen ist, wie auch Röbert [2] nachgewiesen hat.

Die maximale Lastgröße der Probebelastung obs F wird zweckmäßig unmittelbar während der Probebelastung festgelegt. Als Orientierung sollte in Anwendung von (1), mit einem Zuschlag in Form des Faktors k_D, der die Dauerlasteinflüsse kompensiert, und einer eingeschätzten vorhandenen höheren Tragfähigkeit gegenüber der näherungsweise gerechneten von ca. 15 % folgende Größe für die Auslegung der Belastungsmöglichkeit während der Probebelastung gewählt werden:

$$\text{obs } F_{\text{geschätzt}} = 1{,}15 \sum_{i=1}^{i=n} k_{Di} \cdot n_i \cdot \text{ser } F_i \qquad (2)$$

Die Summe ist dann erforderlich, wenn Normlasten ser F_i mit unterschiedlichen Lastfaktoren n_i der Probebelastung zugrunde gelegt werden müssen. Für den Faktor k_D können folgende Werte angenommen werden

n	1,1	1,2	1,3	1,4
k_D	1,21	1,11	1,03	1,0

Damit ergibt sich der mittlere Lastfaktor in (1) zu

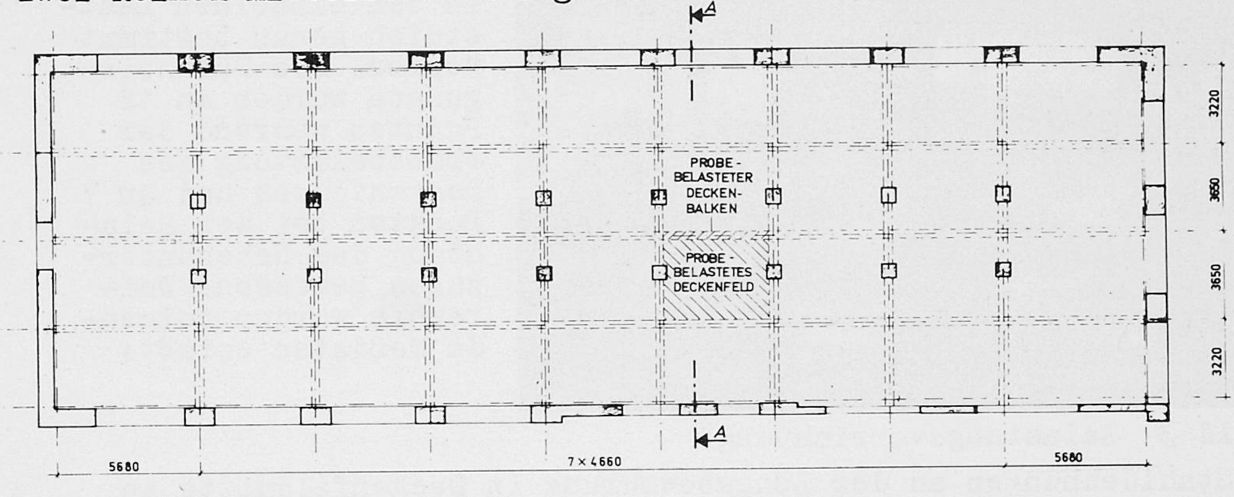
$$n_{tot} = \frac{\sum_{i=1}^{i=n} k_{Di} n_i F_i}{\sum_{i=1}^{i=n} F_i} \qquad (3)$$

In Sonderfällen wird es möglich sein, in (3) die in Vorschriften (z.B. in [4]) festgelegten Lastfaktoren n_i zu reduzieren, wenn mit dem künftigen Nutzer eindeutige und rechtlich verbindliche Festlegungen vereinbart werden können, daß die Normlasten durch geeignete Maßnahmen niemals überschritten werden.

Während der Probebelastung sollen möglichst viele Meßdaten unmittelbar zur Beurteilung der Tragfähigkeit und insbesondere zur Bestimmung der maximalen Lastgröße obs F erfaßt und bewertet werden Hierzu zählen Verschiebungsmessungen in Form von Durchbiegungen. Verdrehungen, Setzungen sowie Dehnungsmessungen an freigelegten Bewehrungsstählen und am Beton der Druckzone in ausgezeichneten Querschnitten sowie die Aufnahme des sich verändernden Rißbildes und der größten Rißweite. Erfahrenen Fachkräften gelingt es in der Regel, aus den für jede Laststufe bereitstehenden Meßdaten während der Probebelastung die richtigen Schlußfolgerungen für eine weitere Laststeigerung oder den Abbruch der Probebelastung zu ziehen. So lassen sich z.B. aus der Linearität oder Nichtlinearität der Verformungen entscheidende Aussagen ableiten. Aus den absoluten Größen der gemessenen Dehnungen kann unter Beachtung der bereits durch die Eigenlasten vorhandenen Spannungen näherungsweise auf die Größenordnung der unter der entsprechenden Laststufe vorhandenen Spannung geschlossen werden. Weiterhin sind das Rißbild und die Rißweiten wichtige Entscheidungskriterien, aus welchen sich die Frage des Abbruches der Probebelastung beantworten läßt.

2. BEISPIEL EINER PROBEBELASTUNG

In einem Industriebetrieb in Dresden machte sich infolge einer geplanten technologischen Veränderung eine Aussage über den baulichen Zustand und die weitere Tragfähigkeit einer in den Jahren 1922/23 ausgeführten Stahlbetondeckenkonstruktion erforderlich. Diese Deckenkonstruktion, im Grundriß 13,75 x 44 m, besteht aus einer 14 cm starken Deckenplatte mit 40 cm hohen, 25 cm breiten Neben- und 68 cm hohen, 30 cm breiten Hauptunterzügen, die auf zwei Reihen Mittelstützen lagern.



SCHNITT A - A

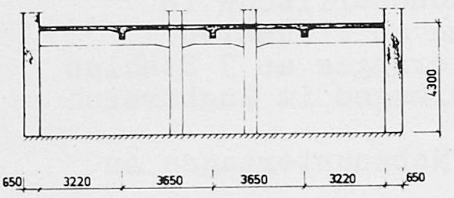


Bild 2 Grundriß und Schnitt der geprüften Deckenkonstruktion

Eine rechnerische Untersuchung erbrachte kein befriedigendes Ergebnis, weil die Betonqualität (es konnten keine brauchbaren zylindrische Prüfkörper erbohrt werden) nicht ausreichend genau bestimmt werden konnte. Mit der vertretbaren Annahme, daß trotz einer relativ geringen Betonfestigkeit für alle Teile der Deckenkonstruktion die gleiche Ausführungsqualität vorausgesetzt werden kann, wurde vom Verfasser vorgeschlagen, die Tragfähigkeit dieser Deckenkonstruktion durch eine Probebelastung eines ausgewählten Decken- und eines Nebenunterzugfeldes zu ermitteln und mit den dabei gewonnenen Meßergebnissen die Tragfähigkeit der nicht direkt probebelasteten Teile der Konstruktion durch Berechnung nachzuweisen.

Unter Beachtung der rechnerischen Untersuchung, einer möglichst geringen Störung der in diesem Gebäude laufenden Produktion und einer einfachen Ausführung der Belastungsvorrichtung mit gesicherter Aufnahme der Reaktionskräfte wurde ein Mittelfeld der über 4 Felder durchlaufenden Deckenplatte und der Abschnitt des dieses Deckenfeld mittragenden Nebenunterzuges (siehe Bild 2) für die Probebelastung ausgewählt.

Zur Krafterzeugung diente ein hydraulischer Heber mit einer größtmöglichen Kraft von 400 kN. Über mehrere statisch bestimmt gelagerte I-Träger wurde die gesamte Einzelkraft in 8 Kräfte für die Deckenfeldbelastung bzw. in 4 Kräfte für die Nebenunterzugbelastung aufgespalten. Die Eintragung dieser aufgeteilten Kräfte in die Stahlbetonkonstruktion erfolgte über in Sand gelagerte Hartholzplatten. Als Widerlager dienten vier I-Träger, die ihre Reaktionskräfte über mittels Schrauben an die Stahlbetonstützen angeklemmte [-Profile durch Reibung abgaben. Das in Querrichtung an die Widerlagerträger angeschraubte I-Profil erlaubte, den Hydraulikheber sowohl in Deckenfeldmitte als auch in der Mitte des Nebenunterzugfeldes anzuordnen. Im Bild 3 ist die Belastungsvorrichtung in Anwendung für das Deckenfeld zu erkennen.

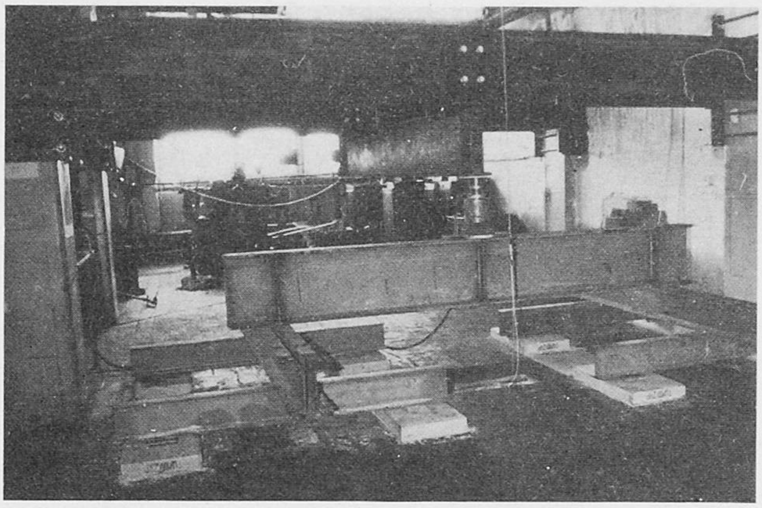


Bild 3 Belastungsvorrichtung

Mit einer eingebauten Kraftmeßdose konnte die jeweils wirkende Kraft in den einzelnen Laststufen genau bestimmt werden. Die Durchbiegungen wurden an 12 Punkten während der Probebelastung des Deckenfeldes und an 7 Punkten bei der Belastung des Nebenunterzuges gemessen. Weiterhin wurden folgende Meßdaten erfaßt:

- Stahldehnungen an der Längsbewehrung in Deckenfeldmitte an drei freigelegten Stählen
- Betonstauchungen an der freigelegten Betonoberfläche in Deckenfeldmitte in Tragrichtung der Decke an 2 Punkten
- Stahldehnungen in Feldmitte des Nebenunterzuges an 3 Stählen
- Stahldehnungen am Auflager des Nebenunterzuges im Zugbereich an 2 Stählen
- Betonstauchungen im Auflagerbereich des Nebenunterzuges an 3 Punkten
- Aufnahme des Rißbildes unter jeder Laststufe

Der vorhandene Fußbodenaufbau mit 30 mm Klinker, 35 mm Mörtelbett, 30 mm Schutzestrich, 1 Lage PVC-Folie und 30 mm Ausgleichestrich ergibt eine Flächenlast von 2,69 kN/m^2. Mit der 14 cm Stahlbetondecke (3,36 kN/m^2) beträgt somit das Deckeneigengewicht 6,05 kN/m^2. Der Auftraggeber wünschte für die künftige Nutzung eine Verkehrsnormlast von 6 kN/m^2.

Von seiten der Staatlichen Bauaufsicht wurde unter Berücksichtigung von möglichen Abweichungen der Eigengewichtslasten in den nicht probebelasteten Bauteilen ein Lastfaktor $n_g = 1,2$ und für die Verkehrslast ein Lastfaktor $n_v = 1,5$ (einschließlich der Dauerlasteinflüsse) gefordert. Außerdem sollte die ungünstigste Laststellung auf die durchlaufenden Konstruktionen Beachtung finden.

Die anzustrebende höchste Einzelprobelast auf das 3,65 x 4,66 m große Deckenfeld ergab somit

$$F_D = (0,2 \cdot 6,05 + 1,09 \cdot 1,5 \cdot 6) \cdot 3,65 \cdot 4,66 - 5,4 = 182 \text{ kN}$$

mit dem Eigengewichtsanteil der Belastungseinrichtung von 5,4 kN und dem Faktor 1,09 für die Simulierung einer Belastung des übernächsten Deckenfeldes.

Aus der rechnerischen Untersuchung konnte unter Beachtung der geringen Betonfestigkeit geschlußfolgert werden, daß die Überschreitung der Biegedruckfestigkeit des Betons im Auflagerbereich für die Nebenunterzüge den Erschöpfungszustand einleiten wird. Deshalb wurde zur Erzeugung eines großen Stützenmomentes das dem probebelasteten Feld des Nebenunterzuges benachbarte Feld mit einer zeitweiligen ruhenden Verkehrslast in Form von Ziegelstapeln von 5,46 kN/m² belastet.

Als Faktor für die Berücksichtigung der Durchlaufwirkung für das Stützmoment ergab sich: $k = \dfrac{0,114 \cdot n_v \cdot F \cdot l^2}{0,054(n_v \cdot F \cdot l^2 + 5,46 \cdot l^2)} \approx 1,3$

und damit die anzustrebende Probelast auf das Nebenunterzugfeld

$$F_N = (0,2 \cdot 6,05 + 1,3 \cdot 1,5 \cdot 6,0) \cdot 3,65 \cdot 4,66 + 0,2 \cdot 1,56 \cdot 4,66 - 5,4 = 2,16 \text{ kN}$$

mit dem Eigengewicht des Nebenunterzuges von 1,56 kN/m.

Nach Aufbringen einer versuchsbedingten Vorlast wurden die Laststufen ähnlich dem Bild 1 realisiert. Bei der Probebelastung des Deckenfeldes wurde die Höchstlast von 200,5 kN mit der elften Laststufe nach reichlich 5 Stunden erreicht und aus Zeitgründen nur 70 Minuten gehalten. Die letzte Ablesung der Meßwerte erfolgte 15 Stunden nach der Entlastung. Für den Nebenunterzug wurde ebenfalls mit 11 Laststufen in knapp 6 Stunden die Höchstlast von 206 kN erreicht und eine Stunde gehalten. Die letzten Ablesungen fanden hier 14 Stunden nach Entlastung statt.

Unter der letzten Laststufe wurden in Deckenfeldmitte maximale Stahldehnungen von 0,41 ‰ gemessen. Durch lineare Interpolation konnte damit die bereits vor der Probebelastung vorhandene Stahldehnung von 0,20 ‰ berechnet werden. Die Stahlspannungen $\sigma_s = (0,41 + 0,20) \cdot 2,1 \cdot 10^2 = 128 \text{ N/mm}^2$ liegen damit noch weit unter der anzunehmenden Streckgrenze von 220 N/mm². Aus den gemessenen Stahldehnungen am Nebenunterzug wurden folgende Gesamtstahlspannungen unter der Höchstlast von 206 kN ermittelt:

$$\sigma_{sFeld} = 60 \text{ N/mm}^2 \qquad \sigma_{sStütze} = 98 \text{ N/mm}^2$$

Die gemessenen Betonstauchungen (unter Höchstlast 0,293 ‰ in Deckenfeldmitte und 0,225 ‰ am Auflager des Nebenunterzuges) erlauben bekanntlich nur eine sehr grobe Abschätzung der vorhandenen Betondruckspannungen. Eine vereinfachte Gleichgewichtsbetrachtung mit den gemessenen Stahldehnungen und Betonstauchungen in Deckenfeldmitte und Vergleiche gemessener mit berechneten Durchbiegungen zeigten, daß der vorhandene Elastizitätsmodul des Betons etwa 8000 bis 12000 N/mm^2 beträgt. Somit konnte nach [5] mit ausreichender Sicherheit eine Betonklasse Bk 5 für die rechnerische Untersuchung der übrigen Teile der Deckenkonstruktion angenommen werden.

Die größten Durchbiegungen betrugen in Deckenfeldmitte 4,2 mm und im Feld des Nebenunterzuges 3,5 mm.Die Zunahme bei Laststeigerung zeigte bis zur Höchstlast noch nahezu linearen Verlauf

Insgesamt konnten bei beiden Erprobungen keine Beweise für ein Versagen unter den Höchstlasten erkannt werden,obwohl von einer weiteren Laststeigerung,insbesondere bei der Probebelastung des Nebenunterzuges, Abstand genommen wurde, weil aus der Einschätzung aller Meßergebnisse geschlußfolgert werden konnte, daß schon eine weitere Lasterhöhung zu bleibenden Schäden an der Konstruktion führen kann.

Aus der höchsten Probelast auf das Deckenfeld obs F_D = 200,5 kN wurde folgende im Experiment ermittelte Normlast adm F ermittelt:

$$\text{obs } F_D = 200{,}5 = (0{,}2 \cdot 6{,}05 + 1{,}09 \cdot \text{adm} F) \cdot 3{,}65 \cdot 4{,}55 - 5{,}4$$

$$\text{adm } F = 6{,}66 \text{ kN/m}^2 > 6{,}0 \text{ kN/m}^2$$

die 11 % über der gewünschten Verkehrsnormlast liegt. Aus der Probebelastung des Nebenunterzuges ergab sich jedoch nur eine Größe für adm F von

$$\text{obs } F_N = 206 = (0{,}2 \cdot 6{,}05 + 1{,}3 \cdot 1{,}5 \cdot \text{adm } F) \cdot 3{,}65 \cdot 4{,}66 + 1{,}45 - 5{,}4$$

$$\text{adm } F = 5{,}71 \text{ kN/m}^2 < 6{,}0 \text{ kN/m}^2,$$

die 5 % kleiner als die gewünschte Verkehrsnormlast ist. Unter Beachtung einer Plastizierung der Betondruckzone über dem Auflager, die eine Momentenumlagerung bewirkt, konnte jedoch insgesamt als Verkehrsnormlast für die geprüfte Deckenkonstruktion 6 kN/m^2 zugelassen werden.

LITERATURVERZEICHNIS

[1] DIN 1045 (Fassung November 1959) § 7 Probebelastung

[2] RÖBERT, S., Kritische Einschätzung der Probebelastung an Stahlbetonbiegeträgern nach DIN 1045, § 7. Bauplanung-Bautechnik, Heft 9 und Heft 10 1958

[3] ACI Standard 318-77

[4] TGL 32274/03 Lastannahmen für Bauwerke-Verkehrslasten

[5] TGL 33403 Betonbau - Festigkeits- und Formänderungskennwerte - verbindlich ab 1.1.1981

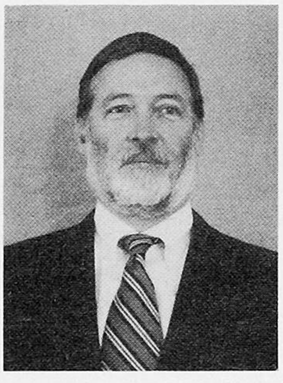
Evaluation of Brick Masonry by Non-Destructive Methods

Evaluation de maçonnerie en briques à l'aide de méthodes non destructives

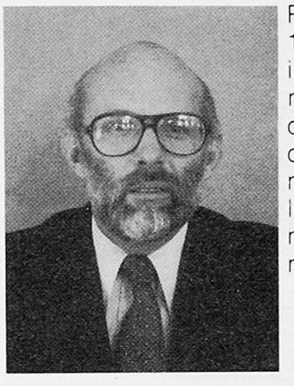
Bewertung von Backsteinmauerwerk mit Hilfe zerstörungsfreier Prüfverfahren

James L. NOLAND
Structural Engineer
Atkinson-Noland & Associates
Boulder, CO, USA

Richard H. ATKINSON
Geotechnical Engineer
Atkinson-Noland & Associates
Boulder, CO, USA



James L. Noland, born 1934, received his Ph.D. in structural engineering from the University of Colorado. He has been involved in research, development and design in many areas related to structures and structural materials with a special interest in masonry.



Richard Atkinson, born 1938, received his Ph.D. in geotechnical engineering from the University of Colorado. Application of rock mechanics techniques to mining problems and to structural masonry have been a major area of his work.

SUMMARY

The project reviews existing non-destructive evaluation methods, which have been applied to other materials, for their effectiveness as methods of strength and condition assessment of masonry. Results are briefly presented with the conclusion that certain methods appear to have potential as practical means of evaluating masonry.

RESUME

Le projet analyse l'application des méthodes non destructives, utilisées pour d'autres matériaux, à l'évaluation de la résistance et de l'état réel d'une structure en maçonnerie. Les résultats sont présentés brièvement. Il semblerait que certaines méthodes puissent être utilisées comme moyens pratiques d'évaluation des structures en maçonnerie.

ZUSAMMENFASSUNG

Der Beitrag untersucht verschiedene zerstörungsfreie Prüfmethoden (NDE-Verfahren) auf ihre Eignung zur Bestimmung der Festigkeit und Güte von Mauerwerk. Die Ergebnisse zeigen, dass sich lediglich gewisse Verfahren für die praktische Bewertung von Mauerwerk eignen.

1. INTRODUCTION

Evaluation of masonry structures in the USA to ascertain strength properties and general condition, i.e., presence of flaws and/or general deterioration, is primarily based on visual observations and destructive tests of specimens removed from the structures [9,31][1]. Visual examination can detect only gross defects while destructive tests of specimens removed from a structure may be time consuming, expensive, and may cause aesthetic or structural damage particularly if the number of destructive test specimens is sufficient to yield a satisfactory degree of statistical confidence.

The research reviewed herein was done to determine whether methods and equipment used for NDE of other materials, primarily rock and concrete [1,2,4,7,11,13,18, 21,26], could be, or have the potential to be, satisfactory NDE methods for structural masonry. The NDE methods evaluated in the research were chosen based on:

- availability and cost of equipment,
- potential for safe and ease of use in field conditions, and
- prior success of use for NDE of other materials.

2. REVIEW OF PREVIOUS NDE OF MASONRY IN THE USA

With the exception of two limited studies on full scale buildings [20,28], no systematic investigation had been conducted in the UDA into the application of NDE methods to masonry prior to the study discussed in this paper [30].

3. NON-DESTRUCTIVE EVALUATION METHODS CONSIDERED

3.1 Schmidt Rebound Hammer

The Schmidt Rebound Hammer is primarily a surface hardness test apparatus developed for concrete testing [13], but has also been used to evaluate rock [1,7] and to provide data used to predict performance of rock tunnel boring mechanics [27].

3.2 Mechanical Pulse Velocity

The mechanical pulse velocity method is based on the correlation of the velocity of an impact-generated stress wave in a material to the properties of that material. Wave velocity is primarily a function of elastic modulus, poisson's ratio, and density, however correlations have been found between strength properties of concrete and wave velocity [13,26]. Wave velocity is affected by flaws in a material [4,13] and was therefore considered potentially amenable to flaw detection in masonry.

3.3 Ultrasonic Pulse Velocity

The ultrasonic pulse velocity method operates on the same principle as the mechanical pulse velocity method. The stress waves are generated, however, by an electroacoustic transducer and are at a high frequency [13,26]. This method has been used to detect flaws in the collar joint of two-wythe masonry walls [29].

---

[1] It should be noted that a form of destructive test, the "shove-test", is being used in parts of the USA for examination of older masonry buildings made using lime-sand mortar or mortars of very low cement content. Post-test repairs completely restore original appearance [30,31]. The "flat-jack" test used in Italy is also a test which temporarily damages the structure, but it can easily be restored to original condition [5,12,17].

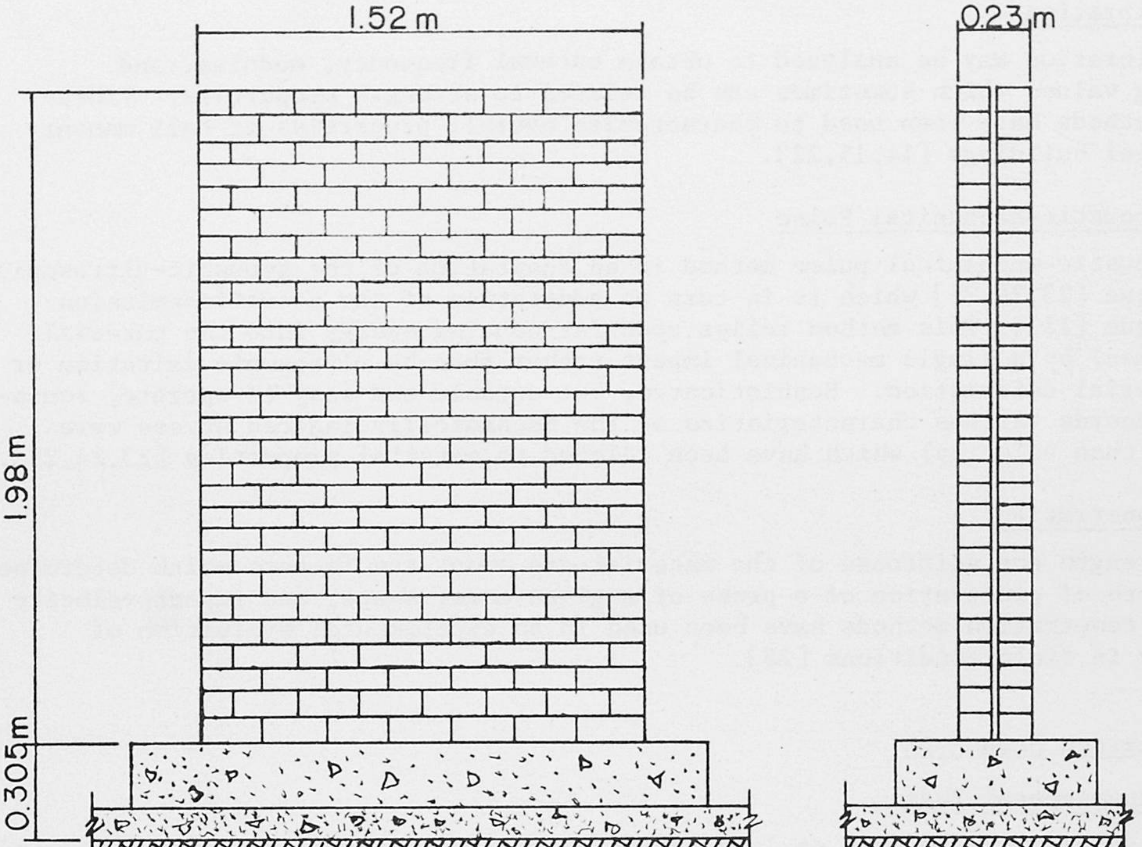


FIGURE 1 - BRICK MASONRY WALL SPECIMEN

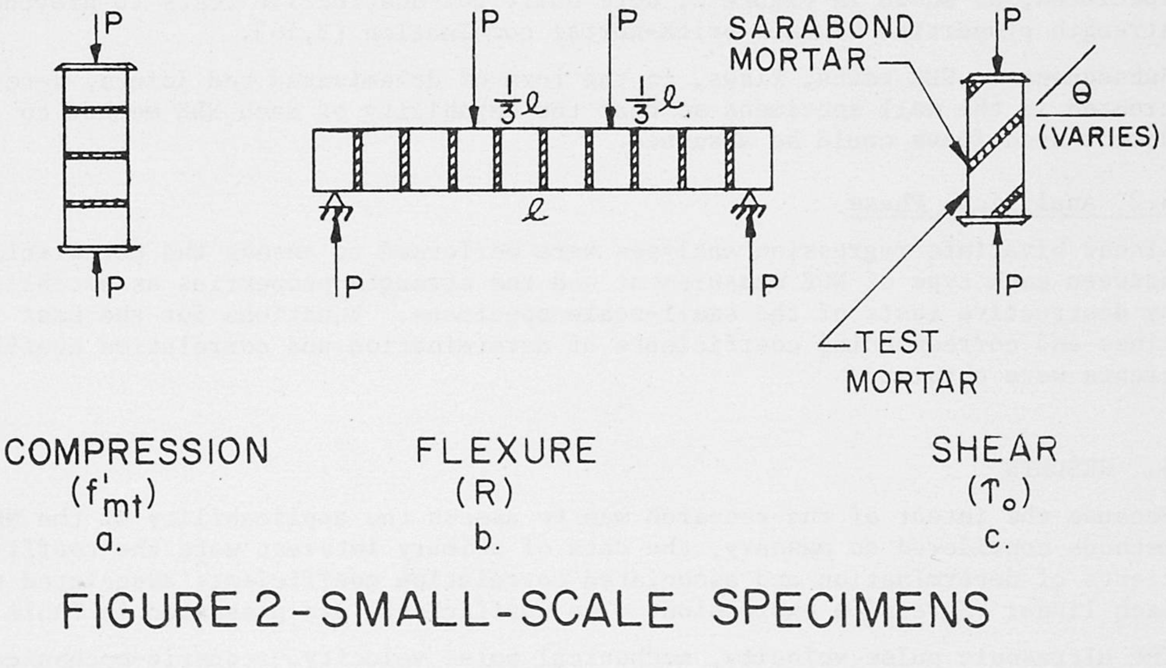


FIGURE 2 - SMALL-SCALE SPECIMENS

3.4 Vibration

Free vibration may be analyzed to obtain natural frequency, modulus, and damping values which sometimes can be related to strength properties. Vibration methods have been used to characterize overall properties of tall masonry and steel buildings [14,15,22].

3.5 Acoustic-Mechanical Pulse

The acoustic-mechanical pulse method is an adaptation of the acoustic-ultrasonic technique [23,24,25] which is in turn an adaptation of the acoustic-emission technique [11]. This method relies upon introducing energy into the material (specimen) by a single mechanical impact rather than by ultrasonic exitation or by material deformation. Sophisticated, but durable and easy-to-operate, equipment records various characteristics of the mechanically induced stress wave (other than velocity) which have been related to material properties [23,24,25].

3.6 Penetration

The strength and stiffness of the material are among the factors which determine the depth of penetration of a probe of a given mass, shape, and impact velocity [13]. Penetration methods have been used in an experimental evaluation of masonry in field conditions [28].

4. RESEARCH CONDUCTED

4.1 Experimental Phase

NDE measurements were made on large brick masonry wall specimens constructed in a laboratory in a cantilever condition as shown in Figure 1. A total of thirty walls were constructed of three types of brick each of a different compressive strength; walls of each kind of brick were built using each of five different mortars. This was done to provide a range of strength properties over which to evaluate the NDE methods considered. The wall specimens built and the NDE tests performed on each are summarized in Table A. Companion small-scale specimens, as shown in Figure 2, were built for destructive tests to provide strength properties of each brick-mortar combination [3,16].

Subsequent to NDE tests, flaws, in the form of delaminated bed joints, were created in the wall specimens so that the capability of each NDE method to detect such flaws could be assessed.

4.2 Analytical Phase

Linear bivariate regression analyses were performed to assess the correlation between each type of NDE measurement and the strength properties as established by destructive tests of the small-scale specimens. Equations for the best fit lines and corresponding coefficients of determination and correlation coefficients were obtained.

5. RESULTS

Because the intent of the research was to assess the applicability of the NDE methods considered to masonry, the data of primary interest were the coefficients of determination and associated correlation coefficients associated with each linear regression expression. The coefficients are presented in Table B.

The ultrasonic pulse velocity, mechanical pulse velocity, acoustic-mechanical pulse, and vibration methods yielded significantly different results when applied to walls with one or two delaminated bed joints. The pulse velocity methods appeared capable of locating the flawed joint, but the acoustic-mechanical pulse and vibration methods could not. The rebound hammer method did not show sensitivity to bed joint flaws.

6. CONCLUSIONS

NDE, at least for the type of masonry tested, appears to have potential for assessment of the strength and condition of masonry. Of the methods considered, the Schmidt Rebound Hammer results were the most closely correlated to compressive strength while the ultrasonic pulse velocity method yielded results most closely related to the modulus of rupture. None of the methods were able to provide results well correlated to joint shear strength.

7. RECOMMENDATIONS

Additional experiments are suggested to evaluate NDE on other forms of masonry. Combining NDE methods applied to masonry may provide better NDE measurement to strength property correlations [6,19].

REFERENCES

1. ATKINSON, R.H., Suggested Methods for Determination of Hardness. Document No. 9, International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Test, 1975.
2. BARTON, N. and CHOVBEY, V., The Shear Strength of Rock Joints in Theory and Practice. v. 10, no. 1-2, Rock Mechanics, December 1977.
3. BAUR, J., NOLAND, J., and CHINN, J., Compression and Tension Bond Strength of Small-Scale Masonry Specimens. Proceedings of the North American Masonry Conference, Boulder, Colorado, August 1978.
4. BRUENING, S.M. and BONE, A.J., Soniscope Applied to Maintenance of Concrete Structures. Proceedings, 33rd Annual Meeting, Highway Research Board, 1954, pp. 210-216.
5. BINDA, L., BALDI, G., CARABELLI, E., ROSSI, G., LANDRIANI, S., Evaluation of the Statical Decay of Masonry Structures: Methodology and Practice. ISMES Report No. 166, Bergamo, Italy.
6. CLIFFON, J. and ANDERSON, E., Nondestructive Evaluation Methods for Quality Acceptance of Hardened Concrete in Structures. NBSIR 80-2163, Center for Building Technology, National Bureau of Standards, Washington, D.C., January 1981.
7. DEERE, D.U. and MILLER, R.P., Engineering Classification and Index Projects for Intact Rocks. Air Force Weapons Laboratory Technical Report No. AFWL-TR-65-116, December 1966.
8. ELLINGWOOD, BRUCE, Analysis of Reliability for Masonry Structures. Proc. Am. Soc. of Civil Eng., Vol. 107, No. ST5, May 1981, pp. 757-773.
9. FATTEL, S.G. and CATTANEO, L., Evaluation of Structural Properties of Masonry in Existing Buildings, NBSIR 74-520, Center for Building Technology, National Bureau of Standards, Washington, D.C.
10. HART, G.C., SABOL, T.A. and HUANG, S.C., Rehabilitation of Masonry Buildings Using Reliability and Decision Theory. Proc. 2nd North Am. Masonry Conf., August 1982.
11. HUCKA, U., A Rapid Method of Determining the Strength of Rocks In-Situ. Int. J. Rock Mech. Mining Science, Vol. 2, 1965, pp. 127-134.

12. JARINA, L., PEANO, A., Characterization of Brick Masonry Stiffness by Numerical Modeling. ISMES Report No. 165, ISMES, Bergamo, Italy.

13. MALHOTRA, V.M., Testing Hardened Concrete: Non-Destructive Methods. Iowa State University Press and American Concrete Institute, 1976.

14. Medearis, K., An Investigation of the Dynamic Response of the Park Lane Towers to Earthquake Loadings. Report to the Colorado Masonry Institute, Denver, Colorado, 1974.

15. MEDEARIS, K., An Investigation of the Strength of Masonry Walls Using Vibration Techniques. Report to the Colorado Masonry Institute, Denver, Colorado, 1976.

16. NUSS, L., NOLAND, J., and CHINN, J., The Parameters Influencing Shear Strength Between Clay Masonry Units and Mortar. Proceedings of the North American Masonry Conference, Boulder, Colorado, August 1978.

17. ROSSI, P., Analysis of Mechanical Characteristics of Brick Masonry Tested by Means of Non-Destructive In-Situ Tests. ISMES Report No. 167, ISMES, Bergamo, Italy.

18. SABNIS, G.M., MILLSTEIN, L., Use of Non-Destructive Methods to Evaluate and Investigate Condition of Buildings and Bridges. Proc. Int'l Conf. on Rehabilitation of Buildings and Bridges Including Investigations, Howard University, Washington, D.C., October 1982.

19. SAMARIN, A., and MEYNINK, P., Use of Combined Ultrasonic and Rebound Hammer Method for Determining Strength of Concrete Structural Members. Concrete International, March 1981, pp. 25-29.

20. SNELL, L.M., Non-Destructive Testing Technique to Evaluate Existing Masonry Construction. Proceedings of the North American Masonry Conference, Boulder, Colorado, August 1978.

21. TARKOY, P.J., Predicting Tunnel Boring Machine Penetration Rates in Selected Rock Types. Ninth Canadian Symposium on Rock Mechanics, Montreal, December 1973.

22. TRIFUNAC, J., Wind and Microtremor Induces Vibrations of a Twenty-Two Story Steel Frame Building. California Institue of Technology EERL Report, Pasadena, California, 1970.

23. VARY, ALEX, A Review of Issues and Strategies in Nondestructive Evaluation of Fiber Reinforced Structural Composites. NASA Technical Memorandum 79246, 1979.

24. VARY, A., BOWLES, K., Ultrasonic Evaluation of the Strength of Undirectional Graphite-Polymide Composites. Proceedings, 11th Symposium on Nondestructive Testing, American Society for Nondestructive Testing and Southwest Research Institute, San Antonio, Texas, 1977, pp. 242-258.

25. VARY, A. and LARK, R., Correlation of Fiber Composite Tensile Strength with the Ultrasonic Stress Wave Factor. J. Testing and Evlauation, Vol. 7, No. 4, 1979, pp. 185-191.

26. WHITEHURST, E.A., Evaluation of Concrete Properties from Sonic Test. American Concrete Institute Monograph No. 2, 1966.

27. WHITMAN, R., HEGER, F., LUFT, R., KRIMGOLD, F., Seismic Resistance of Existing Buildings. Jn. Struc. Div., ASCE, July 1980, pp. 1573-1592.

28. Non-Destructive Test Procedures for Brick, Block, and Mortar. Victor Wilburn Associates, Report to the Dept of Housing and Urban Development, Contract #H-2540, July 1979.

29. Investigation of Sonic Testing of Masonry Walls. Aerojet-General Corp. Report to the Dept. of General Services, Office of Architecture and Construction, State of California, 1967.

30. An Investigation into Methods of Nondestructive Evaluation of Masonry Structures. Report by Atkinson-Noland & Associates, Inc. to National Science Foundation, Grant No. PFR-7916572, February 1982.

31. Earthquake Hazard Reduction in Existing Buildings. Los Angeles City Building Code, Division 68, 1982.

Table A

Wall Specimens*

Wall Series No.	Clay Unit Type	Mortar Type	Nondestructive Tests Performed
IA	A.R.	0:1:3	SH, V, UP, MP
IB	A.R.	0:1:3	SH, V, UP, MP
IC	A.R.	0:1:3	SH, V, UP, MP
ID	A.R.	0:1:3	SH, V, UP, MP, DP
IIA	A.R.	1:¼:3	SH, V, UP, MP
IIB	A.R.	1:¼:3	SH, V, UP, MP
IIC	A.R.	1:¼:3	SH, V, UP, MP
IID	A.R.	1:¼:3	SH, V, UP, DP
IIIA	A.R.	1:1:6	SH, V, UP, MP
IIIB	A.R.	1:1:6	SH, V, UP, MP
IIID	A.R.	1:1:6	SH, V, UP, MP
IVA	A.R.	1:2:9	SH, V, UP, MP, DP, AMP
IVB	A.R.	1:2:9	SH, V, UP, MP, AMP
IVC	A.R.	1:2:9	SH, V, UP, MP, AMP
IVD	A.R.	1:2:9	SH, V, UP, DP
X	A.R.	1:3:12	SH, V, UP, MP, DP
VA	I.S.	1:¼:3	V, UP, MP, DP
VB	I.S.	1:¼:3	V, UP, MP
VC	I.S.	1:¼:3	V, UP, MP
VIA	I.S.	1:1:6	SH, V, UP, MP, DP
VIB	I.S.	1:1:6	SH, V, UP, MP
VIC	I.S.	1:1:6	SH, V, UP, MP
VII	I.S.	1:2:9	SH, V, UP, MP, DP
VIII	I.S.	1:3:12	SH, V, UP, MP, DP
IX	I.S.	0:1:3	SH, V, UP, MP, DP
XI	H.P.	1:¼:3	SH, V, UP, MP, DP
XII	H.P.	1:1:6	SH, V, UP, MP, DP
XIII	H.P.	1:2:9	SH, V, UP, MP, DP
XIV	H.P.	1:3:12	SH, V, UP, MP, DP
XV	H.P.	0:1:3	SH, V, UP, MP, DP

*Code:

SH = Schmidt Hammer
V = Mechanical Vibration
UP = Ultrasonic Pulse Velocity
MP = Mechanical Pulse Velocity
DP = Densicon Penetrometer
AMP = Acoustic Mechanical Pulse
A.R. = Antique Rustic Brick (f_b = 86.1 MPa)
I.S. = Iron Spot Brick (f_b = 91.2 MPa)
H.P. = Hard Pressed Brick (f_b = 46.7 MPa)

Table B

Results of Destructive Tests on Small Specimens vs. NDE Measurements on Unflawed Wall Specimens

Dependent Variable y'	Independent* Variable x	Correlation Coefficient R	Coefficient of Determination R^2
Compressive Strength f'_{mt}	NR	.890	.792
	WD	.768	.590
	HUV	.772	.596
	VUV	.849	.721
	UV	.776	.603
	VMV	.703	.495
	HMV	.734	.539
	BP	.620	.390
	MP	.530	.280
Modulus of Rupture R	HUV	.690	.476
	VUV	.675	.455
	UV	.672	.452
	WD	.644	.415
	NR	.738	.545
	VMV	.447	.200
	HMV	.910	.828
Shear Strength τ_o	NR	.518	.268
	WD	.177	.031
	HUV	.257	.066
	VUV	.392	.154
	UV	.220	.049
	VMV	.340	.115
	HMV	.326	.106

Code:

NR = Rebound number
WD = Natural frequency of vibration
HUV = Ultrasonic velocity - horizontal direction
VUV = Ultrasonic velocity - vertical direction
UV = Ultrasonic velocity - thru wall
HMV = Mechanical pulse velocity - horizontal direction
VMV = Mechanical pulse velocity - vertical direction
BP = Penetration test - brick
MP = Penetration test - mortar
f'_{mt} = Compressive strength of prism
R = Modulus of rupture
τ_o = Shear strength as measured using the inclined bed joint specimen [16]

Effects of Steel on Ultrasonic Measurements for Concrete Members

Effets de l'acier sur les mesures par ultrasons dans les éléments en béton

Einfluss der Bewehrung auf Ultraschallmessungen an Betonelementen

John BUNGEY
Senior Lecturer
University of Liverpool
Liverpool, U.K.

John Bungey, born 1944, studied at St. Andrews University and Imperial College London. Following several years of design and construction of concrete structures and motorways he now teaches structural design. He has undertaken extensive research and consultancy work on the subject of in-situ concrete assessment.

SUMMARY

Embedded reinforcement may have a significant effect on ultrasonic pulse velocity measurements taken through concrete members. If test locations cannot avoid the influence of reinforcement it is essential that reliable corrections be made. This paper demonstrates that currently accepted allowances are unsatisfactory in practice and confirms that bar diameter is an essential variable to be considered. A correction procedure is proposed which has been developed from the results of laboratory testing.

RESUME

Les barres d'armature peuvent avoir un effet important sur les mesures de vitesse de pulsation ultrasonique au travers d'éléments en béton. Si les emplacements de mesure ne peuvent pas éviter l'influence de l'armature, il est de toute nécessité de faire des corrections sûres. Cet étude démontre que les rectifications couramment acceptées ne sont pas satisfaisantes en pratique et confirme que le diamètre de la barre est une variable essentielle dont il faut tenir compte. Un procédé correctif a été développé à partir des résultats d'expériences.

ZUSAMMENFASSUNG

Eingebettete Bewehrung kann einen grossen Einfluss auf die Ultraschallmessungen an Betonelementen haben. Wenn mittels der Messposition der Einfluss der Bewehrung nicht vermieden werden kann, ist es notwendig, Korrekturen einzuführen. Der Beitrag zeigt, dass die zur Zeit in der Praxis akzeptierten Korrekturfaktoren nicht befriedigend sind und bestätigt, dass der Durchmesser der Bewehrung eine wesentliche Variable ist. Ein aus den Laborversuchen entwickeltes Korrekturverfahren wird vorgeschlagen.

1. INTRODUCTION

1.1 Significance of Reinforcement

It is well established that embedded reinforcement which is located along, or close to, the line of ultrasonic pulse velocity measurements will influence the measured values. This is recognised by National and International Standards which recommend that reinforcement should be avoided whenever possible when selecting test locations. Such an approach is clearly the most reliable. With the aid of cover measurement devices this may often be practicable, but there will also be circumstances in which this proves to be impossible. In these cases it is necessary to make a correction to the measured value to provide an estimate of the velocity of the pulse in the plain concrete. Corrections of this type are not easy to establish because of the nature of the variables involved, and the steel influence may dominate over the concrete properties. This will inevitably reduce the confidence that can be placed in the value obtained, but careful examination of the parameters involved may help to reduce the uncertainty.

1.2 Existing Allowances and their Shortcomings

The current recommendations given by British Standards [1] and RILEM [2] for this are essentially similar and involve only the two basic parameters of concrete pulse velocity and relative pulse path lengths within the steel and concrete. An average pulse velocity through embedded steel (greater than through concrete) is assumed, and factors are given to allow for the maximum possible influence of the steel. In practice it has been suggested [3] that the diameter has a considerable effect on the pulse velocity within the steel bar. This value is further affected by the velocity of a pulse through the concrete surrounding the bar, and the condition of the bond between steel and concrete may also be important. The presence of cracking in the concrete will further complicate the situation.

The use of correction factors which do not allow for these features may result in a significant underestimate of the true pulse velocity within the concrete and lead to subsequent misinterpretation of the test results. This is a major shortcoming of the currently recommended allowances which only provide an indication of the maximum possible effects of reinforcement. These are of limited value, and may be very misleading for practical situations with common bar sizes.

1.3 Aims of Investigation

The results presented in this paper have been obtained in the course of a laboratory investigation to examine the influence of a range of variables upon the effects of embedded steel on pulse velocity measurements. The purpose of the work was to confirm and extend the basic findings of Chung [3] in relation to the currently recommended corrections, and to consider the significance of bond defects and cracking. More realistic correction procedures can thus be identified and their reliability assessed.

2. THEORETICAL BACKGROUND

2.1 Basic Theory

The influence of steel may be of importance whenever it is possible for a pulse to arrive more quickly at the receiving transducer by taking a path passing partly through the steel rather than through the concrete alone. The important features are therefore:-

a) location of reinforcement relative to the transducer positions.
b) pulse velocity within the concrete (V_c km/s).
c) pulse velocity within the steel (V_s km/s).

By reference to Fig. 1 it can be shown that for a bar lying parallel to the proposed path, the steel will potentially influence the results when

$$\frac{a}{L} < \frac{1}{2}\sqrt{\frac{V_s - V_c}{V_s + V_c}}$$

in which case, the pulse velocity in the concrete is given by

$$V_c = \frac{2a\,V_s}{\sqrt{4a^2 + (TV_s - L)^2}} \text{ km/s}$$

provided that $V_s \geq V_c$, where
T = measured transit time (sec.).

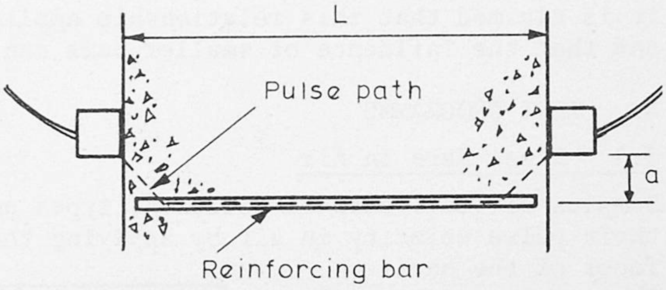


Fig. 1 Influence of longitudinal bar

If the measured pulse velocity in such circumstances is V_m, a correction factor (k) can be developed such that $V_c = kV_m$, with V_s and a/L as variables such that

$$k = \gamma + 2\left(\frac{a}{L}\right)\sqrt{1 - \gamma^2} \quad \text{where } \gamma = \frac{V_c}{V_s}$$

It can be shown that the above expressions will only hold when the offset (a) is large in relation to the end cover (c). If a < 2c (approximately) then the pulse will theoretically pass through the full length of the bar (L_s) and the velocity through concrete is given by:

$$V_c = \frac{2V_s(\sqrt{a^2 + c^2})}{(TV_s - L_s)} \text{ km/s}$$

and for the case where a = 0 (i.e. bars directly in line with pulse) the correction factor k may be obtained from

$$k = 1 - \frac{L_s}{L}(1 - \gamma)$$

This same expression will apply to the case of bars transverse to the pulse path, as shown in Fig. 2, in which case the total path length in the steel (L_s) is taken as the sum of the diameters of the individual bars.

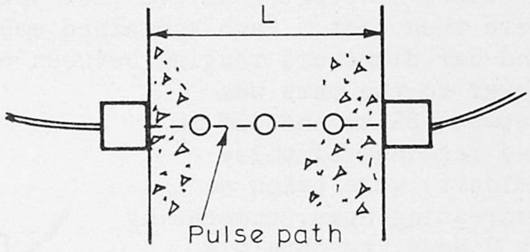


Fig. 2 Transverse bars

The above expressions are based on the assumptions that concrete is a uniform homogenous material, and that the pulse is transferred between the concrete and steel with total efficiency. They also require a value for V_s, the pulse velocity in the steel embedded in the concrete, to be available. Although the value will be influenced by the pulse velocity in the surrounding concrete, no detailed information concerning this is currently available in Standards. B.S. 4408: Pt. 5 [1] provides correction data based on V_s = 5.5 km/s for longitudinal bars.

2.2 Effect of diameter and surrounding concrete

Chung [3] has proposed an effective velocity concept to account for the fact that the pulse velocity in steel varies according to the surrounding medium.

For a bar embedded in concrete, the effective pulse velocity along the bar will be less than for the bar in air and is dependent upon its diameter. Chung has developed the following empirical expression to allow for these combined effects

$$V_s = 5.90 - 10.4(5.9 - V_s)/\Phi \quad \text{where } \Phi \text{ is the bar diameter.}$$

It is claimed that this relationship applies to bars of 10 mm diameter or over, and that the influence of smaller bars can scarcely be detected.

3. TEST PROGRAMME

3.1 Steel Bars in Air

Samples of steel bars of different types and diameters were tested to determine their pulse velocity in air by applying the transducers to the smoothed end faces of the bars. These values are plotted in Fig. 3 for transducers with a frequency of 54 kHz which is the type most commonly used for insitu concrete investigations. Measured values using a frequency of 82 kHz were found to be approximately 2% higher.

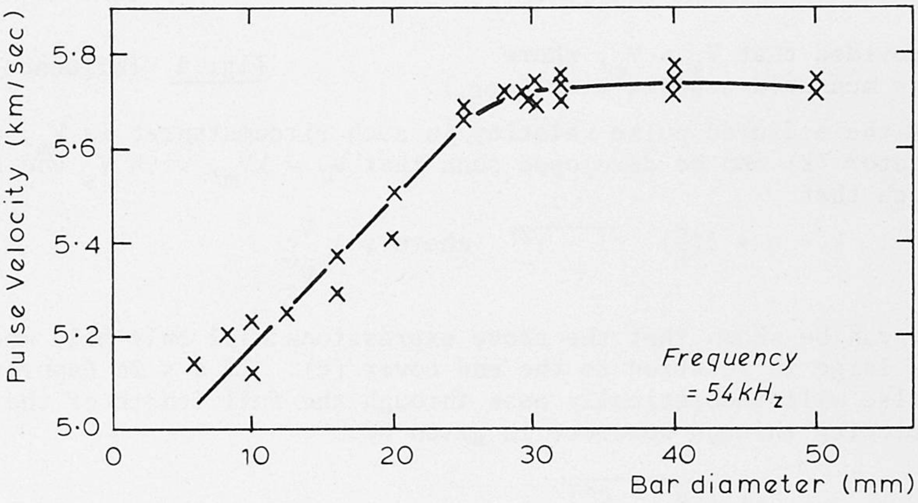


Fig. 3 Results for bars in air

3.2 Steel Bars Embedded in Concrete

A series of eight concrete test specimens with dimensions 490 x 250 x 150 mm were then cast. Each contained embedded reinforcement as shown in Fig. 4, and bar diameters ranging between 6 mm and 50 mm were used. Initially the end cover to the bars was between 85 mm and 130 mm and readings of pulse velocity were taken at increasing ages, commencing at 2 days, to obtain a range of concrete pulse velocities between 3.9 km/s and 4.5 km/s. These measurements were taken directly along the line of each bar, and also transversely across the block in line with each bar. Subsequently the ends of the blocks were sawn off to give end covers in the range 20-25 mm and the longitudinal measurements were then repeated. Round mild steel was generally used since earlier preliminary tests had indicated no significant differences due to steel type, and a pulse velocity of 54 kHz was adopted for these tests.

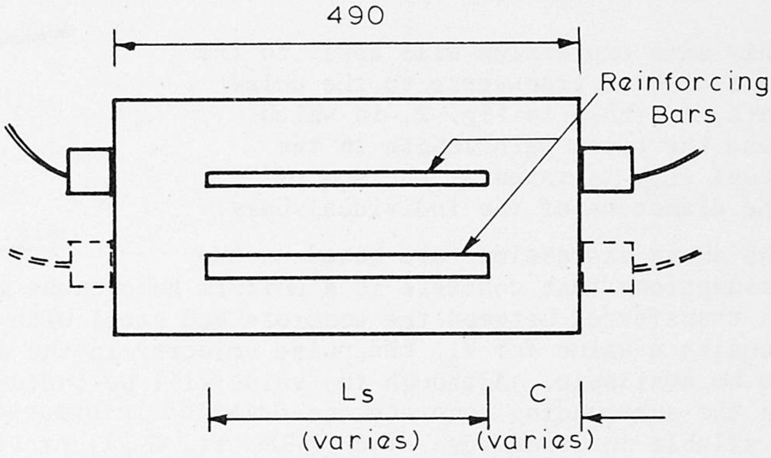


Fig. 4 Test block

3.3 Effects of Cracking and Bond

Following the longitudinal readings with 25-30 mm end cover, a single substantial crack was induced across the full cross-section of each block approximately at the midpoint of its length. The longitudinal readings were then repeated for comparison with the previous values. In addition to two blocks of the series which contained bars liberally coated with grease prior to casting, further specimens in the form of 150 mm cubes were made with comparative greased and ungreased bars cast-in with their ends projecting from the cubes. 10 mm and 32 mm bars were examined in this way.

3.4 Beam containing reinforcement

Following the above tests, an extensive series of readings was taken across the 150 mm width of a 4 m long reinforced concrete beam which had been cast in the laboratory by undergraduate students. Readings were taken at 3 levels, including that of the 12 mm main steel, and were spaced to be in line with, and midway between, the 6 mm links.

4. DISCUSSION OF RESULTS

4.1 Steel Bars in Air

It is clear from Fig. 3 that the pulse velocity along a reinforcing bar in air is reasonably constant for bar diameters of 30 mm and above, with an average value of approximately 5.72 km/s. This reduces with bar diameter below that size in an approximately linear manner.

4.2 Embedded bars parallel to pulse path

The effective velocity in each reinforcing bar has been calculated from the measured values by allowing for the concrete end cover and γ evaluated. Comparisons of the results with established theories are summarised in Fig. 5 which shows the ratio of theoretical/measured velocities in the steel for concrete pulse velocities between 4 km/s and 4.5 km/s. This covers the most commonly occurring range of concrete quality and indicates that the experimental values of V_S generally agree with Chung's predictions for bars of 20 mm diameter or over. For smaller bars, Chung's theory underestimates the steel influence and his contention that bars of 10 mm or less may be ignored is not confirmed.

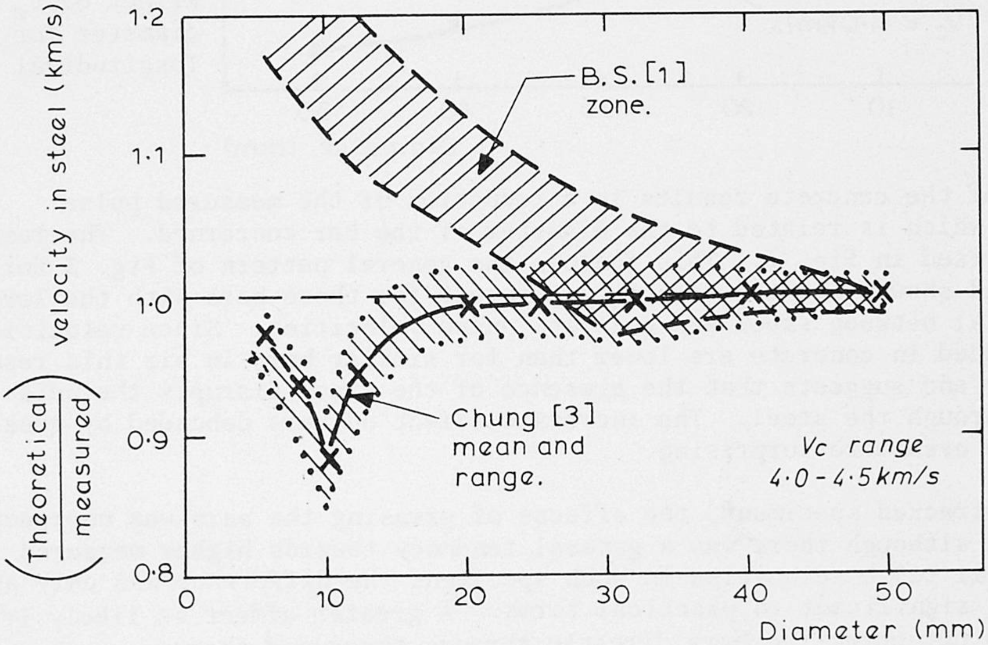


Fig. 5 Results for longitudinal bars

The British Standard values provide a broad band for the range of V_c since V_s is assumed to be constant. The prediction is good for 50 mm bars, and remains reasonable for bars of 30 mm or over in good quality concrete. Below this size it has been shown (Fig. 3) that the bar velocity decreases significantly, and this corresponds well with the features of Fig. 5, which demonstrates the inadequacy of the currently accepted corrections.

The overall range of measured values was approximately 5% for any particular combination but it must be remembered that the generally accepted accuracy of measured pulse velocities on a concrete member is about ± 2%. It was found that the measurements with reduced end cover to the bars yielded higher apparent steel pulse velocities (2% to 3% on average) than those for larger end covers. This may be due to reduced pulse attenuation within the cover concrete, but the influence of differences in contact surface (moulded and sawn) may also have contributed. It should be noted that Chung [3] used 32 mm end cover with moulded contact surfaces.

Measured values of γ are plotted in Fig. 6 for two levels of concrete pulse velocity, and may be used as the basis of a correction factor when used in conjunction with the expression given in section 2.1. The importance of bar diameter is clear, and it will be noted that for small diameter bars the velocity of pulses in the surrounding concrete is of reduced significance. It is also clear that bars of only 6 mm diameter may be detected although their effect is small, and the links of this size could not be identified in the tests on the beam when V_c = 4.4 km/s and $L_s \approx 0.67L$.

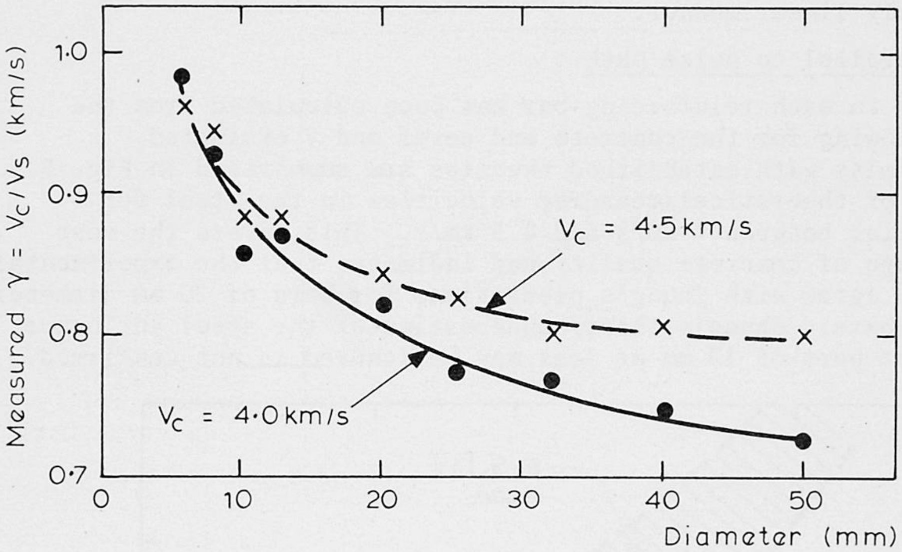


Fig. 6
Effect of V_c and diameter for longitudinal bars

Cracking of the concrete results in a reduction of the measured pulse velocity, which is related to the diameter of the bar concerned. The results are summarised in Fig. 7. This follows the general pattern of Fig. 3 for bars in air, and shows that the effect is greatest for those bars with the largest differential between steel and concrete pulse velocities. Since velocities in bars embedded in concrete are lower than for similar bars in air this result is surprising and suggests that the presence of the crack disrupts the pulse passing through the steel. The increased effect on bars debonded by greasing is perhaps even more surprising.

For the uncracked specimens, the effects of greasing the bars was unexpectedly small, and although there was a general tendency towards higher measured longitudinal pulse velocities in such specimens the difference was only about 1% and not significant in practical terms. A greater effect is likely if the pulse does not enter the bars directly through these end faces.

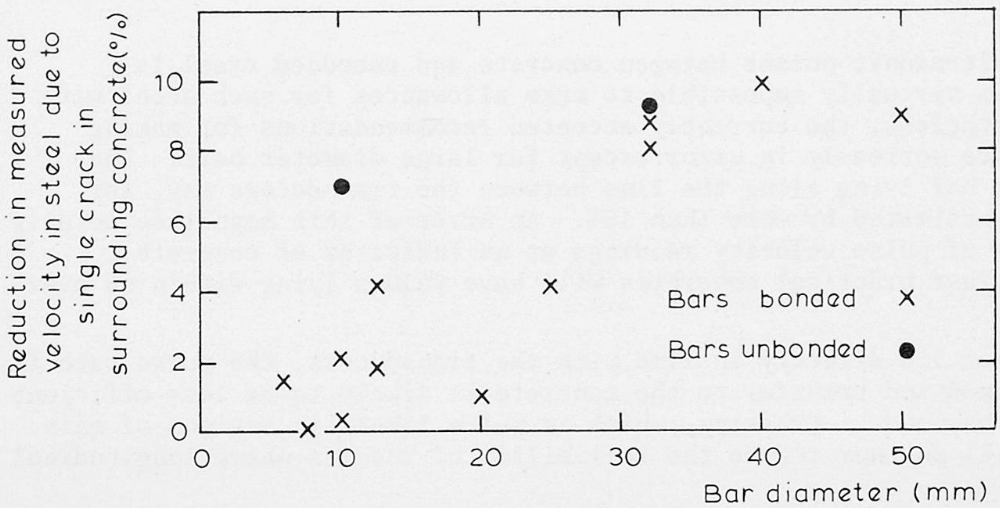


Fig. 7
Effects of cracking

4.3 Transverse Bars

Calculated effective pulse velocities in the steel yield values of low accuracy due to the small path in steel. Consequently, these results have been presented directly in the form of the correction factor k required to convert measured pulse velocity to concrete pulse velocity. For ease of presentation, the values corresponding to $L_S/L = 0.2$ have been computed from the measured readings and are plotted in Fig. 8 for two values of concrete pulse velocity.

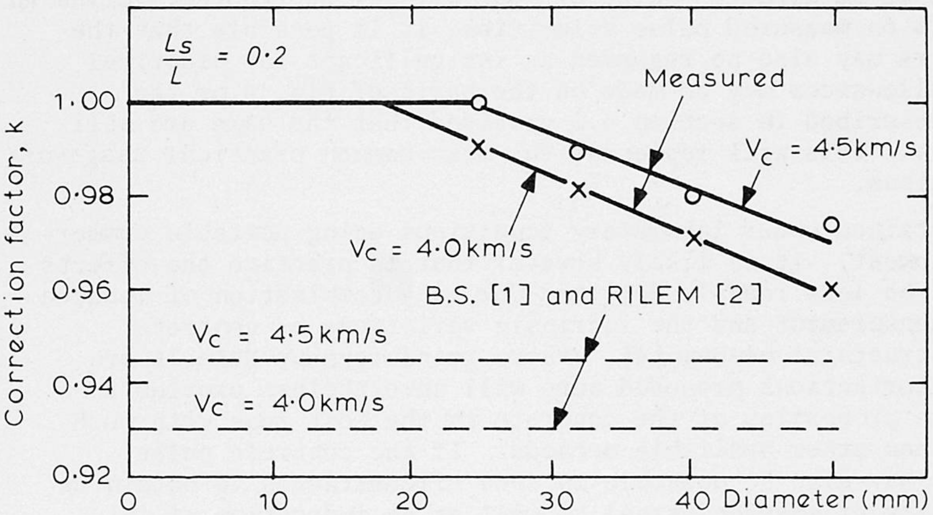


Fig. 8
Correction factors for transverse bars

The inadequacy of existing recommended corrections is again obvious. The effect of reinforcement in this orientation is much smaller than longitudinally, but is nevertheless related to the bar diameter with bars of 20 mm or smaller scarcely detectable. Tests on the beam (section 3.4) showed no influence from the 12 mm main bars. Earlier tests by the Author [4] have indicated that a theoretical estimate of the effect of a transverse bar may be obtained by consideration of the bar as having an equivalent longitudinal length equal to the diameter, but with an effective diameter equal to one half of the true value. Use of the expression proposed by Chung [3] will then yield a value of V_S and hence correction factor k. If this approach is applied to this series of tests, excellent agreement is obtained with the measured values.

The effect of greasing on transverse readings was found to be dramatic, with the result that the influence of the bar disappeared completely. The pulse in this situation is unable to effectively enter the steel, which becomes equivalent to a void of insufficient size to be detected.

5. CONCLUSIONS

The transfer of ultrasonic pulses between concrete and embedded steel is complex, and it is virtually impossible to make allowances for such steel with precision. Nevertheless, the currently accepted recommendations for making such allowances are seriously in error except for large diameter bars. The effect of a 10 mm bar lying along the line between the transducers may, for example, be over-estimated by more than 15%. An error of this magnitude totally negates the value of pulse velocity readings as an indicator of concrete properties since most practical concretes will have values lying within an overall range of 20%.

If the bar does not lie directly in line with the transducers, the pulse path is less clearly defined and transfer to the concrete is likely to be less efficient than through the bar ends. Cracking, which is quite likely in regions of main reinforcement, will further reduce the reliability of results where longitudinal steel is present.

Small diameter bars, such as those commonly used for links and binders may have a significant influence, and where longitudinal steel cannot be avoided it may be allowed for by the use of Fig. 6 to obtain a suitable correction factor. An estimate of concrete pulse velocity obtained in this way may be expected to be accurate within ± 3% provided that cracking is not present and precise details of the steel are known.

Transverse steel has been found to have a much smaller effect than predicted by existing methods, and 20 mm diameter bars, or smaller, may be ignored. Given an accepted error of ± 2% on measured pulse velocities, it is possible that the influence of 25 mm bars may also be regarded as insignificant for practical purposes. Reliable allowances may be made on the basis of Fig. 8 or the numerical procedure described in section 4.3 provided that the bars are well bonded to the concrete. This will represent the most common practical usage of reinforcement corrections.

These results were obtained under laboratory conditions using portable commercially available equipment. It is likely however that in practice the effects of reinforcement will be less readily detected due to a combination of reduced accuracy of on-site measurement and the intrinsic variations of concrete properties within a structural member [4]. Where reinforcement details are known the use of the corrections proposed here will nevertheless provide an indication of the true properties of the concrete in the test zone with much greater reliability than other available methods. If the concrete pulse velocity is known, it may also be possible in some circumstances to obtain an estimate of the quantity of embedded steel as well as an indication of its presence.

REFERENCES

1. B.S. 4408 : Pt. 5, Measurement of the velocity of ultrasonic pulses in concrete. B.S.I. 1974
2. RILEM Recommendation N.D.T.1. Testing of Concrete by the ultrasonic pulse method. RILEM, December 1972
3. CHUNG, H. W., Effects of Embedded steel bars upon ultrasonic testing of concrete. Magazine of Concrete Research, Vol. 30, No. 102, March 1978
4. BUNGEY, J. H., The validity of ultrasonic testing of in-place concrete for strength. N.D.T. International, December 1980, pp. 296-300

Assessment of Concrete Strength by Means of Non-destructive Methods

Evaluation de la résistance du béton au moyen de méthodes non destructives

Ermittlung der Betonfestigkeit mit Hilfe zerstörungsfreier Prüfmethoden

Antonio DI LEO
Civil Eng.
Univ. of Bologna
Bologna, Italy

A. Di Leo took his civil engineering degree at the Bologna University. He has worked in the theoretical and experimental fields of fracture mechanics, constitutive laws, destructive and non-destructive testing methods.

Giovanni PASCALE
Assist. Professor
Univ. of Bologna
Bologna, Italy

G. Pascale received his civil engineering degree at the University of Bologna in 1973. He has carried out his research in the fields of elastic properties of concrete, steam curing, statistic and probabilistic methods applied to civil engineering, non-destructive testing methods.

Erasmo VIOLA
Assoc. Professor
Univ. of Bologna
Bologna, Italy

E. Viola received a civil engineering degree in 1973 at the Naples University. He has authored and co-authored over thirty papers in the fields of elastic stability, fracture mechanics, masonry structures and probabilistic methods applied to damaged materials.

SUMMARY

The purpose of this paper is to present a method for assessing hardened concrete strength by means of non-destructive testing methods. According to the Bayesian approach, the mean and the variance of the concrete compressive strength as well as its characteristic value are considered as random variables. The information of prior distribution supplied by non-destructive testing methods may be up-dated by means of data obtained by core tests. One example of the procedure is reported.

RESUME

L'exposé présente une méthode pour évaluer la résistance du béton au moyen de méthodes d'essai non destructives. D'après la méthode de Bayes, la moyenne et la variance de la résistance à la compression du béton aussi bien que sa valeur caractéristique sont considérées en tant que variables aléatoires. L'information de la distribution initiale obtenue moyennant des méthodes d'essai non destructif peut être mise à jour sur la base de données obtenues à l'aide d'essais sur des carottes. Un exemple du procédé employé est présenté.

ZUSAMMENFASSUNG

Zweck dieser Ausführungen ist die Veranschaulichung einer Prüfmethode zur Ermittlung der Betonfestigkeit durch Anwendung nicht-destruktiver Verfahren. Laut Bayes'Methode sind der Mittelwert und die Varianz der Druckfestigkeit von Beton sowie dessen Eigenwert als stochastische Variablen zu betrachten. Die Informationen aus der herkömmlichen Verteilung, ergänzt durch nicht-destruktive Prüfmethoden, können wirksam durch die Daten aktualisiert werden, die mittels Kernproben ermittelt werden.

NOTATIONS.

$n, \overline{x}, s^2$	=	size, mean and variance of the core sample respectively;
$n', \overline{x}', s'^2$	=	parameters of the prior joint density function;
$n'', \overline{x}'', s''^2$	=	parameters of the posterior joint density function;
$L(m, \sigma / x_1 \ldots x_n)$	=	likelihood function;
$\mu_{(\ )}, s^2_{(\ )}$	=	mean and variance of the variable indicated by the suffix ( );
M, Σ, Ψ^*	=	random variables denoting mean value, standard deviation and characteristic value of the strength;
m, σ	=	values of M and Σ respectively;
$\overline{R}_c, \delta, R_{ck}$	=	deterministic mean, standard deviation and characteristic value of the compressive strength.

1. INTRODUCTION.

Non-destructive methods to assess the mechanical properties of concrete in reinforced concrete structures are generally used when one or more compliance controls with specifications (Model--Code FIP/CEB, D.M. 26.3.1980) result negative or in case of a change of destination or damage in the structure caused by an earthquake(*). The mechanical parameter currently used to measure the properties of the material is the "characteristic compressive strength" R_{ck} defined by the following relation:

$$R_{ck} = \overline{R}_c - k\delta \tag{1}$$

where $\overline{R}_c$ = mean value and δ = standard deviation.

An investigation carried out by means of non-destructive methods should therefore provide the values of the above statistical parameters, considering that they must relate to concrete of "homogenous mixture". On the basis of experiments carried out in the past by other researchers besides the authors, reliable assessment of the casts'homogeneity can be obtained by limiting the coefficient of variation $C_v = \delta / R_c$ by 10%. For this purpose, the empirical correlations (Swedish SS 137352, Czechoslovack CSN 732411-76, Rumanian C.30-67, RILEM 1973 Standards) existing between compressive strength and non-destructive parameters can be used, which relate to the following methods:

- ultrasonic pulse test;
- surface hardness test (rebound hammer);
- penetration resistance test (Windsor Probe System);
- pull-out test (Lok Test, Capo Test, PI Test, Chabowski's method);
- combined methods.

Such correlations, however, are not generally valid to assess compressive strength, since they are determined by means of tests on concrete of standard mixture different from the one to be examined. First-estimate values are currently modified on the basis of a total coefficient of influence, which can be defined according to the information, in case it is available, about the mixture of in-situ concrete or on the basis of the results of tests carried out on a limited core sampling [1 - 4].

(*) The use of non-destructive methods for this purpose, for instance, is provided for by the final document required by the University of Bologna - Emilia Romagna Region - Basilicata Region Convention.

When data obtained by core tests become available, they provide additional information and uncertainty associated with estimate decreases as the core sample size grows. However, the damage of the structure and the cost of the experiments increase steadily as the independent observation $(x_1, \ldots, x_n)$ grows.

Thus, when there is uncertainty, it may not be necessarily economical to obtain more information. In many situations, the size of the sample minimizing the difference between the value of information and its cost, may be determined [5].

The results provided by any set of observations on cores drilled in-sity may be dealt with according to Bayes'theorem. In the Bayesian approach the unknown parameters (the mean value $\overline{R}_c$ and the variance δ in this case) are considered as random variables. The argument of Bayes requires a *prior probability distribution* on the parameters. The distribution is assigned by the engineer on the basis of his professional assessments of all related information available. Experimental observations, which are completely summarized by the likelihood function, are used in order to modify the prior distribution of the parameters. The distribution obtained after taking the sample is called *posterior distribution.* The two density distributions are related, according to Bayes'theorem. By denoting with $\underset{\sim}{\Lambda} = (\overline{R}_c , \delta)$ the vector of the parameters, we have:

$$\begin{pmatrix}\text{Posterior proba-}\\ \text{bility of } \underset{\sim}{\Lambda}\text{, gi-}\\ \text{ven the sample}\end{pmatrix} = \begin{pmatrix}\text{normalizing}\\ \text{constant}\\ \ \end{pmatrix} \times \begin{pmatrix}\text{likelihood of}\\ \text{the sample, gi-}\\ \text{ven } \underset{\sim}{\Lambda}\end{pmatrix} \times \begin{pmatrix}\text{prior proba-}\\ \text{bility of } \underset{\sim}{\Lambda}\\ \ \end{pmatrix} \qquad (2)$$

in which the sample likelihood function $L(\underset{\sim}{\Lambda}/x)$ is the probability of the observed sample $\underset{\sim}{x} = (x_1, \ldots, x_n)$, given $\underset{\sim}{\Lambda}$. The Bayesian treatment permits us to deal with uncertainty connected with the determination of the strength of in-situ concrete as well as to take a decision on the basis of the posterior distribution instead of the prior one. Information on prior distribution can be supplied by the results of a non-destructive testing method.

In the general case of a normal process where no parameter is known, the form of the joint--likelihood function of $M = \overline{R}_c$ and $\Sigma = \delta$ may be expressed as the product of a normal density and a gamma density. It can be shown, in this way, that the prior $f'_{M,\Sigma}(m, \sigma)$ and the posterior $f''_{M,\Sigma}(m, \sigma)$ joint-density functions are of the same form.

The posterior distribution will conform just to the likelihood function when prior information is vague and the sample size is large, or when the prior distribution is relatively flat compared to the sample-likelihood function.

The marginal distributions on the unknown mean value $M = \overline{R}_c$ as well as standard deviation $\Sigma = \delta$ may be found by integration and the posterior synthetic values of the above random variables M and Σ can be determined.

The characteristic value of the strength will also be treated as a random variable.

As a conclusion of the work, examples of application of the above procedure on the basis of investigations carried out in-situ by the authors is reported.

2. MATHEMATICAL THEORY.

When neither parameter m or σ of the normal distribution is known, the likelihood function may be written as follows [6]:

$$L(m, \sigma / x_1, x_2, \ldots, x_n) = \left\{ \frac{1}{\sigma} \exp \left[-\frac{1}{2} \left(\frac{m - \bar{x}}{\sigma \sqrt{m}} \right)^2 \right] \right\} \left[\frac{1}{\sigma^{n-1}} \exp \left(-\frac{n-1}{2} \frac{s^2}{\sigma^2} \right) \right] \tag{3}$$

in which $\bar{x}$ and s^2 are the sample statistics:

$$\bar{x} = \Sigma\, x_i / n \ , \qquad s^2 = \frac{1}{n-1} \Sigma\, (x_i - \bar{x})^2 \tag{4}$$

and n is the number of observations.

The joint conjugate prior distribution on m and σ is:

$$f'_{M,\Sigma}(m, \sigma) = \left\{ \frac{1}{\sqrt{2\pi}\ \sigma / \sqrt{n'}} \exp \left[-\frac{1}{2} \left(\frac{m - \bar{x}'}{\sigma / \sqrt{n'}} \right)^2 \right] \right\} x$$

$$x \left\{ \frac{\left(\frac{n'-1}{2} \right)^{\frac{n'-2}{2}}}{\Gamma \left(\frac{n'-2}{2} \right)} \ \frac{2}{s'} \left(\frac{s'^2}{\sigma^2} \right)^{\frac{n'-1}{2}} \exp \left(-\frac{n'-1}{2} \frac{s'^2}{\sigma^2} \right) \right\} \tag{5}$$

with parameters n', $\bar{x}'$, s'^2, where n' may be interpreted as an equivalent prior sample size.

The posterior joint density function is of the same form as the prior one:

$$f''_{M,\Sigma}(m, \sigma) = N \ \ f'_{M,\Sigma}(m, \sigma) \ \ L(m, \sigma / x_1, x_2, \ldots, x_n) \tag{6}$$

in which N is a normalizing constant with parameters n'', x'', s''^2 which may be calculated in the sequential order:

$$n'' = n + n'$$

$$\bar{x}'' = (n\bar{x} + n'\bar{x}') / (n + n') \tag{7}$$

$$s''^2 = [(n-1)s^2 + (n'-1)s'^2 + n\bar{x}^2 + n'\bar{x}'^2 - n''\bar{x}''^2] / (n'' - 1)$$

The marginal posterior distributions of M and Σ may be obtained from Eq. 5 by integration.

The posterior mean and variance of M are:

$$\mu_M = \bar{x}''$$

$$s_M^2 = s''^2 \frac{n'' - 1}{n''(n'' - 2)} \tag{8}$$

The posterior mean and variance of the standard deviation are:

$$\mu_\Sigma = s'' \sqrt{\frac{n'' - 1}{2}} \ \frac{\Gamma[(n''-3)/2]}{\Gamma[(n''-2)/2]} \qquad n'' > 3 \tag{9}$$

$$s_{\Sigma}^2 = s''^2 \frac{n''-1}{n''-4} - \mu_{\Sigma}^2 \qquad n''>4 \tag{10}$$

From a classical point of view, the characteristic value R_{ck} of the strength may be expressed as in eq. (1), in which k = 1,64 when the strength is normal-distributed and a 5% fractile is considered. From a Bayesian point of view, the strength parameters M, Σ are treated as random variables, each of which possesses a distribution with parameters μ_M, s_M and μ_Σ, s_Σ respectively. Consequently, the characteristic value $\Psi^* = R_{ck}$ of the strength has also to be considered as a random variable with parameters $\mu_{\Psi*}$ and $s_{\Psi*}$. When the random variables M and Σ are supposed to be normal-distributed and independent, on the basis of the (1) we have:

$$\Psi^* = M - k\,\Sigma \tag{11}$$

and consequently:

$$\mu_{\Psi*} = \mu_M - k\,\mu_\Sigma \tag{12}$$

$$s_{\Psi*} = \sqrt{s_M^2 - k^2 s_\Sigma^2} \tag{13}$$

3. APPLICATIONS.

The example here reported refers to an investigation carried out by the authors on a building, the plan of which is reported in Fig. 1. The building is composed of 19 stories and is made up of elements cast in-situ as well as of precast elements. In this work we have reported only the results concerning beams and columns cast in-situ, which belong, according to the instructions provided by the design, to the same strength class. The homogeneity of the concrete was confirmed by the results.

Three positions for each structural element were tested by means of the following methods:

- ultrasonic pulse test (frequency 50 KHz);
- surface hardness test (sclerometer Schmidt type N);
- penetration test (Windsor Probe System)

in accordance with the ASTM Standards.

In order to assess the strength, the results of the first two methods were combined according to the following relation [8]:

$$R_c = 7.55 \times 10^{-11} \times I_r^{1.4} \times V_L^{2.6} \tag{14}$$

where R_c is the compressive cubic strength in $N \cdot mm^{-2}$, I_r the rebound index and V_L the ultrasonic pulse velocity in $m \cdot sec^{-1}$ (Fig. 2). Such relation reproduces the constant strength curves proposed in [6] for a concrete with standard mix proportions. The estimate of the compressive cubic strength by means of the penetration test was carried out on the basis of the tables provided by the builder. The results are reported in Fig. 3. Thirty structural elements were tested in all, to each of which we

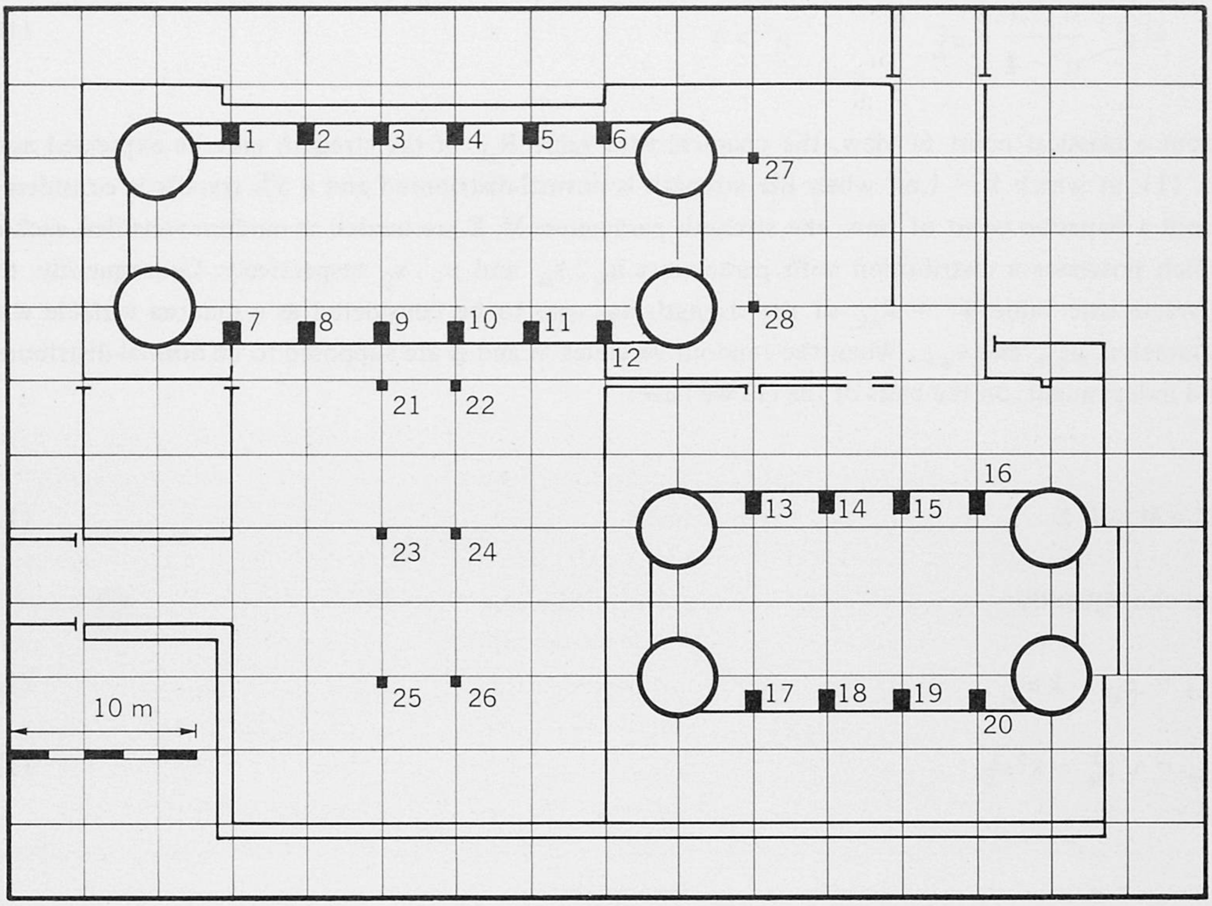


Figure 1

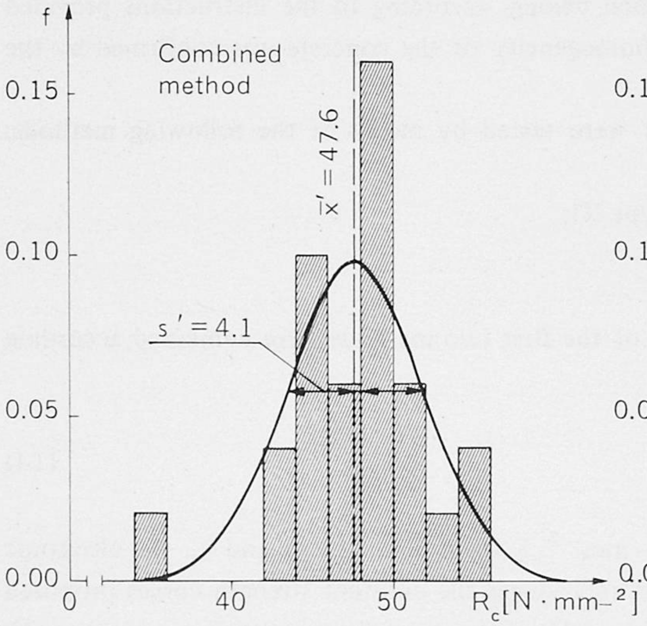


Figure 2

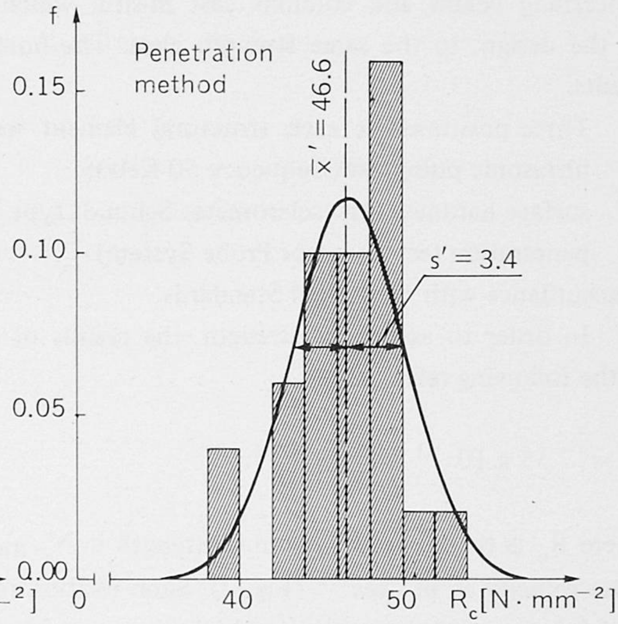


Figure 3

associated the two strength values estimated on the basis of the values of the non-destructive parameters calculated as means of the values determined in the three positions. The results are reported in Figures 2 and 3 in terms of histograms and respective frequency density curves.

By observing the synthetic parameters $\overline{x}'$ and s' relative to the two methods, almost equal mean values and more differentiated standard deviations may be noted. In particular, in the case of the penetration test, the standard deviation is lower since, in general, the method shows a lower sensitivity to the strength variations with respect to the combined method. For both methods, the values of s' are rather low, since the concrete is of good quality. In fact, as well-known, the sensitivity of the non-destructive methods decreases as the strength increases, even if in a different manner.

Besides the determinations by means of the non-destructive methods, four cores with size $\emptyset$ 15 x 30 cm were drawn which, in a second time, were subject to compressive tests. The strength values thus determined, transformed into cubic strength values, have provided the following synthetic parameters: $\overline{x} = 47.2\ N \cdot mm^{-2}$, $s = 1.9\ N \cdot mm^{-2}$.

Figures 4 ÷ 9 report the distributions of the parameters μ_M, s_M, μ_Σ, s_Σ, $\mu_{\Psi*}$, $s_{\Psi*}$ with respect to n' for different values of n estimated by means of the combined method. On the basis of Figures 4 and 6, it may be noted that, as n' increases, the curves tend to the values of the prior distribution parameters $\overline{x}'$ and s' while, as n increases, they approach the values of the sample parameters $\overline{x}$ and s. Figures 5 and 7 show that the uncertainty level of the true value of the parameters M and Σ decreases as the information defined by $n'' = n + n'$ increases. Figures 10 and 11 report the comparison between the values of the parameters μ_M and μ_Σ relative to the two testing methods considered for n = 1, 4, 20. It may be noted that the influence of n and n' in the case of the penetration test is analogous to that already found in the case of the combined method.

REFERENCES.

1. G., PASCALE, *Non-destructive testing methods of concrete,* INARCOS, No. 413, November 1980 (In Italian).
2. A., DI LEO, *Evaluation of concrete elastic properties by means of vibration methods,* INARCOS, No.415, Jenuary/February 1981 (In Italian).
3. A., DI LEO, G., PASCALE, *Strength evaluation of in-situ concrete by means of non-destructive methods,* INARCOS, No.415, January/February 1981 (In Italian).
4. A., DI TOMMASO, A., DI LEO, G., PASCALE, *Procedures to estimate mechanical properties of concrete structures' materials in buildings damaged by earthquakes,* Emilia-Romagna Region - Basilicata Region - University of Bologna Convention (in press). (In Italian).
5. E., VIOLA, *On the Bayesian approach to estimate compressive strength of concrete in-situ,* ICASP 4, Florence, 13 - 17, June 1983.
6. F., CIANFRONE, I., FACAOARU, *Study for the introduction in Italy of the combined non--destructive method for in place evaluation of the concrete strength,* LIPE, No. 6-7, August 1979 (In Italian).
7. J.R., BENJAMIN, C.A., CORNELL, Probability, Statistics and Decision for Civil Engineers. M. Graw-Hill, New York (1970).
8. R., GIACCHETTI, V., LACQUANITI, Technical note, I.S.T.C., University of Ancona (1980). (In Italian).

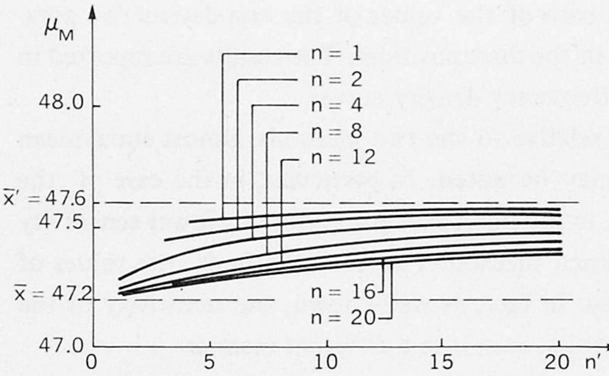


Figure 4

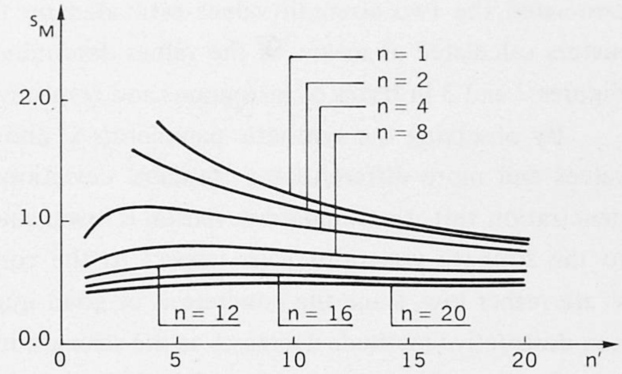


Figure 5

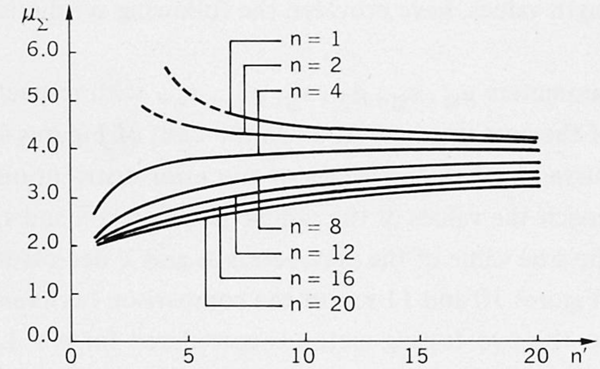


Figure 6

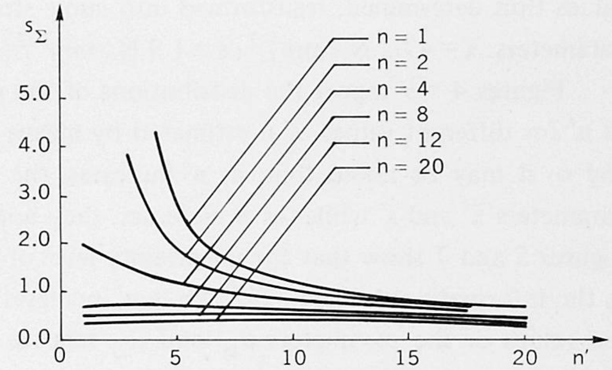


Figure 7

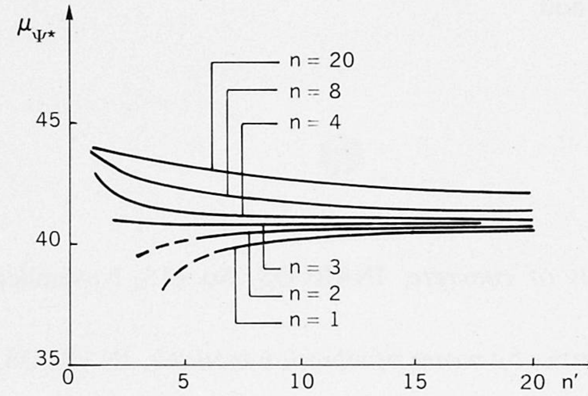


Figure 8

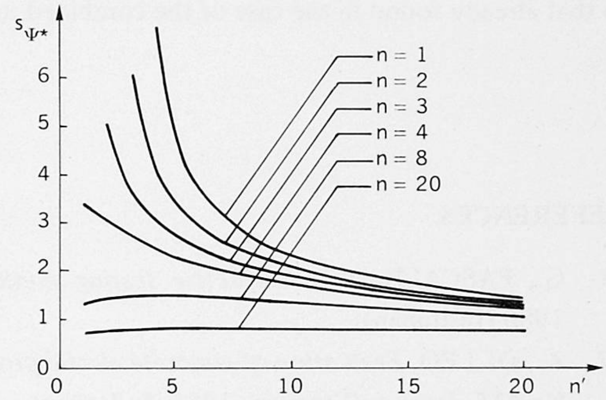


Figure 9

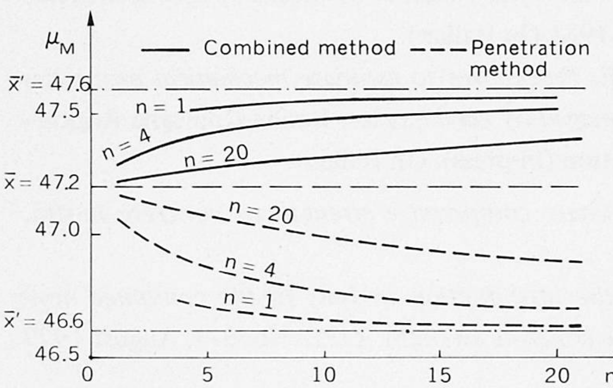


Figure 10

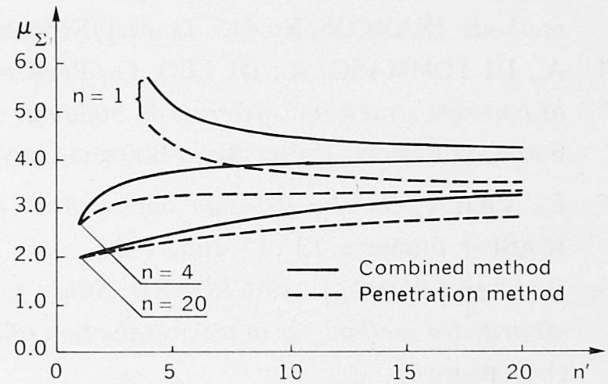


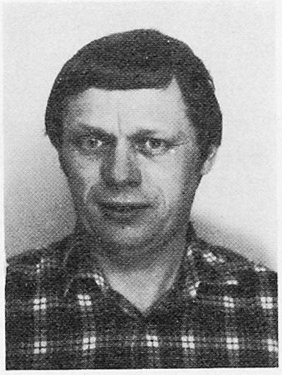
Figure 11

Investigation of Concrete by Analysis of Thin Sections

Examen du béton par analyse de préparations en couche mince

Untersuchung von Beton durch Analyse von Dünnschliffpräparaten

Arne D. JENSEN
Civil Engineer
Technological Institute
Taastrup, Denmark



Arne Damgaard Jensen, born 1942, received his civil engineering degree at the Technical University of Denmark, Copenhagen, 1969. For three years he was involved in the construction of concrete bridges as a consulting engineer. Since 1972 at the Technological Institute, Copenhagen, dealing with inspection and investigation of concrete structures.

SUMMARY

This paper deals with the investigation of damaged concrete and with quality control of concrete. The technique of using microscopy on thin sections is described and results of such microanalyses are listed.

RESUME

L'exposé traite d'études faites sur des bétons endommagés et du contrôle de la qualité des bétons. La technique d'analyse au microscope de préparations en couche mince est décrite et les résultats de ces microanalyses sont résumés.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die Untersuchung von schadhaftem Beton und zeigt Erfahrungen mit der Qualitätskontrolle von Beton auf. Die mikroskopische Analyse von Dünnschliffpräparaten wird beschrieben und die Ergebnisse dieses Verfahrens werden zusammengefasst.

1. INTRODUCTION

1.1 Normal practice

In connection with investigations of concrete buildings and other concrete constructions showing sign of deterioration after influence of the weather or other harmful action, it is normal to set up the conclusions about the type and the extent of the deterioration only based on macroscopical observations, maybe various NDT-tests, some empirical results from strength testing and chemical analyses.

The background for carrying out restoration works therefore often has been doubtful. This has led to wrong selection of repair methods and unexpected high costs.

1.2 Micro analysis

By microscopical analysis of thin sections it is possible to look inside a hardened concrete, thus obtaining a precise picture of the concrete composition and the condition of the concrete.

The technique has been developed in Switzerland. In Denmark it has been used since 1977. It is the Technological Institute which has imported and further developed some of the principles.

Micro analysis of thin sections can be used for:

- stipulation of concrete quality (young and old concrete)
- investigation of damaged concrete, mortar, tiles, bricks, natural stones, etc.)
- estimation of new products (fibre reinforced concrete, concrete with silica dust, etc.)
- improvement of concrete
- quality control of concrete, etc.

1.3 Quality control

From 1982 micro analysis has in some cases been used as a test method for quality control of newly cast concrete constructions. From the construction, a concrete element, a pavement area, a part of a bridge, one or several cores are drilled. From each core there are prepared two thin sections, which are analysed in the microscope. The following parameters are investigated: porosity, pore structure, cracks, homogeneity of the cement paste, bleeding tendency and entrapped air. An overall investigation of for instance cement type, composition can easily be made. The result of the analysis will be available one to two weeks after casting.

2. CONCRETE INVESTIGATION

2.1 General

The normal investigations of concrete structures are as follows:

- visual inspection of the structure
- examination of data on the concrete, the structure and the environment
- visual inspection of the cores, sawn longitudinally (plane cut)
- crack detection on plane cut
- air pore analysis on plane cut
- micro analysis on thin sections.

In situ testing by NDT-tests, strength testing on cores, chemical analysis, SEM and X-ray analysis, etc. may be carried out, if necessary.

2.2 Macro analysis

This analysis will reveal features about the concrete. The core is sawn longitudinally. On the plane cut the following can be estimated: stone type, stone quantity, stone segregation, type of sand fraction, colour of paste, possible air entraining, entrapped, homogeneity of mortar, effect of compaction, visible cracks, carbonation, crystallization on the surface, etc. See Fig. 1.

A more detailed knowledge of the cracks can be obtained by crack detection on the plane cut. The cut is impregnated with a fluorescent epoxy and polished: In UV-light cracks can be clearly seen now. See Fig. 2.

Information about the air pore structure is obtained by automatic image analysis of a contrast impregnated plane and polished cut. The information can be air volume, specific surface and spacing factor. See Fig. 3.

2.3 Micro analysis

The micro analysis is carried out by microscopical investigation of fluorescent impregnated thin sections as described in the next paragraph.

3. MICRO ANALYSIS

3.1 Equipment

3.1.1 Thin sections

A thin section is a 20 µm thin slide of concrete, glued on to a glass plate and covered by a cover glass. The area is normally about 50 x 30 mm. See Fig. 4.

The thin sections are prepared by grinding - step by step - an epoxy impregnated piece of concrete. A fluorescent pigment is added to the epoxy. The concrete is dried by saturation in alcohol and by evaporation to avoid crack propagation.

3.1.2 Microscope

Polarization and fluorescent microscopes are used for the analysis of the thin sections.

In the polarization microscope the crystalline components are determined, i.e. aggregates, cement particles, cement gel, different crystalline products as ettringite, calcite, calcium hydroxyde, etc. Air pores, alkali-silica gel, etc. can be identified.

In the fluorescent microscope the porosity of the different components can be identified due to the fluorescent impregnation. It is possible to see cracks down to a few µm as well as the capillary porosity of the cement paste.

3.2 The analysis

Figs. 5 - 12 show different photos from the microscope.

3.2.1 Cement paste

The volume of the cement paste in the hardened concrete is the volume of the water and the cement in the mix. The amount of cement paste can be measured by point counting, and the water-cement ratio in the hardened concrete can be determined in the fluorescent microscope. From this the original cement content can be determined.

By measuring the air content (point counting) and using the Férét formula, the strength of the concrete can be determined optically.

Normally, the cement type can be determined. The nature and the extent of different unnatural recrystallizations can be seen. A possible addition of fly ash or silica dust to the concrete can be seen. The homogeneity of the cement paste and silica dust dispersion, the formation of calcite, ettringite, alkali-silica gel, etc. can be estimated.

3.2.2 Water-cement ratio

The Technological Institute has developed a fluorescent microscopical method to estimate the water-cement ratio in a hardened concrete. In the fluorescent microscope the cement paste will shine because of the fluorescent pigment in the capillary pores. High capillarity will give a bright colour and low capillarity will give a dark colour. The method is very accurate for water-cement ratios between 0.40 and 0.60. On the basis of the homogeneity of the capillarity the efficiency of the concrete mixer can be estimated.

3.2.3 Air pore system

The air pore system can be seen in the microscope and the homogeneity can be estimated.

A concrete affected by humidity, water, dissolved salts, etc. is slowly (sometimes rapidly) transformed into ettringite, calcite, alkali-silica gel, etc. and the transformation products can often be seen in the air pores. This results in a deterioration of the concrete and a reduction of the free air pore volume, giving a reduced frost resistance and tightness.

3.2.4 Cracks

Normally, concrete contains cracks which can be observed in the fluorescent microscope. The cracks are caused by bleeding, plastic settlements, shrinkage, temperature, loading, different deteriorating actions as frost, alkali-silica reactions, sulphate attack, etc. Often the type of action causing the crack can be estimated. The cracks can be filled with the same products as the air pores and with rust from corrosion of the reinforcement.

3.2.5 Aggregates

A normal thin section contains up to about 10 coarse aggregates and 2000-3000 fine aggregates (sand particles). The ingredients of the aggregate can be estimated and it can be seen, if the aggregate has given rise to harmful deterioration in the concrete.

3.2.6 Various

The microscopical analyses of thin sections can be used to investigate different materials as mortars, tiles, bricks, natural stones, etc. This can be done in connection with investigations of deterioration, with determinations of quality and with quality control.

4. CONCLUSIONS

Up till now the Technological Institute has made about 4000 thin sections for microscopical analyses of concretes, mortars, tiles, bricks, natural building stones, clinkers, etc.

Project work is done in connection with the use of silica dust in concrete, fibre reinforced concrete and mortar, maintenance of natural building stones and monuments, etc.

The method is necessary for an internal investigation of the different materials both in connection with investigation of damages and in connection with quality control of new products, including control with repair works and restorations.

Fig. 1. Sawn cut of a concrete core with internal crack and low stone content.

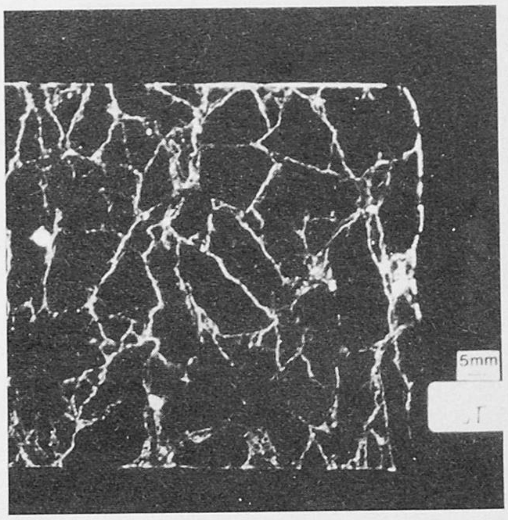


Fig. 2. Crack detection on prepared sawn cut. Heavily cracked concrete.

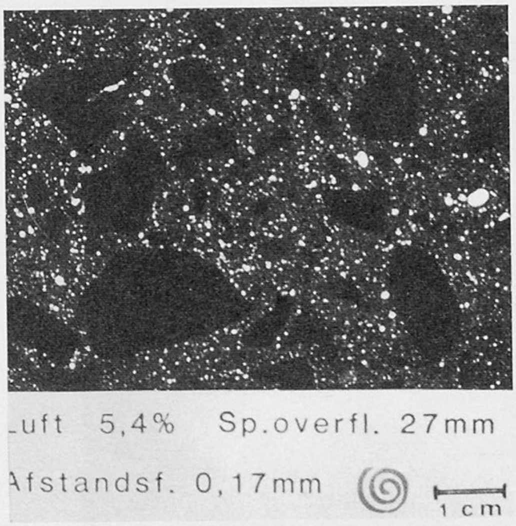


Fig. 3. Air pore analysis on contrast impregnated plane cut. A concrete with good air pore distribution.

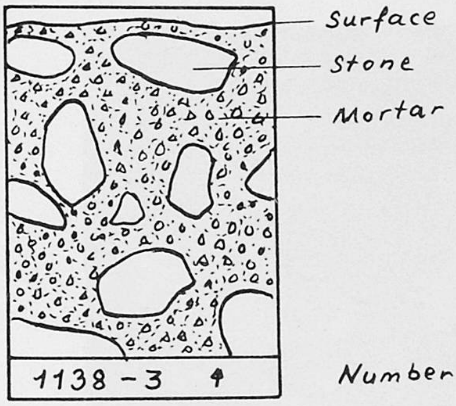


Fig. 4. Sketch of a thin section, 1:1.

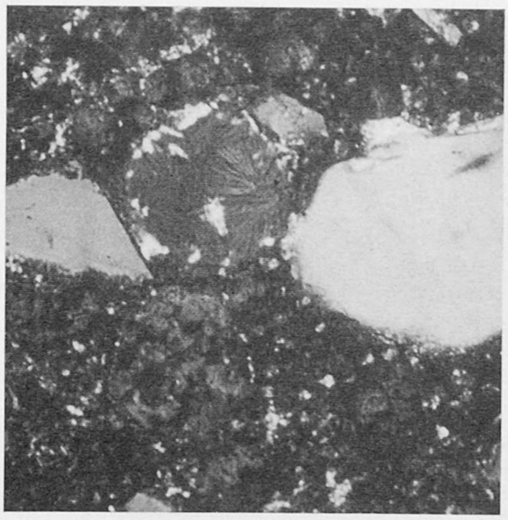


Fig. 5. Polarization. 160x.
Air pore filled with $Ca(OH)_2$ and ettringite. $Ca(OH)_2$-crystals in the cement paste.

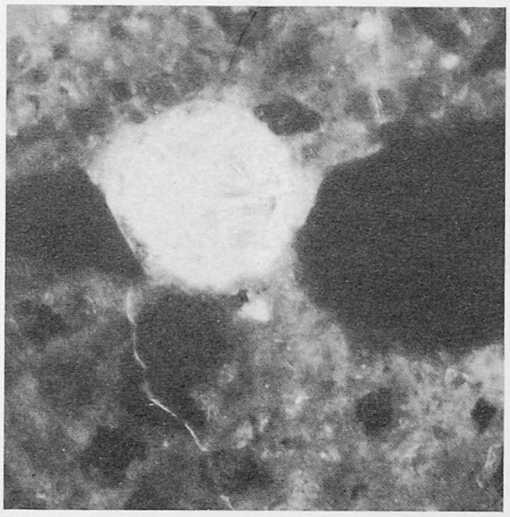


Fig. 6. Fluorescence. 160x.
The same area as Fig. 5. Small cracks in the cement paste.

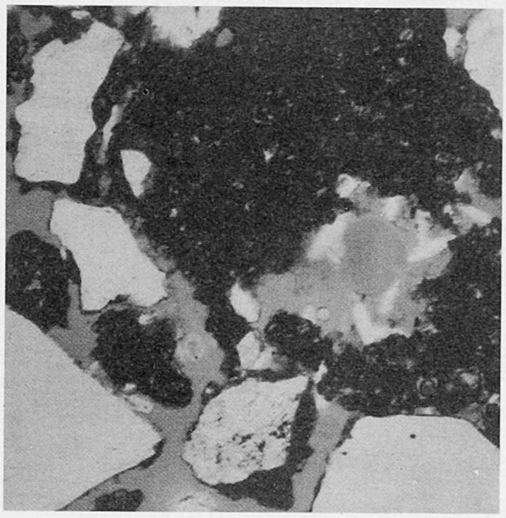


Fig. 7. Polarization. 65x.
Badly mixed cement mortar with cement paste lumps and entrapped air. Big $Ca(OH)_2$-crystals in the air pores.

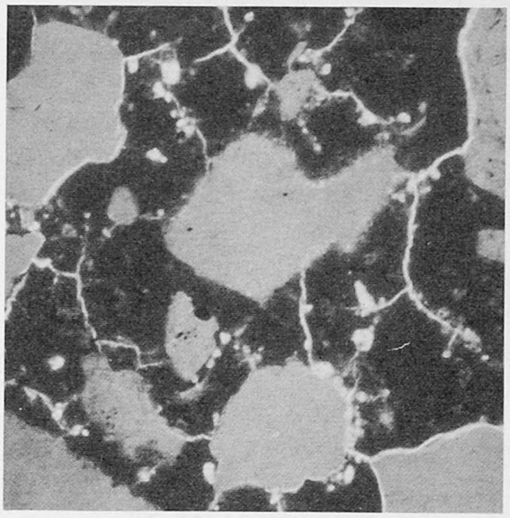


Fig. 8. Fluorescence. 65x.
Dense cement paste with many micro cracks in a concrete containing silica dust.

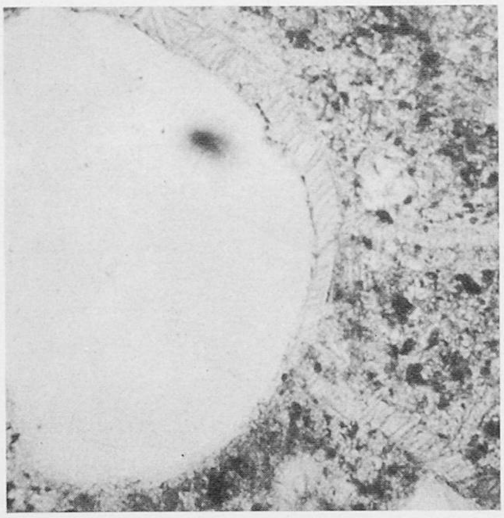


Fig. 9. Polarization. 160x.
Micro cracks filled with ettringite.

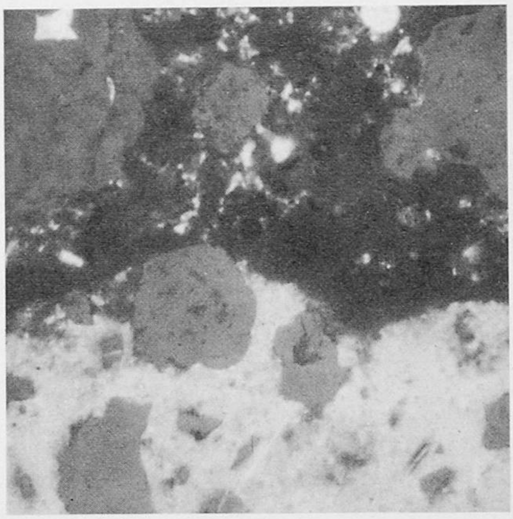


Fig. 10. Fluorescence. 65x.
Joint between old (porous) and new (dense) concrete.

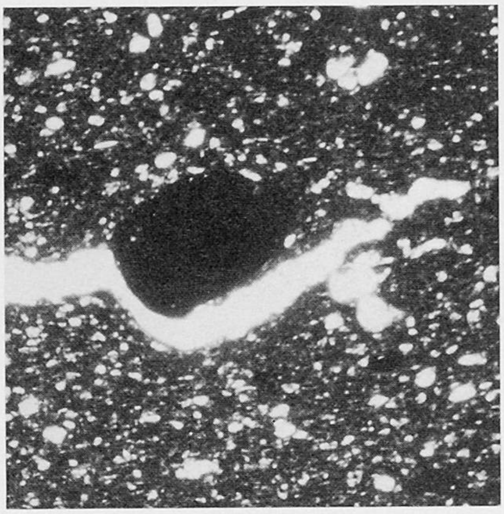


Fig. 11. Polarization. 65x.
Production crack in a tile with bad frost durability.

Fig. 12. Fluorescence. 65x.
Surface parallel cracks in a clay containing limestone used as building stone.

Leere Seite
Blank page
Page vide

Thermographie infrarouge appliquée à des bâtiments anciens

Anwendung der Infrarot-Thermographie bei Altbauten

Application of Infrared Thermography to Old Buildings

Daniel LAURENS
Assistant
Laboratoire Régional, C.E.T.E.
Aix-en-Provence, France

Daniel Laurens, né en 1937, a débuté sa carrière au Laboratoire Régional des Ponts et Chaussées en 1963. Il est depuis 1975 responsable de la section Métrologie où sont utilisées les méthodes les plus modernes.

RESUME

Parmi toutes les méthodes non destructives d'investigation, la thermographie infrarouge a fait ses preuves dans les domaines militaire et médical, mais aussi dans l'évaluation de la qualité de l'isolation des bâtiments. Cette méthode peut être appliquée à des bâtiments anciens pour mettre en évidence certaines particularités ou certains désordres.

ZUSAMMENFASSUNG

Unter den zerstörungsfreien Prüfverfahren hat sich die Infrarot-Thermographie im militärischen wie auch im medizinischen Bereich bewährt. So auch bei der qualitativen Einschätzung der Wärmeisolierung von Gebäuden. Dieses Prüfverfahren kann ebenfalls bei Altbauten eingesetzt werden, um gewisse Besonderheiten oder Schäden hervorzuheben.

SUMMARY

Among the methods of non destructive testing, infrared thermography has proved its worth not only in the military and medical fields, but also in the evaluation of the quality of building insulation. This method can be applied to old buildings to show up certain irregularities or defects.

I. GENERALITES

Le rayonnement infrarouge émis par une structure peut fournir certains renseignements sur l'agencement de cette structure et son histoire récente.

Les ondes électromagnétiques du spectre infrarouge se situent entre les ondes visibles et les ondes radio. Le spectre infrarouge est lui même divisé en trois grandes régions :

U.V	visible	Infrarouge proche	Infrarouge moyen	Infrarouge lointain	onde radio

0,35 0,75 1,5 20 1 000

Longueurs d'ondes en microns.

Le soleil dont la température apparente est de 6000°K à son maximum d'émittance spectrale à max = 0,5 micron ; un objet à la température ambiante T 290°K présente un maximum pour max = 10 microns. Ceci implique l'utilisation de détecteurs pouvant fonctionner dans l'infrarouge moyen.

2. APPAREILLAGES

Les systèmes infrarouge disponibles sur le marché sont équipés de détecteurs sensibles dans la gamme 3 à 5 microns ou 8 à 14 microns. Ces détecteurs capables de convertir le rayonnement reçu en signaux électriques sont associés à des systèmes de balayage qui scrutent la scène observée point par point à une très grande vitesse.

Ce dispositif permet après traitement des signaux électriques de restituer sur un écran cathodique une image thermique où les zones froides apparaissent en noir et les zones les plus chaudes en blanc avec, entre ces deux extrêmes les teintes de gris intermédiaires.

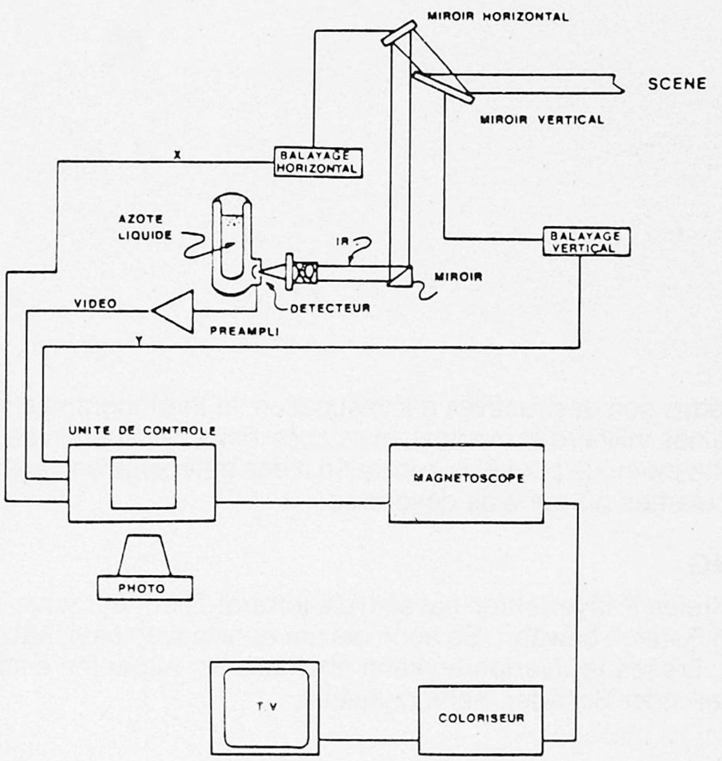


Fig.1 Schéma de fonctionnement

L'image thermique peut être photographiée directement sur l'écran cathodique, mais elle est aussi enregistrée sur magnétoscope. L'avantage de cette méthode est de pouvoir visualiser la bande enregistrée sur un moniteur T.V couleur, de grande dimension à travers un coloriseur permettant de faire apparaître les isothermes en huit couleurs.

3. APPLICATIONS

Le flux rayonné par une structure observée peut fournir de nombreux renseignements sur cette structure. L'application la plus connue de la thermographie appliquée au bâtiment est la détection des pertes d'énergie dues à des défauts d'isolation ou à des ponts thermiques. Le rayonnement de surface est aussi influencé par la composition même de la structure. Certaines hétérogénéités s'opposent à l'écoulement du flux ou au contraire peuvent le favoriser. L'examen de surface peut révéler ces hétérogénéités qui délimiteront des zones rayonnant de façon différente.

Une ouverture murée dont le remplissage est moins dense que les matériaux environnants se détachera sur l'image, un enduit décollé enfermera une lame d'air qui s'opposera au cheminement du flux. La présence d'humidité se traduit par une évaporation de surface qui crée des points froids sur la zone considérée.

Ces particularités apparaîtront d'autant mieux que les gradients de température seront importants. On considère qu'une différence Intérieur/Extérieur de 20°C est satisfaisante ; ceci implique que les mesures soient faites par temps froid et que les bâtiments soient chauffés.

4. RESULTATS OBTENUS

L'investigation effectuée sur les palais communaux de RIMINI fournissent des exemples des possibilités de la méthode. Les photos des pages suivantes illustrent certaines particularités relevées. Les résultats sont qualitatifs mais la souplesse d'emploi du matériel permet de faire très rapidement une évaluation à distance, non destructive qui pourrait être utilisée pour optimiser d'autres mesures ou d'autres contrôles.

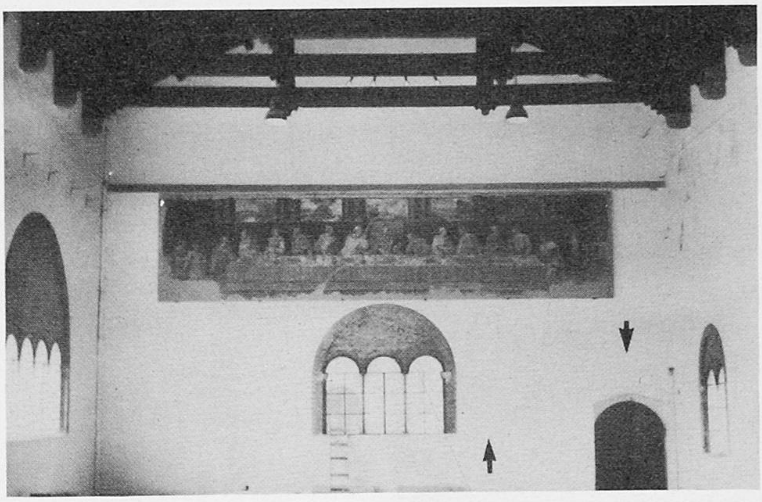


Fig. 2 Vue intérieure

La photo N°2 montre le mur Ouest de la Salle de Conseil. Sur cette photo sont repérés les emplacements des particularités relevés en infrarouge.

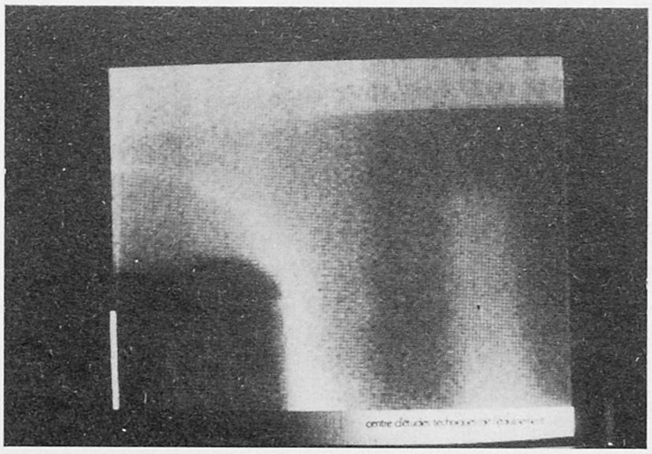


Fig. 3 Conduit de chauffage

De part et d'autre de la fenêtre située sous la fresque, on peut voir en infrarouge des marques claires verticales qui correspondent à d'anciens conduits de chauffage qui sont aménagés dans la maçonnerie. L'air enfermé dans ces cavités s'oppose à l'échange thermique avec la salle et en ces points la structure apparait plus chaude.

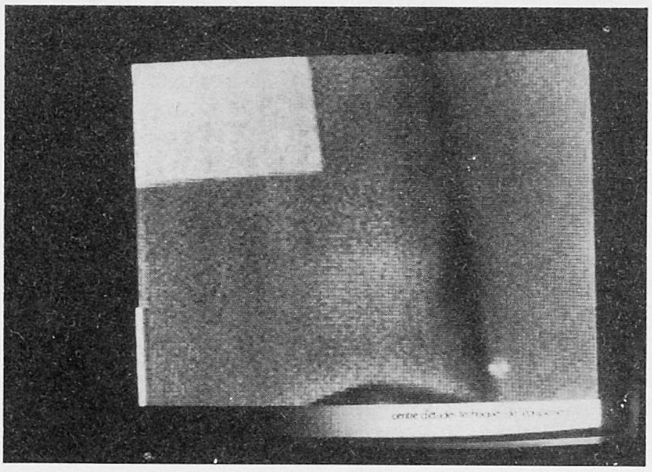


Fig. 4 Ouverture murée

Au-dessus de la porte repérée sur la figure N°2, l'image infrarouge révèle une hétérogénéité dont la forme fait penser à une ancienne ouverture murée

Fig. 5 Mur Nord dans le visible

La photographie du mur Nord dans le visible montre une marque d'humidité sur la partie gauche du document. Le reste de la structure semble sain.

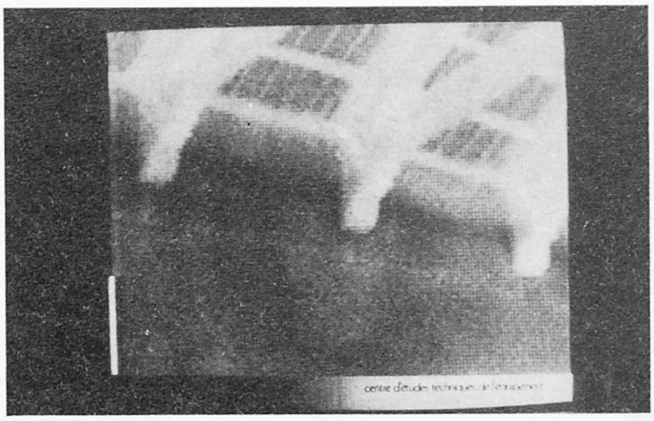


Fig. 6 Thermogramme du mur Nord

Les clichés infrarouge sur les mêmes parties de la structure révèlent la présence d'une trace rectiligne entre les corbeaux.
Cette trace très bien délimitée fait penser à une poutre.

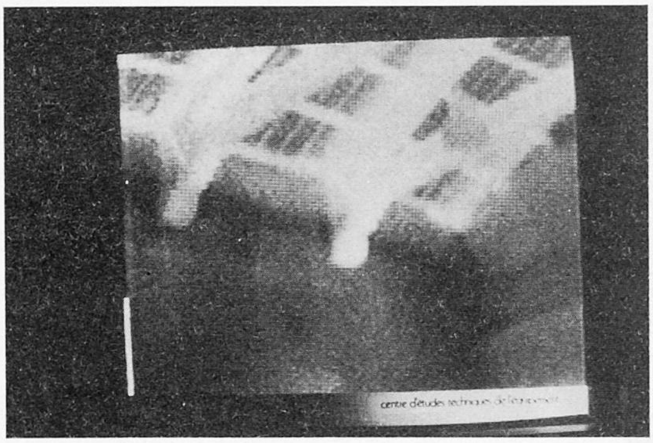


Fig.7 Extrémité du mur Nord

On note que la marque observée se prolonge jusqu'au mur Est.
La présence d'une zone sombre dans l'angle de la pièce révèle que cet angle est pollué par l'humidité.

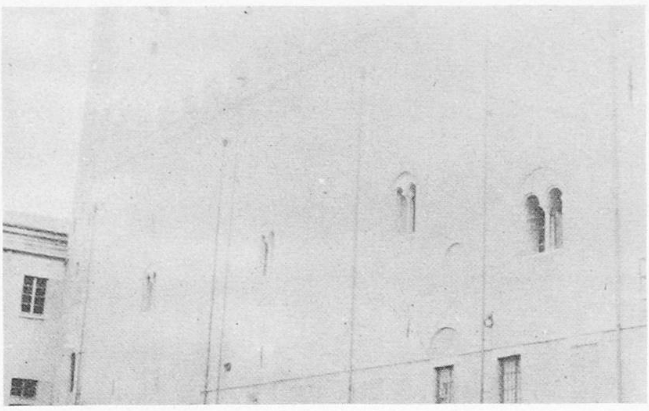


Fig. 8 Façade Nord

La photo ci-contre représente la façade Nord de l'édifice dont la Salle de Conseil occupe toute la partie supérieure.
L'angle de la pièce montré sur l'image infrarouge précédente correspond à l'emplacement de la gouttière la plus à gauche.

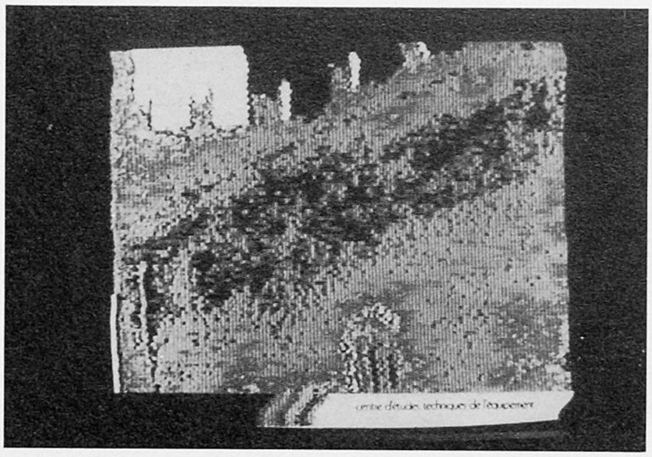


Fig.9 Traces d'humidité

La prise de vue infrarouge sur cette partie de la façade montre une zone sombre, localisée dans la partie supérieure du mur, qui révèle la présence d'humidité et confirme les constatations faites à l'intérieur.

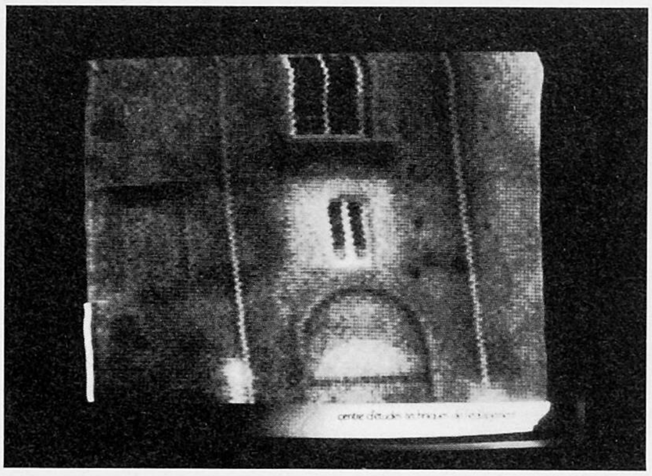


Fig.10 Différence de structure

Sur la même façade, ce point qui rayonne de façon importante correspond à un couloir de liaison entre deux bâtiments.
La nature du mur est certainement différente et il y a un point de chauffage sous la fenêtre.

Stress and Deformability in Concrete and Masonry

Contrainte et déformabilité dans le béton et la maçonnerie

Spannungszustand und Verformungsfähigkeit von Beton- und Backsteinbauwerken

Charles ABDUNUR
Dr. Sc.
Lab. Central Ponts & Chaussées
Paris, France

After receiving his civil engineering degree, Charles Abdunur worked for five years on the design and supervision of several big projects. He prepared his doctorate and obtained it in 1974. Since then, he directs several research programs at a civil and structural engineering laboratory.

SUMMARY

In the assessment of concrete or masonry structures, it is essential to determine their absolute stresses and deformability. A method by partial stress release and compensating flat jack pressure was developed. After miniaturisation and improvement, this method is now accurate and operational. It can directly measure the actual absolute stress, separate its components, situate the general state of the assessed medium and estimate its elastic properties and bearing capacity.

RESUME

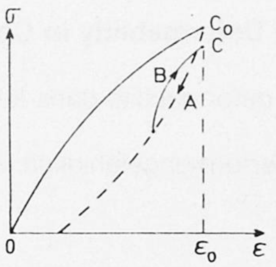
Dans l'auscultation des ouvrages, en béton ou en maçonnerie, il est essentiel de déterminer leurs contraintes absolues et leur déformabilité. Une méthode par libération partielle et compensation par vérin plat a été développée. Après miniaturisation et amélioration, cette méthode est désormais précise et opérationnelle. Elle peut mesurer directement la contrainte absolue réelle, séparer ses composantes, situer l'état général du milieu ausculté et estimer ses caractéristiques élastiques et sa portance.

ZUSAMMENFASSUNG

In der Beurteilung von Beton- oder Backsteinbauwerken ist es wesentlich, ihre absoluten Spannungen und die Verformungsfähigkeit zu bestimmen. Eine Methode mit flachen Druckmessdosen wurde entwickelt. Nach Miniaturisierung und Verbesserung ist diese Methode jetzt genau und einsatzfähig. Hierdurch ist es möglich, die wirkliche absolute Spannung direkt zu messen und in ihre Komponenten aufzuteilen, den allgemeinen Zustand des untersuchten Bereichs zu beurteilen und seine elastischen Eigenschaften und Tragfähigkeit zu bewerten.

1. INTRODUCTION

Engineers often encounter concrete and masonry structures showing signs of damage or, in any event, requiring strengthening. In such cases, the determination of the actual state of absolute stress is essential for diagnosis, for the evaluation of the residual strength and for stress control during an eventual repair operation. However, as these absolute stresses cannot possibly be determined through mere strain measurements, a direct method by partial stress release was considered. It consists in a local elimination of stresses, followed by a controlled stress compensation (fig. 1). In practice, the displacement reference field is first determined; a slot is then cut in a plane normal to the desired direction of stress determination; finally, a very thin flat jack is introduced into the slot and used to restore the initial displacement field. The amount of cancelling pressure is an indication of the compressive stress in the direction normal to the slot.



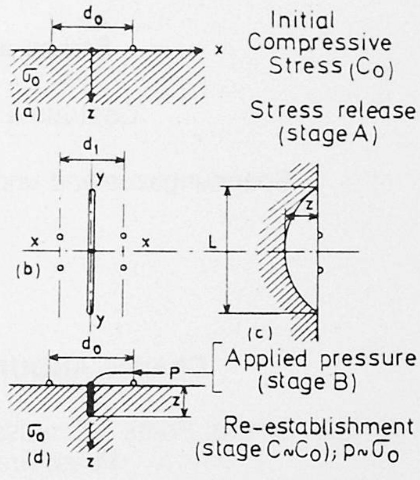


Fig. 1
Different stages of stress measurement

2. APPLICATION TO CONCRETE; EXTENSION TO MASONRY

Although this principle was easily applied in rock mechanics, its mere transfer to concrete has been repeatedly disappointing for two main reasons: The difficult nature of concrete as a material and the complications arising from the absolute need for the miniaturisation of this somewhat destructive operation.

The extension to masonry has raised specific problems. In such a composite material, with fragile bond and complex stress distribution, the main concern is the stress representativeness at the point of measurement and the reversibility of the stress - strain relationship under the cancelling pressure.

3. ORIENTATION OF THE RESEARCH

To overcome these difficulties, the method was thoroughly re-examined from all its basic aspects. This led to several general and specific imperatives.

For both materials:

- Behaviour analysis of the slot vicinity from a purely mechanics viewpoint.
- Dry cutting and simultaneous thermal stability to avoid distorted results.

For concrete:

- Miniaturisation, considered absolutely imperative, but resulting in measurements increasingly sensitive to heterogeneity, hair-cracking, cut clearness and, above all, to the quality of the jack.
- Evaluation of tensile stresses occuring in zones where the applied compression is sufficiently low to allow the prevailance of internal stresses (mainly tensile).
- Break down of the measured absolute stress to its two main components: applied and internal, the latter being rather complex.
- Correction for cutting through reinforcement during stress release.

For masonry:

- Location of points, on the stone-joint assemblage, where measurement may best reflect the actual average stress.
- Keeping the stone-joint disturbance to a minimum.

4. BASIC OBJECTIVES

- To analyse the disturbance in the displacement, strain and stress fields induced by the presence of a slot in a medium subject to known stresses.
- To evaluate the restoration of these fields, using an ultra-fine jack.
- To establish a correlation between the cancelling pressures in the jack and the initial stresses.

The governing parameter is the depth of the slot.

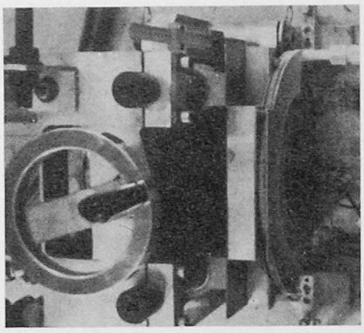


Fig. 2
Cutting machine

5. EXPERIMENTAL PROGRAM

It followed three lines of attack where different tests often had to overlap:

- Development of specific instruments and procedures.
- Tests on plexiglass, using photo-elasticity and Moiré, to study the effect of the slot in a homogeneous and isotropic material.
- Tests on concrete and masonry models, using mechanical and electric gauges, to simulate actual cases and study local effects.

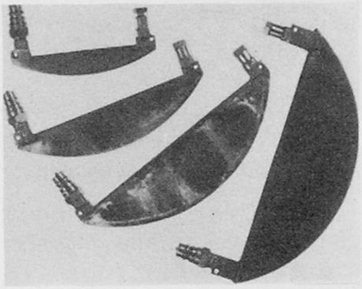


Fig. 3
Flat jacks

6. DEVELOPMENT OF THE EQUIPMENT

The equipment included the release and cooling apparatus, the flat jack and the measurement apparatus.

Cutting is carried out by successive passes which alternate with measurements. A special apparatus was constructed for this purpose (fig. 2). A clear cut, with a very uniform thickness, was thus obtained. Furthermore, a special cooling system was developed to favour dry cutting; a good thermal stability was achieved. The flat jacks had to reconcile miniaturisation with strength and flexibility. Prototype jacks (fig. 3) were designed by the author and successfully tested in the laboratory. They are 4 mm thick and have a maximum depth of 60 mm. In the measurement apparatus (fig. 8), the displacement field is materialised by a set of mechanical gauge bases. It is complemented by strain gauges forming the strain field for instantaneous measurements. Immediately prior to cutting, the readings taken of these two sets constitute their respective initial reference fields.

7. THE IDEALIZED MODEL

The surface behaviour of a three-dimensional model was analysed on a series of plexiglass blocks by two complementary optical methods.

7.1 Qualitative analysis

A plexiglass block, containing a slot and provided with photo-elastic coating, was subjected to pure compression in successive stages. Fig. 4 a reflects the extent of the disturbance in the stress field. Fig. 4 b, on the other hand, shows the high degree of its re-establishment, using a prototype flat jack. At each stage of loading, the cancelling pressure proved to be exactly equal to the corresponding applied stress. Other tests on similar models have shown the possibility of measuring absolute bi-axial stresses.

a)

c)

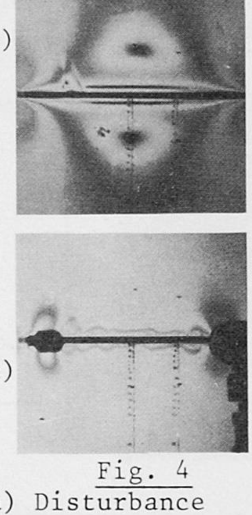


Fig. 4
a) Disturbance
b) Restoration

a)

b)

c)

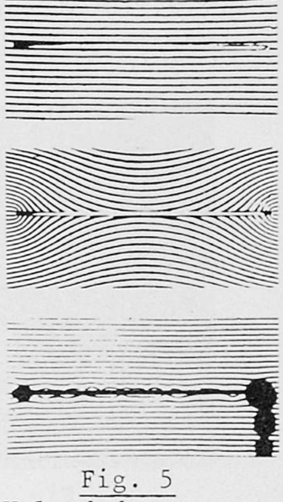


Fig. 5
a) Unloaded
b) External stresses
c) Superposed equivalent jack pressure

7.2 Quantitative analysis

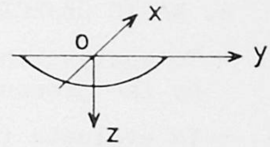
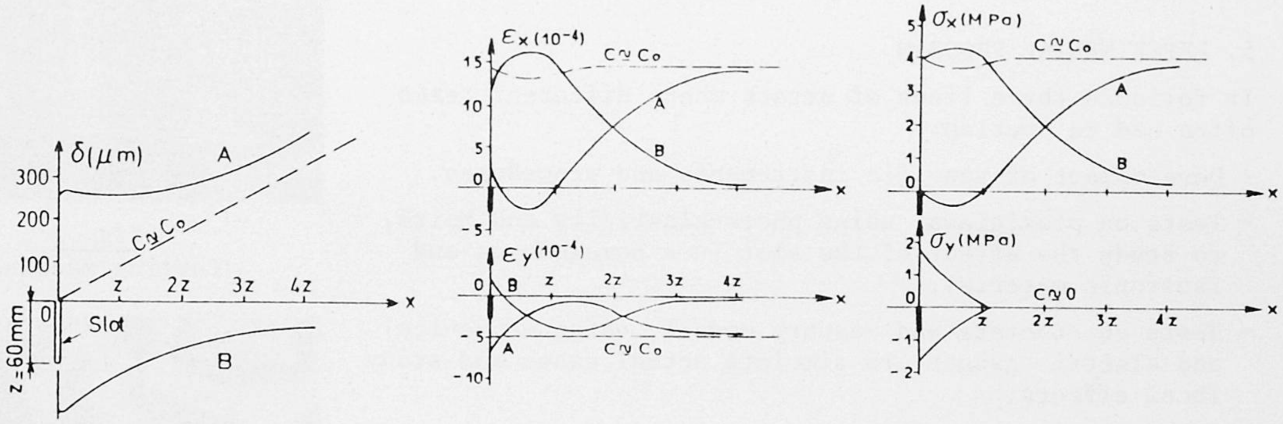


Fig. 6
Co-ordinate axes

Another block, containing a slot, was equipped with a double Moiré-grating (fig. 5 a). It was subjected to pure compression, then a uniform jack pressure was superposed in the slot. Fig. 7 a, b, c show the results obtained and the efficacy of the flat jack in restoring the field. These results also confirm and complement those of a calculation carried out previously by finite elements.



a) Displacements; assumed positive towards the slot

b) Strain; contraction assumed positive

c) Stresses; compression assumed positive

Fig. 7 - Displacement, strain and stress distribution (along an axis normal to the plane of the slot) obtained on a plexiglass model by the Moiré technique. Cases of loading: A - Pure compression (σ_x = 4MPa); B - Pressure (4MPa) in the slot; C - Combination of A and B; C_o - Pure compression in a model without a slot.

8. TESTS ON CONCRETE MODELS AND ACCURACY OBTAINED

Now that the theoretical, technological and metrological problems are resolved, the way is paved for application.

8.1 Compressive stresses

The tests undertaken on plexiglass by optical methods, were repeated on uncracked concrete models by extensometry (fig. 8). They were carried out within the elastic limit and at different slot depths. The results were in close agreement with those already obtained on the idealised models. Fig. 9 shows the essentials of this study, i.e. for a model subjected to pure compression, a maximum error margin of 0.3 MPa was found between the cancelling pressures and the actual applied stresses.

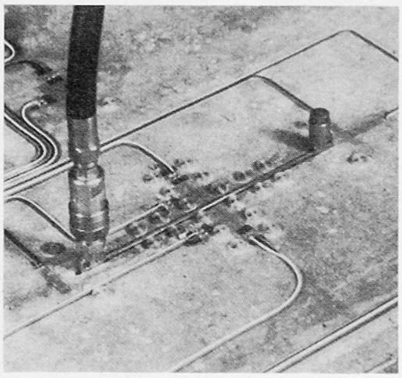


Fig. 8
Measurement apparatus on concrete

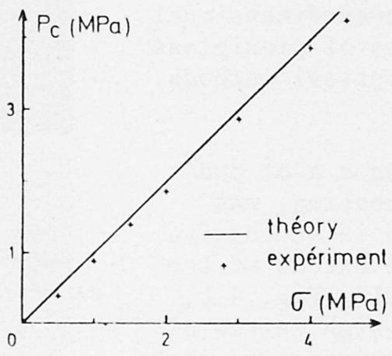


Fig. 9
Cancelling pressure compared with actual applied stresses

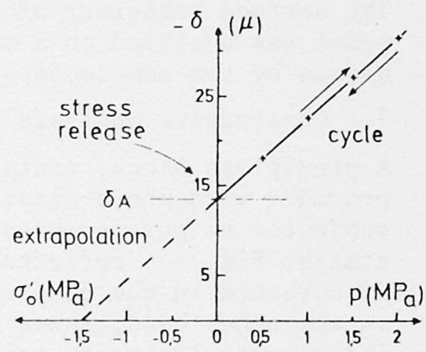


Fig. 10
Tensile stress deduction from a pressure cycle p and corresponding displacements δ

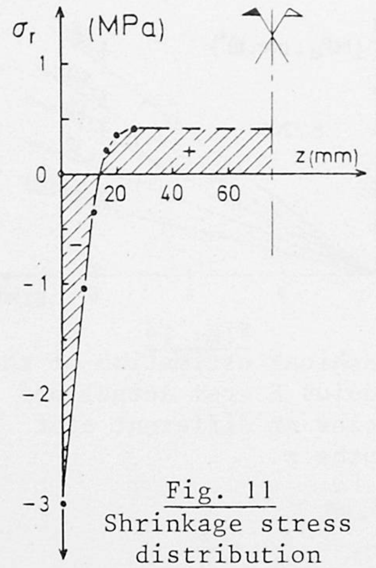


Fig. 11
Shrinkage stress distribution

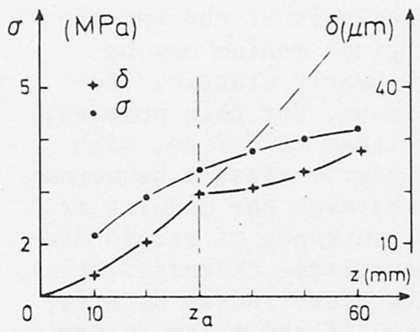


Fig. 12
Cutting through reinforcement (depth z_a): immediate effect on displacement field, none on measured stress profile

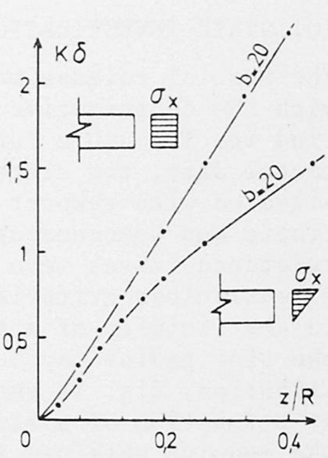


Fig. 13
Dimensionless reference curves evaluating the relative state of a medium through release criteria

8.2 Tensile stresses

The principle of estimating compression, by measuring the cancelling pressure, could have the following corollary: Tension can be estimated through measuring a subsequent forced extension and the pressure that caused it, then by extrapolating down to zero displacement (fig. 10).

8.3 The stress gradient

Other tests were carried out on models subjected to flexure. At each slot depth, the measured stresses agreed fully with the theoretical ones, acting at the centre of gravity of the jack. Hence, by measuring the stresses at closely successive depths of the same slot, the gradient can be determined with a good approximation.

9. THE ACTUAL CASE ON SITE

In all tests carried out so far, cutting has been taking place before applying external loads to eliminate internal stresses and offer a straightforward test of the release method. Now that the accuracy of this method is established, the more complex actual case on site can be tackled. It involves two supplementary problems: The internal residual stresses and the eventual cutting through reinforcement.

9.1 Internal stresses; separation of stress components

The progress achieved in measuring a tensile stress and its gradient paved the way for the comprehension of internal residual stresses in concrete. A series of tests were carried out to analyse the effects of shrinkage. Fig. 11 shows an example of the stress distribution obtained. Shrinkage, as any other internal stress, obeys the law of the equilibrium of forces. When shrinkage and applied external stresses were both involved, the superposition principle proved to be valid. The absolute stress, measured on site, can thus be broken down to its components: The applied and the internal.

9.2 Presence of reinforcement

All possible steps were taken to avoid cutting through reinforcement during the stress release operation. However, as exceptionally unfavourable cases may arise, tests were carried out to study the effect of such cutting. Fig. 12 summarises the results. It shows a clear and immediate discontinuity in the evolution of the displacement field, but hardly any effect on the continuity and accuracy of the measured stress profile. It goes on as if the cutting through reinforcement had furnished the given medium with a higher rigidity facing the same stress field.

10. STATE INVESTIGATION OF A MEDIUM

The partial release method can now not only dispense with the constitutive law, but would even help to find it. Through a further analysis of the experimental data, the state of a given medium may be situated with respect to a linearly elastic, isotropic and homogeneous continuum. For this purpose, reference curves were established to define, with dimensionless criteria, a linearly elastic behaviour in the vicinity of a slot, whatever the modulus or the slot radius may be. For two types of stress distribution, fig. 13 shows the release characteristics as a function of a slot depth z and radius R; E is the modulus obtained from fig. 14 and σ the stress assumed equal to the cancelling pressure p_c. For the same slot of different depths, fig. 15 shows, as a function of position x, the response of the displacement field to a unit pressure of the jack. The actual curves can thus be traced and compared with their references. If a serious disagreement is observed, the medium may be cracked or plastified, depending on the direction and sense of deviation.

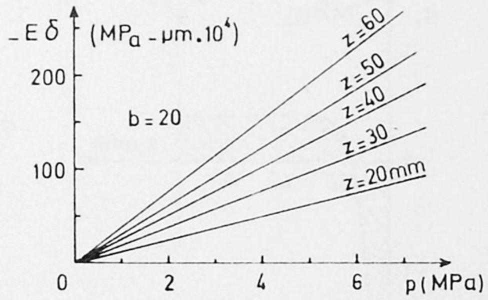


Fig. 14
Graphical estimation of the modulus E from actual p-δ cycles at different slot depths z

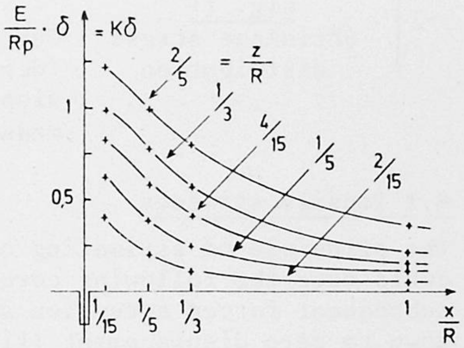


Fig. 15
Dimensionless reference curves for exploring a medium through a unit pressure in the slot (p = 1)

11. LOAD BEARING CAPACITY

In a sound medium, the ultimate load can be estimated through an imposed deviation (from the curves of figs. 14 and 15) under excessive pressures of the jack. In a damaged medium, a divergence from fig. 13 already occurs on stress release; the jack can then be used to confirm the damage and supply the information leading to the ultimate load estimation.
Where the compressive stress is not sufficiently high, excessive jack pressures may cause two successive deviations from the curves of figs. 14 and 15: The first, a sharp one, marks a tension failure of the slot extremities; the second, much more gradual, reflects the approach of ultimate compressive strength.

12. EXTENSION TO MASONRY

Having obtained positive results with concrete, the release method is now being extended to composite materials. No technological changes were necessary and the same procedures, already put forward, were retained.

At an early stage, it was observed that, unlike concrete, masonry units originally have very low internal stresses, which facilitates access to stress components. However, this considerable advantage is annulled by the fact that, in such a composite medium, the mortar joint is very flexible and fragile. The resulting distribution of axial and lateral stresses is a complex function of position.
Hence, the specific questions facing stress measurement in masonry concern the choice of the appropriate slot positions, where:

- stress is most representative of the actual average value in the structure,
- the stress-strain relationship is adequately reversible under the jack pressure,
- the bearing capacity may be estimated.

These problems can mainly be handled through experiment. Even in the case of the first question, where attempts by finite elements are underway, a correct theoretical stress analysis is subject to assumptions on boundary conditions, which only experiment can confirm.

13. MASONRY MODELS; LOCATION OF MEASUREMENT POINTS

Testing involved two full-scale models, a column and a wall. Stones with high elasticity modulus were chosen for better comparison with concrete. In each model, three slot positions were considered (fig. 16):

S1: entirely in the mortar joint
S2: entirely in the stone
S3: partially in two adjacent stones and cutting their common joint at right angles.

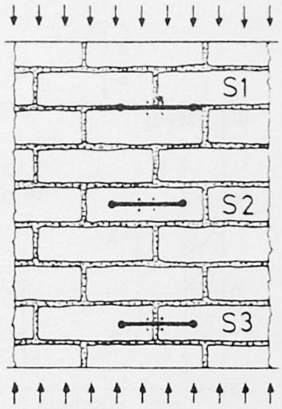


Fig. 16
Proposed location of stress measurement in masonry

14. TESTS, RESULTS AND COMMENTS

An axial load was applied to the column. The wall was subjected firstly to axial, then to accentric loading. The deformability was estimated by extensometry and the actual stresses by the release method.

14.1 General and local deformability

Prior to stress release, each model was loaded in successive stages to examine its general behaviour and establish the various stress-strain relationships.

As expected, plane cross sections do not remain plane even under a well distributed load; the joint cannot prevent normal and tangential relative displacements of stone units. Fig. 17 compares the overall stress-strain relationship to that of each constituent. For stresses higher than 0.6 MPa, the stone and overall moduli are respectively five and four times that of the mortar.

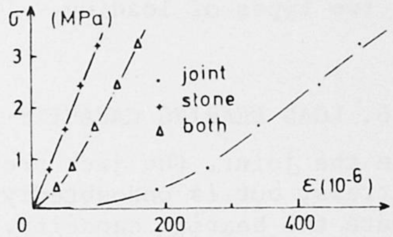


Fig. 17
Stress-strain diagram for a masonry model and its constituents

On site, the deformability is estimated by inserting, in the joints, two jacks facing each other at an easily calculable distance.

14.2 Stress determination. Accuracy obtained

Under constant loading, this operation was carried out by alternating cut and measurement. Fig. 19 summarises the results obtained under two stress profiles: σ_a, pure compression and σ_b, compression and flexure. It shows a high accuracy for slots S2 and S3 (entirely or partially in the stones), but reveals approximate and even singular values for slots S1 (entirely in the joints). The error in S1 is always in excess. In fact, the accuracy of the release method depends on the degree of cohesiveness of the slot's immediate vicinity facing, after release, a high stress concentration on the borders. The mortar being too fragile to meet this condition, a possible though invisible crushing, with uncontrolled disturbance, can lead to an erroneous interpretation of the measured cancelling pressure. Consistent results might be obtained and lead to an empirical calibration formula, but only for a well prepared model and not on site where the mortar has deteriorated. Hence, wherever a relatively high accuracy is required, it is advisable to measure the stress in position S2 or preferably in S3.

Finally, with regard to the curved stress gradients observed, they are probably caused by the confinement of the mortar joint and its resulting lateral stresses.

15. STATE OF A MASONRY MEDIUM

Figs. 13 and 15 have already set standards for situating the state of a monolithic cohesive material. However, their application is rather delicate in a composite medium, where the modulus varies with position and stress (fig. 17). Pending further research, the actual field reactions, to release and pressure, are compared for S1, S2, S3 and with reference to concrete (figs. 19 and 20). These curves confirm the different behaviour of the joints.

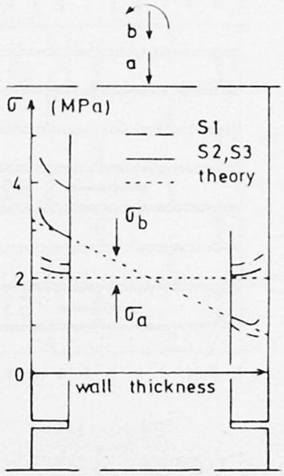


Fig. 18
Stress measurement under two types of loading

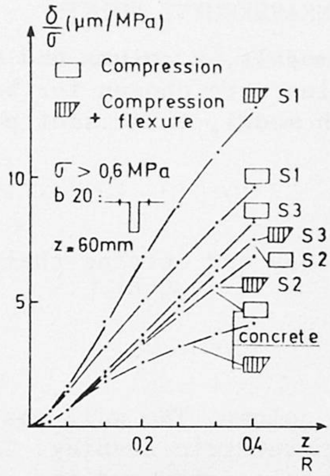


Fig. 19
Relative field disturbances per unit stress release

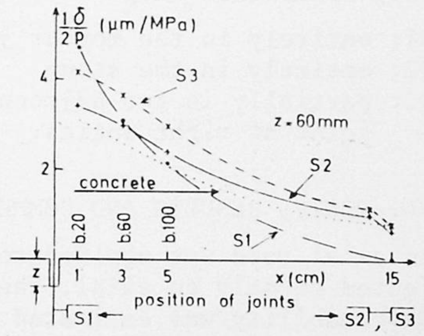


Fig. 20
Relative response to a unit pressure in the slot

16. LOAD BEARING CAPACITY

In the joint, the jack pressure may lack accuracy in measuring the stress, but is undoubtedly in the only position where it can estimate the bearing capacity. This is carried out by following the same procedure used to determine the deformability on site (§ 14.1) and then by spotting the excessive pressure causing a clear deviation in the strain growth.

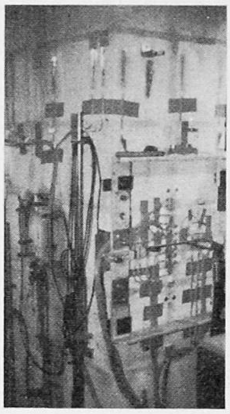


Fig. 21
Masonry model and test set-up

17. IN BRIEF

Successful applications on site have been made of the release method.

Automatic stress measurements are being introduced to study the time-dependent behaviour of a structure or to monitor its evolution during eventual repair.

Although miniaturisation has sharply reduced the method's destructive character, post-operational repair techniques, to statical and esthetic ends, are being developed.

18. CONCLUSION

Substantial progress has been achieved in the direct evaluation of the actual stresses in concrete and masonry structures. After miniaturisation and development, the partial stress release method is now operational for instantaneous and, possibly, long-term measurements. This method can also situate the general state of the assessed medium and estimate its elastic properties and load bearing capacity.

BIBLIOGRAPHY

ROCHA, LOPES, DA SILVA, "A new technique for applying the method of the flat jack in the determination of stresses inside rock masses"; LNEC, Lisbon, 1969.

ABDUNUR, "Mesure de contraintes par libération"; International Conference on Inspection, Maintenance and Repair of Road and Railway Bridges, Brussels - Paris, April 1981.

ABDUNUR, "Direct measurement of stresses in concrete structures"; IABSE Symposium on Maintenance, Repair and Rehabilitation of Bridges, Washington D.C., September 1982.

Diagnostic Analysis of Masonry Buildings

Analyse et diagnostic d'un bâtiment en maçonnerie

Diagnostische Analyse von Backsteinbauten

Luigia BINDA MAIER
Associate Professor
Politecnico di Milano
Milan, Italy

Luigia Binda, born in 1936, received her architecture degree at the Politecnico of Milan - Associate Professor, Dept. of Structural Engineering, Politecnico of Milan, she is involved in restoration, statical problems and durability of building materials.

Pier Paolo ROSSI
Engineer
ISMES
Bergamo, Italy

Pier Paolo Rossi, born in 1942, received his engineering degree at the University of Bologna. Head of Rock Mechanics Division of ISMES since 1968, he is involved in rock mechanics, physical models and brick masonry tests.

G. SACCHI LANDRIANI
Professor
Politecnico di Milano
Milan, Italy

Giannantonio Sacchi Landriani, born in 1930, received his architecture degree at the Politecnico of Milan, Doctor in Applied Sciences (F.P.Ms-Belgium). Full Professor, Dept. of Structural Engineering, Politecnico of Milan, he is involved in plastic analysis and design of structures and behaviour of anisotropic materials.

SUMMARY

This paper describes operative criteria for the stress analysis of masonry buildings whose data on local geometrical configuration, physical properties of materials and loading history are incomplete or only partly available. The use of experimental data obtained from non-destructive in-situ tests concerning material mechanical characteristics and local stress and strain states (stress relaxation methods) are discussed.

RESUME

Le rapport présente des critères d'analyse des contraintes d'ouvrages existants, dont la connaissance de la configuration géométrique, des caractéristiques physiques des matériaux et de l'histoire de chargement n'est pas complète. Il est fait mention de l'utilisation de données expérimentales obtenues à l'aide d'essais non destructifs «in situ» (méthode de libération des contraintes).

ZUSAMMENFASSUNG

Dieser Artikel beschreibt operative Grundsätze und Methoden für die statische Berechnung von bestehenden Backsteinkonstruktionen, von denen die physikalischen Eigenschaften der Materialien, die geometrische Beschaffenheit des Aufbaues und die Geschichte der Belastung nur teilweise bekannt sind. Wir diskutieren die Verwendung der experimentellen Daten die aus zerstörungsfreien «in-situ»-Versuchen über die Materialeigenschaften und die lokalen Beanspruchungszustände entnommen werden können.

1. DESCRIPTION OF ACTUAL SITUATION

This research, as in previous paper [1], aims at a systematic use of in situ investigation procedures capable of collecting helpful data for structural analysis.

As is known, the determination of geometrical features and their graphic display (Figs. 1, 2) together with an appropriate photographic campaign (Fig.3) are needed to supply a knowledge of the statical behaviour of the construction.

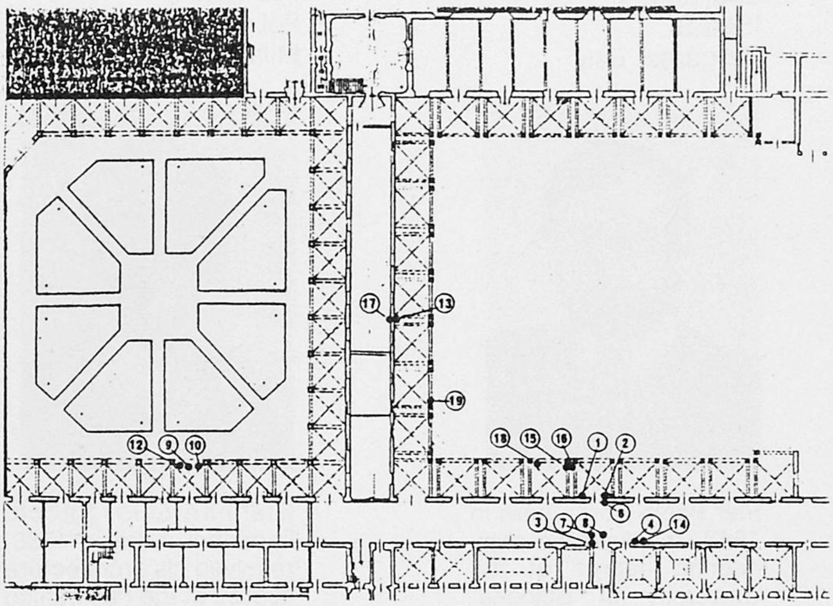


Fig. 1 Plan of the building.

Changes undergone by structure as a result of soil or structural element settling or else of rehabilitation works, cause modification in the stress-strain state of the structure. Relevant information may be given by historical research.

The photogrammetric survey and the crack pattern representation (Figs. 4, 5) may be quite useful: they aid in defining the actual static behaviour.

A direct measurement of stabilized settlements undergone by the structure in the past is often practically impossible. The setting-up of monitoring systems allows to control displacements

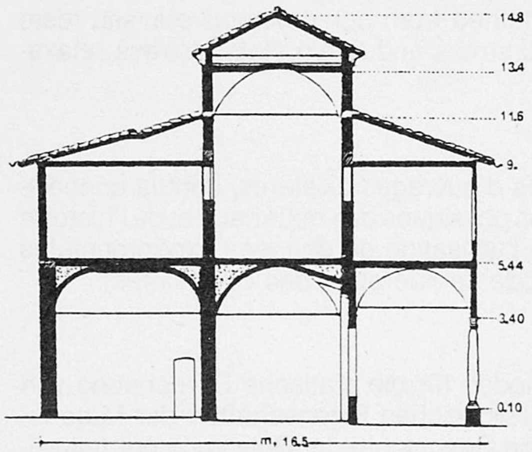


Fig. 2 Vertical section.

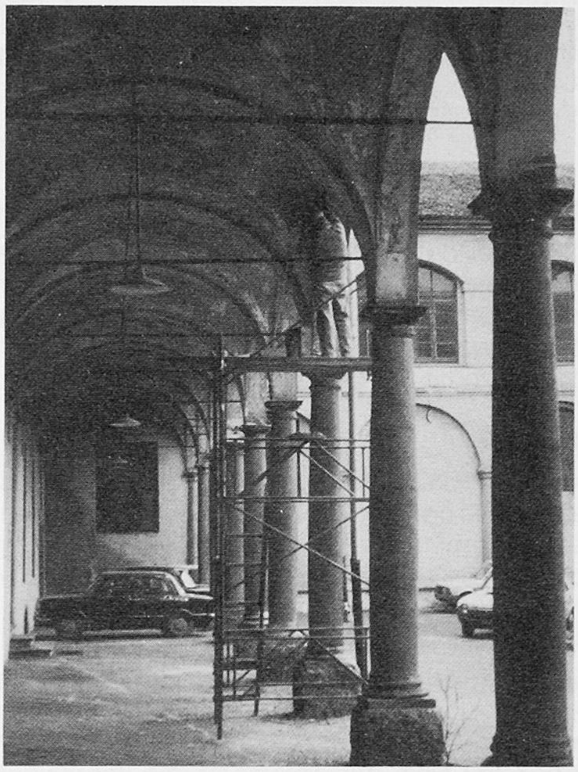


Fig. 3 Photographic survey.

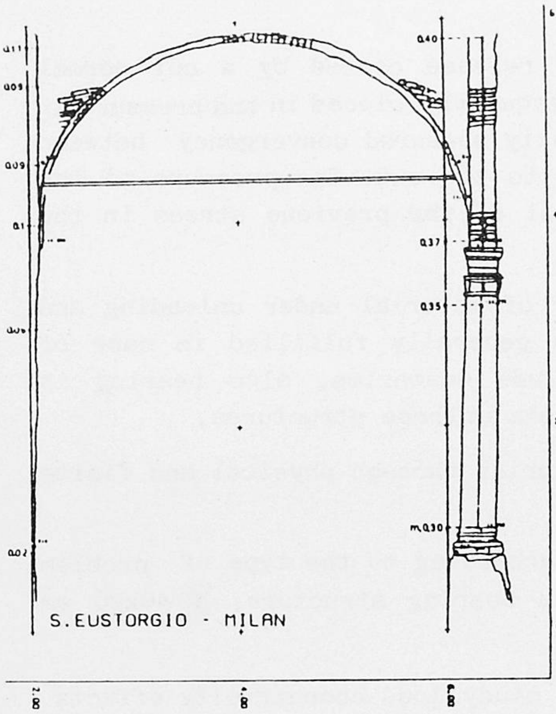


Fig. 4 Photogrammetric survey.
(Inst.of Topogr.& Photogr., Politecnico of Milan)

in progress. Of course, control should include soil behaviour.

Alteration of materials, especially of mortar joints, decreases the load carrying capacity of the structure. Knowledge of actual deformability characteristics and strength limit of materials enables the designer to appraise both the structure response to loads and its safety margin. Laboratory tests call for large-size in situ sampling, which might be of no use if the sample is disturbed.

Non destructive in situ mechanical tests based on the insertion of special flat jacks in the masonry represent a useful tool to determine the mechanical behaviour of the material without extracting samples.

An experimental investigation on the masonry static condition of S. Eustorgio Cloister in Milan, was conducted through non-destructive testing technique.

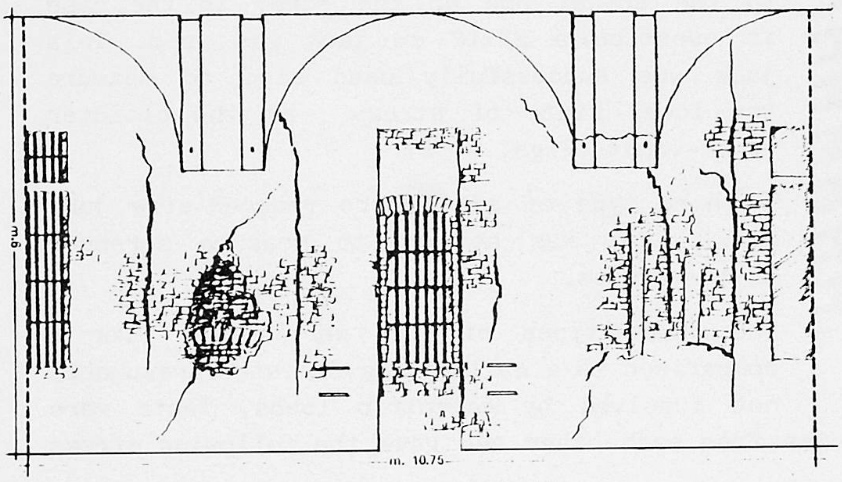


Fig. 5 Crack pattern representation.

Two are the S. Eustorgio cloisters, attached to the homonimous church. The construction of the first cloister dates back to the first half of 13th century; the construction of the second one started in 1380. Both cloisters were repeatedly damaged and restored in the following centuries during Spanish and French domination and rebuilt in 1600. After that time they underwent further damages and rehabilitations, the latest of which around 1950, when wooden floors were replaced with tile lintol floors and r.c. beams were placed. The superposition of the various ages may be seen in the surveys (Fig. 5).

2. MECHANICAL NON-DESTRUCTIVE TESTS

Mechanical in situ testing makes it possible to determine both the local stress state in the masonry and deformability properties and may also provide an appraisal of failure resistance of material.

2.1 Stress state measurement

Stress state measurement is based on stress release caused by a cut normal to the masonry surface. A flat jack is subsequently placed in and pressure increased gradually up to compensate the previously measured convergency between two points which are symmetrical in relation to the cut. The pressure of the properly calibrated jack provides an appraisal of the previous stress in the masonry in normal direction to cut plane.

The assumption that the mechanical behaviour of material under unloading and reloading conditions is of reversible type, is generally fulfilled in case of slightly fractured rock masses and may include masonries, also bearing in mind the low stress levels which generally exists in these structures.

Test reliability was checked at ISMES laboratories through physical and finite element tridimensional models [2].

Flat jack dimensions may easily be changed according to the type of problem to be tackled. In the case of high-thickness bearing structure, a 40x20 cm jack is generally used (Fig. 6).

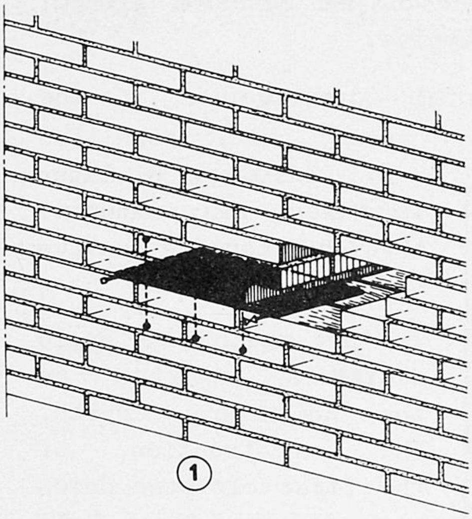


Fig. 6 Flat jack test.

If we wish to study load eccentricity effects through tests carried out on the two opposite faces of the masonry, the jack depth should be reduced to measure a stress value nearest to the actual one on the edge. In the case in question, a 24x12 cm jack was used. This jack was successfully used also to measure the local state of stress at the cloister cross-vault (Figs. 7, 8).

A third type of still more reduced-size jack (12x12 cm) was set up to measure stresses in the arches.

The three types of jack were set for a comparison in a wall of the cloister presumably not involved by eccentric loads. Tests were carried out at short distancies from each other and gave the following stress values:

JACK 1	(40 x 20 cm)	σ = 1.04 MPa
JACK 2	(24 x 12 cm)	σ = 1.12 MPa
JACK 3	(12 x 12 cm)	σ = 1.17 MPa

The good agreement between measurements made it possible to ascertain the reliability of the test carried out by using small-size jacks. The correct working of the 40x20 cm jack was indeed fully checked during calibration tests and several investigation campaigns conducted on monumental buildings (Palazzo della Ragione - Milan; "Classense" Library - Ravenna; building at "Piazzale Dateo" - Milan).

2.2 Comparison between measured and calculated stress values

In situ tests were carried out at the points shown in Fig. 1. Stresses

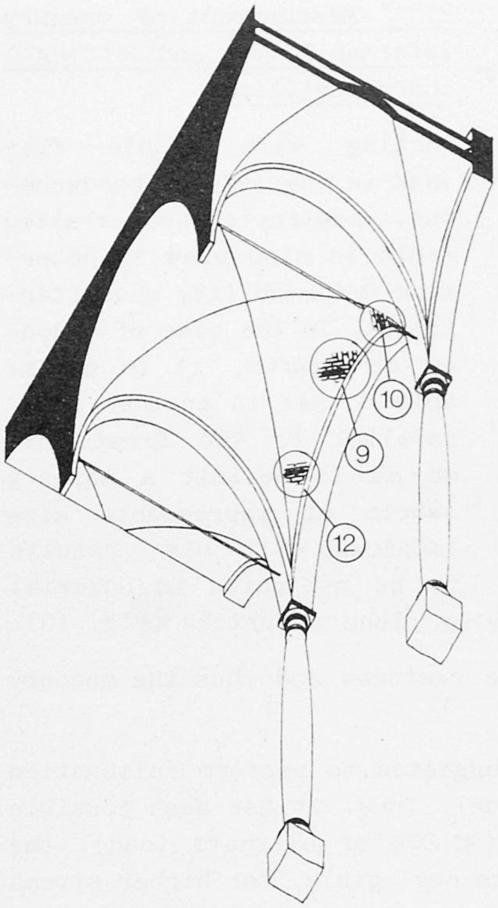


Fig. 7 Tests points on the cross vault.

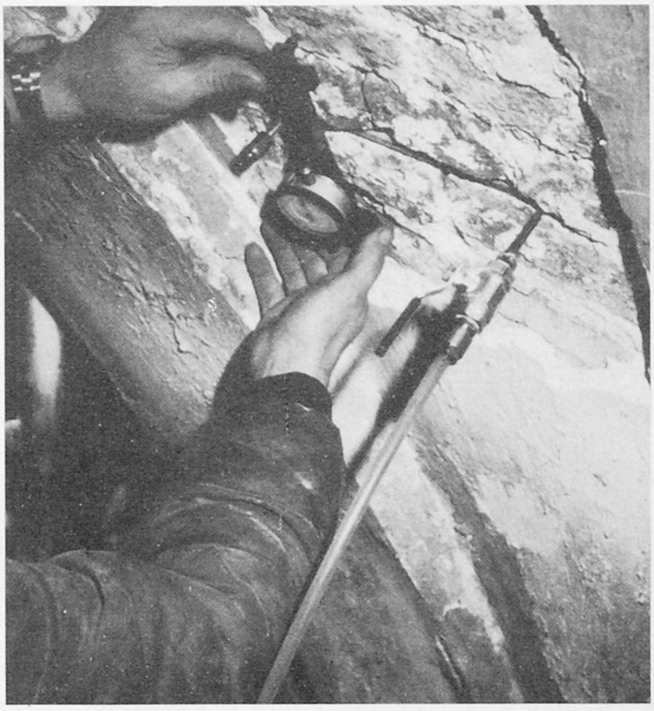


Fig. 8 Tests on the cross vault with a 24x12 cm flat jack.

corresponding to a statically determined solution, suggested by the crack pattern, were computed. In particular, Fig. 9 shows the solution adopted for the cross vaults. The results are reported in the Table I.

TABLE I: Comparison between in situ measured stresses and calculated stresses

Test no.		Calculated stress (MPa)	In-situ measured stress (MPa)
1	wall		0.4
2	wall	0.51	0.56
6	wall		0.4
3	wall		1.04
4	wall	1.16	1.12
14	wall		1.05
13	wall	0.78	1.6
17	wall		0.48
15	pulvino	1.73	2.0
16	pulvino		1.46
7	barrel vault		0.8
8	barrel vault		0.72
9	cross vault	0.06	0.16
10	cross vault	0.37	0.32
12	cross vault		0.24
18	cross vault		1.3

The evident correspondance between experimental and calculated values confirms the good choice of the statical solution.

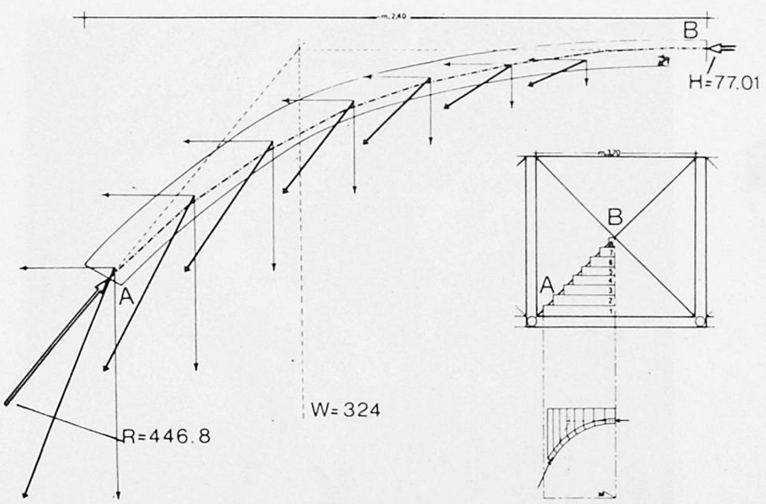


Fig. 9 Static solution adopted for cross vault.

2.3 Measurement of masonry deformability and strength characteristics

Testing with simple flat jack in the case of homogeneous, isotropic and elastic media is also used to determine deformability characteristics. In the case of masonry structures, it is advisable to set in another jack, parallel to the first one, so as to delimit a masonry sample of appreciable size (40x50x20 cm); this results to be subjected to uniaxial compressive test in direction normal to the laying plane of bricks (Fig. 10).

After the test was performed, mortar layers are restored and thus the masonry is returned to its original conditions.

Lateral confining conditions of sample have suggested to perform calibration tests by means of physical and mathematical model. Thus, it has been possible to ascertain that , for slight stress values ($<$ 20% of ultimate load), the lateral confining effect may be considered quite negligible. For higher stress levels ($<$ 50% of ultimate load), an increase in deformability modulus equal to about 10% of the value determined by unconfined compression tests was noticed.

Fig. 11 shows stress-strain diagrams of the sample delimited by the two jacks.

To estimate the strength limit, a test was carried out up to the appearance of the first cracks in the bricks (Fig. 12).

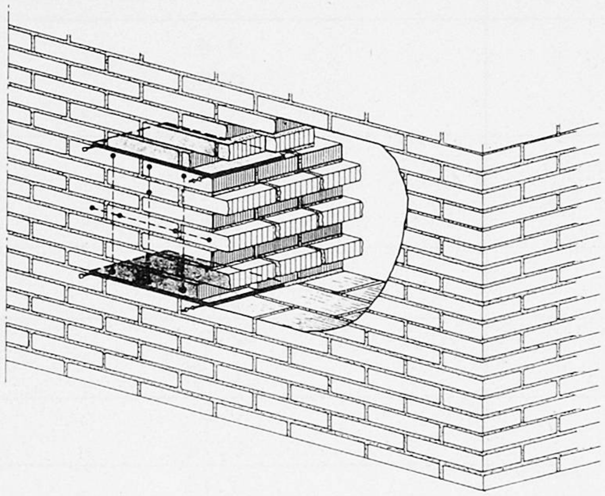


Fig. 10 Deformability test.

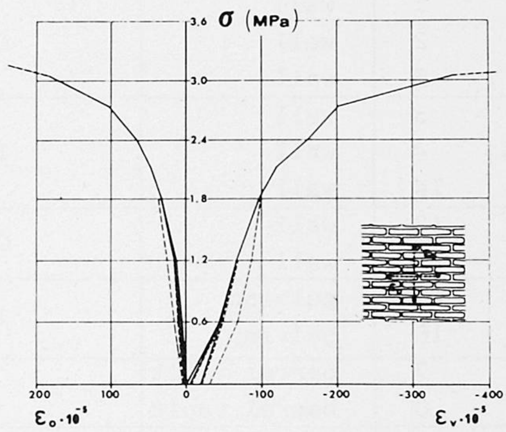


Fig. 11 σ - ε diagram.

The ultimate stress value measured was equal to 3.1 MPa.

Fig. 12 Photo showing the appearance of first cracks.

Ratio of deformability modulus E_d, computed in the load interval 0 - 0.5 σ_R to ultimate stress σ_R is (Fig. 10):

$$\frac{E_d}{\sigma_R} = \frac{1,990}{3,1} = 609$$

It may be pointed out that the evaluated ratio is included in the usual range for this kind of materials.

3. ANALYSIS OF A REAL STRUCTURE

Experimental in-situ investigation, which deals especially with geometric and mechanical aspects of the structure, is of particular importance because it makes it possible: to follow the soil settlements (monitoring); to notice whether geometric alterations exist, such as to change the original load condition; to appraise the strength of the material and the local stress state. These latter two conditions are particularly useful, as they provide an important hint for the formulation of equilibrated solutions.

The following remarks mainly emphasize the know-how degree of the structure stress state and therefore of its safety, versus available information on the behaviour of the material, on constraint conditions and on soil behaviour, and on the stationary condition or progress of alteration phenomena.

a) In case all this information is clear - which is an exceptional case - a step-by-step non-linear analysis may be applied. This, in spite of the numerical difficulty, will provide exhaustive solutions.

b) Constraint settlements are unknown, constraint conditions are uncertain; however, the material shows a behaviour, up to failure, characterized by ductility and associated flow rule. In this case, the classical limit analysis may be applied. The structural safety domain may be defined in the load space. For every load condition, the correspondent kinematic load P_K will be an upperbound on the limit load P_L; in turn, the latter is an upperbound on any statically admissible load P_S [3] [4]. A solution $P_S = P_L = P_K$ is proved to exist.

c) Preceeding conditions occur, but the material has no-tensile strength. Limit analysis may still be used, assuming the cracks as fictitious ductile strains [5], the normality rule being still considered existent.

d) A non-associated flow rule is known. Two fictitious standard domains f_G, f_F [6] [7] can be defined; f_F contains f_G and, using limit analysis techniques, inequalities $P_{LG} \leqslant P_L \leqslant P_{LF}$ can be proved. In short, an upper and a lower bound of P_L can be calculated; however, it will be impossible to know the exact value of P_L.

e) The material has no-tensile but very high compressive strength; that is the particular case of stone-masonry structures. Limit analysis may still be applied and is essentially reduced to a geometric problem [8] [9] .

f) The failure criterium of material is known as a function of stresses f_F; however, the flow rule is unknown. Referring to item d), we may say that the domain f_G is no longer defined and we only state that $P_L \leqslant P_{LF}$. Load domain built up with f_G is then to be considered only as "potentially safe". Points outside the domain f_F certainly represent collapse states. Therefore P_L may be accessed to only for kinematic approach - which involves a good experience on failure mechanisms of the various structures [10]. The capability of the static approach decreases, although it remains still useful. A coefficient of "true failure" can be defined - which is important in case a modification of the load conditions of the structure is required [11].

REFERENCES

[1] BINDA L., BALDI G., CARABELLI E., ROSSI P.P., SACCHI G., Evaluation of the Statical Decay of a Masonry Structure: Methodology and Practice, Proc. of 6th IBMaC, Rome, May 1982

[2] ROSSI P.P., Analysis of mechanical characteristics of brick masonry by means of non-destructive in-situ tests, Proc. of 6th IBMaC, Rome, May 1982

[3] PRAGER W., An Introduction to Plasticity, Addison Wesley, 1959

[4] SAVE M., MASSONER Ch., Plastic Analysis and Design of Plates, Shells and Disks, North-Holland Pub. Co., Amsterdam, 1972

[5] FRANCIOSI V., Calcolo a rottura. Lo stato limite ultimo da meccanismo, Liguori, Napoli, 1979

[6] RADENKOVIC D., Théorèmes limites pour un matériau de Coulomb à dilatation non standardisée. C.R.Ac.Sc., Paris t. 252, 4103-4104, 1961

[7] PALMER A.C., A Limit Theorem for Materials with Non-associated Flow Laws, J. de Méc. 5, 217-222, 1966

[8] HEYMAN J., The Stone Skeleton, Int. J. Solids Struct. 2, 249-279, 1966

[9] HEYMAN J., The Masonry Arch, Ellis Harwood Ltd., London, 1982

[10] SALENÇON J., Cours de calcul des structures anélastiques: Calcul à la rupture et analyse limite, ENPC, Paris, 1981

[11] DELBECQ J.M., Les ponts en maçonnerie: évaluation de la stabilité, SETRA, Min. des Transports, Paris, 1982.

Techniques for Testing, Analyzing and Rehabilitation of Terra-Cotta

Techniques d'essai, d'analyse et de réparation de la terre cuite

Versuchsmethoden, Analyse and Instandsetzung von Terrakotta

Sven E. THOMASEN
Consultant
Wiss, Janney, Elstner Assoc.
Emeryville, CA, USA

Carolyn S. EWART
Project Engineer
Wiss, Janney, Elstner Assoc.
Emeryville, CA, USA

Sven Thomasen has a structural engineering degree from the Technical University of Denmark. While with WJE, he has investigated building failures and material deterioration. He has taught at California Polytechnic in San Luis Obispo and has designed many large projects.

Carolyn Ewart received a BS in Architectural Engineering from California Polytechnic State University, San Luis Obispo, and a MS in Structural Engineering from University of California, Berkeley. She has done field investigations and analysis of structural failures and evaluation and rehabilitation of masonry, marble and terra-cotta facades.

SUMMARY

Terra-cotta was often used as wall cladding for the early American skyscrapers. The material went out of fashion around 1930, but many ornate terra-cotta structures are still in service. Deterioration has resulted from design deficiencies and lack of understanding of material properties. Field investigation of terra-cotta failures and tests of the material are discussed. The cause of distress is analyzed and techniques for rehabilitation introduced.

RESUME

La terre cuite a été très employée comme revêtement de façade de gratte-ciels aux Etats-Unis. L'emploi de ce matériau a diminué vers 1930, mais il reste en service un grand nombre de structures en briques. La détérioration a été le résultat de déficiences dans le projet et d'une mauvaise compréhension des caractéristiques de la terre cuite. La méthode d'investigation insitu et des essais en laboratoire sont décrits. L'origine des dégâts et les techniques pour la remise en état sont présentés.

ZUSAMMENFASSUNG

Die Terrakotta wurde beim Bau der ersten amerikanischen Hochhäuser oft als Wandverputz verwendet. Um 1930 kam der Gebrauch des gebrannten Tons aus der Mode aber viele mit Terrakotta-Verputz gezierte Bauten versehen noch ihren Dienst. Infolge unsachgemässer Gestaltung und fehlendem Verständnis für das Material resultieren Verfallserscheinungen. Die Untersuchung der Terrakotta-Verfallserscheinungen und Materialversuche werden diskutiert. Die Ursache der Schäden werden analysiert und Verbesserungsvorschläge für die Problemlösung werden vorgeschlagen.

1. INTRODUCTION

The use of terra-cotta as an architectural material coincided with the building of the great American cities and the rise of the skyscrapers. The usage declined with the change in building technology, but a large number of outstanding terra-cotta structures are still in service [1]. Terra-cotta is durable and permanent because of the excellent weathering properties and the hard surface of the glaze, but the cladding on many high-rise buildings has deteriorated due to inherent deficiencies in the design and a disregard and lack of understanding of the material. The historical significance of these buildings and the unmatched richness of their detailing and vivid coloring of the terra-cotta make preservation of these structures important.

Terra-cotta was sometimes used as load-bearing masonry, but more often as a cladding anchored to the structural framing system. No provisions were made for movement, either absolute or differential, in the back-up framing or in the cladding. Moisture expansion and thermal fluctuations of the clay body and the glaze often resulted in cracking of the terra-cotta blocks. Water entering the cracks then accelerates the weathering and deterioration of the terra-cotta and causes corrosion and distress in the complex support and anchoring system [2].

Rehabilitation originates with a field survey and a program of field testing to detect the extent and the nature of the distress. Strain measurements reveal stress concentrations in the masonry and a stress map evaluates if cutting of expansion joints will relieve the build-up of stresses. Laboratory tests of terra-cotta samples measure properties of the clay body and the glaze and indicate the extent of decay and deterioration of the material.

Replacement of damaged pieces with new terra-cotta or with substitute materials requires a careful match of appearance and material properties. Laboratory tests establish strength, expansion and rate of absorption of the replacement material and accelerated weathering tests question the long-term performance and rate of degradation.

2. TERRA-COTTA CONSTRUCTION

The typical terra-cotta cladding material is about 10 cm thick and from 1000 to 2000 cm^2 in surface area. The blocks were fabricated by hand pressing the clay into wood forms. The back is open with internal webbing to stiffen the block. The glaze, an aqueous solution of metal salts, was sprayed or brushed on the unfired clay. It was fired to cone 4 to 5 (about 1130°C), resulting in good glaze hardness and high clay strength. The blocks were supported vertically at each floor level by shelf angles. Z-shaped steel straps were fastened into slots in each block and anchored the terra-cotta horizontally to the back-up walls of masonry or concrete. Ornamental units generally had multiple anchors, as seen in Figure 1.

The terra-cotta was installed with solid joints of cement/lime mortar. Joints were narrow, often about 5 mm. The walls had no expansion joints either in the back-up or in the cladding and they had no internal flashings or weepholes.

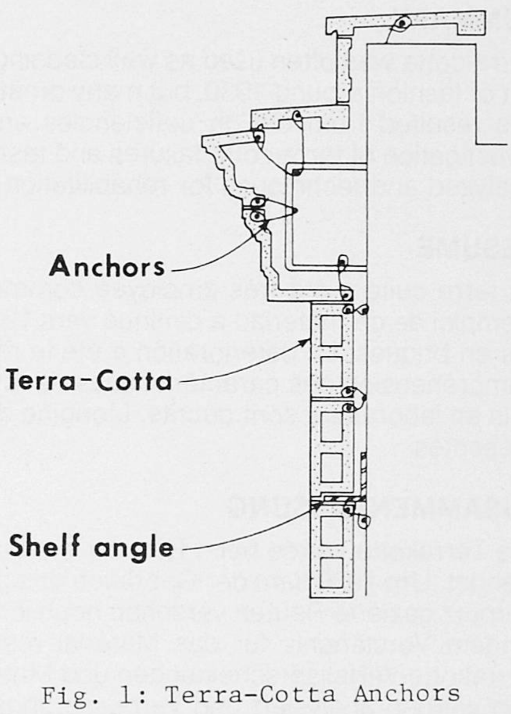


Fig. 1: Terra-Cotta Anchors

3. TERRA-COTTA FAILURES

Terra-cotta failures are often interrelated and progressive in nature. Water entering through cracks caused by differential expansion accelerates the weathering of the terra-cotta and corrodes the metal anchors and shelf angles [3].

3.1 Glazing Failures

Environmental exposure to temperature fluctuations can result in glaze crazing if the thermal coefficient of expansion of the glaze and the clay body is incompatible. Crazing, or the formation of small random cracks in the glaze, allows water to enter the clay body causing pinhole spalling of glaze when water in the clay pores freezes. More water will then enter the block resulting in general glaze spalling and the loss of the entire glazed surface [4].

3.2 Expansion Failures

Terra-cotta buildings with concrete frames often experience long-term shrinkage of the frames, but most terra-cotta failures result from temperature and moisture expansion of the clay body. Thermal and wet/drying cycles are often associated with permanent lengthening of the terra-cotta blocks, and this, plus the absence of expansion joints in the facade, creates high compressive stresses. Failure can be in the form of buckling of block units, as seen in Figure 2, or crushing at the base of the facade where expansion stresses are combined with compressive gravity loads.

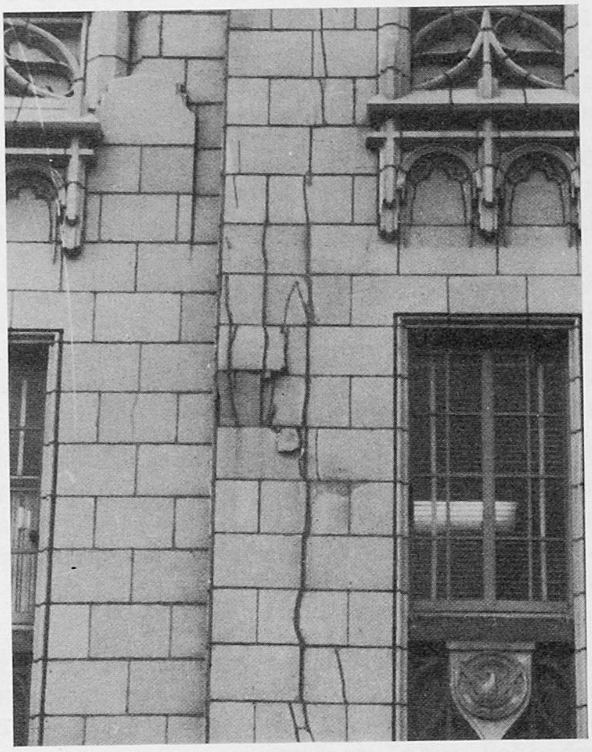


Fig. 2: Expansion failure

3.3 Moisture-Related Failures

Spalling of the glaze and cracks caused by expansion allows water to enter the wall and results in further damage to the terra-cotta from freeze/thaw action. The moisture corrodes the anchors and the pressure from the volume expansion of the rust cracks the blocks, as seen in Figure 3.

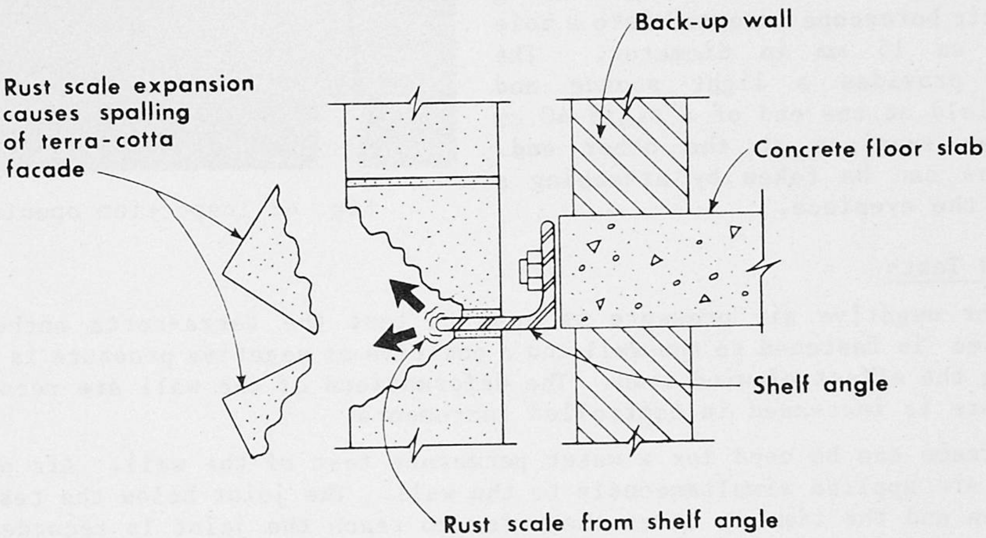


Fig. 3: Terra-cotta spalling caused by corrosion of shelf angle.

4. FIELD INVESTIGATION

The extent and the nature of the distress is evaluated by a field survey and by field testing of the terra-cotta.

4.1 Field Survey

Visual observations are made of the entire structure from the ground with binoculars or close-up from scaffolding. Photographs and drawings document the observatons and aid in detecting failure trends.

Tapping the wall with a wooden mallet can detect internal cracking. A terra-cotta block with cracks sounds different from an undamaged block when hit with a hammer. Metal hammers ruin the glaze and should not be used.

Infrared image systems have been used to detect internal cracks and delaminations. Infrared scanning or infrared photographs detect sources of heat loss in the building facade. Internal cracks in the terra-cotta or voids in the masonry backup create a temperature difference which become evident on the infrared scan. Metal anchors can also be detected this way because of their high thermal conductivity.

Crack movements are measured by installing gages or by monitoring the crack width. The cracks can have both daily and seasonal changes, as well as long-term growth. Long-term surveillance is often required to detect any significant movement.

Soniscopes have been used to locate internal cracks in the terra-cotta, but only with limited success. The many voids in the blocks tend to create patterns very similar to those produced by cracks.

Pachometers are used to detect embedded steel members and terra-cotta ties. The metal detector is used to verify the location of shelf angles and structural steel supports.

Inspection openings, as seen in Figure 4, are the best method to verify the location and the condition of the wall supports, and especially, to detect corrosion of the embedded metal anchors.

The inside of a wall can be examined using a fiberoptic borescope inserted into a hole as small as 15 mm in diameter. The borescope provides a light source and viewing field at one end of a rigid 40 cm rod and an eyepiece at the other end. Photographs can be taken by attaching a camera to the eyepiece.

Fig. 4: Inspection opening

4.2 Field Tests

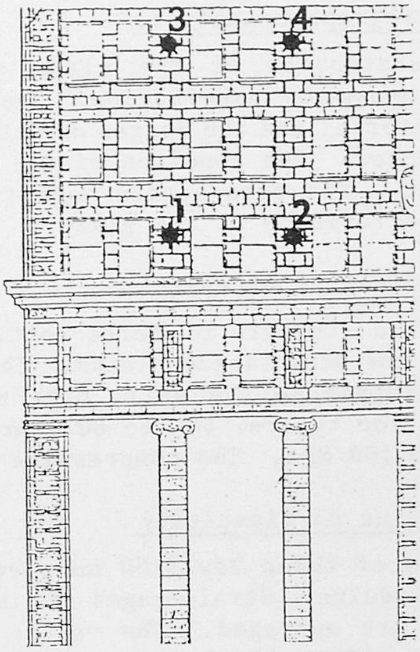
Positive or negative air pressure is used to test the terra-cotta anchors. A closed frame is fastened to the wall and a positive or negative pressure is applied simulating the effect of wind load. The deformations of the wall are recorded as the pressure is increased in controlled increments.

The same frame can be used for a water permeance test of the wall. Air pressure and water are applied simultaneously to the wall. The joint below the test frame is cut open and the time it takes the water to reach the joint is recorded. The absorbed quantity of water is measured as the weight loss of water circulating in the frame.

5. STRESS MEASUREMENT

Expansion of the terra-cotta and absence of expansion joints often cause high compressive stresses in the cladding. Strain relief testing is the best method to measure the magnitude and direction of the built-up stresses, as seen in Figure 5.

Electrical resistance strain gages are attached to the terra-cotta surface and the gages are read. Then the terra-cotta block, with the gages attached, is cut loose from the wall and the gages are read again, as seen in Figure 6. The change in gage reading is a measure of the strain in the block. The stress in the block is found by multiplying the measured strain difference by the modulus of elasticity, as determined by laboratory tests.



STRAIN RELIEF TEST DATA				
	VERTICAL		HORIZONTAL	
GAGE	STRAIN µ cm/cm	STRESS kPa	STRAIN µ cm/cm	STRESS kPa
1	450	9315	50	345
2	283	5858	24	497
3	320	6624	30	621
4	171	3540	7	145

Fig. 5: Strain relief test

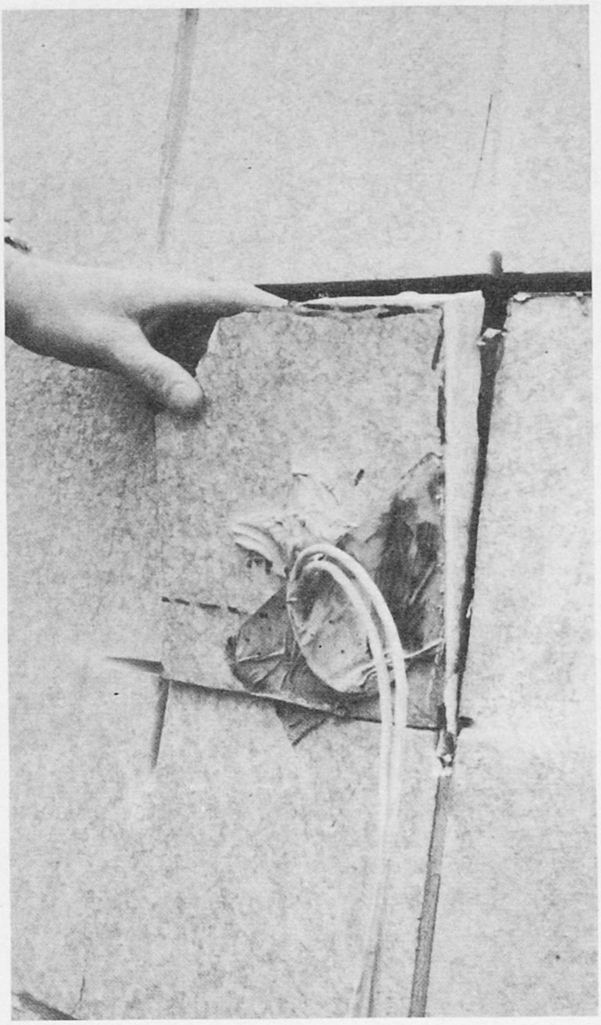


Fig. 6: Terra-cotta block, with strain gages attached, is cut loose.

The glaze in the test area should be firmly attached to the clay body and no glaze cracks should occur under the gage. Temperature variations during the day can affect the readings and should be established before the block is cut.

Compressive vertical stresses as high as 23 MPa have been recorded. This is close to the compressive strength of the terra-cotta wall. From multiple strain measurements, a stress map can be made for the exterior elevation. The map is used to evaluate if cutting of expansion joints into the facade will relieve the built-up terra-cotta stresses.

6. LABORATORY TESTS

6.1 Petrographic Analysis

The consistencies of the glaze and the clay body are evaluated through a stereomicroscopic examination. The density of the glaze surface, the composition of the material and the degree of deterioration can be established by an experienced petrographer. The conditon of the boundary layer between glaze and clay body is important. The nature, the magnitude, and the depth of the cracks predict the future performance of the glaze.

6.2 Compressive Strength

Compressive strength of terra-cotta is tested in accordance with ASTM C67 on 25 mm cubes cut from facade blocks. The load is applied in the two directions of the major stresses in the wall, both parallel to the glazed face. The compressive strength for typical 40- to 80-year old terra-cotta has been found to range from 40 MPa to 100 MPa. The compressive strength appears not to deteriorate with time.

6.3 Modulus of Elasticity

A minimum of three 25x25x50 mm specimens are tested in compression to determine Young's Modulus. Strain gages are mounted on three faces of each specimen and the readings are averaged. The values of the modulus of elasticity for terra-cotta range from 13 to 40 GPa.

6.4 Absorption

The absorption test is a modificaton of ASTM C67 and it compares the performance of glazed and unglazed specimens. On one sample, the glaze is ground off while an identical sample has the glaze intact. The sides are soaked face down in water for 24 hours. The weight gain of the glazed compared to the unglazed sample is a measure of the absorption characteristics of the glaze. Ideally, a glazed specimen should produce zero absorption if the glaze is intact, sound and craze-free. New glazes are normally impervious, but tests of 40- to 80-year old terra-cotta structures found that, at best 67%, and in the worst case, only 10% of the moisture was prevented by the glaze from penetrating the clay body [5].

6.5 Thermal Coefficients

Tests are performed both on the complete terra-cotta block and on separate pieces of glaze and clay body. Strain gages are mounted on the samples which are then subjected to temperature ranges representing the normal wall exposure. Strain readings are taken at the high, the low, and an intermediate temperature. The thermal coefficient of expansion can be established after a few cycles, but to find evidence of permanent elongation from thermal fluctuations requires a minimum of 25 cycles.

A difference in thermal coefficients for the glaze and the clay body causes a state of tension when temperature varies from normal. Such tension often causes glaze crazing [6]. The terra-cotta glaze can also experience crazing during the kiln cooling if the thermal coefficients of the glaze at high temperatures are greater than that of the clay even though the coefficients are similar in the normal range of temperature exposures.

6.6 Chemical Resistance

Different portions of glazed terra-cotta are exposed to a 10% solution of hydrochloric acid or a 10% solution of potassium hydroxide. After 3 hours, the specimens are rinsed, dried, and examined for change of color.

6.7 Moisture Expansion

During the firing, the free water is removed from the terra-cotta. As it again absorbs moisture, the clay body expands. Some of the expansion is cyclic, but a portion is nonrecoverable. A general magnitude of the moisture expansion can be determined by a reheat test. A terra-cotta sample is measured at 21°C. The sample is then heated to 1000°C, again allowed to cool to 21°C and then measured. The measured shrinkage is an indication of some of the long-term moisture expansion of the terra-cotta.

Moisture expansion of the clay body is the cause of much of the cracking, spalling, and build-up of stresses in the terra-cotta walls.

7. TERRA-COTTA REHABILITATION

Restoration of terra-cotta cladding is both difficult and expensive. The causes of distress must first be eliminated by cutting of expansion joints. The wall is made watertight by installation of flashing and joint sealants. Corroded anchors and supports are repaired and damaged terra-cotta blocks are replaced.

7.1 Expansion Joints

Vertical expansion joints are generally cut near corners and wall returns, while horizontal expansion joints are cut below reliveing angles,as seen in Figure 7. The joint location is determined from stress measurements of the terra-cotta. The joints are made watertight with elastomeric sealants. Cutting of expansion joints can disturb the fastening of the terra-cotta and will often require the installation of new anchors.

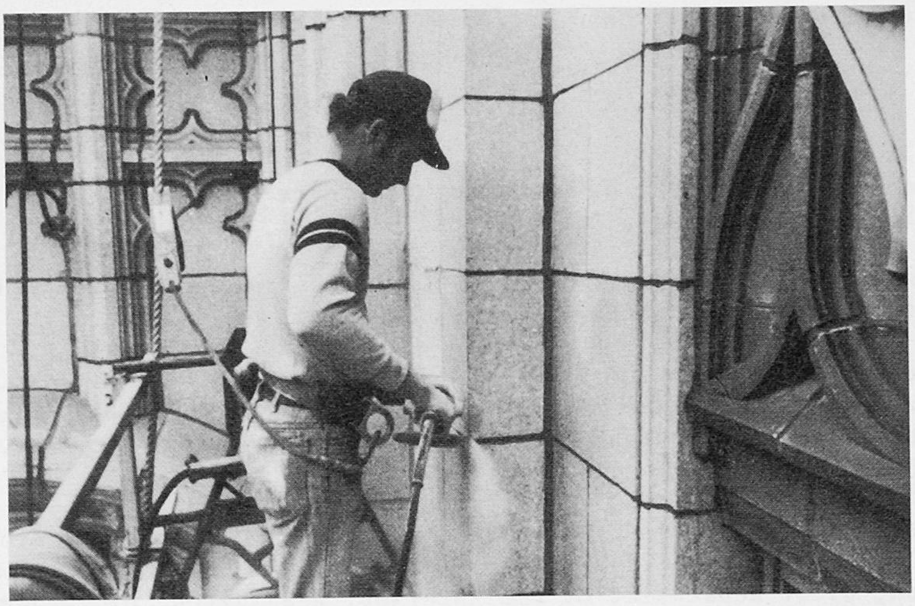


Fig. 7: Cutting of new expansion joints.

7.2 Terra-Cotta Supports and Anchors

Corroded or missing shelf angles and wall anchors are replaced. New anchors and angles should be given rust protection or be of stainless steel, especially in walls with past leakage problems. Consideration should also be given to installing flashing and weep holes in the wall cladding.

7.3 Joints

Sources of water penetration should be corrected. Roofing, flashing and capping are repaired and deteriorated sealant around windows and doors recaulked. The joints between terra-cotta blocks should be tuckpointed and not caulked since the mortar allows trapped water to dissipate without appreciably increasing absorption. Tuckpointing should be with a mortar of lower compressive strength than the terra-cotta. Application of waterproofing coatings to the terra-cotta, and especially the mortar, is not recommended. Such coating will prevent trapped water to dissipate through the joints. This will cause spalling and deterioraton of the terra-cotta.

7.4 Replacement with New Terra-Cotta

New terra-cotta is the preferred replacement material from the point of esthetics and durability. Plain ashlar blocks can be extruded, but decorative pieces require field forming and hand casting. Matching of color and texture should be verified by test firing of several full scale terra-cotta blocks. Quality testing of the material includes compressive strength, absorption, imperviousness, chemical resistance, acid and base resistance, crazing tests and glaze adhesion tests.

7.5 Replacement with Other Materials

Materials other than terra-cotta have sometimes been used for reason of economy. Fiberglass is lightweight and lends itself to decorative pieces, but it will discolor with age and is not fireproof. Precast concrete, sometimes reinforced with glass fibers, can be cast to reproduce terra-cotta details. The concrete color is matched to the existing material and surface coatings will duplicate the gloss of the terra-cotta glazing. The long-term performance of these replacement materials should be checked using accelerated weathering tests with samples subjected to repeated wet/dry and hot/cold cycles and to exposure of ultraviolet light.

REFERENCES

1. **Berryman, N. C.,** History of Architectural Terra-Cotta. National Council on Education for the Ceramic Arts (NCECA), Journal 1981.

2. **Thomasen, S.**, Degradation of Terra-Cotta on the Atlanta City Hall. ACI Convention, Atlanta, Georgia, January 1982.

3. **Thomasen, S.**, Degradation and Rehabilitation of Terra-Cotta. Second International Conference on the Durability of Building Materials and Components, National Bureau of Standards, Gaithersburg, Maryland, September 1981.

4. **Patterson, Tiller**, de Teel, The Preservation of Historic Glazed Architectural Terra-Cotta. Preservation Briefs No. 7, Technical Preservation Services Division, U. S. Department of the Interior, Washington, D.C., June 1979.

5. **Stockbridge, J. G.**, Evaluation of Terra-Cotta on Inservice Structures. ASTM Special Technical Publication 691, American Society for Testing and Materials, Philadelphia, PA, 1980.

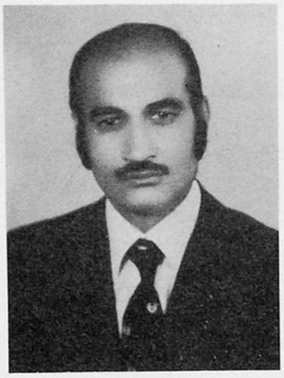
6. **Tindall, S. M.**, Architectural Terra-Cotta Restoration, National Council on Education for the Ceramic Arts (NCECA), Journal 1981.

Corrosion Deterioration of Reinforcement in Concrete Structures

Corrosion des armatures de structures en béton armé

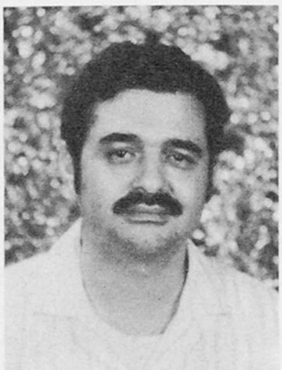
Korrosion an der Bewehrung von Stahlbetonbauten

RASHEEDUZZAFAR
Professor
Univ. of Petr. & Miner.
Dharan, Saudi Arabia



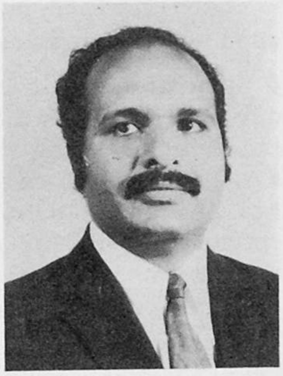
Rasheeduzzafar, born 1939, obtained his Ph.D. degree in Structural Engineering from University of London. Presently he is Professor of Structural Engineering at the University of Petroleum and Minerals, Dhahran, where he is teaching design of concrete structures.

Fahd H. DAKHIL
Vice Rector
Univ. of Petr. & Miner.
Dharan, Saudi Arabia



Fahd H. Dakhil, born in 1942, obtained his Ph.D. degree in Structural Engineering from Pennsylvania State University. Presently he is Vice Rector for Research and Graduate Studies at UPM, Dhahran. Interests are related to durability studies of concrete construction.

Fouad A. AHMAD
Assoc. Professor
Univ. of Petr. & Miner.
Dharan, Saudi Arabia



Fouad A. Ahmad, born in 1941, obtained his Ph.D. degree in Surveying from University of New Brunswick, Canada. Presently he is Associate Professor at the UPM Dhahran. His research interests are in the areas of photogrammetric engineering and geodetic surveying.

SUMMARY

Concrete construction along the seaboard of the Arabian Gulf is showing an alarming degree of deterioration within a span of 10 to 15 years. Corrosion of reinforcement associated with concrete spalling outweighs other causal factors. The paper describes the methodology of condition surveys and sampling of concrete from field structures. Spalled area is measured by a photogrammetric technique and evaluations of cover, chloride content, electrical resistivity and concrete quality show strong correlation between these factors and the extent of rebar corrosion deterioration.

RESUME

La construction en béton armé au Golfe Arabique présente un degré de détérioration alarmante après une durée de 10-15 ans. La corrosion des armatures associée avec les fissures est la cause principale des dégâts. L'article décrit la méthode de mesures et d'échantillonage du béton des structures. Les fissures sont mesurées avec la méthode photogrammétrique. Le contenu de chlorure, la résistance électrique et la qualité du béton montre une corrélation forte entre ces facteurs et le degré de corrosion.

ZUSAMMENFASSUNG

Viele Betonbauten entlang der Arabischen Golfküste zeigen ein alarmierendes Mass an Zerfall nach einer Zeitspanne von 10 bis 15 Jahren. Allmähliche Korrosion der Bewehrung die die Absplitterung des Betons verursacht überwiegt andere Kausalfaktoren. Der Beitrag beschreibt die Methoden der Lagevermessung und der Entnahme von Betonproben. Abgesplitterte Betonoberflächen wurden photogrammetrisch vermessen. Der Chloridgehalt, der elektrische Widerstand und die Betonqualität wurden bestimmt. Die Resultate zeigen eine starke Korrelation zwischen diesen Faktoren und dem Grad der allmählichen Korrosion der Bewehrung.

1. INTRODUCTION

Studies on the field performance of concrete in the Gulf area provide a unique opportunity to evaluate parameters bearing directly on the durability characteristics of concrete construction in an aggressive service environment. The boom in the construction activity for the last two decades has brought in its wake the concrete frame and the concrete block as the most popular form of construction. There has been an unprecedented demand for concrete buildings of all kinds and the local construction industry, beset by an inadequate infrastructure, shortage of suitable materials, equipment, skilled manpower and inadequate specifications and construction practices has succeeded only in producing structures which are showing an alarming degree of deterioration within a short span of 10 to 15 years. The deterioration is accentuated by the environmental conditions which are characterized by high temperature-humidity regimes combined with severe ground and ambient salinity (1).

The data collected in this investigation is part of a large scale durability study being presently undertaken at the University of Petroleum and Minerals with objectives to evaluate the deterioration problem in the area, to comprehend its magnitude in relation to affecting parameters, to develop quantitative estimates and indices of causal factors and their relative importance, to develop recommendations for obtaining improved durability of new concrete construction and to make proposals for retarding and repairing the existing deterioration. What appears most significant is a quantitative understanding of the interactive causal factors and to develop specific rating of concrete to withstand the severe exposure conditions.

2. METHODOLOGY OF INVESTIGATION

2.1 Framework of Investigations

The first phase of concrete deterioration research planned by the authors for the Gulf region is primarily based on collection, analysis and interpretation of field data from deteriorated field structures. The methodology adopted for these investigations is sequentially based on:

(i) Condition surveys of concrete buildings in order to establish a reasonable overall picture of the extent and severity of concrete deterioration.

(ii) Case studies related to special modes of deterioration employing destructive and non-destructive testing techniques, as well as chemical and petrographic examination of concrete samples obtained from the deteriorated field structures.

(iii) Analysis and interpretation of the data.

2.2 Sampling in Field Studies

A sampling plan for evaluating the deterioration of concrete in service is bound to be significantly different from the sampling at the production and placing stages for the purpose of material control or acceptance. Whereas in case of the later a viable mechanism in compliance with statistical procedures can be devised without incurring excessive cost and effort such is not the case with the former. Sampling of field structures and subsequently concrete from service conditions which may be considered truly representative of the properties and characteristics of the relevant population seems unrealistic even on the face of it. The inherent and well recognized original variability of the fresh concrete manufactured even under significantly controlled conditions is greatly compounded by subsequent variables in placement, consolidation, curing, protection and exposure when obtained from service structures. To this factor must be added regional characteristics such as variable and poor workmanship, lack in doctrination of the work force involved and lack of supervision and control.

In view of this position the sampling plan was devised so as to examine and evaluate a sufficiently large number of samples, each of which, even when it represents only itself as part of a larger population, will yield data which, taken together for all samples and subjected to careful analysis and interpretation, will enable the investigators to estimate relevant characteristics of the larger population. Two criteria were, however, invariably invoked in the sampling procedure:

(i) the samples were always obtained in a random manner so that the selection was based strictly on the element of chance rather than the choice or decision of the investigators;

(ii) a sufficiently large sample was obtained covering as far as possible the whole range of variation in terms of intensity of a particular form of deterioration.

In terms of actual logistics the following procedure was followed:

(i) The whole area under study comprising three cities of Dhahran, Dammam and Al-Khobar was delineated on maps in terms of smaller housing localities. A random selection was made of the localities wherein structures would be located for survey.

(ii) Each randomly selected locality was surveyed for comparatively old construction and in the first round information was gathered regarding the age of the construction only. In the execution of two surveys, concrete buildings falling in age groups of 15 to 20 years and 22 to 27 years were identified. Again a random selection of two to three buildings was made from each locality under study.

(iii) For each randomly chosen structure the owner was approached for permission and in most cases his cooperation was forthcoming. Each structure was first condition surveyed and the observations were recorded on two types of survey forms as suggested by Idorn (2). Proforma I is designed to record background information related with structure identification, construction type, site topography, exposure conditions, material data and relevant information about contractor, equipment, field force and, the degree and quality of workmanship and supervision. Proforma II recorded the general condition rating of the structure on a six point scale, extent and type of deterioration and the degree of repairs needed. Eighteen types of possible forms of deterioration were listed in Proforma II inviting related observations; the definitions of all these forms of deterioration were adequately established and illustrated with the help of photographic documentation.

Feedback at the initial stages of survey revealed difficultues in obtaining information on the initial quality of concrete, constituent materials, compositional aspects and construction practices. In the face of this difficulty, wherever necessary, initial concrete quality was rated on a three point scale as "poor", "good", and "excellent". These terms have been defined in relation to strength and concrete denseness. Reasonably settled standards of the degree of severity of each deterioration type were established by visual inspection with the help of preliminary case studies undertaken only for the prupose of practice inspections and by supplementing with photographic documentation. A little observational skill, developed with some measure of practice, provided satisfactory and meaningful rating of a particular deterioration.

(iv) Of the 62 surveyed structures 20 structures were selected randomly for an in-depth investigation of the causal factor identified from the condition survey. In the case of rebar corrosion studies, follow up investigations comprised:

(i) measuring precisely the area affected by corrosion deterioration in terms of concrete spalling and delineating regions according to deterioration ratings on the six point scale;

(ii) drilling concrete cores from randomly selected areas of deterioration in such a manner as to provide samples which represented individual points in the structure covering in each case, as far as possible, the range of deterioration spanned by the six point scale;

(iii) obtaining cover measurements non-destructively on partially spalled concrete floor slabs using an electromagnetic covermeter;

(iv) developing strength, concrete absorption and pulse velocity data from the cores removed from field structures;

(v) determining chloride content of the cored concretes; and

(vi) measuring the electrical resistivity characteristics of concrete.

2.3 Techniques

2.3.1: Photogrammetric Technique for Measuring Area and Depth of Concrete Spalling at Inaccessible Locations:

A simple pictorial technique has been developed to measure the three dimensions of an object at any distance from 5 to 20 meters. A mirror stereoscope is mounted in front of a normal hand-held non-metric camera as shown in Fig.1. Two pictures will appear on the same photographic slide. Since the two photographs are not taken from the same point the two pictures will not be exactly identical and a part of the object will appear twice.

Referring to Figs.1 and 2 the distance between the centers of the two apertures of the mirror stereoscope is the stereoscopic base B. This produces a parallax P_x which is the difference between the two locations of the same photographed object. It is equal to (M - N) where M is the distance between the centers of the two pictures and N is the distance between the two object photographs. If the focal length of the camera is F and the coordinate system is chosen as shown in Fig.2, the space coordinates of a point object are computed from:

$$Y = \frac{FB}{P_x} \qquad (1)$$

$$X = \frac{Y}{F}\, x \qquad (2)$$

$$Z = \frac{Y}{F}\, z \qquad (3)$$

where x and z are the photo-coordinates of the object in the left photograph. The first equation is used in the determination of the scale factor S=Y/F for a body object at an average distance of Y which includes the distance C between the two parallel mirrors. The components of the object dimensions can be obtained from the first derivatives of equations (1) to (3).

$$dY = \frac{F}{(M - N)}\, S \;\; dP_x \qquad (4)$$

$$dX = S\, dx \qquad (5)$$

$$dZ = S\, dz \qquad (6)$$

dP_x, dx and dz can be measured from the pictures by using a special 0.1mm optical scale. If the photographic slide is placed on an illuminated table the measurements can be conveniently carried out to the nearest 0.02mm by estimation.

The view can be observed stereoscopically by placing the slide in a special viewer which operates in a reverse concept.

Nikon F3 with a normal 50mm lens was used in the present investigations. If it is focussed to infinity the minimum photographing distance, corresponding to a circle of confusion of 0.05mm, is 5m. The base B of the mirror stereoscopic adapter was chosen approximately to be 70cm. Experiments were carried out to find with high precision the most probable values of the three constants, namely; the focal length F, the base B of the mirror stereoscope and the distance between the centers of the slide stereopairs M. A test area was established where distances to 25 scales were measured precisely with a steel tape. The total length of each scale was 4 meters and residual equations are formed from equations (1), (2) and (3). The required unknowns have been determined from the least squares solution and found to be: F = 4.975mm, B = 70.740mm and M = 18.641mm. These values are used in Equations 1, 2 and 3 to determine the object dimensions. Experiments show that the standard error (in mm) of the object dimensions in a plane perpendicular to the camera axis is given by 4 X Y (X and Y are in meters).

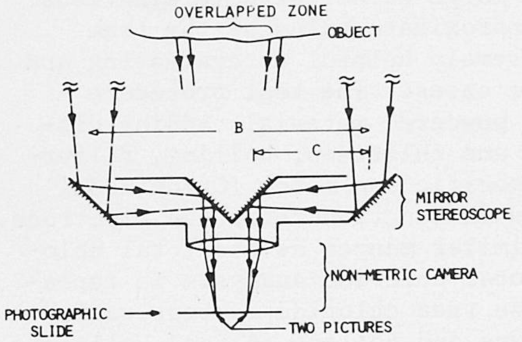


Fig.1: The mirror stereoscopic photographic arrangement

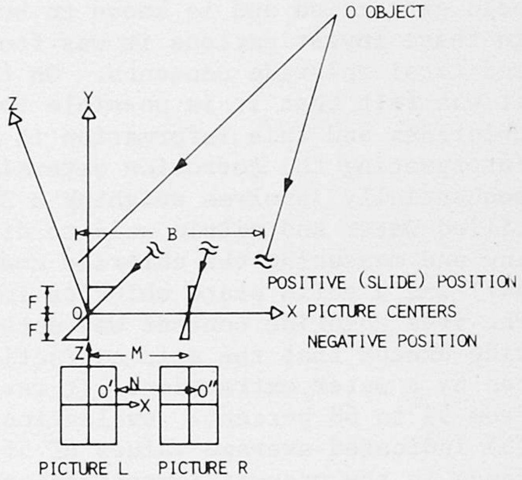


Fig.2: The geometry of the photogrammetric technique

2.3.2: Depth of Concrete Cover:

Results of condition surveys show that in most cases the corrosion and severe concrete spalling could be attributed to atleast one of the many possible parameters - insufficient concrete cover to reinforcement. Cover measurements were made non-destructively on partially spalled concrete slabs using an eletromagnetic covermeter which requires only a knowledge of the rebar diameter. Cover measurements were made on a grid system over the concrete surface and this enabled equi-depth of cover contours to be drawn for the area under study.

2.3.3: Chloride Analysis of Concrete Samples:

In the coastal flats of the Gulf calcium, magnesium and sodium salts of sulfates, chlorides and carbonates extensively contaminate the ground, groundwater and the moisture-laden environment. Consequently, salts permeate concrete, firstly, through the contaminated aggregates, brackish mix and curing water and subsequently as a result of ingress through cracks and pores. Concrete construction on the Gulf seaboard is continually exposed to ground and atmosphere charged with salt. Aided by capillary action, dew and high humidity conditions the salt contaminated groundwater and the salt-laden airborne moisture find an easy ingress in the exposed concrete matrix. The unusual high incidence of rebar corrosion against the backdrop of a highly salt-polluted environment puts chloride ion as the most important potential cause for reinforcement corrosion. Chloride ion, inducted into concrete through salt permeation, is a specific destroyer of the protective gamma ferric oxide film surrounding the reinforcement and is especially effective in eliminating passivity against corrosion. This activates the electrochemical process wherein corrosion cells are formed due to non-uniformities either in the rebar material structure or in its enveloping environment. Chloride ion determinations in the sampled concrete, therefore, constitute an important part of the evaluation test program. Chlorides can be present in concrete in three forms (3):

(i) Free chloride ion

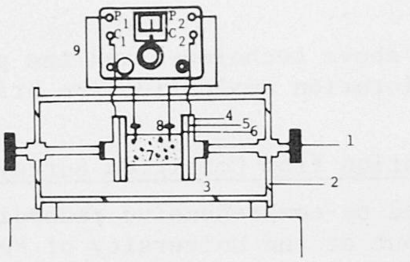
(ii) Chloride bonded strongly with the calcium silicate hydrates

(iii) Chloride combined in compounds such as calcium aluminate chlorides.

Although it is the quantum of free and weakly bonded chloride which is the operative parameter in the corrosion process, its determination in concrete is rendered uncertain by the sensitivity of the test to a large number of procedural parameters such as sample size, extraction medium, soaking time, temperature, etc. As against this the total chloride determination involves a nitric acid extraction and is known to be unaffected by the aforesaid factors. However, in these investigations it was found useful to determine both the free chloride and total chloride contents. On the basis of a large number of determinations it was felt that it is possible to measure an approximate value of the free chlorides and this information is sometimes extremely helpful in evaluating and interpreting the corrosion determination in some cases. The test procedure sequentially involves weighing a 3-gm sample of powdered material, adding distilled water and nitric acid to dissolve cement and chlorides, boiling, filtering and measuring the chloride content with a specific ion meter (Orion Model 407) and a solid state chloride ion activity double junction reference electrode. The free chloride content was determined in a similar manner as the total chloride except that the acid extraction used for total chloride analysis is replaced by a water extraction. It was found that the free chloride content varied from 52 to 68 percent. Evaluations made by Browne and Bolling (4) and Lolivier (5) indicated average values of 55 and 65 percents respectively. The variation range in the present investigations is based on the fact that the proportion of free chlorides depends on the total chloride content and on the source of the chlorides i.e. whether inducted at the mixing stage or at a later stage by ingress through pores and cracks.

2.3.4: Electrical Resistivity Measurements:

The electrical resistivity of concrete ranges from around 10^3 ohm-cm when saturated to 10^{11} ohm-cm when oven dried. Rebar corrosion is an electrochemical reaction necessitating a potential difference along the bar in order to activate a current from the anode to the cathode. The magnitude of the corrosion current is primarily controlled by the resistivity of the concrete. High resistivity reduces current and also the probability of corrosion. Resistivity measurements on concrete samples were made according to a set up shown in Fig.3, using an integrated and compact instrument commercially available as Nilson 400. The instrument is a 4 terminal null balancing ohm-meter (0.01 ohm to 1.1 meg-ohms) which generates a low voltage 97 HZ square wave current between the binding posts C_1 and C_2 (Fig.3). The detector with its input connected between binding posts P_1 and P_2 is only responsive to 97 HZ and remains insensitive to any spurious potentials or stray currents. This eliminates a frequently occuring source of error. Copper pins were inserted in concrete cores and were directly connected to binding posts P_1 and P_2 for voltage drop measurements. The cores were held horizontally in a steel frame with end plates made of copper and covered with a cloth pad which was kept wetted with water. The copper plate-wet cloth combination ensured a uniform distribution of current density across the face of the core.



1. Screw for gripping the block
2. Steel frame
3. Insulator
4. Steel plate
5. Copper plate
6. Cloth pad
7. Concrete core
8. Copper pins inserted in concrete cores
9. Nilson 400 resistivity meter

Fig.3: Electrical Resistivity Measurement of Concrete.

2.3.5: Tests for the Evaluation of Concrete Quality:

As pointed earlier, sufficient information was not available on the concrete samples from field structures in terms of mix specifications and constructional parameters such as placement, consolidation, curing, etc. which significantly influence concrete quality. Therefore concrete quality was evaluated in terms of pulse velocity and water absorption measurements corroborated, wherever possible, with compressive strength determination on cores.

Proprietary Pundit equipment was used with 82 KHZ transducers coupled to the smooth end faces of a core with the help of sticking grease. Care was exercised in ensuring good acoustic coupling between the transducer face and the surface of the core as well as in measuring the path length and transit time. In general the higher the velocity of the pulse the higher the concrete quality.

Concrete permeability is the pre-eminent criterion governing its durability performance in aggressive environments. Mechanisms of rebar corrosion and sulfate attack which dominate concrete deterioration in the Gulf environment are extremely permeability oriented. It follows that in the Gulf conditions concrete should be sufficiently dense and impervious for high durability performance. Concrete permeability determination in the field has met with little success and even in the controlled conditions of the laboratory it is a difficult test to perform. Water absorption measurements being known to be intimately connected with the permeability characteristics, these investigations have largely relied on a 30-minute absorption test and on initial surface absorption test developed by Levitt (6). Both these tests are fully described in BS 1881 Part 5. Although there is a degree of arbitrariness associated with the results of these tests, on the basis of a large number of measurements Table 1 has been formulated for an interpretation of concrete quality. Whereas a broad correlation has been obtained between the two techniques, experience shows the initial absorption test to be more consistent compared to the 30-minute absorption test.

Ratings on the basis of 30-min Absorption Test		Suggested limits of ISAT for good quality concrete	
30-minute Absorption Value %	Concrete Quality	Duration of Test (Minutes)	Max.ISAT limits ml/sq.m/s
2.5 and lower	Good	10	0.40
Between 2.5 and 5	Medium	60	0.20
Between 5 and 10	Poor		
Above 10	Very Poor		

Table 1: Absorption values and concrete quality.

3. RESULTS

Typical applications of the above techniques and the possible interpretations in terms of corrosion deterioration evaluation are briefly discussed in this section.

3.1: Evaluation of Deterioration from Condition Surveys:

Condition surveys accompanied by comprehensive recordings and photographic documentation were carried out at the University of Petroleum and Minerals on 42 concrete framed structures located in Al-Khobar, Dhahran, and Dammam habitations along the Gulf coast in the Eastern Province of Saudi Arabia. The objectives were limited to an establishment of the boundaries and parameters of the concrete deterioration problem and an investigation of the relative importance of the operative causal factors. The results show an alarming condition of structures constructed 15-20 years hence. The surveyed structures were constructed during the years 1960-64. Although the general condition of each structure was recorded as comprehensively as possible, the detailed deterioration recordings were made only on concrete exposed to the ambient environment. This made it possible to hold at least two variables - age and inservice exposure - reasonably constant in order to study the effect of other variables on concrete deterioration. Fig.4 classifies on a six point scale the general condition rating of 168 study areas from these 42 structures. The figure shows that 48% of the observations group in the classifications 5 and 4 which corresponds to a very unsatisfactory condition range of the rating and about 19% in classifications 3 which is also far from satisfactory. Only 36% manifest slight or no deterioration. Illustrations of classifications 4 and 2 are shown in Figs.5 and 6 respectively.

Legend on Classification of Deterioration

Classification	Extent of Deterioration
0	No visible sign of deterioration.
1	Slight in severity and extent
2	Severe but limited in extent
3	Slight but extending over major portion of area under study.
4	Moderate over whole area of inspection
5	Severe, overall deterioration

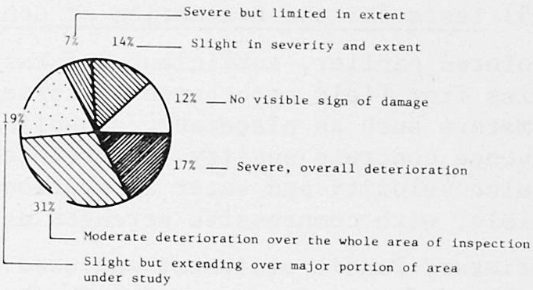


Fig.4: Classification of condition of 168 observations from concrete structures located in Eastern Saudi Arabia.

3.2: Evaluation of Concrete Spalling Areas due to Rebar Corrosion:

Figs. 5 and 6 show two structures which were condition surveyed and where concrete spalling has occurred as a direct outcome of visible heavy rebar corrosion. In each of these figures two pictures appear of the same structure on one photographic slide. Photogrammetric technique has been used for measuring the area of spalling and the results are shown in Table 2.

Structure No.	Total area under observation (m^2)	Area showing cracking and initiation of Spalling (m^2)	Spalled Area (m^2)
1	4.65	1.19 (26%)	1.77 (38%)
2	4	0.13 (3%)	1.35 (34%)

Table 2: Evaluation of concrete spalling areas due to rebar corrosion.

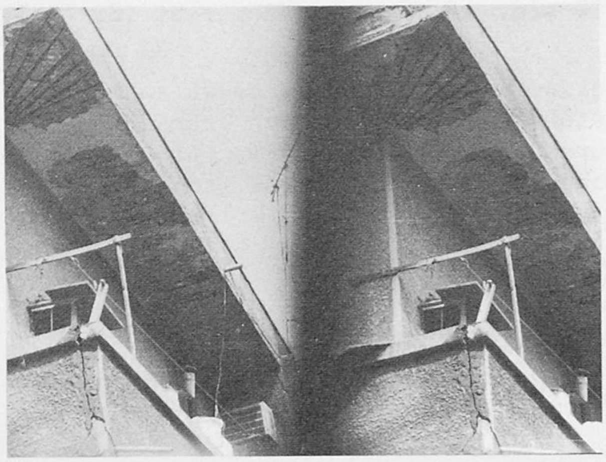


Fig. 5 Structure 1 showing concrete spalling due to rebar corrosion

Fig. 6 Structure 2 showing concrete spalling due to rebar corrosion

3.3: Metal Loss of Reinforcement and Chloride Content:

Concrete cores from exposed slabs of 24 structures 17-22 years old were obtained for laboratory analysis of chloride levels at the steel-concrete interface. Fragments of concrete were removed from the steel-concrete interface and were analyzed for chloride content. For each sample six to ten points were obtained and standard deviations worked out for this data. Steel corrosion was measured by removing from each specimen six to ten 7.5cm pieces of rebars. The loose rust was removed by an emery paper and the weight loss was measured. The results of this analysis for selected samples pertaining to 1.25cm cover are plotted in Fig.7. As concrete quality, cover, electrical conductance characteristics, and the soluble chloride content are the four supposedly dominant parameters affecting corrosion, samples were selected for the plot which enabled the first three variables to be held reasonably constant. Understandably, from among one hundred available observations only a few met the aforesaid criterion (Fig.7). Concrete

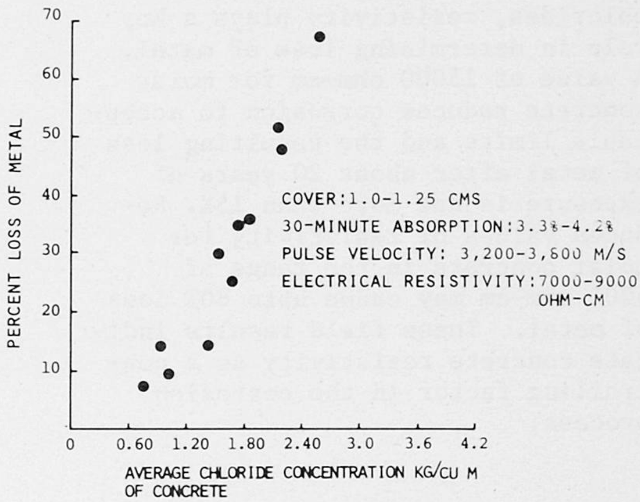


Fig.7: Relationship between Chloride content and metal loss.

quality was ascertained on the basis of a 30 min. water absorption test (BS 1881) and the pulse velocity measurements.

It would be difficult and an oversimplification to read obvious cause and effect relationships between certain causative factors and deterioration from this presentation. However, certain broad features of the deterioration problem as affected by the presence of chlorides in concrete can be inferred from this data analysis.

(i) The chloride ion and its concentration in concrete have a very definite influence on corrosion deterioration.

(ii) For a 1.25cm cover the threshold chloride concentration for the exposure conditions of Eastern Saudi Arabia is about 0.6 kg/m^3. This value is close to the concentration of chlorides permitted by ACI Committee 201 on Durability of Concrete.

(iii) Gulf concretes generally show a high chloride concentration.

3.4: Electrical Resistivity of Concrete and Rebar Corrosion Deterioration:

The significant effect of concrete resistivity on corrosion is seen in Fig.8 where the loss of metal is plotted against resistivity for narrow variations in the values of cover to reinforcement, chloride content and concrete quality. The results show that given sufficient chlorides, resistivity plays a key role in determining loss of metal. A value of 15000 ohm-cm for moist concrete reduces corrosion to acceptable limits and the resulting loss of metal after about 20 years of exposure is not more than 15%. Reduced values of resistivity for moist concrete in the range of 6000 ohm-cm may cause upto 80% loss of metal. These field results indicate concrete resistivity as a controlling factor in the corrosion process.

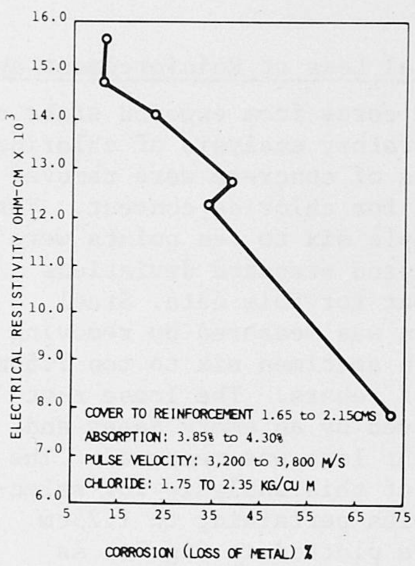


Fig.8: Relationship between electrical resistivity of concrete cored from field structures and metal loss.

3.5: Cover to Reinforcement and Corrosion Damage in Terms of Concrete Spalling:

In the condition survey carried out on 42 concrete framed structures 15-20 years old, 76 spalls of varying dimensions and severity were observed during 168 observations covering approximately 112 sq.m. of concrete area. In 68% of the observed spalls the thickness of concrete cover was less than 1.25 cms, in 53% it was less than 0.95 cms and in 18 observations it was less than 0.65 cms. There were 7 cases (9.2%) where there was almost no cover to steel reinforcement. Fig.9 shows the distribution of concrete cover observed in spalled concrete. Fig.10 is the typical presentation of cover measurements on floor slabs with partial spalling. The spalls were found to be invariably located in regions of insufficient concrete cover.

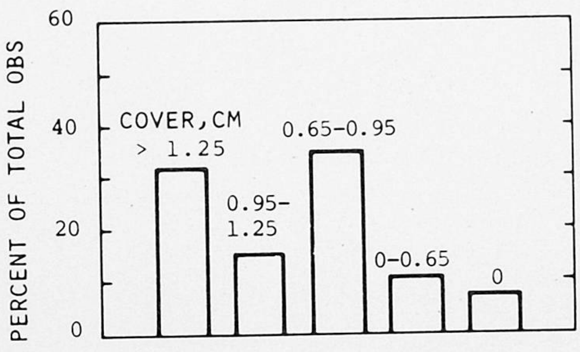


Fig.9: Distribution of concrete cover in spalled concrete

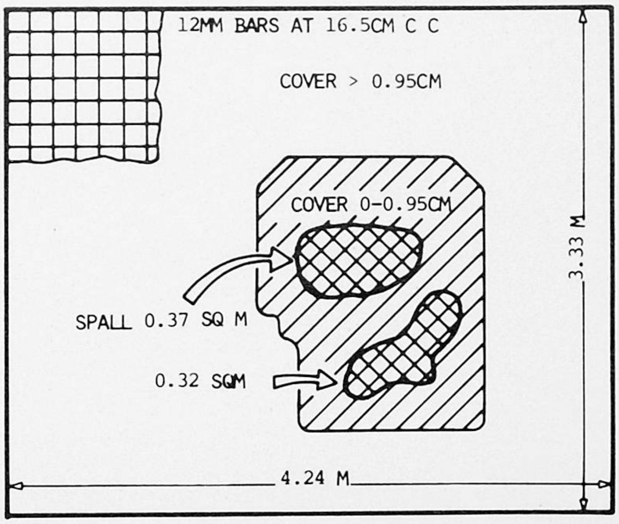


Fig.10: Concrete spalls related to inadequate cover.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the support provided by the Saudi Arabian National Center for Science and Technology (SANCST) and the University of Petroleum and Minerals, Dhahran, for research on the durability of concrete in Eastern Saudi Arabia.

REFERENCES

1. Rasheeduzzafar, Dakhil, F. H., Al-Gahtani, S. A., The Deterioration of Concrete Structures in the Environment of Eastern Saudi Arabia. The Arabian Journal for Science and Engineering, Vol.7, No.3, 1982.
2. Idorn, G. M., Durability of Concrete Structures in Denmark. Danish Technical Press, Copenhagen (1967).
3. Berman, H. A., Determination of Chloride in Hardened Portland Cement Paste, Mortars and Concrete. Report No. FHWA-RD-72-12, Federal Highway Administration, September 1972.
4. Bolling, N. B., and Browne, F. P., A New Technique for Analysis of Chlorides in Mortars. Journal of Materials, Vol.6, No.3, September 1971.
5. Lolivier, J., Determination of Chlorides in Hardened Concrete. International Symposium on Admixtures in Mortars and Concrete, Brussels, 1967, RILEM-ABEM, Report, V/II, Vol.V.
6. Levitt, M., The ISAT - A Non-destructive Test for the Durability of Concrete. British Journal of Non-destructive Testing, July, 1971.

Leere Seite
Blank page
Page vide