Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 56 (1987)

Rubrik: Session 1. Criteria for development and operation of monitoring systems

SESSION 1

Criteria for Development and Operation of Monitoring Systems

Critères pour le developpement et l'utilisation des systèmes de surveillance

Kriterien für das Entwickeln und Anwenden von Überwachungsystemen

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Preservation of Historically Important Buildings

Conservation de bâtiments d'importance historique

Erhaltung von historisch wertvollen Bauten

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Fritz Wenzel is a structural engineer. He lectures on structures at the faculty of architecture, Univ. of Karlsruhe. He is concerned with the diagnosis and therapy of old building structures, in research and practice. As a consulting engineer he participated in the restoration of many historic buildings and he has reported on this work in numerous publications, lectures and seminars.

SUMMARY
In the summer of 1985 the Special Research Programme «Preservation of Historically Important Buildings» took up work at the University of Karlsruhe with research projects carried out by the Institute of Bearing Structures, the Departments of Building Material Technology, Timber Construction, Steel Construction, Soil Mechanics and the Institute of Mineralogy together with the Institute of Architectural History and the Provincial Monument Protection Agency. This article gives some information about the programme and describes first results.

RESUME
L'été 1985 l'institution de recherche spéciale «Preservation des Monuments Historiques» a été établie à l'Université de Karlsruhe. Elle consiste de l'institut de statique, l'institut des Matériaux de construction, de l'institut des constructions de bois, l'institut des constructions de fer, l'institut de minéralogie, l'institut d'histoire de l'architecture et de l'autorité de préservation des monuments historiques de l'Etat de Baden-Württemberg. Cette contribution renseigne sur le programme de recherche et représente les premiers résultats.

ZUSAMMENFASSUNG
Im Sommer 1985 nahm der Sonderforschungsbereich «Erhalten historisch bedeutsamer Bauwerke» an der Universität Karlsruhe seine Arbeit auf. Er wird getragen von den Instituten für Tragkonstruktionen, Baustoff-technologie, den Lehrstühlen für Ingenieurholzbau, Stahl und Leichtmetallbau, dem Lehrstuhl für Bodenmechanik und Grundbau, dem Institut für Mineralogie, sowie dem Institut für Baugeschichte und dem Landesdenkmalamt Baden-Württemberg. Dieser Beitrag gibt Auskunft über das Programm und schildert erste Resultate.

1. EXCERPTS OF THE RESEARCH PROGRAMME 1985

The SFB 315 at the University of Karlsruhe "Preservation of historically important buildings" is intended to help revive work with old constructions and building materials. It will focus on the cause of decay and on methods and procedures to preserve and improve the substance. Solutions are to be found that are compatible with the monumental value of old buildings by minimizing both the destruction of the original substance and any addition of technical aids. The costs can be reduced accordingly by carefully-aimed measures. Besides basic and applied research, contributions are to be made to our knowledge of the history of construction and technology.

1.1 PARTICIPATING DISCIPLINES, INSTITUTIONS AND INDIVIDUALS

The SFB 315 concerns the faculties of

- architecture
- civil engineering and surveying
- biology and geology

as well as the

- "Landesdenkmalamt Baden-Württemberg."

It is conducted by the following institutions and invividuals:

- Institut für Tragkonstruktionen
  Prof. Dr.-Ing. Fritz Wenzel (Speaker of the SFB)
- Institut für Baugeschichte
  Prof. Dr.-Ing. Wulf Schirmer
- Institut für Massivbau und Baustofftechnologie,
  Abteilung Baustofftechnologie
  Prof. Dr.-Ing. Hubert Hilsdorf
- Versuchsanstalt für Stahl, Holz und Steine,
  Abteilung Stahl- und Leichtmetallbau
  Prof. Tekn. dr Rolf Baehre
- VersuchsanstaltfürStahl,Holz undSteine,
  Abteilung Ingenieurholzbau und Baukonstruktionen
  Prof. Dr.-Ing. Jürgen Ehlbeck
- Institut für Bodenmechanik und Felsmechanik,
  Abteilung Bodenmechanik und Grundbau
  Prof. Dr.-Ing. Gerd Gudehus
- Mineralogisches Institut
  Prof. Dr. Egon Althaus
- Landesdenkamt Baden-Württemberg
  Prof.Dr. August Gebeßler,Präsident des Landesdenkmalamtes
  Baden-Württemberg

1.2.1 PROJECT FIELD A, BUILDING INVENTORY, HISTORY, MONUMENTAL VALUE

The project field A has been titled "Building Inventory, History, Monumental Value" and is conducted by the "Landesdenkmalamt", the Institute of Architectural History and the Institute of Bearing Structures. As suppliers and recipients of data and information, the three subprojects will be linked especially close to each other and to the other groups in the more technically oriented project fields B and C.
Within subproject A1, the "Landesdenkmalamt" will name those buildings suitable as exemplary research objects for the different areas of interest to the SFB and will make them accessible. It will analyse and determine the monumental value, will accompany the engineer-technical examinations from its history-related point-of-view and analyse the securing measures proposed in respect to their compatibility with the monumental value.
The subproject A2 is intended to contribute to the knowledge of the history of building construction and technique by examining exemplarily selected buildings under the aspect of architectural history. This work deals with general historical questions concerning foundations and mortar composition. The understanding of the particular time, the past ways and means in working with constructions and material should be considered. The architectural historians will be able to compile a sequence of research and examination results by participating in the work done by the other subprojects. By completing and arranging them systematically, an important contribution to technical and constructional history could be achieved.
Within subproject A3, examinations are to be carried out on existing repaired buildings to examine how effective the methods and techniques used to secure old building structures and constructions have been after a number of years or decades have passed. The strong and weak elements in current repair techniques are to be determined to bring about purposeful future improvements.

1.2.2 PROJECT FIELD B BUILDING MATERIAL, CHARACTERISTICS, WAYS OF IMPROVEMENT

The four subprojects of the project field B are summarized by the title "Building Material, Characteristics, Ways of Improvement". These projects are carried out by the Departments of Buildings Material Technology, Timber Construction and Soil Mechanics as well as by the Institute of Mineralogy.
The aim of subproject B1 is to develop effective damp protection methods. First steps will be to work on the physical phenomenon of moisture travel and on the damage caused by dampness. Using data derived from experiments, the moisture protection of porous building material should be characterized mathematically to be able to describe dampening and drying procedures with model calculations.
Within subproject B2, wood-testing methods are to be developed that do little or no damage to the substance. They are to be applied in practice to achieve information on the load-bearing and deformation behaviour of old timber. This subproject will continue, in follow-up work, by systematically examining old timber in buildings and establishing rules for judgement.
The subproject B3 is concerned with those subsoil properties, weaknesses of foundations and environmental effects that to this day cause subsidence and movements of historical buildings with shallow foundations. At the beginning, the continuous settlement of heavily loaded foundations on clayey soil will be of greatest concern as well as

continuing movements and deformation of retaining walls and buildings on clayey slopes. Later, questions will be of interest dealing with the effects of traffic vibration in connection with granular soils and with buildings on expansive soil.
In subproject B4, the Mineralogical Institute, acting as a central laboratory for the whole SFB, is to carry out the experiments needed within the other subprojects dealing with fabric composition and structural properties of materials as far as mineralogical methods can be of any help. It is to be explored which mineral components and processes lead to the observed damages in buildings and which mineralogical and geochemical procedures could help repair them or preserve the repaired condition of the particular building monuments.

1.2.3 PROJECT FIELD C CONSTRUCTIONS, LOAD-BEARING PERFORMANCE, STRENGTHENING TECHNIQUES

The project field C "Constructions, Load-bearing Performance, Strengthening Techniques" covers four subprojects of the Departments of Timber Construction, the Institute of Bearing Structures and the Departments of Steel Construction and Soil Mechanics.
The topic of subproject C1 will be joints and connecting means in old timber constructions. The most important types used in the past are to be recorded. Then, experiments are to be made to determine the bearing and deforming performance. Finally, criteria of judgement are to be derived which allow for the bearing security of old wooden joints and connections to be determined without reference to modern building regulations which apply to new buildings.
The subproject C2, continuing from previous work, is to deal with procedures to determine material strength and deformation values of old masonry that vary greatly. These procedures should be more descriptive than those used when examining stone and mortar samples individually and they should be less destructive than those where complete small pillars are removed. Later on, work should be done using these results and improving the load-bearing and deformation performance of single and multi-shell masonry. The possibilities to stabilize severely torn masonry walls by applying prestressing forces without bonding activity should be investigated now.
Subproject C3 is concerned with iron and steel structures of the 19th century. They date from a time at which the profession of master builder split into those of architect and civil engineer and they therefore represent an important phase in the development of engineering construction. Together with the architectural historians and the conservationists a catalogue of objects worthy of preserving is to be compiled. Material tests and experiments on the load-bearing performance are to help achieve a basis to assess the load-bearing ability, the remaining time for further use, possible alterations in use and suitable securing and repair measures for the structures.
The subproject C4 deals with construction activity within, in the vicinity of and underneath historical buildings. Methods are to be developed to predict the behaviour of the soil and the building in such cases. The investigations give priority not only to achieving or preserving structural stability through such activity but also to preserving the historical building substance as far as possible including the foundations.

1.2.4 PROJECT FIELD D
DOCUMENTATION, INFORMATION, ADMINISTRATION

The documentary and transfer office is responsible for the establishment of a commented bibliography, for building documentation and subsequently for the compiling of a register of those valuable constructions dealt with in the selected buildings within the SFB, furthermore for central inventory and archives within the SFB. In addition it is reponsible for the publication series containing the research results of the SFB as well as the recommendations and case studies for practice. Another task will be organizing meetings and workshops and co-ordinating the work of the University Institutes and the "Landesdenkmalamt", especially when buildings are to be examined, and bringing work in the line with the other research groups.

2. FIRST RESULTS

2.1 SUBPROJECT A3: ENGINEERING EXAMINATION OF EXISTING BUILDINGS PREVIOUSLY REPAIRED
FRITZ WENZEL, MICHAEL ULLRICH, HELMUT MAUS:

For decades damaged masonry of historically important buildings has been structurally strengthened by injecting mortar suspension consisting generally of cement mortar, by inserting reinforcement steel and prestressing bars. Reports on these methods exist from as early as the 1920s. These repair techniques are to improve the masonry, handle local tensile stress and re-establish the ability of torn walls to transfer loads.
There are obvious stabilizing effects achieved by grouting and reinforcing old masonry but there are also indications coming from several sources that unsuitable grouting material and corrosion of steel may lead to damages.
Systematic follow-up examinations concerning possible subsequent damages had not been undertaken until recently. Now two groups, in Karlsruhe and Münster, have compiled first results of their work. These were achieved by conducting inspections of old masonry buildings that had been structurally repaired, by exposing reinforcement members, by core-boring and by using an extendable endoscope. There were positive as well as negative results.
On the one hand cavities were found in grouted masonry and old mortar was not entirely surrounded by injection material, additionally steel bars were found lacking sufficient protection against corrosion due to inadequate cement covering. On the other hand the investigations also revealed completely filled cracks and gaps, thoroughly mixed and strengthened old mortar as well as sufficiently covered reinforcement bars without any trace of rust. Measuring the tensile forces in the reinforcement bars, first carried out 11 years ago, was resumed and shows the longterm performance of inserted prestressing members.
Quite different the results of more than 20 buildings in Lower Saxonia. These buildings with walls made of gypseous brick work are in a very bad shape after they were injected with cement mortar 15 to 20 years ago. These injections caused inside the walls a crystallization with volume increase and as a result of this: cracked walls. The flow of injectionmaterial, the position of needles and anchors within the drillholes could be checked in situ after opening walls for repair. Furthermore samples of stones and mortar - decomposed by cristallization - could be collected for laboratory tests. In one building resin-based-mortar was injected 12 years ago, but after opening the wall the material still was liquid and dropped out of the opening.

2.2 SUBPROJEKT B2: LOAD-BEARING AND DEFORMATION BEHAVIOUR OF OLD CARCASSING TIMBER

JÜRGEN EHLBECK, RAINER GÖRLACHER:

The specific density of wood is an important property to assess the quality of wood. This value correlates closely with the wood strength especially with the compressive strength parallel to the fibre. The determination of the specific density is generally conducted according to the DIN 52182 (German Code). For the procedure square or oblong samples are recommended depending on the objective of the examination. The specific density of in-situ building timber (in-situ-measurement) can only be determined by removing samples and therefore material is destroyed if this method is applied. Since it is usually most interesting to examine the properties of highly-strained building members a non-destructive method becomes necessary that will not or at least not substantially damage the members.

Such a non-destructive testing method to determine the specific density of in-situ timber is the Pilodyn technique.
In applying the wood-testing-device PILODYN 6J a steel pin penetrates the wood with a specified amount of energy whereby the depth of penetration is measured. The specific density can then be estimated with the help of regression analysis taking into account the moisture content of the wood and the number of measurement attempts. There was no proof that the angle between the direction of penetration and the course of the annual rings had any influence on the results. First measuring attempts on old timber showed no systematic deviations from measurements on new timber. The number of tests conducted to date is not sufficient to draw any final conclusions. In certain cases, though, this procedure may represent a valuable support in examining the condition of existing timber structures.

2.3. SUBPROJECT C3: IRON AND STEEL STRUCTURES OF THE 19TH CENTURY ROLF BAEHRE, RUDOLF KÄPPLEIN

The cast-iron hollow columns widely used in construction during the second half of the 19th century are expected to reveal considerable irregularities in their form originally not intended. The eccentricities in the cross section occurred during casting in horizontal position when the mould core slipped out of position. There are two techniques at hand to determine the circumferential course of wall thickness of the column shafts - one working with ultrasonics, the other using a mechanical measuring device developed within subproject C3.

These developments from the basis of determining the quantity of eccentricities to consider them when calculating the stability of cast-iron hollow columns.

References: Erhalten historisch bedeutsamer Bauwerke, Forschungsprogramm 1985, Berlin 1987
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BILDUNTERSCHRIFTEN

fig. 1 Unsuccessful result of grouting. Photograph taken through an endoscope. Cavities are shown that were not reached by grouting mortar. The old mortar had not been stabilized. It was washed away during the survey drilling.

fig. 2 Successful result of grouting. The joint is completely filled. The cement mortar is well interlocked with the porous stone and the lime mortar.

fig. 3 Negative example of protection against corrosion. Reinforcement bar without enough protective cement mortar covering. The diameter of the drill hole was to small.

fig. 4 Good example of protection against corrosion. Photograph taken through an endoscope. A steel reinforcement bar in the middle which has been cut at an angle during survey drilling. Sufficient protective covering with grouting mortar.

fig. 5 The Pilodyn 6J wood tester including loading rod and 3 spare striker pins.

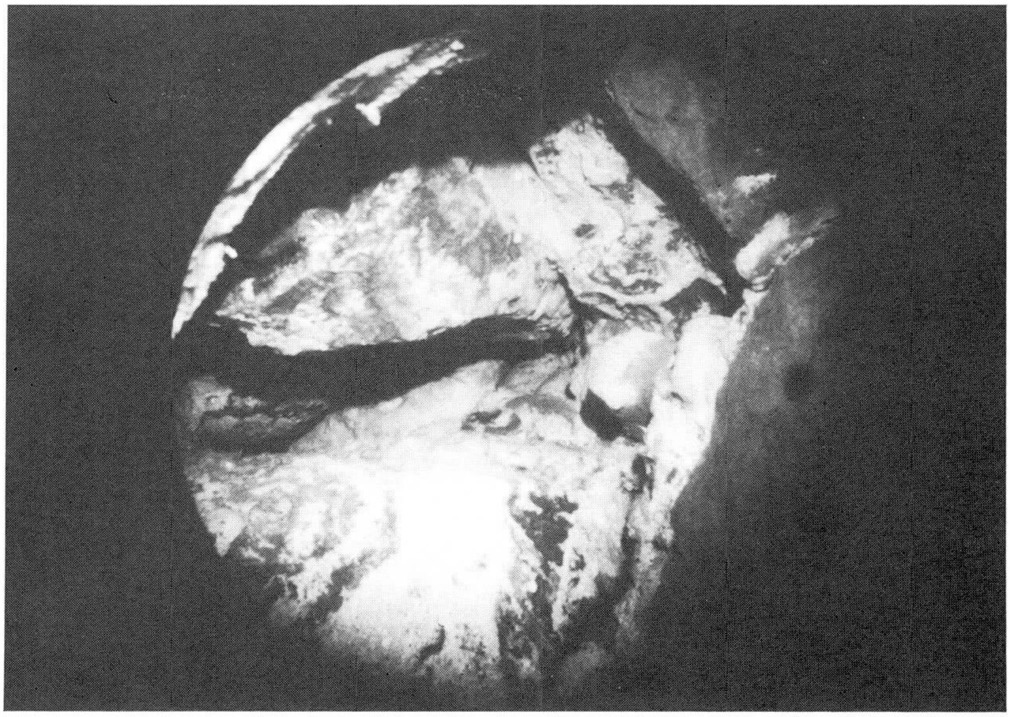
fig. 6 Relationship between density and Pilodyn 6J penetrations in spruce (Piceae abies). + = average of 16 measurements.

fig. 7 Relationship between density and Pilodyn 6J penetrations. Oak, pine and fir from the 18th century.

fig. 8 Cast-iron hollow columns with varying wall thickness due to slipping of the mould core.

fig. 9 Cross sections of cast-iron hollow columns.

fig. 10 Mechanical wall-thickness-measuring-device.



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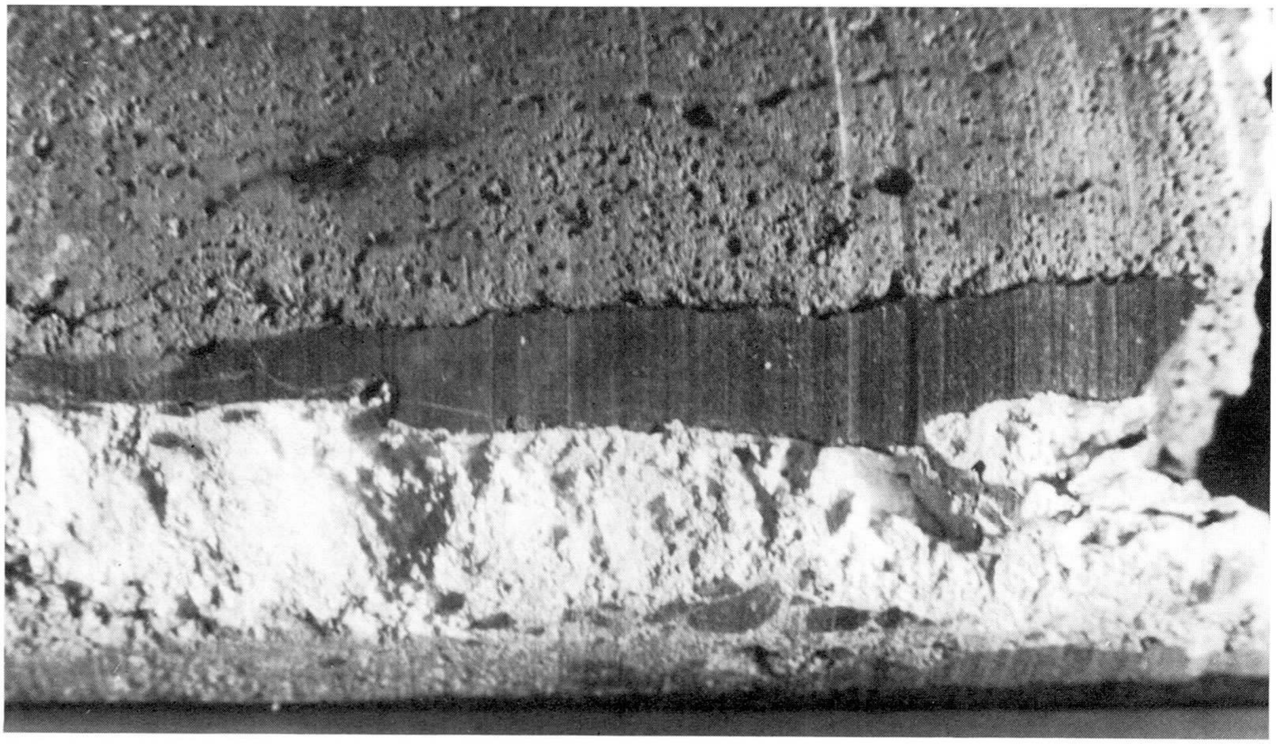

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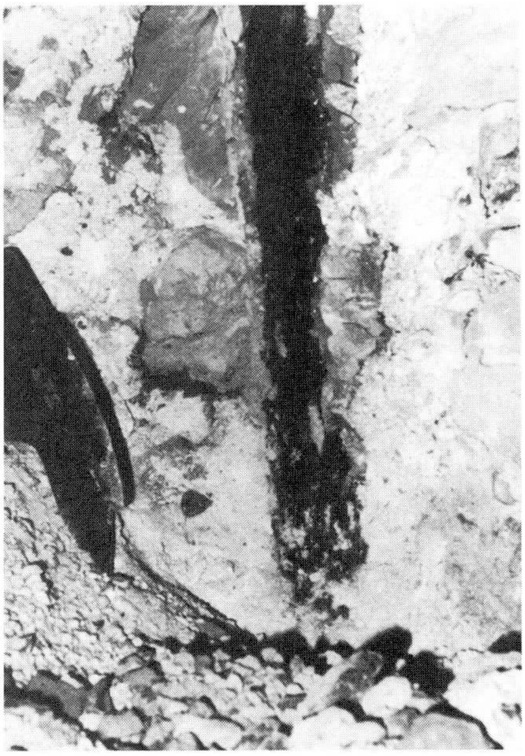

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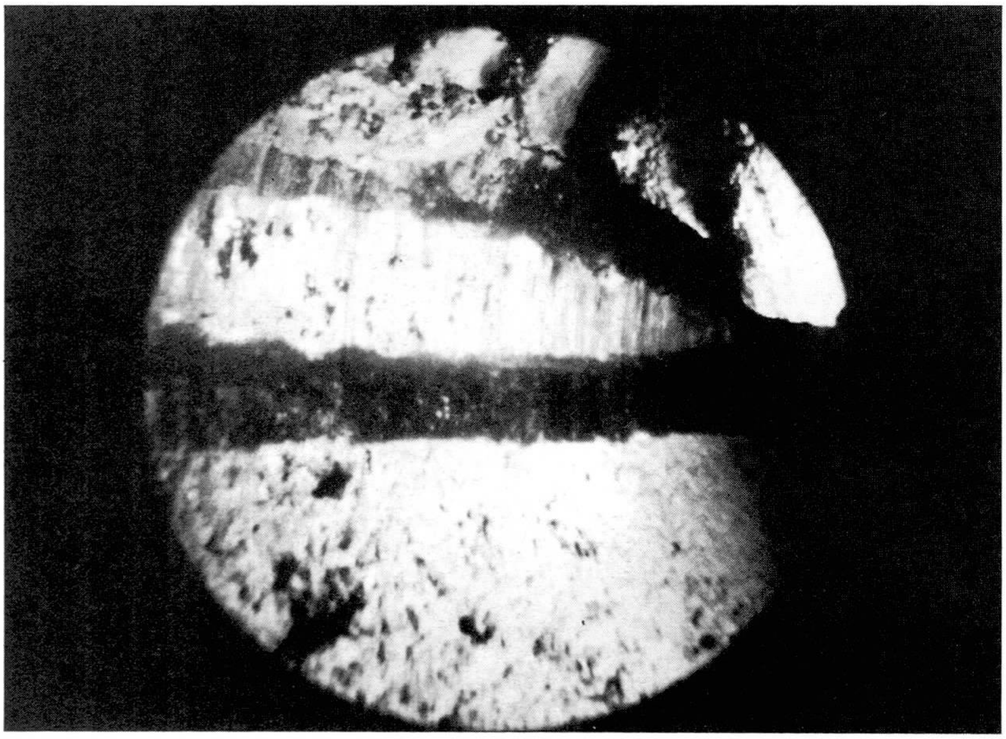

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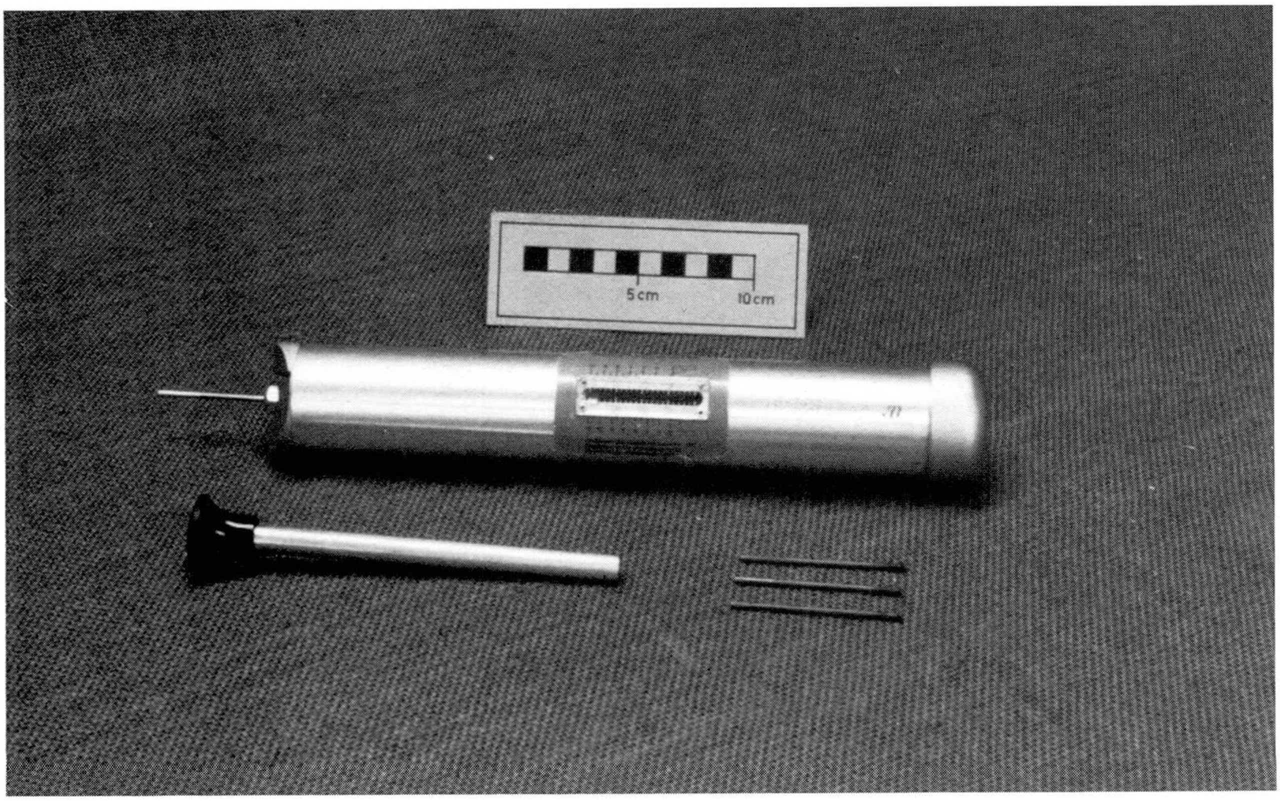

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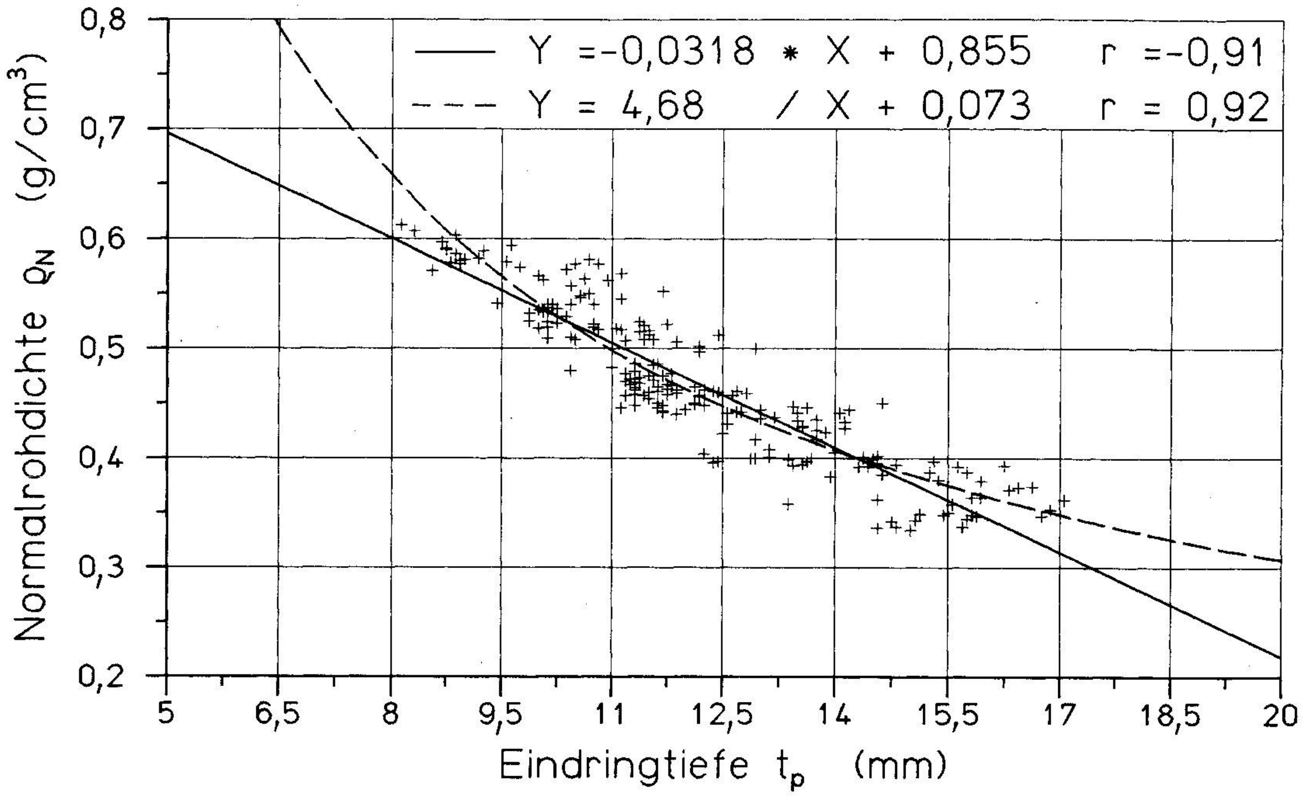

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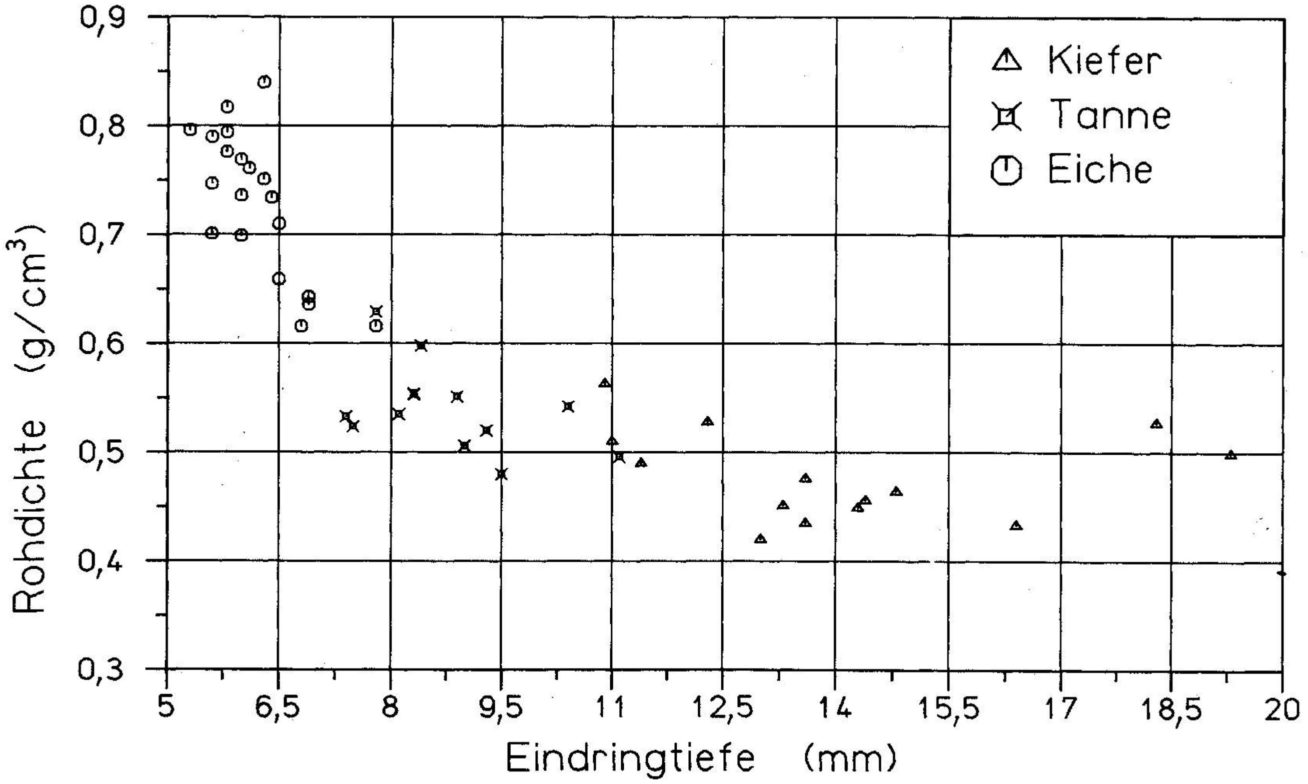

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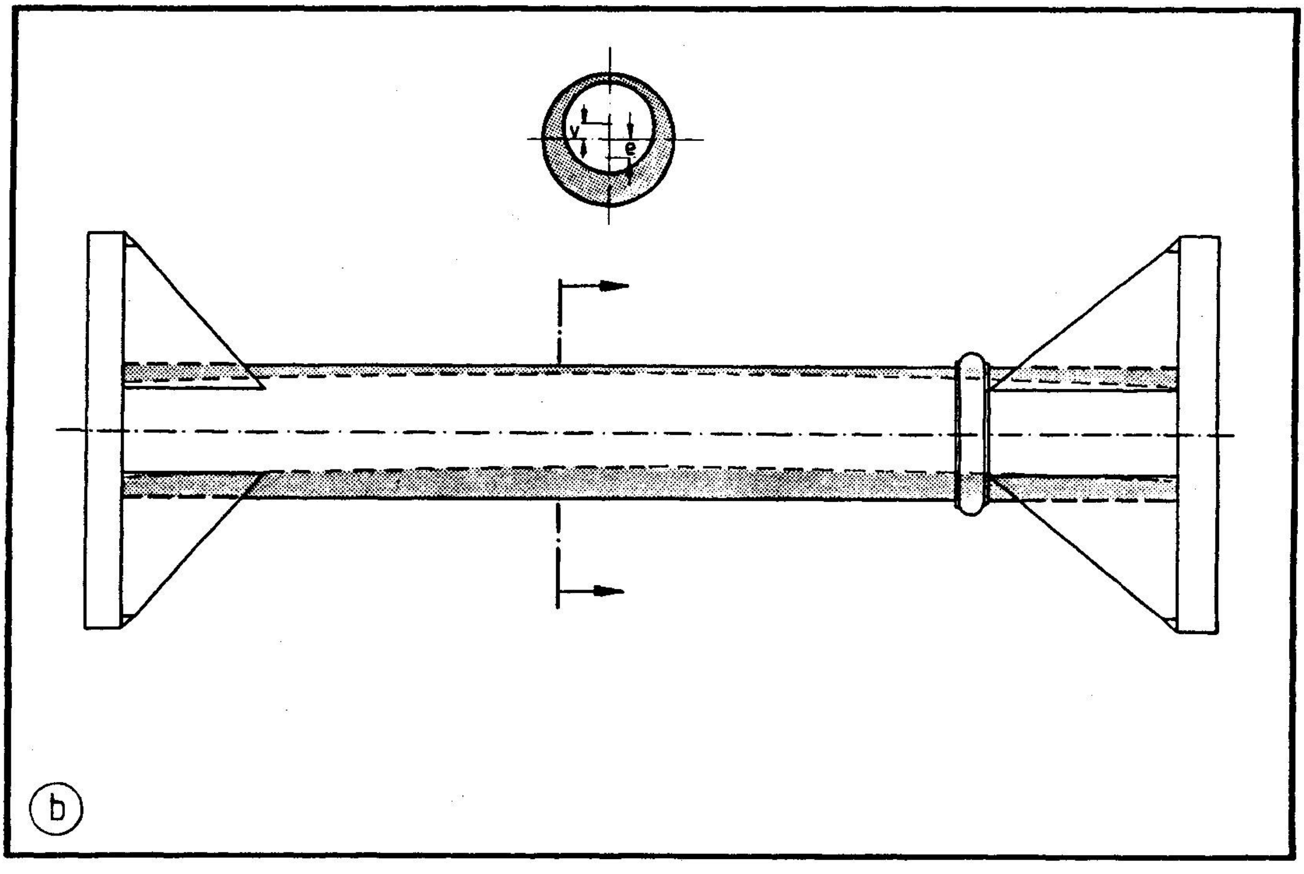

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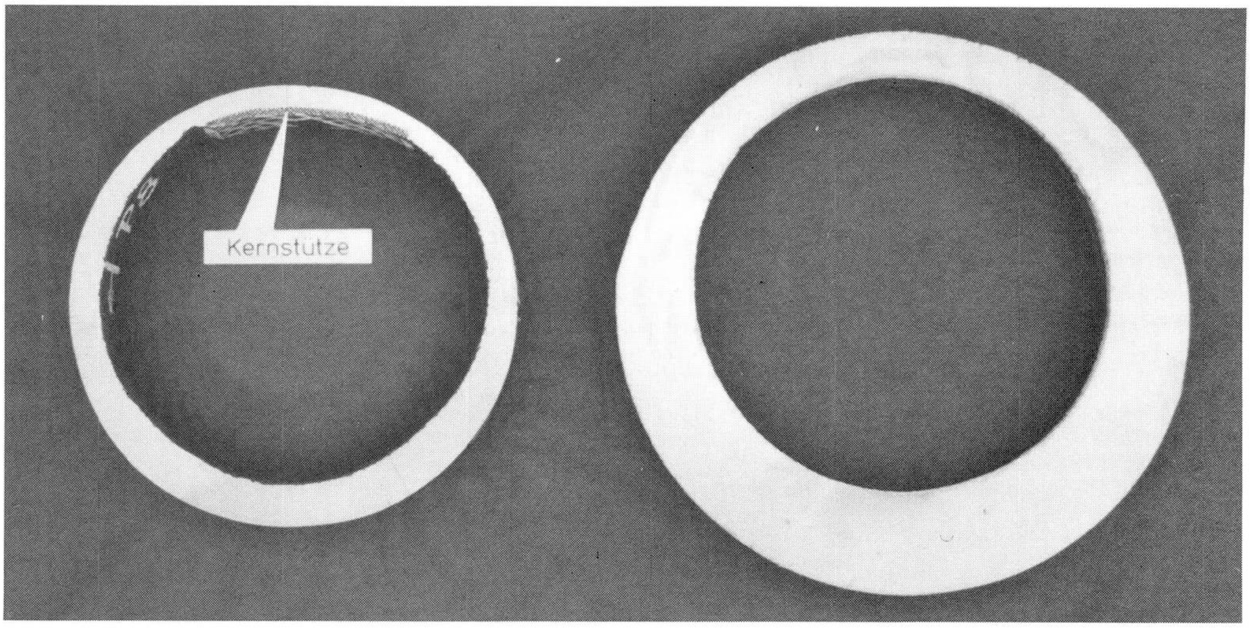

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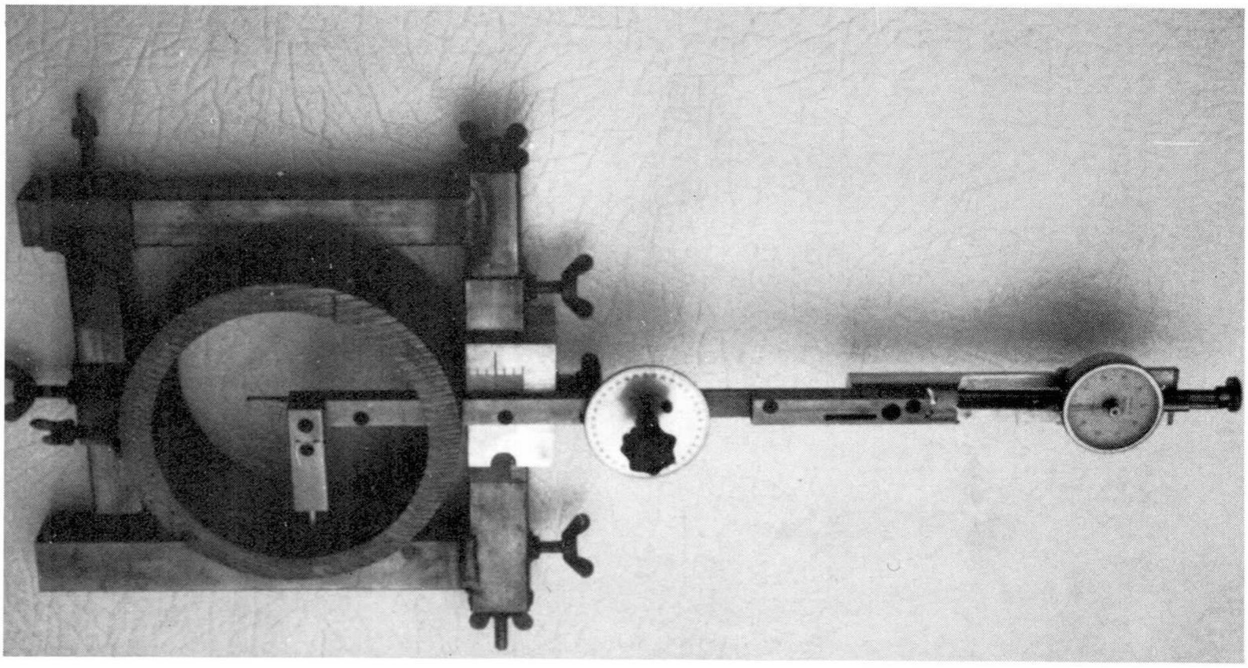

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Static Monitoring System for the Brunelleschi Dome in Florence

Système de contrôle statique pour le Dôme de Brunelleschi à Florence

Statischen Kontrollsystems für die Brunelleschi Kuppel in Florenz

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SUMMARY
A structural control system is important when a global and simultaneous analysis and comparison of the process is required. The diagnostics of the structural safety based on the ensemble of the quantities — and not on single or group of variables — is a main requirement of the Brunelleschi Dome and basement control system. The article describes the purpose and requirements involved in the project, together with the measurement instrumentation used and its functions.

RESUME
L'application d'un système pour le contrôle structurel est essentiel là où il faut considérer en même temps et entièrement tous les processus d'analyse et de comparaison des observations. La prévision de la sécurite structurelle fondée sur l'ensemble des grandeurs relevées et non pas sur chaque variable ou groupes de celles-ci - représente la condition fondamentale du système pour le contrôle du Dôme de Brunelleschi et de ses fondations. L'article présente les objectifs et les conditions du système, les instruments de mesure employés et les fonctions effectuées pas le système.

ZUSAMMENFASSUNG
Die Hilfe eines Strukturpruefungsystem ist wesentlich für eine Gesamtbeurteilung die ganze Analyse- und Vergleichsverfahren zu berücksichtigen hat. Die Diagnose der strukturellen Sicherheit, die nicht auf einzelnen oder Gruppenvariablen, sondern auf die Gesamtheit der entnommenen Grössen abgestützt ist, bestimmt die Grundlage des Kontrollsystems der Brunelleschi Kuppel und ihre Fundationen. Der Artikel beschreibt die Ziele und Forderungen die zum Systementwurf führten, sowie Art und Funktion der gewählten Instrumentierung.

1. INTRODUCTION

The dome of the Santa Maria del Fiore in Florence presents a complex crack pattern that has the significant aspect in the four through lesions involving the even sectors of the structure.

The wealth of existing historical documentation on the monument confirms the assumption that the first lesions had already commenced in the early years after the building of the dome and that, subsequently, the process of deformation underwent a constant evolution up to the present time.

The large number of studies undertaken over the centuries make it possible to reconstruct this evolution in broad terms.

The construction of the Cathedral began in 1295, starting out with the facade and proceeding slowly through diverse vicissitudes over the span of a whole century, with the transept with pulpits and the chapels being completed only at the beginning of the 15th Century.

The first signs of failure in the lateral walls of the building can be traced back to 1434, that is to say, with the completion of the building of the dome.

For what concerns this latter, the oldest document - which clearly refers to small cracks that however provide a passageway for "air and the wind" - is a report dating from 1639, some two hundred years after the construction of the dome [1].

The first scientifically valid document however is dated 1757 and is the work of the astronomer Leonardo Ximenes, who in his studies for the sun dial of the Cathedral set out an exact description of the existing failures via observation of no less than 13 cracking phenomena [2]. Ximenes also carried out the first geometric measurements that indicated that both the church as well as the bell-tower were slightly out-of-plumb towards the south in the direction of the river.

No other significant document was issued until 1895, a year that witnessed a violent earthquake. That same year, however, saw the execution of maintenance operations, that reasonably lead to the conclusion that perhaps new cracks had shown up [3].

The Commission appointed by the Opera del Duomo in 1934 verified a condition of cracking that was similar to the one at present [4]. The Commission laboured for three years, studying with great care the relationships existing between the air temperature and the opening/closure of the cracks, and was able to show how the cracks undergo variations of amplitude linked not only with seasonal thermal cycle but also with the daily ones.

Recently the "Superintendence of the Beni Ambientali ed Architettonici" of Florence has carried out a new and detailed study of the state of cracking, that sets out in a reference document a series of tables giving the position and the number of all the main cracks in the structure [5].

The present Commission with the task of studying the renewal of the Santa Maria del Fiore structural complex, with special reference to the static problems of the dome, issued in 1985 a document [6], that puts forward a number of assumptions on the structural behaviour of the monument that, supposedly, underly the formation of the cracks. Considerable assistence towards the interpretation of the mechanical phenomenon has been provided in recent years by the analysis carried out by ENEL-CRIS with the employment of finite element mathematical models that have facilitated in examining various causes of the damage [7].

In view of the complexity of the problem, the Study Commission has deemed it necessary to check for a suitable period of time all the quantities that are held to be the most significant in terms of the structural behaviour of the monument. As such, in 1986 the Superintendence of the Beni Ambientali assigned ISMES the task of supplying and installing a system for the control of the dome, the start-up of the system being scheduled for September, 1987.

The article refers to the most important design aspects and describes the technical choices adopted for what concerns both the measurement instrumentation as also the data acquisition system.

2. DESIGN OF THE CONTROL SYSTEM

2.1 Aims of the System

Two essential objectives were aimed at with the decision to install a monitoring system on the dome.

The first and the more immediate one was that of keeping under control the evolution of the deformation process of the dome by means of measurements of both the variations of amplitude of the main cracks as also of the movements of the structure, and thereby to be able to issue a warning in the event of these quantities exceeding a pre-established threshold.

In these terms, the system represents the mean for the safeguarding of the Cathedral.

The system was entrusted with a further task as well: that of identifying the correlations between the quantities that describes the deformation process and the quantities that may be considered to be the causes - thermal variations, foundation settlements, etc.

Such correlations can be of great help in interpreting the on-going phenomenon, thus facilitating the setting up and calibration of mathematical interpretative models.

2.2 System Requirements

From the definition of the objectives of the system are derived a series of requirements connected partly with the structural problems and partly with the functional aspects that have involved a precise design and the adoption of particular technical solutions.

The main considerations that have characterized the design of the system are as follows:

- Type, number and position of the sensors are defined by the need to gather in a comprehensive manner the on-going phenomenon, while at the same time, containing the cost of the system within acceptable limits and therefore the number of sensors. The choice was made on the basis of a preliminary survey of the deformation condition and a entirely qualitative model of the structural behaviour.

- Sensitivity, accuracy and stability over time, full scale of the various measurement networks are obviously established on the basis of the expected variations in the values of the quantities under observation, thus involving the choice of sensors with high characteristics.

- Measurements process must be completely automatic, making it possible to gather data with the desired periodicity and their transfer to mass memory: the autonomy of the system must be in the order of several months.

- System to offer highly flexible and expandable characteristics, to enable enlargement or at any rate modification, subsequently and on the basis of the initial indications, of the sensor network and of the data gathering mode.

- The reliability of the system to receive special attention via choice of special components and adoption of a madular construction. These criteria facilitate in reducing not only the number of breakdowns but also maintenance time.

3. MEASUREMENT INSTRUMENTATION

3.1 The Structure and The Deformation Condition

As already emphasized, the choice of the quantities to be monitored was made with the aim to ensuring a reliable interpretation of the on-going deformation phenomenon.

In this choice, the knowledge of the structural scheme and the crack condition is of fundamental importance.

3.1.1 The Structure

Examining the structure of the dome-base complex, it is possible to recognize three abses and the nave of the Cathedral, which though not called on to act as a real support of the vertical forces transmitted by the structural masses above it nevertheless play a determinant role of buttress of the complex (Figs. 1 and 2).

The structural elements making up the base of the dome consist of four polygonal cross-section supporting pillars. The inner side coincides with that of the sides of the inner octagon of the base (sides 2, 4, 6, 8).

The load-bearing capability of these massive supports, some 30 metres in height, is weakened by reductions of resistent cross-sections. In fact, the two sacristies are located in the apse supports (sides 4 and 6), while those in adjacent to the body of the Cathedral are traversed by accesses to the side naves that lead to the octagonal space under the dome (sides 2 and 8).

From the height of the first inner gallery, a structural element - drum - in the shape of an octagonal parallelepipedon joins the supports with the springer of the dome. While found to be weighted above on every side of the dome, the lower part presents the edge alternatively supported by the pillars or free with curved soffit. In the upper part of the drum, that is, between the second and third inner gallery, the circular windows at the centre of each side weaken the resistent cross-section.

The octagonal dome made up of two inter-connected canopies is set on the upper cross-section of the drum, coinciding with the third inner gallery.

The skylight completes the construction.

3.1.2 The Deformation Condition

The main phenomenon involving the dome, the drum and the supporting pillars is to be met with, though with varying amplitude of the cracks, on all four even sides of the octagon of the base (Fig. 3).

Starting out from the top, the cracks are to be found in the dome at a height between the last inner walkway and the base of the skylight. They then run downwards almost vertically to the centre of the gables and are found to pass through both the inner and outer canopy; extending to the drum, where they present the maximum amplitude, they cut through its entire thickness and involve the cornices of the circular windows. Still proceeding downwards, they take in the outer structures of the semicircular chapels. Finally, they are still identifiable in the pillars inside the Cathedral, below the first gallery.

Quantitatively, the phenomenon is more emphasized in sides 4 and 6; while in side 2 rather than concentrating in one or more branches the cracking is more spread out, so much so that it is difficult to recognize it in the canopies of the dome.

In the drum, corresponding to the odd sides, the cracks start out from the circular windows at an angle of about 60 degrees vis-a-vis the horizontal, then reaching down to the second gallery they proceed towards the soffit of the arch that defines the free edge of the drum.

Inside the construction, in the eight angle areas, cracks are encountered that with essentially vertical progression involve the drum and the dome up to an intermediate height between the second and third inner walkways. They are identifiable in the drum areas also in the stretch of the inner helicoidal stairs that rise up in the vicinity of the corner.

In the dome springer area they involve the corner ribs and are found to pass through inasmuch that they are visible also on the inner and outer walls of the opening that the first inner walkway produces in the corner ribs.

Again in correspondence to the odd gables at the soffit of the inner canopy of the dome, systematic crack formation is identifiable at the centre of the gable itself, with an essentially vertical progression, from a height of the second inner walkway to the third.

The inclined symmetrical crack systems are identifiable on both sides of the walls of the main nave adjacent to the base of the dome and cut downnwards through the first gallery. In this strecht a vertical relative settlement of the pillars of the dome respect to the pillars of the nave is to be noted.

In the area of contact between the side chapels and the pillars, marked inclined cracks are identifiable.

3.2 Monitoring of the Deformation Condition

3.2.1 Monitoring of the Cracks (Fig. 3)

The main attention was focused on the through-cracks that have developed vertically in the event sectors of the dome. For these cracks it was deemed important to measure the opening and closing movements - in the circumferential direction - at five different heights. The radial components of the movement were instead neglected, since notwithstanding their presence, they are limited in magnitude.

The control of the behaviour of the cracking on the dome was then completed by measuring the opening and closing movements of the cracks in the lower edge of the circular windows - in the uneven sectors - in as much that these are of particular interest for clarifying the behaviour of the drum and those of the cracks corresponding to the corners.

The sensor adopted for these measurements was an inductive type transducer that, mounted astride the two opposite edges of the cracks, ensures an accuracy in the order of +/- 0.02 mm.

3.2.2 Monitoring of the Displacements of the Structure

The measurement of the displacements of some of the structural elements is of particular significance in view aboveall of the setting up and calibration of a mathematical model capable of explaining the behaviour of the structure.

For this standpoint it was deemed essential to measure displacements - vertical and horizontal - of the drum and the pillars, which could provide an indication of the manner in which the vertical loads and the thrust of the dome are transferred to the ground.

a. Horizontal displacement of the drum and the pillars below it (Fig. 4):

   The measurements were effected via a system made up of a plumb-line and a telecoordinometer.
   The plumb-line, whose point of fixture is located in correspondence to the third gallery, represents the line of vertical reference for the measurement. The telecoordinometer, connected to the structure via a rigid support, measures the position of the line by means of a system of optical interception, and thus provides the measurement of the horizontal components of the displacement of the mounting point vis-a-vis the point of fixture of the plumb-line.
   Eight plumb-lines were installed in proximity of the corners of the dome and three telecoordinometers were mounted for each of these, respectively at heights below the drum, of the first gallery and at floor level.
   It is obvious that the indication provided by the telecoordinometer at floor level coincides with the measurement of the displacement of the point of fixture of the plumb-line.

b. Vertical settlements of the drum and pillars (Fig. 4):

   For these measurements, a hydraulic levelling system was resorted to, this being made up of 8 levellometric tanks positioned at the height of the second gallery and approximately at the centre of each sector.
   This arrangement facilitates the gathering of two fundamental aspects for the analysis of the structural behaviour: on the one hand, the relative settlements of the 4 pillars - and therefore a possible overall rotation of the dome; on the other, the settlements of the centre line of the "beam-wall", to which may be ideally similarized the side (uneven) of the drum above the arch of the lateral naves.
   In view of the press in the field of electronic instrumentation - in the present case the measurement of the level of the liquid in the tank is of the capacitive type -

the levellometric measurements achieve high sensitivity and optimal accuracy, in the order of +/- 0.04 mm.

3.3 Temperature Monitoring

Owing to the essential role placed by the variation in temperature in the context of the possible evolution of the cracking condition, it was thought important to acquire detailed knowledge on temperature distribution in the structure over the annual cycle.

As such, a network of sensors was installed, the numbers and position of the latter being chosen with the aim of both identifying the value of the temperature in some of the significant positions, as well as of gathering a series of data against which to set up a "thermal" model of the dome (Fig. 5).

The measurements - a total of 60 points - were carried out:

- along the octagonal perimetre in correspondence to the second walkway;

- along two meridians in the sectors facing North and South-West, which obviously are subjected to conditions of minimum and maximum sunlight, respectively.

For each position along the directions cited above temperature measurements were effected of the inner and outer surfaces and in several points along the masonry of the inner canopy, the air space in between and the masonry of the outer canopy.

The thermometers employed for the measurement of the temperature of the air and the masonry mass were of the electrical resistance type, with platinum sensitive element and capable of offering an accuracy of measurement equal to +/- 0.1 degrees C.

4. MONITORING SYSTEM

The number and the distribution of the sensors, especially the considerable distance separating the various measurement points, obliged the choice of a monitoring system based on the concept of distributed measurement. This system is composed mainly of two types of physically and functionally separate units linked with digital data transmission lines for the exchange of information: one or more peripheral units and the central control unit (Fig. 6).

The peripheral measurement units (Fig. 7), deployed centrally vis-a-vis a group of measurement points, carry out the task of electrical conditioning of the sensors, scanning of the measurement channels, analog/digital conversion of the signals, temporary memorization of the values measured and, finally, of transferring the data to the central control unit.

Based on the use of a microcomputer, this latter unit (Fig. 8) is instead entrusted - via suitable programmes - with all the control functions of the system.

As against a centralized type traditional system, this conception facilitates in limiting the lengths of the connecting cables of the sensors, thereby reducing costs, bettering measurement accuracy, increasing signal/disturbance ratio, and providing immunity from the overvoltages induced by electrical discharges.

The control software of the system was realized on the basis of previous experience and facilitated the carrying out of periodical type measurement cycles, as also of the manual type and of those meant to updated the measurements, and of controlling the instrumentation.

The periodical cycles can be effected at well-defined times of the day or else uniformly distributed over the span of the day, with regular time intervals.

As such, the system provides for the execution of a complete measurement cycle, which involves the acquisition of all the pre-established quantities within four minutes, the recording of the data on magnetic cassette with a maximum capacity of 90 cycles, and the print-out of the information related to the acquired values.

The execution of a cycle of manual measurement is provided for, which is similar to that described above and offers various possibilities of visualization: on video-monitor, with print-out and recording on magnetic support.

Moreover, updating measurement cycles are executed hourly, the acquired data being used for the possible identification of irregularities vis-a-vis the components of the system.

5. CONCLUSIONS

The automatic system of monitoring of the dome and its base is expected to provide a response that can offer the interpretation of the on-going structural phenomena and facilitate the setting up of a reference mathematical model.

This model will subsequently represent the basis for designing interventions of structural consolidation of the monument and for the verification of their effectiveness.

In any event, the activity will bear precise historical witness of the life of the monument and should provide an useful reference for future studies.

6. REFERENCES

[1] GUASTI C., "La Cupola di Santa Maria del Fiore", Florence 1857

[2] XIMENES L., "Del vecchio gnomone fiorentino ......", Stamperia Imperiale, Florence 1757

[3] DEL MORO L., "Relazione sui danni arrecati ....", Atti per la conservazione dei monumenti della Toscana compiuti dal 1 luglio 1834 al 30 giugno 1895, Florence 1896

[4] "Rilievi e studi sulla Cupola del Brunelleschi eseguiti dalla Commissione nominata il 12 gennaio 1934", Opera di Santa Maria del Fiore, Florence 1939

[5] "Catalogo dei plessi fessurativi della Cattedrale di Santa Maria del Fiore in Firenze", Ministero per i Beni Culturali ed Ambientali di Firenze, July 1984

[6] "Rapporto sulla situazione del complesso strutturale Cupola - basamento della Cattedrale di Santa Maria de Fiore in Firenze", Ministero per i Beni Culturali ed Ambientali - Commissione di studio per la salvaguardia del monumentale complesso della Cattedrale, con particolare riguardo a problemi di statica della Cupola (DD.MM. 1.2.1983, 10.3.1983, 9.3.1984), Florence, March 1985

[7] CHIARUGI A., FANELLI M., GIUSEPPETTI G., "Analysis of Brunelleschi - Type Dome Including Thermal Loads", IABSE Symposium: Strengthening of Building Structures, Venice, September 1983.

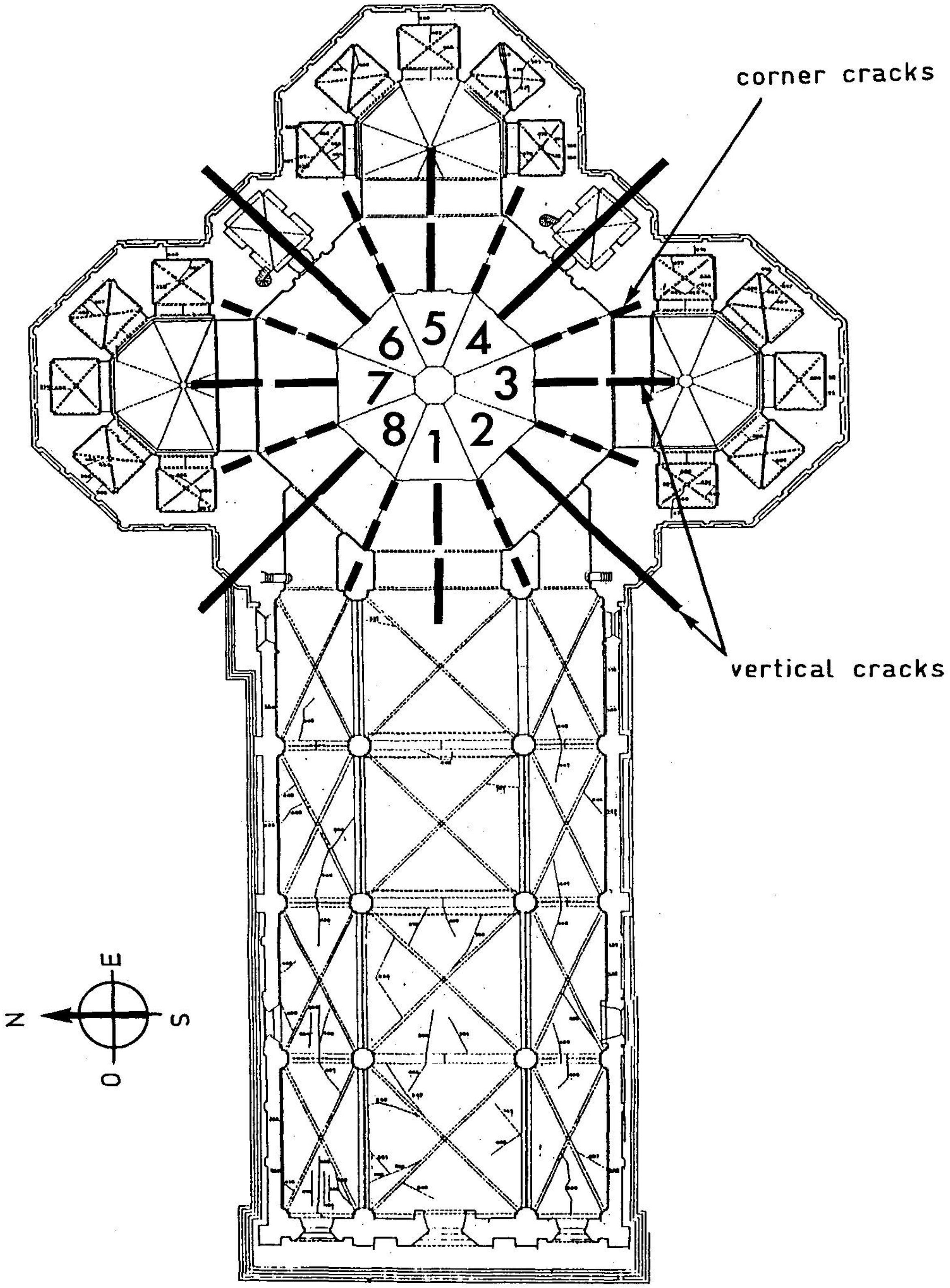

Fig. 1 Plan of the cathedral basement : crack location

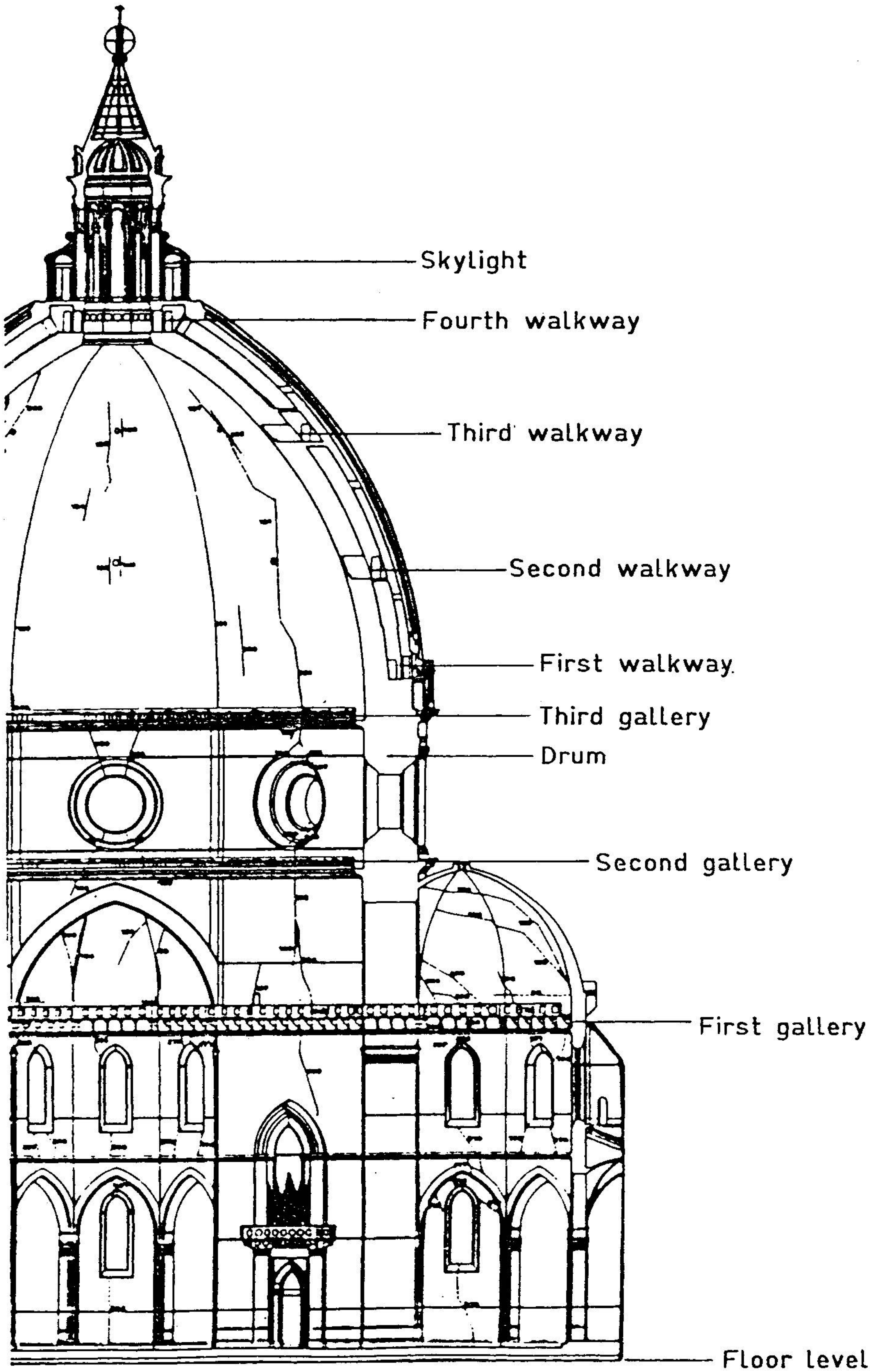

Fig 2 Dome and basement transversal section

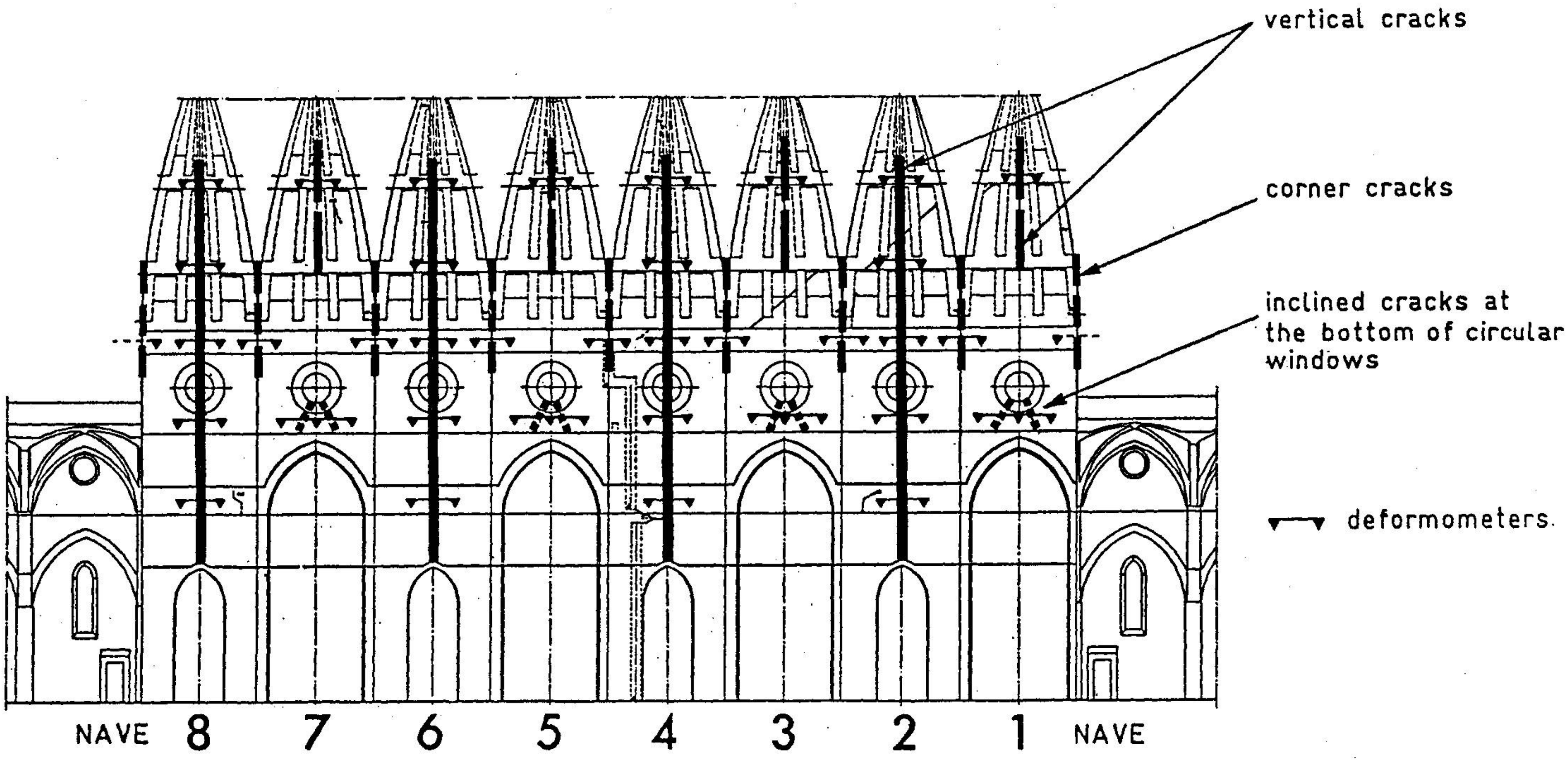

Fig. 3 Crack and deformometer location

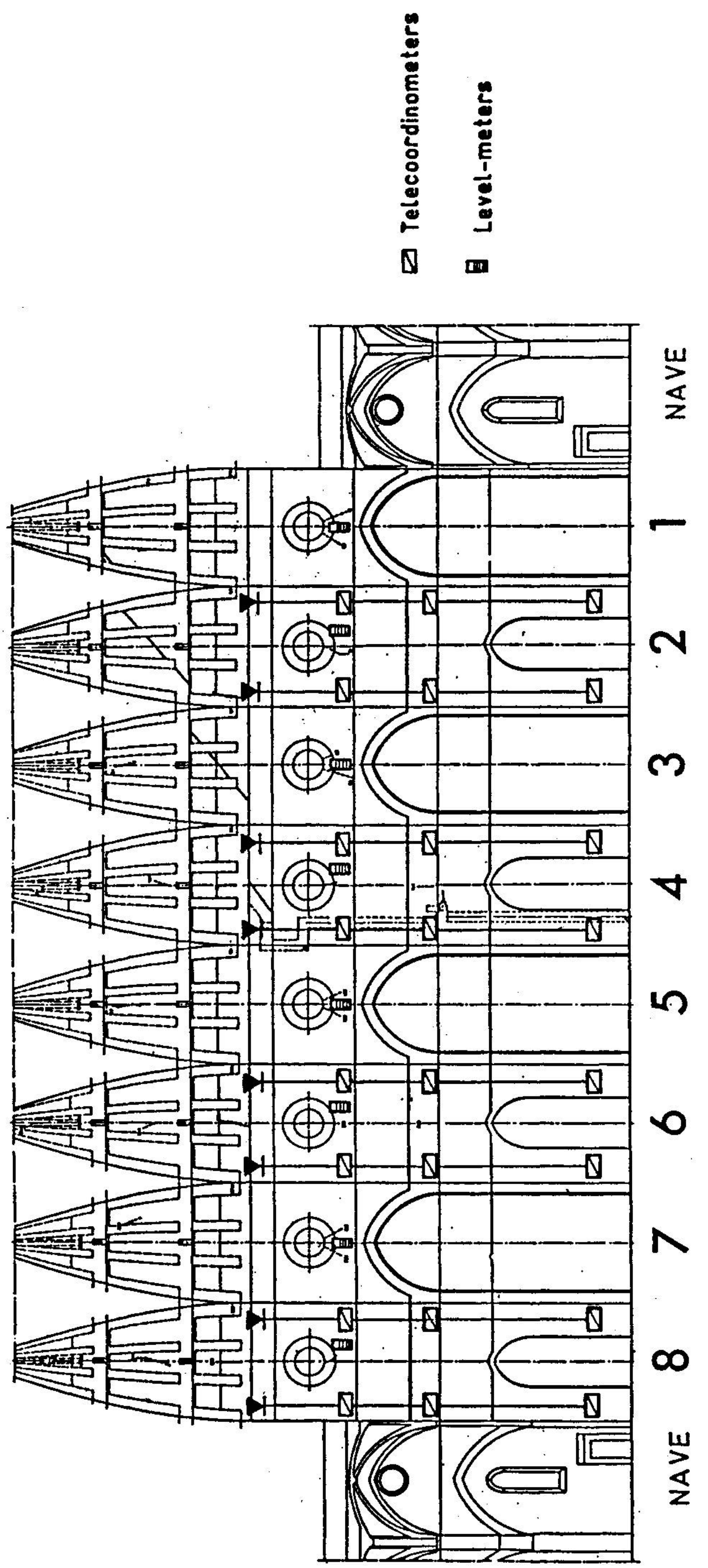

Fig. 4 Telecoordinometer and level- meter position

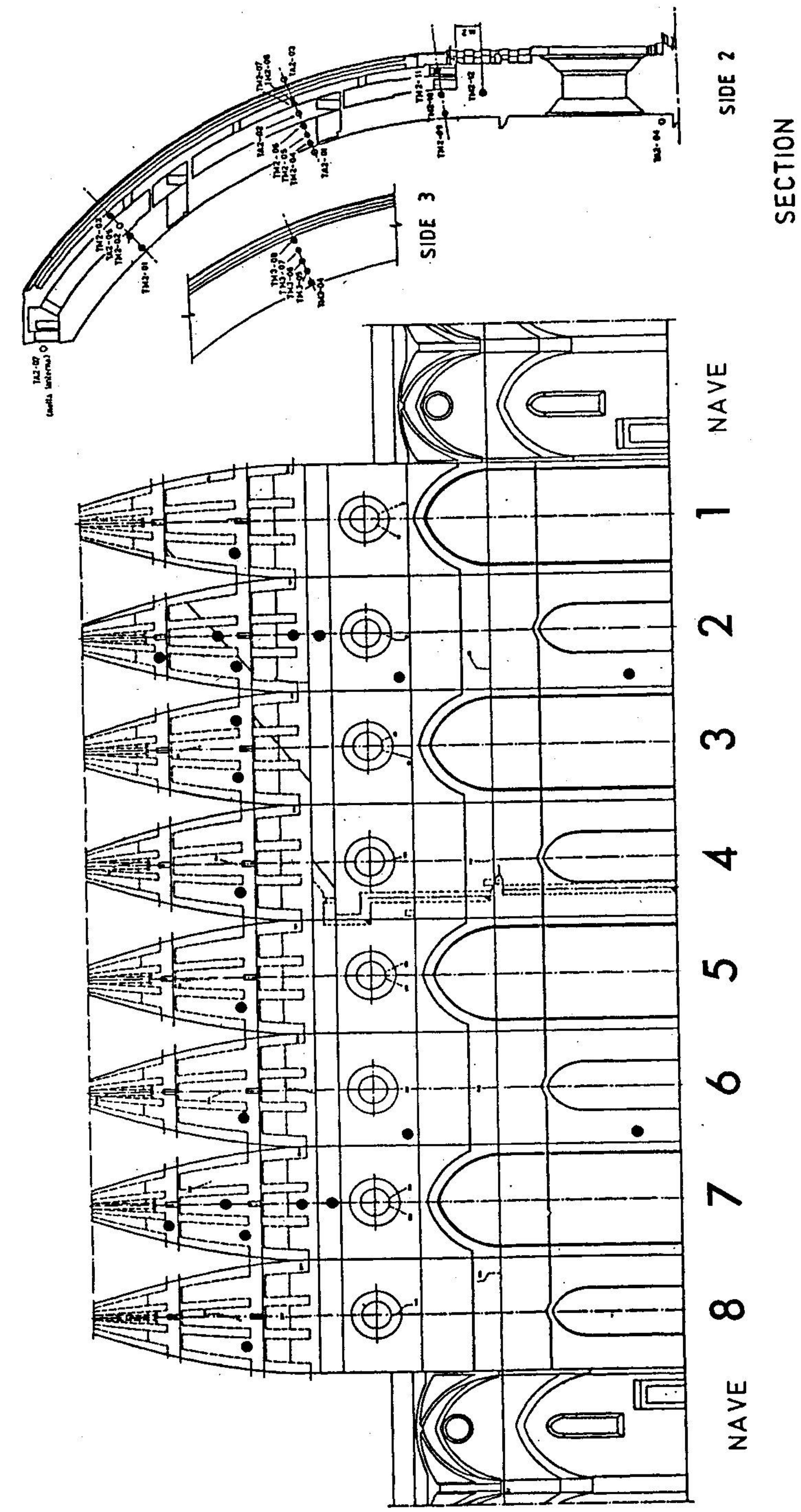

Fig. 5 Thermometer position

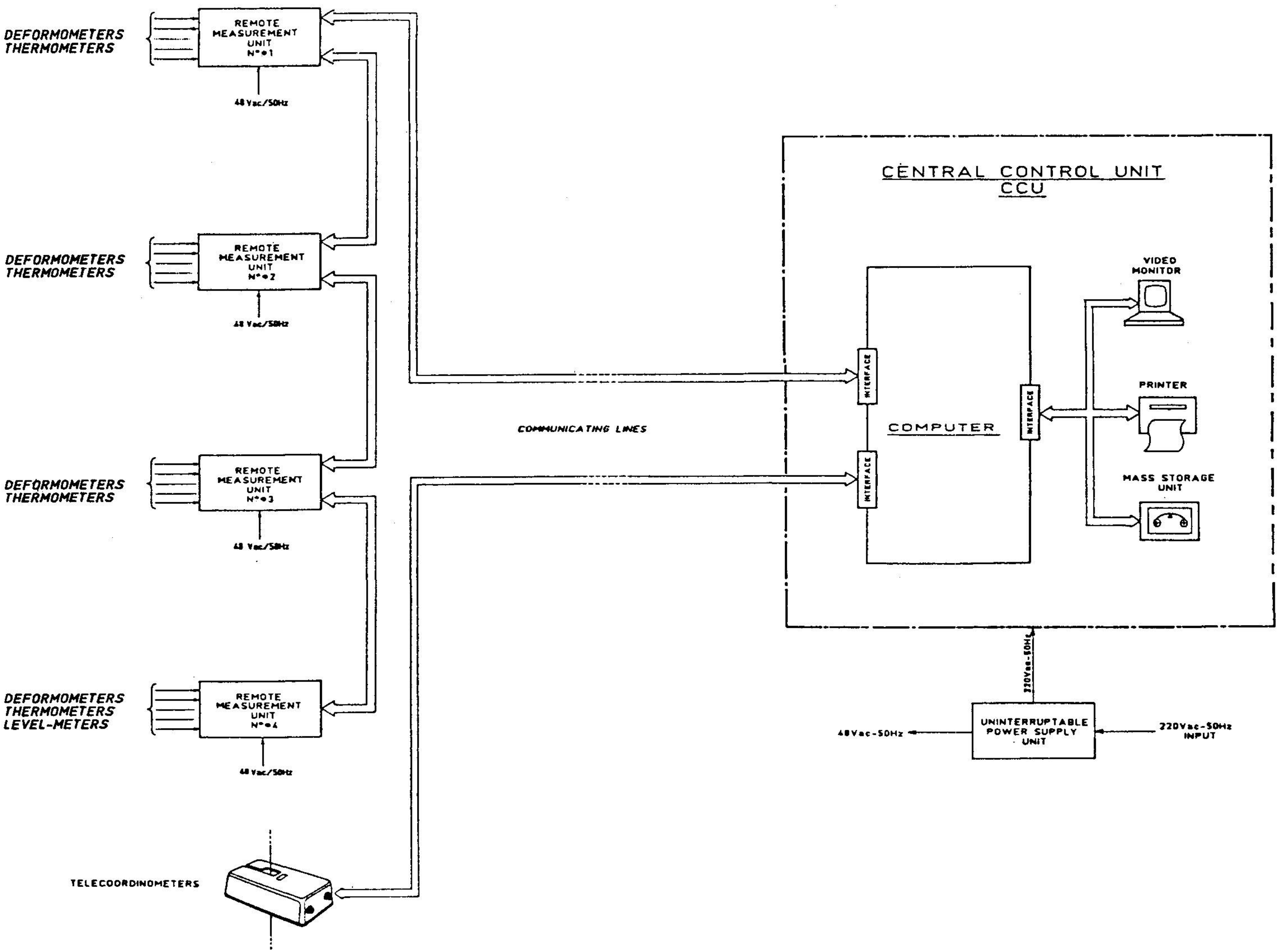

Fig. 6 Static monitoring system - block diagram

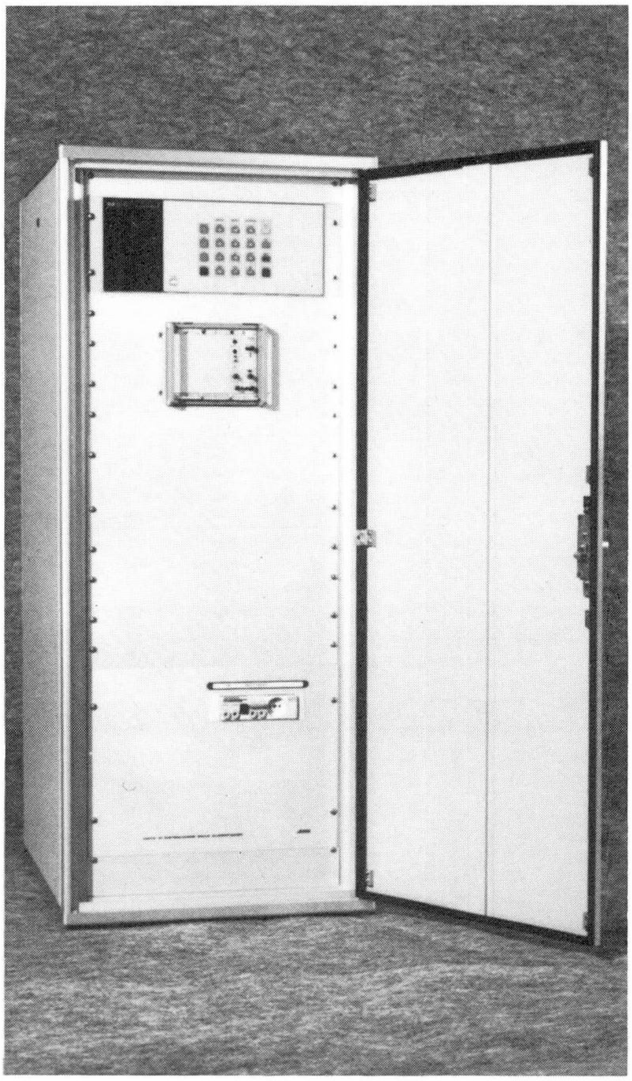

Fig. 7 Remote measurement unit

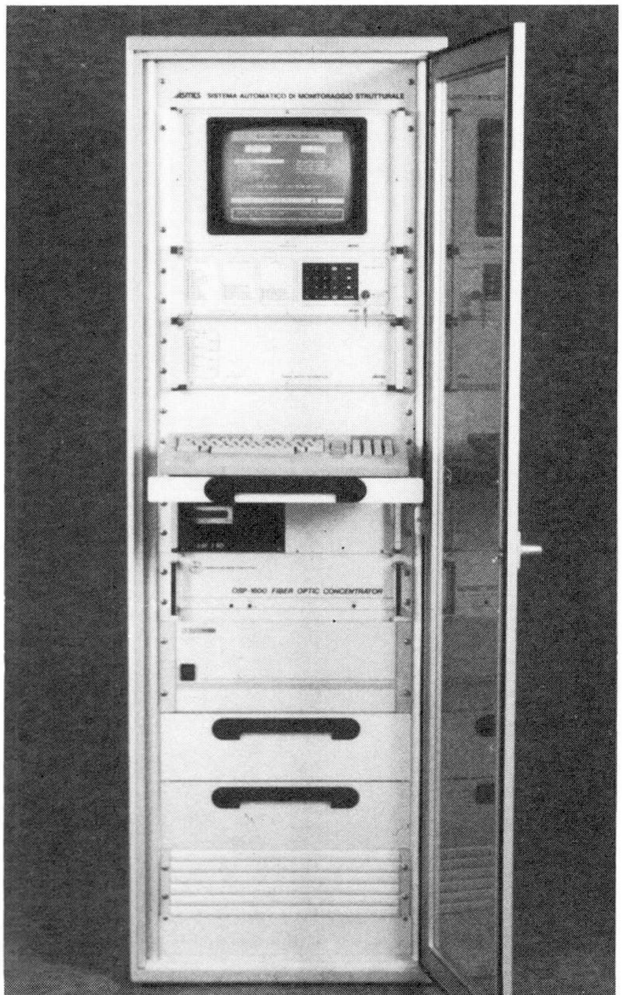

Fig. 8 Central control unit

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The Role of Monitoring for the Knowledge of the Behaviour of Structures

Le rôle de la surveillance dans la connaissance du comportement des structures

Die Rolle der Überwachung für ein besseres Kenntnis des Bauwerken-verhaltens

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SUMMARY
This paper presents different criteria to use advantageously a monitoring system to verify the structural behaviour of constructions; three kind of functions are presented: control of the construction during its execution; control of an existing construction; control of a construction in relation to perturbations produced by external works.

RESUME
Cet article présente divers critères pour utiliser avantageusement un système de surveillance dans le but de vérifier le comportement structural des constructions; on souligne trois types de fonctions: le contrôle de la structure durant sa construction; la surveillance d'une construction existante; le contrôle d'une structure à la suite de perturbations dues à des travaux extérieurs.

ZUSAMMENGFASSUNG
Dieser Beitrag stellt verschiedene Kriterien zum günstigen Verwenden von Überwachungssystemen für das Prüfen des Bauwerkenverhaltens vor; es werden drei verschiedenartige Tätigkeiten hervorgehoben: Das Überwachen von Bauwerken während deren Errichtung; Das Prüfen von bestehenden Bauten; Das Überwachen von Störungen durch äussere Arbeiten beeinflusste Bauwerken.

1 PRELIMINARY REMARKS

The use of a monitoring system, to obtain data (strain and stresses in the material; the amplitude of cracks, and joints; temperature; movements of the foundation; inclinations, ...), represents a very usefull way to verify the structural behaviour. This facility appears of particular interest in the following situations:

1) Control of the construction's behaviour during its execution;
2) Control of an existing construction, that is interested by damages, which can be related to ciclic or progressive phenomena;
3) Control of the construction's behaviour in relation to perturbations produced by external works of different kinds.

In this Note are presented some examples, that illustrate the three situations which we described before, and the criteria on the base of which it is possible to correlate the experimental data with the analytical models representative of the construction.

2 CONTROLS DURING EXECUTION

2.1 General aspects

This type of controls allows to verify the correspondence between the reality and design, either during execution of new constructions, or during interventions of strenghtening or consolidation; the analysis of the results can be usefull not only to modify the design in relation with the obtained data, but also like a testin during the works.

2.2 A prestressed concrete bridge on the Tiber River

The monotoring was related with the control of the bridge during its execution (fig. 1), and in particular it regarded the verticals displacements (due to elastic strains, creep shrinkage and temperature variations) for comparing theorical and effective phenomena. The difference between them were analyzed by a computer program in order to correct the elevation of the structural elements before to cast them.
The vertical displacements were correlated to the level of a liquid (mercury), contained in some small vases (fig. 2), designed by the Author in 1970, and connected between them with a net of tube. The level was measured by a inductive transducer that trasmitted the signal to a computer. The diagrams of the displacements' evolutions, for different sections of the bridge, are shown in fig. 3. and 4.

2.3 Santomarco tunnel in the Paola -Cosenza railway.

The behaviour of the tunnel lining depends from the radial pressure distribution, from the value of these pressure and their variation with time increasing. The definition of the rules and parameters governing this problem is always affected by uncertainty related with the indispensable extrapolations of the results of tests to complexes situations. In these extrapolations an important rule is assumed by the anisotropy of the rock and by the alterations due to exavation. In order to obtain more complete informations on the real behaviour of the soil-structure system, and to have the capabiity of adjust during

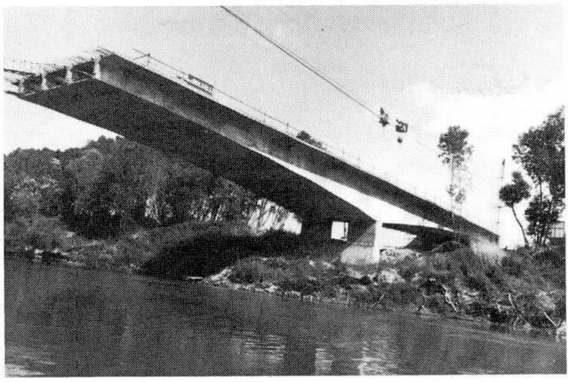


Fig. 1 The bridges on the Tiber River during its ececution.

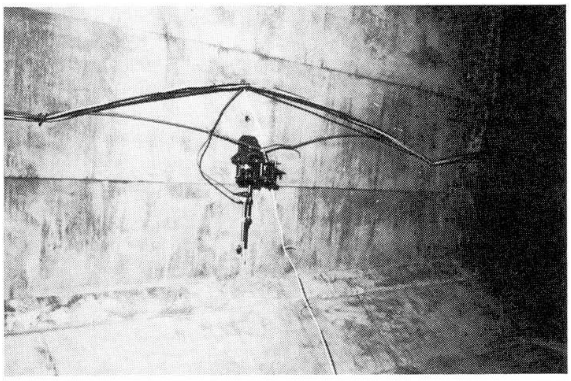


Fig. 2 Levellometric vases.

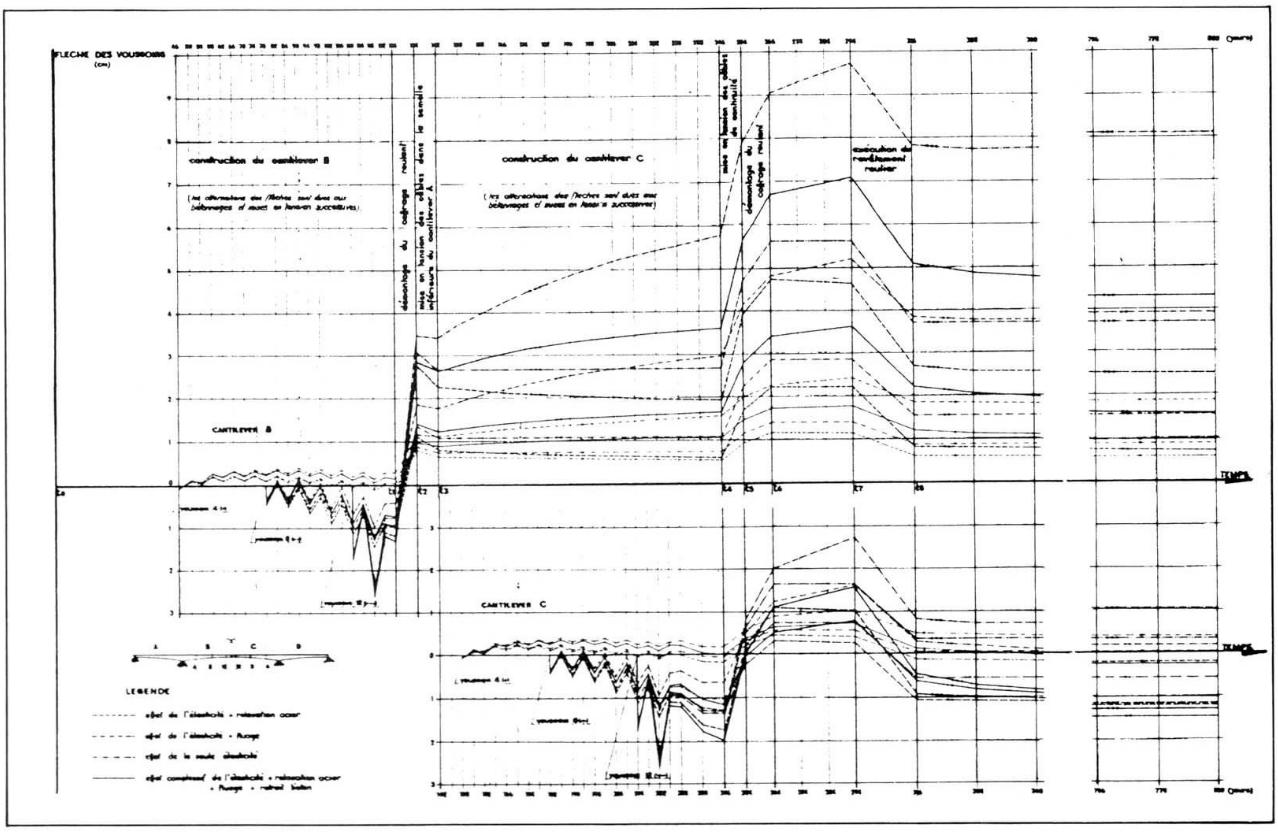
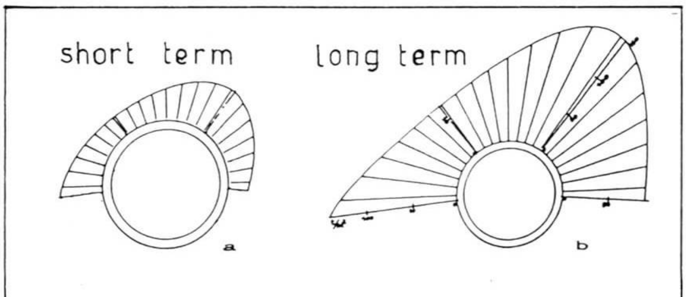


Fig. 3-4 Diagrams of the displacements' evolution for different sections of the bridge.



Diagrammi polari di sollecitazione sul rivestimento dopo 2.5 mesi dalla posa in opera (a) ed a situazione stabilizzata (b).

Fig. 5 Diagram of the soil-liner interface pressures at short and long term.

the works the construction modality in relation with situations different by the design assunptions, was planed an instrumentation system,that was compose sed by:

- pressure ganges to determine the rock-liner interface pressure;
- strain ganges into the concrete segments and into the additional centres;
- rock extensimeters placed in radial direction from the liner, anchored in the rock, with different lenght (6,9 and 15 meters), to control the width of surroinding soil yelded, in consequence of exavation. The figure 5 shows,

for one of the instrumeted rings of the liner, the diagram of the soil-liner interface pressures at short and long term. In the figure 6 is showed the evolution of the average pressures with the time increasing. It appears the comparison among the theorical exponential curve (a), the curve related with the graphic extrapolation of the adimensional curves (b) and the experimental exponential interpolation curve(c). The concordance among the results is goood at long term, while in the first months the theorical previsions had under-estimated the phenomenon. The experimental control has pointed out the necessity of gear up the times of insertion of the strengthening steel centres, that in the design assumptions should have been inserted in place more later. The applied methodology can be considered from a general point of view. Infact, every time there are complex problems, and phenomena based upon highly uncertain parameters the experimentation during the construction is a rational and convenient way in a more large evaluation costs-benefits. The best knowledge in fact, not only permit to receive the allarm warning and then operate in safety conditions, but also let the engineer to develop the design in a more flessible and articulate way. Many unfavorable situations are not unforeseeable, but only more one less probable conditions: a correct methodology of design must anticipate at the begin the possibility to adjust the solutions in consequence of the progressively pointed out situations, without engage in undiscriminately conservative intervetions, and then unjuistifavely expensive

2.4 The intervetions in the Major and Minor Council hall into the Ducal Palace of Genova

The Vaults of the two halls see to Figure (7 and 8) presented important deformations, that are probably appeared since during his erection and successively and unlarged in spite of the intervetions made in threee centuries. The characteristics of the Vaults, conformed as barrel Vault in the central part, much lowered and thin (about 1/120 of lenght of the span) make the structure much deformable; and are sufficient little displacements and rotations of the facade to preduce the great present deformations (more than 40 cm of relative vertical displacement between the two anterior and posterior portons, Figure 9). Such as interpretation of the vault's behaviour is confirmed by the theorical analysis of a finite element model of the structure (Figure 10). The design choices to a final recovery should consider the two following requirement:

- create a global connection between the two facades of the two halls;
- create a strtucture to support the vaults, without to intervent directly on the vaults, as to mantain the effective characteristics. The structural solution is reperesented by a system of stell beams jointed theirselves and

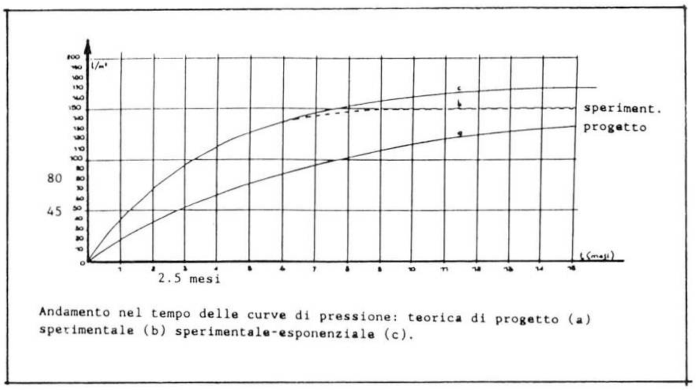


Fig. 6 The evolution of the average pressure with time increasing.

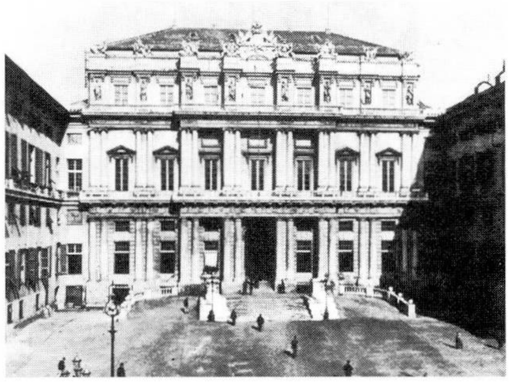


Fig. 7 View of the Ducal Palace of Genova.

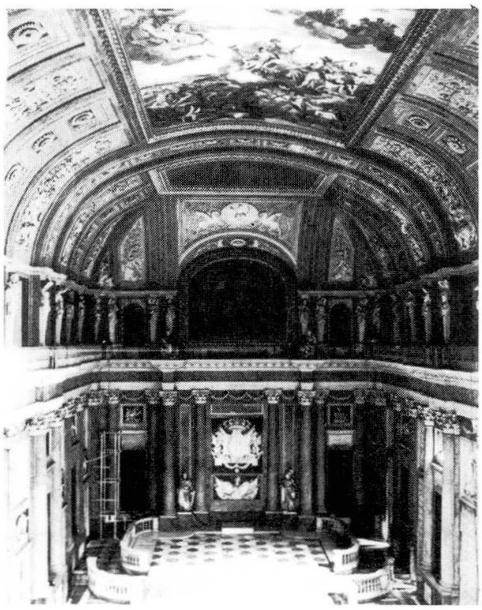


Fig. 8 The Major Council Hall.

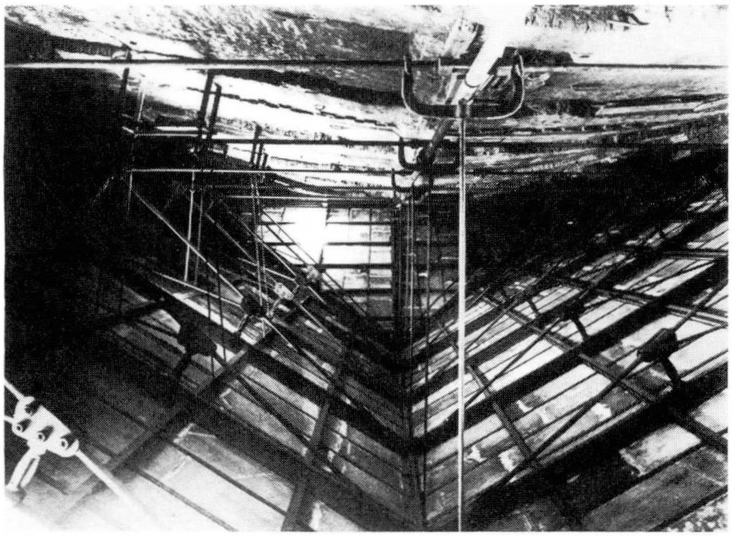
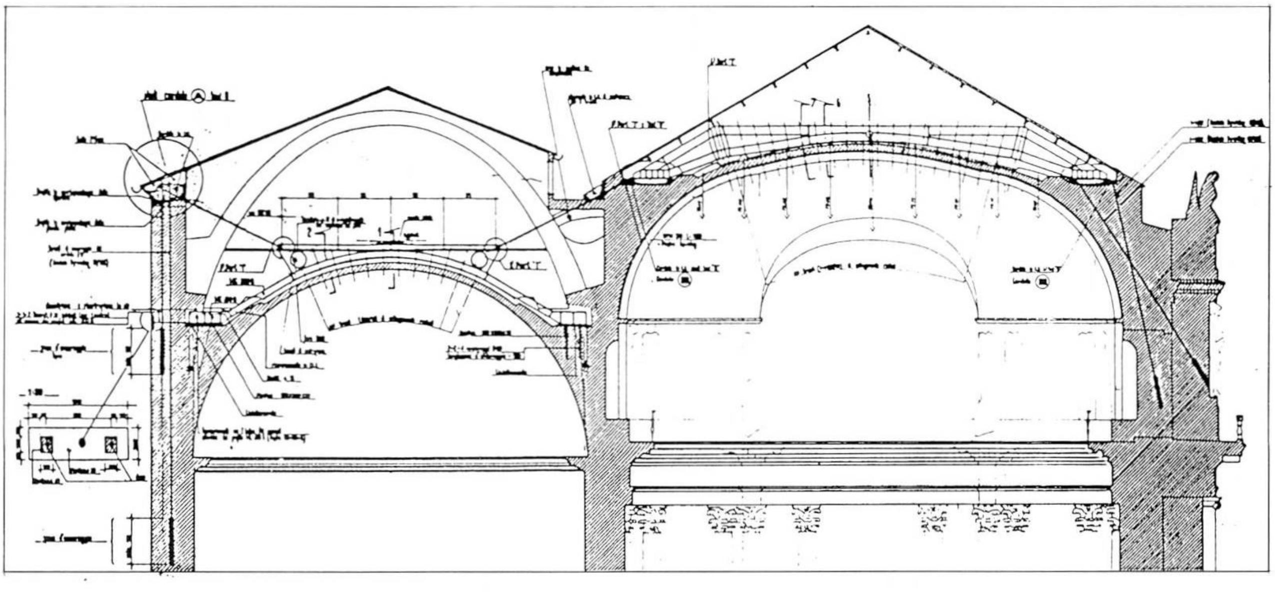
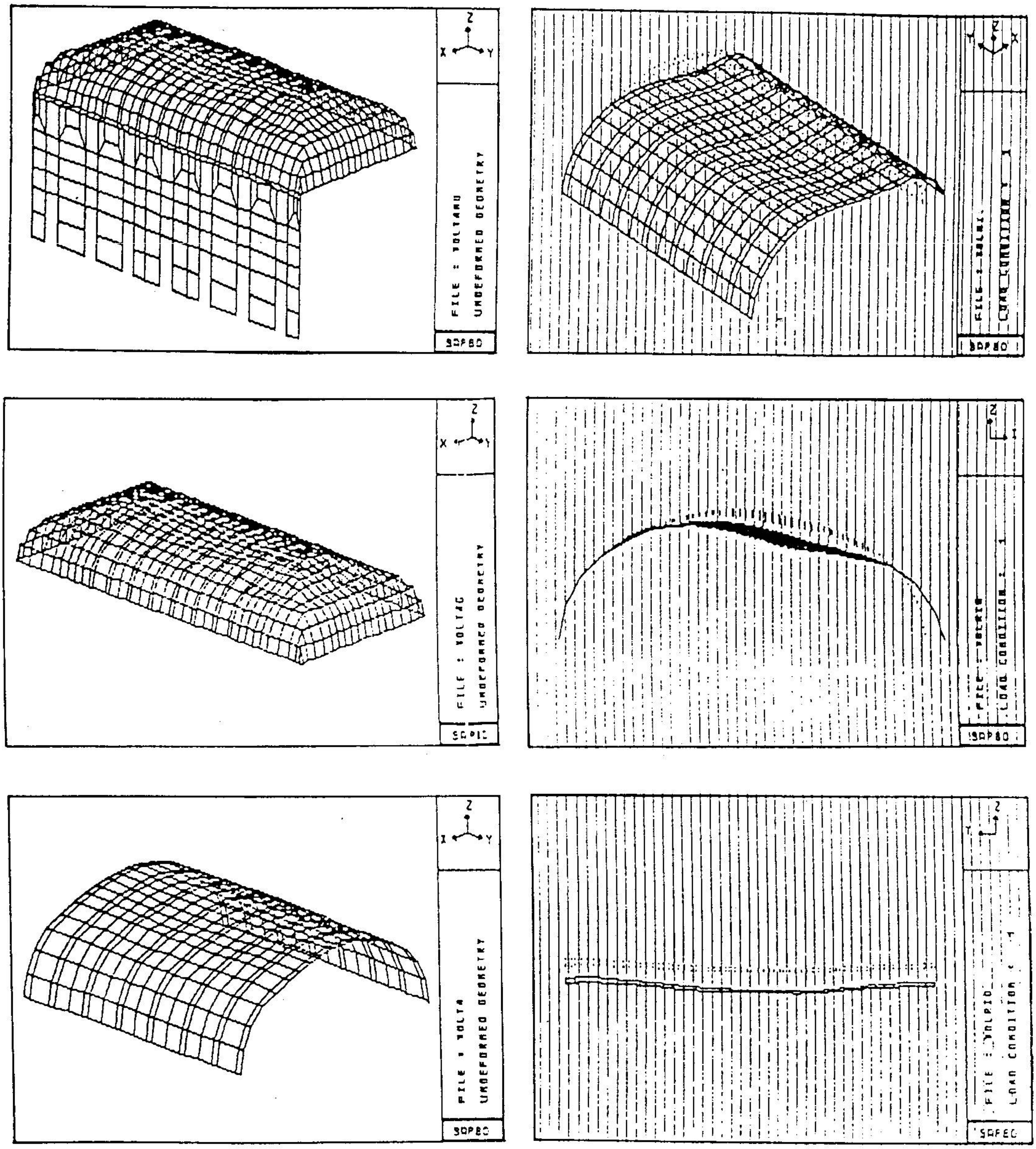


Fig. 9 The extrados of the Vault of the Major Council hall.

Fig.11 The intervetions on the masonary structures.





The finite element models used to analyze the vaults.

anchored at the facades (Figure 11); the Figure 12 shows the stell beams, placed at the extrados of the vault of the Major Council Hall. The connection between the steel structures and the vaults is provided by prestressed bars. The value of the pretension has been accurately evaluated ; in fact the bar can provide two different functions: (A) to provide support forces, (B) to impose deformations that allow to partially ricovery the old displacements and to reinstate then into the vaults a bearing capability due to curvature. The installation of an instrument net, integrant part of the restore intervetion, was necessary to evaluate which of the two behaviours should be spontaneansly established in dependence of the actual vault's stiffness.
The maintenance in activity of the instrumentation system for a suitable period, permits to verify the behaviour in the time increasing and then the final validity of the intervetion.
The monitoring system is composed by an automatic remote data acquisitor connected with a computer (Figure 13) in which are recovered the values measured by the different instruments: strain gunges on the bars to evaluate the stresses (Figure 14), transductors into the vault to measure the crack's width, and the deformations of the vault's intrados (Figura 15) and at least termometers placed on the steel strucuture and on the mansory vault.

2.5 Monitoring during the construction of the great concrete buildings

The construction of the great concrete buildingseven emphasizes the opportunity to control the corrispondence between the actual behaviour and the design hipothesis that can be of particular importance for the constructions that use precast structural elements, stiff as a whole and weak in the assembly; more subject then the other to the effects of the indirected actions, as the soil deformations and the thermic variations. For this purpose, have been placed some instrumentation system in threee Service Centers and Offices of the Ministry of Finance in Venice, Genova and Alessandria (Figura 16, 17). There are the most delicate problems in the center in Genova, where are been erected impressive retaining structures (Figure 18 ;the load cells placed on the rear of the retaining structures and the load cells placed on the head of the anchorage bars permit, in particular,to have ever the effective values of the earth thrust and any relaxation of the anchorage bars with time increasing.

3 MONITORING OF AN EXISTING BUILDING, IN WHICH ARE PRESENT FAIULERS DUE TO CYCLIC AND EVOLUTIVE PHENOMENA.

3.1 General aspects

Such as kind of controls permit to have usefull data to a better understanding the actual phenomena. So it's possible to make a reliable diagnosis and operate a choise about the intervetion criteria.
This stage of the study is of particular importance in the case of the monuments, in which an accurate preventive cognitive investigation makes it possible to optimize the intervetions in respect for the historic-artistic value of the building.

Fig. 12 The steel beams placed at the extrados of the vault of the Major Council Hall.

Fig. 13 The monitoring system

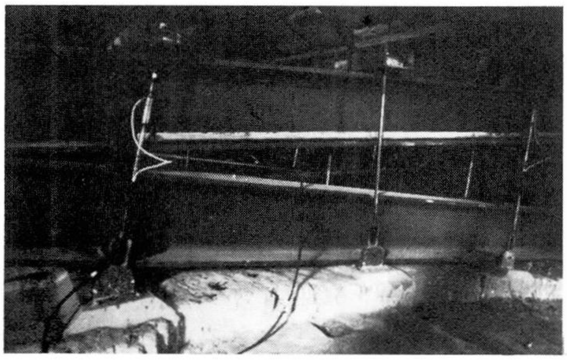


Fig. 14 The strain-gunges on the bars to evaluate the stresses.

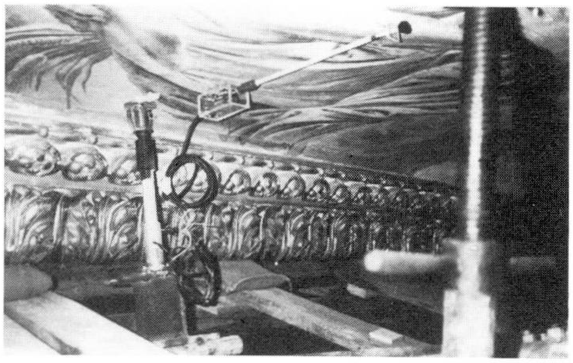


Fig.15 The instruments placed at the intrados of the vault.

Fig. 16 The Service Center in Venezia

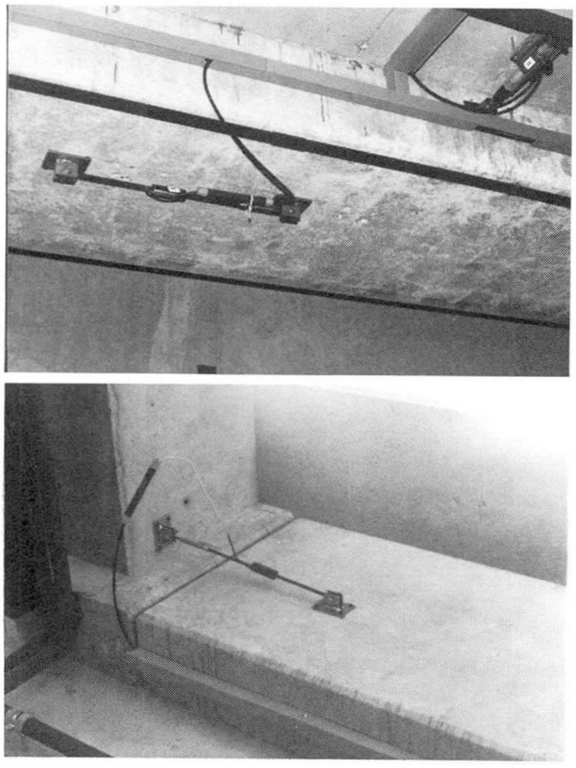


Fig. 17 Some of the instruments placed into the Service Center in Venezia.

3.2 The monumental building of the Tabularium - Senatorial Palace in Campidoglio

The monument present serious damages and it's now in very dangerous conditions. The first part of the study was concerned individualization of the phenomena with the ascertainment of the causes (Figures 19 and 20).
Apart from the investigation to ascertain: the connection between walls and slabs (assaies), the sthrenght (test with flat jacks), the diffusion of cracks (ultrasound, ...), ecc. are been carried out other measurement on time increasing, to understand the effects of the temperature and of the soil settlements. The results of these measurement are shown in the following Figures:
Figure 21: Temperature distribution through the Facade wall looking on the Roman Forum;
Figure 22: variations of the span of the archs that are in perpendicular direction with respect the Facade of the Figure 19, due to the temperature variations;
Figure 23: development,on time increasing, of the out of plumb of the bell tower that now presents at the top an horizontal displacement of about 50 cm. These deformations not only depend, on the season's differences of temperature, but also due to settlements of the foundations. The settlements are probably a consequence of the excavations carried out in the first half of our century, to uncover the temple of Veiove.

3.3 The bridges of the Roma-Viterbo railway

These bridges present, apart from a widespread state of decay, some important longitudinal cracks that have disconnected the archs from the external walls (Figure 24 and 25). The monitoring in the time domain of this phenomennon was carried out by the use of joint-gunges and extensimeters and permitted to acertain the stabilization of the relative displacements.Then it was possible to plan out, for the successive bridges, a much simple consolidation intervetion: instead of empting out all the bridges and create a reinforced concrete caisson, working together to the masonry structure (Figure 26), the design was restricted to carried out an effective connection between external walls and archs, inserting a prestressed Dywidag bars system (Figure 27).

4 CONTROL OF BEHAVIOUR OF AN EXISTENT BUILDING RELATED WITH THE DISTURBANCES DUE TO EXTERNAL WORKS

4.1 General aspects

This is the case in which excavations are carried out or other buildings are erected in the closeness of the considered building. In these problems the monitoring can minimize the preventive intervetions, planning out an action strategy thet will be carried out gradually depending by the evolution of unfavorable events (removal of the people, insertion of cribs or chains, close to traffic, ecc...). An instrumentation net has, in these cases, its

Fig. 18 Service Center in Genova, the retaining structures.

Fig. 19 The facade looking on the Roman Forum, in the present state.

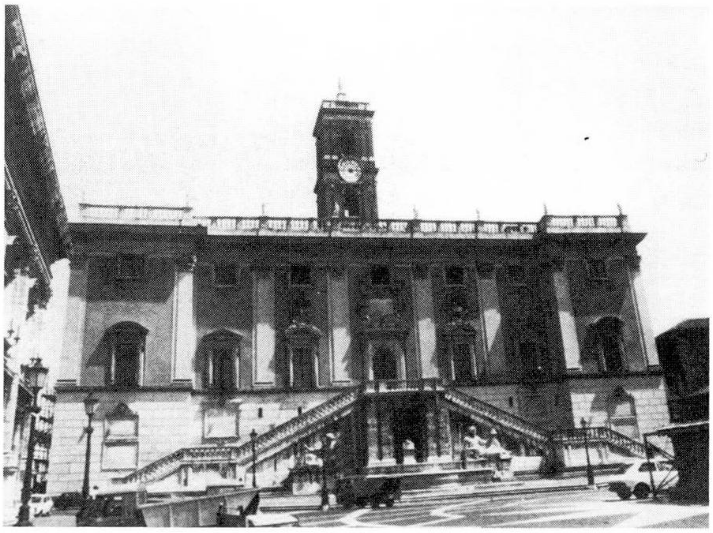


Fig. 20 The facade looking on Campidoglio square in its present state.

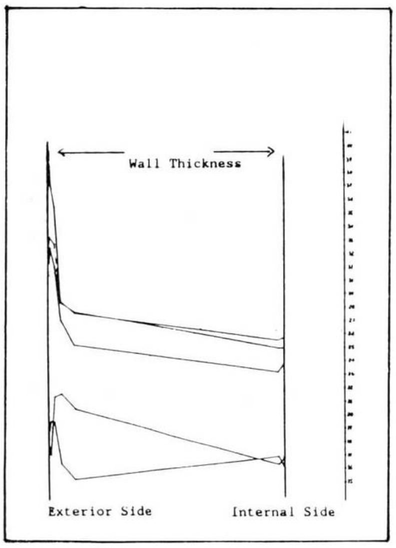


Fig.21 Temperature distribution trough the facade wall looking on the Roman Forum.

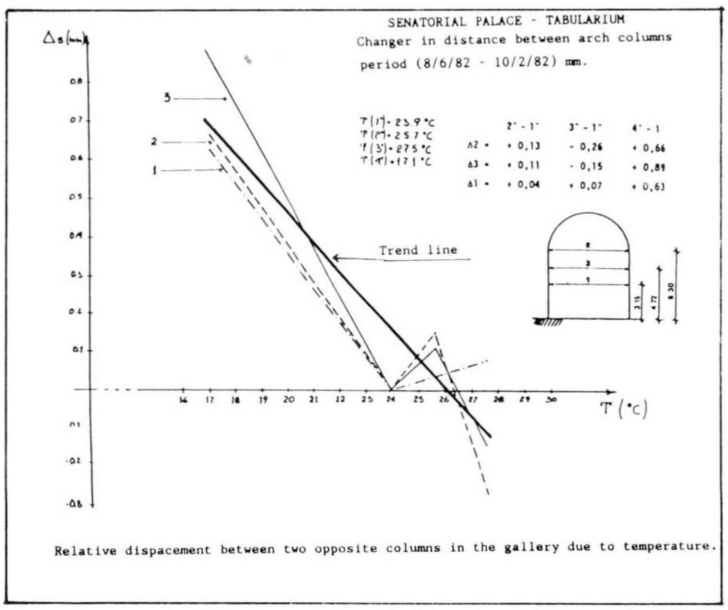


Fig. 22 Variations of the spans of the archs, due to temperature variations.

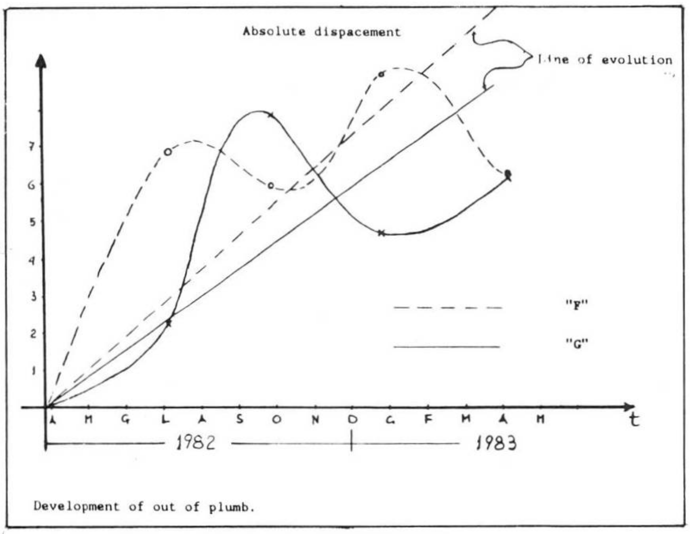


Fig. 23 Development, on time increasing, of the out of plumb of the bell tower.

Fig. 24 A bridge of the Rome-Viterbo railway

Fig. 25 The lesion in the masonary structures of the bridge.

Fig. 26 The realization of a reinforced concrete caisson.

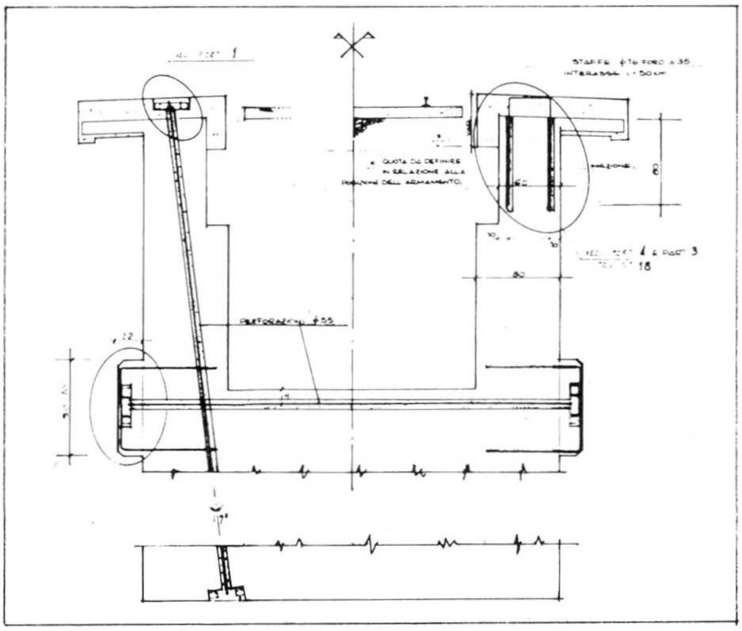


Fig. 27 The intervetions based upon the insertion of prestressed Dywidag bars.

independently by the data measured and by their scientific significance, assuming the role, in a sense, of an insurance policy; it can be represent, in fact, a warning bell that intervenes only in case of unfavorable or risks events.

4.2 The construction of a tunnel of the subway of Rome, under some building the Castro Pretorio area.

The instruentation net, that is composed by temperature sensors, cracks and strain meters, was placed in the most signifative locations on the buildings (Figure 28, 29 and 30); the remote for the data recording was connected with an alarm system. The good course of the works had avoided, in any case, the trasmission of alarm signals by the system, because nothing of the quantities measured, exceeded the preset alarm levels. So the buidings could remain occupied during all the work time without any risk.
The measurement carried out also revealed theirselves of great scientific interest permitting the clear knowledge of happened phenomena; the comparison between these phenomena and those theorically foreseable by a finite element model, was soddisfacent (Figure 31). A more detailed study is reported on another paper (1).

4.3 The excavations into the court of Ex Massimo Institute, to realize a bunker caveau containing artistical and archeological finds of the Superintendence of Rome.

The realization of a great excavation (Figure 33) in the court of the building (Figure 32) to realize a bunker cavèau, obliged to several precautionary measures during the excavation (chains ...). Nevertheless the importance of the excavation and the values of the forces related with the work recommended to strictly control all the stages of the intervention,by a monitoring net, to opportunely determine differential soil settlements, wall rotations, changes of tension in the chains, ecc... The Figure 34 showes some of the installed instruments.
The monitoring system, still upon office, showed an actual stationary situation and so permitted a quick work development.

REFERENCE.

1 CANGIANO M., CROCI G., Control and detecting system apllying to some buildings located in Rome and connected with works relating to the construction of the underground line.

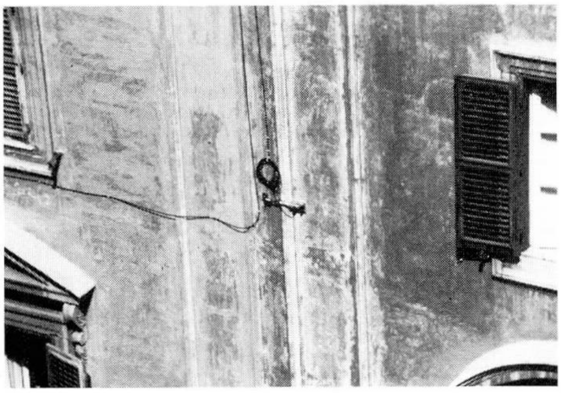


Fig. 28

Fig. 29

Fig. 30

Fig. 28, 29, 30 The instruments installed on the most significative locations on the building.

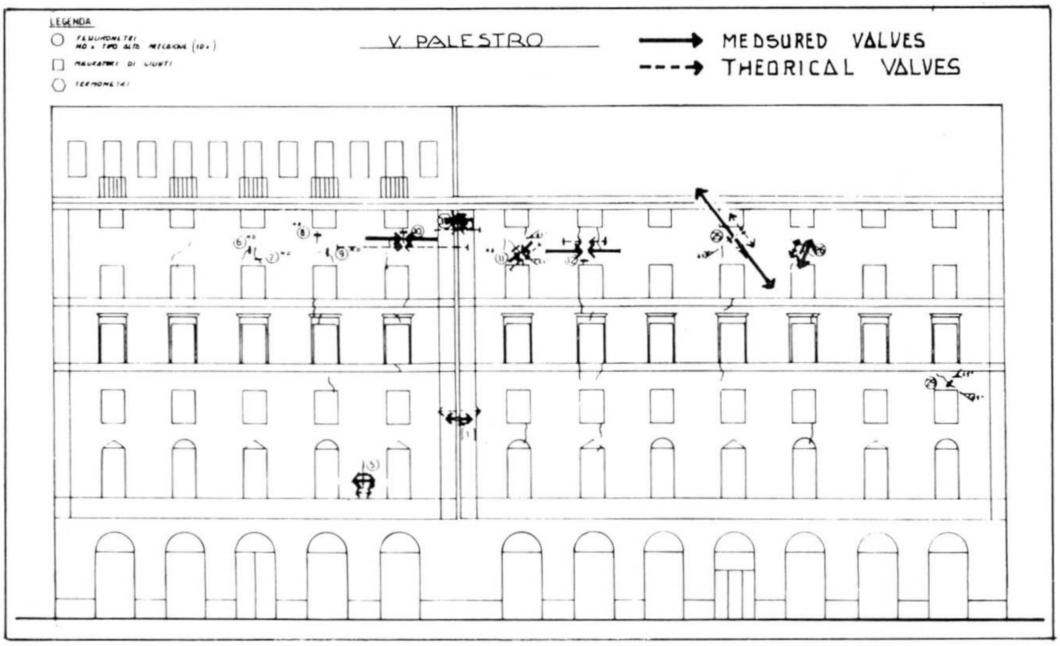


Fig. 31 Comparison between measure of phenomena and the results of theorical analysis.

Fig. 32 View of the Ex Massimo Insti-tute

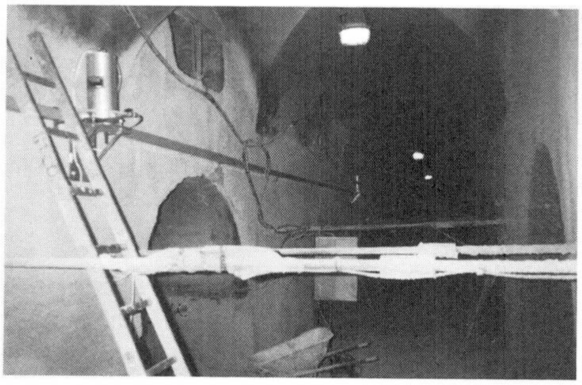
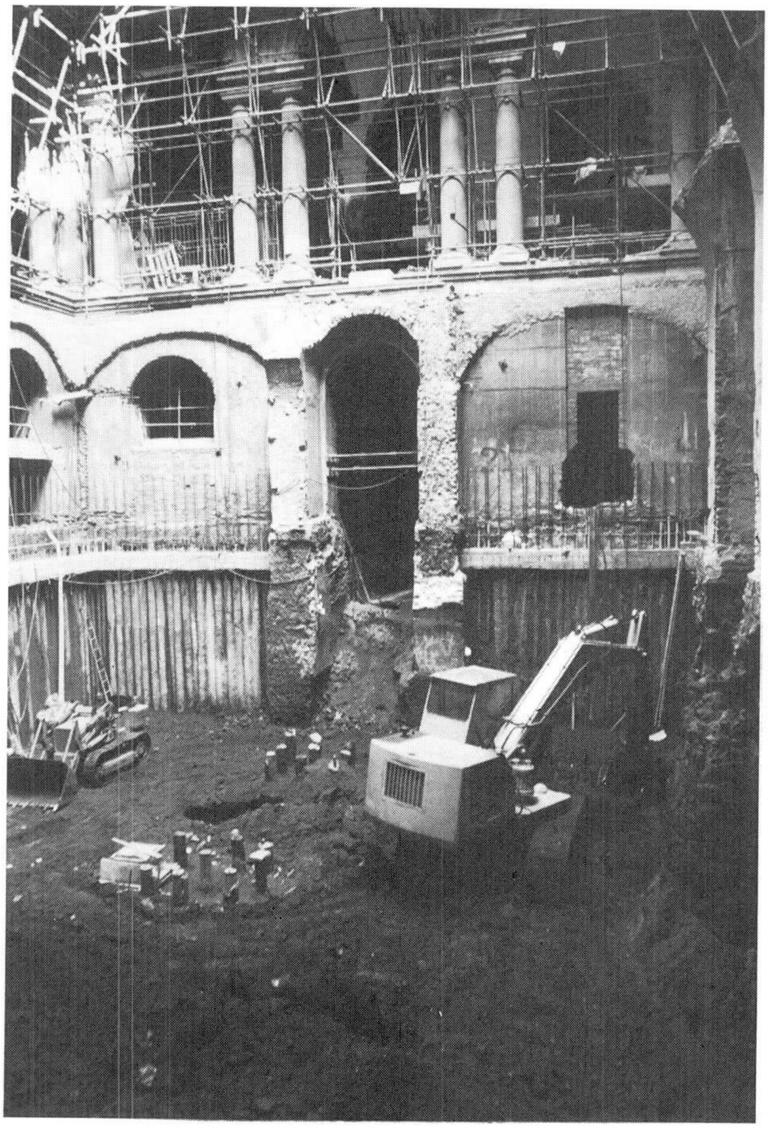


Fig. 34 Some of the installed instru-ments.

Fig. 33 The realization of the excavation to realize a bunker caveau in the court of the building.



Systematic Approach to the Structural Diagnosis of Reinforced Concrete

Approche systématique pour la détermination de l'état de structures en béton armé

Systematische Vorgehen zur Beurteilung des Zustandes von Stahlbeton

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SUMMARY

The assessment of the conditions of RC and PRC structures calls for the combined use of several non-destructive or partially destructive techniques. After a brief examination of the deterioration processus affecting concrete and steel properties, available investigation methods are systematically examined and new techniques currently being developed are illustrated pointing out their applicability within a systematic structural diagnosis procedure.

RESUME

L'évaluation de l'état des structures en béton armé et précontraint doit se faire au moyen des plusieurs techniques non destructives ou partiellement destructives. Après une brève description des processus de détérioration du béton et de l'acier, on passe en revue les méthodes d'essai disponibles et on présente quelques techniques nouvelles actuellement en cours d'expérimentation. Les techniques indiquées font partié d'un procedé systéematique pour le diagnostic des structures.

ZUSAMMENFASSUNG

Die Zustandsermittlung von Beton- und Spannbetonbauwerken erfordert eine ganze Reihe von zerstörungsfreien und teilzerstörenden Techniken. Nach einem Ueberblick über die Zersetzungsvorgänge bei Beton und Stahl werden die verschiedenen Prüfmethoden besprochen. Die Anwendung neuer, noch in Entwicklung stehender Techniken bei systematischen Untersuchungen wird beschrieben.

INTRODUCTION

Non destructive diagnoses can be applied both to new structures and to works that have been in service for a long time. In the first case, the diagnosis serves as a quality control of the execution, in the other, it can be used to identify execution defects as well as damages which may have been produced later on.
In this report these two aspects will be examined together as a number of the available techniques are applicable to both, whilst others are more suited to either the one or the other.
It should be pointed out that in the case of newly erected structures, the current approach to durability tends to define various classes of exposure and to lay down different criteria for each. Hence, quality control will consist of checking whether these requirements have been met.
In this connection we attach Tables 1 and 2 taken from a proposal developed jointly by the CEN and by the editorial committee of the Eurocode.
As for structures that have been in service for a long time, the aim is to identify the deterioration processes they have been subjected to as well as the ones under way.
These processes are also strictly tied up with the exposure classes referred to above, and naturally enough, with the design and execution of the structure.

1. A FEW OBSERVATIONS ON DETERIORATION PROCESSES

1.1. Physical processes

The physical processes leading to the deterioration of concrete properties are:
- cracking
- freezing and thawing effects
- erosion

While the last two are usually easy to identify, cracking calls for an accurate study aimed at identifying its causes.

1.2 Chemical processes

1.2.1 Carbonation

The carbon dioxide (CO_2) present in the air penetrates into the concrete pores and may react with the calcium hydrowide ($Ca(OH)_2$) present in the cement mix via a process which can be simplified in these terms:

$$Ca(OH)_2 + CO_2 \longrightarrow CaCO_3 + H_2O$$

This reaction begins on the outer surfaces exposed to the air and penetrates inward at a rate governed essentially by the permeability of concrete.
The phenomenon lowers the pH of the material, from the habitual values of 12.5 ÷ 13.5 to 8.3 ÷ 9, enabling reinforcement corrosion to take place.
From the viewpoint of the mechanical characteristics of concrete it has no negative repercussions, as a rule, actually this phenomenon gives rise to an increase in surface hardness; thus, it should be regarded as a deterioration process solely as concern the durability of metal reinforcement.

Table 1: Exposure classes related to environmental conditions

Exposure class		Environmental conditions
1 dry environment		e.g.: interior of buildings for normal habitation or offices
2 humid environment	a without frost	e.g.: - interior of buildings where humidity is high (≥ 60 %) - exterior components - components in non-aggressive soil and/or water
	b with frost	e.g.: exterior components exposed to frost or non-aggressive soil and/or water
3		Humid environment with frost[1]) and de-icing agents e.g.: exterior components exposed to frost or non-aggressive soil and/or water and frost and de-icing chemicals
4 seawater environment	a without frost	e.g.: - components in splash zone or partially emerged from seawater - components in saturated salt air (direct coast area)
	b with frost	
The following classes may occur alone or in combination with the above classes:		
5 chemical environment	a	- slightly aggressive (gas, liquid or solid)[2]) - aggressive industrial atmosphere
	b	moderately aggressive (gas, liquid or solid)[3])
	c	highly aggressive (gas, liquid or solid)[4])

[1]) Under moderate European conditions

[2]) ISO-classification (ISO DP 9690): A 1 G, A 1 L, A 1 S

[3]) ISO-classification (ISO DP 9690): A 2 G, A 2 L, A 2 S

[4]) ISO-classification (ISO DP 9690): A 3 G, A 3 L, A 3 S

Table 2: Requirements for reinforced concrete

Exposure class	1	2a	2b	3	4a	4b	5a	5b	5c
max w/c ratio	0.65	0.60	0.55	0.50	0.50	0.55	0.55	0.50	0.45
min cement content in kg/m³	260	280	280	300	300	300	280	300	300
Strength class	If the strength classes below are reached, the values above are assumed to be fulfilled								
	C 16/20	C 20/25	C 25/30	C 30/35	C 25/30	C 30/35	C 25/30	C 30/35	C 30/35
air content	—	—	+	+	—	+	—	—	—
frost resistant aggregates	—	—	+	+	—	+	—	—	—
Types of cement	CE I 32.5/42.5 CE II 32.5/42.5 CE III 32.5/42.5 CE IV 32.5/42.5	the same as 1	CE I 32.5/42.5 CE II 32.5/42.5 CE III 32.5/42.5 CE IV 42.5	CE I 32.5/42.5 CE II 42.5 CE III 42.5	the same as 1	the same as 2b	the same as 1	the same as 1	the same as 1
					however sulfate resistant cement for sulfate contents of: > 500 mg/kg in water > 3000 mg/kg in soil				
water impermeability	—	—	+	+	+	+	+	+	+

1.2.2 Chloride penetration

The penetration of chlorine ions into concrete has a de-passivating effect on the reinforcing steel; this phenomenon is due exclusively to the free Cl^- ions that come into contact with concrete through the water contained in its pores, whilst the ions that are chemically bound with the constituents of the mix play no part in it.
Carbonation enhances this phenomenon, since it reduces the binding capacity of cement constituents for chloride compounds. The de-passivating effect of chloride is counteracted by concrete alkalinity, and, all other conditions being equal, it develops only if the concentration of Cl^- ions exceeds a certain threshold value which depends upon concrete pH.
In this case too, the fundamental parameter influencing this phenomenon is concrete permeability.
The structures most exposed to this type of attack are those located in a sea-water environment and roads on which deicing salt is scattered.

1.2.3 Sulfate attack

Under the attack of the sulfates present in water (high concentration of SO_4^{--} ions) concrete swells and its surface turns whitish, soft and pasty.
The compounds that have the most detrimental effects on concrete are Ammonium, Calcium, Magnesium and Sodium sulfates.
These substances react with hydrated calcium aluminate and with calcium hydroxide that are present in hardned cement, resulting in the formation of Ettringite and of other compounds which crystallize and expand in doing so, creating a considerable pressure which leads to the disintegration of concrete.
The severity of this phenomenon hinges largely on concrete permeability.

1.2.4 Acid attack

This phenomenon can be summarized as follows: when an acid (HCl, H_2SO_4, HNO_3, etc.) penetrates into the interior of hardened concrete through its pores it may react with the calcium-based compounds contained in the material, leading to the formation of salts which are usually highly soluble and therefore can be washed away by water.

1.2.5 ASR induced deterioration

Essentially, ASR is a chemical reaction between some forms of silica contained in the aggregate and the alaklis contained in cement; the outcome of the reaction is a gel that tends to expand and give rise to cracks.
Tle latter may be considerable in terms of both length and width. In general, this phenomenon occurs within a year since the casting, but it has also been observed to occur at a later date. It can only take place, however, under the following concurrent conditions:

- considerable humidity in the environment
- a sufficient quantity of alkalis in the cement
- the presence of a significat amount of reactive aggregate.

The visual outcome is an extensive network of cracks forming a random pattern (like a spider web) independent of static conditions.

1.2.6 Reinforcement corrosion

This is an electro-chemical phenomenon which develops through two processes: an anode and a cathode one.

- Anode process

$$2Fe = 2\ Fe^{++} + 4e$$

(dissolution of iron and release of electrons)

- Cathode process

$$O_2+2H_2O+4e = 4OH^-$$

(consumption of electrons and reduction in the amount of oxygen in contact with the reinforcement)

The overal reaction is as follows:

$$2Fe+O_2+2H_2O = 2Fe^{++}+4OH^- = 2Fe(OF)_2$$

(formation of rust)

The phenomenon is affected by the potential and the pH of the environment in which the reinforcement is placed, as can be seen in fig. 1: hence the importance of the carbonation and chloride penetration phenomena described above which influence both concrete pH and the potential.

1.2.7 Hydrogen embrittlement

This phenomenon has been observed in high yield point type steel (prestressing steel).

It evolves through the extension of the cracks occurring through failure at the root as the adjacent material becomes brittle owing to the migration of atomic hydrogen towards it, this migration being induced by the cathodic reaction complementary to the iron dissolving anode process.

Steel sensitivity to this attack increases with strength and is a function of its composition and structure.

This phenomenon will not take place when the steel is embedded in concrete of a suitable composition, with high pH value, free from chloride, sulfate and thiocyanates.

2. INVESTIGATION METHODS

A visual examination perfomed by a structural specialist is of fundamental importance for the identification of construction flaws and visible deterioration processes; this will be the basis of all subsequent investigations.

It is essentially aimed at:

- the identification of cracks and their causes. There are many different types of cracks and in many cases it is not easy to recognize the phenomena that produced them; in dealing with these deterioration phenomena special importance should be attached to even minor cracks located along the reinforcement or along the path of prestressing tendons. The significance of transverse cracks with an opening of less than 0.3 mm is controversial;
- the identification of rust stains, especially in the areas adjacent to the prestressing tendons. This is one of the crucial tasks of visual inspection;
- the identification of saline efflorescence revealing water permeable areas.

When obvious signs of local corrosion are detected it will be adviseable to proceed to a direct examination of ordinary reinforcement by removing a portion of the concrete cover; for a larger scale investigation, instead, especially where prestressing tendons are concerned, it will be necessary to resort to more complex methods, as described below.

When a structure has an iterative configuration, the visual examination will generally enable the information obtained from one representative element to be extended to the entire structure.
When visual observation suggests the need of a more through investigation, it will be necessary to resort to special testing instruments and techniques.

2.1 Special investigation techniques for concrete

2.1.1 Initial approach tests

Let us consider first of all non-destructive or partially destructive testing methods which might be defined as 'initial approach' tests because of their relative ease of execution: cores, microcores, on-site compressive tests, pull-out, pull-off, Windsor probe, pH measurements.
By means of microcores (Ø = 30 mm) specimens can be obtained from the areas deemed to be most significant for visual examination purposes and subjected to laboratory tests designed to determine the mechanical characteristics of concrete at various points and at different depths. The results should be interpreted taking into account the impact of core drilling which can be considerable. A viable alternative consists of on-site compressive tests (10). This is done by loading in compression, till failure, portions of the reinforced concrete structural members (beams, slabs, etc..) without removing them from the overall structure they are part of. The testing apparatus is shown in figure 2; a basic requirement is to make use of plates sufficiently rigid to reduce to the minimum the risk of uneven distribution of the load and to this end the tierods should be placed as close as possible to the edges of the element being tested.
When interpreting the results, however, it should be kept in mind that a portion of the compressive load effects might have been carried by the adjacent areas.
Other, moderately destructive testing methods, such as pull-out, pull-off, Windsor test, provide an indirect measurement of concrete strength on the surface.
The rebound hammer, an instrument widely used on account of the ease with which it can be applied, yields rather uncertain results, especially when dealing with carbonated concretes having a hardened outer layer.
All these techniques as a whole, however, cannot provide failsafe results, since they are primarily intended for use in comparative assessments. They can be instrumental both for quality control purposes and to evaluate the damages affecting a reinforced concrete structure that has been is service for a long time. Concrete pH measurements, instead, are applicable only to the latter type of investigation, i.e., on pre-existing structures. They can be performed on cores or microcores taken from the structure as well as on fragments chiselled off starting from the outer surface of the structure. The specimens must be tested right away to prevent them from remaining in contact with carbon dioxide for a sustained period ot time, as the CO_2 contained in the air could alter the results (an exposure time of a few hours is acceptable).
The following procedures can be employed:
- spraying with phenolphthalein in ethanol solution
- pH-meter

The alcoholic phenolphthalein solution in 1% ethanol turns pink/purple when it comes in contact with materials having a pH of over 9.8 and stays colourless in the case of lower values (photos 1 and 2).
The pH-meter should be used in doubtfus cases and whenever one wishes to obtain more detailed pH data as a function of depth. It can be applied by taking concrete samples and breaking them up into a powder. When mixed with distilled water the samples will provide an accurate measurement of pH values at various depths.

2.1.2 More rigorous testing methods

Among the so-called "second approximation" tests let us mention ultrasonic pulse velocity tests, tomography and dynamic pulse tests. We shall not describe these techniques in detail as they are illustrated by other reports dealing specifically with this subject.
Finally, there are other tests - still being perfected - that seem very promising. One is the on-site permeability test with which the local permeability of a structure can be assessed and, as we have seen, this is a fundamental parameter influencing almost all deterioration phenomena. In the case of new structures it can be profitably used to evaluate execution quality, since the results provide data not only on the composition of the casting aggregate size and grading, water/cement ratio, cement quantity) but also on on-site compacting methods.
As for older structures, it can be used to substantiate the data provided by pH measurements, especially when the regularity of the diffussion of the chemical phenomenon - that may be interrupted by the formation of calcium carbonate - is doubtful. The testing method, developed within the framework of a joint research project by the Politecnico di Torino and ISMES, is similar to a technique devised a few years ago by Figg (6).
The test we have developed consists of making a small diameter hole in the structure to be examined and introducing in it air, water or gas under pressure: then the reduction in pressure over time is carefully observed.
Fig. 3 provides, as an exemplification, several diagrams illustrating the results of tests performed on various concrete samples. The assessment of concrete permeability must take into account the possibility of an uneven diffusion of the fluid owing to local defects or pore clogging produced while making the hole.
Another method, currently being tried out within the framework of the above mentioned research project, is based on thermographic measurements (7). As is known, thermography can be used to map out surface temperature values, with an accuracy of about 0.1 °C. Thus, in the presence of a heat gradient, this reveals differences in conductivity levels corresponding to different materials or local disuniformity.
A first application may consist of combining visual thermographic inspection with on site permeability tests to check the even diffusion of the fluid inside a structure. Notably, this provides information as to the presence of preferential directions (cracks, etc.). Furthermore, the thermographic procedure can be used to detect local disuniformity in the material or in hygrometric conditions.
Laboratory tests have resulted in the identification of gaps and holes in masonry structures, water infiltrations in concrete slabs, grouting defects in prestressing tendons, segregation areas (photo 3).

In all cases, a thermal gradient was artificially induced in the elements to be checked.

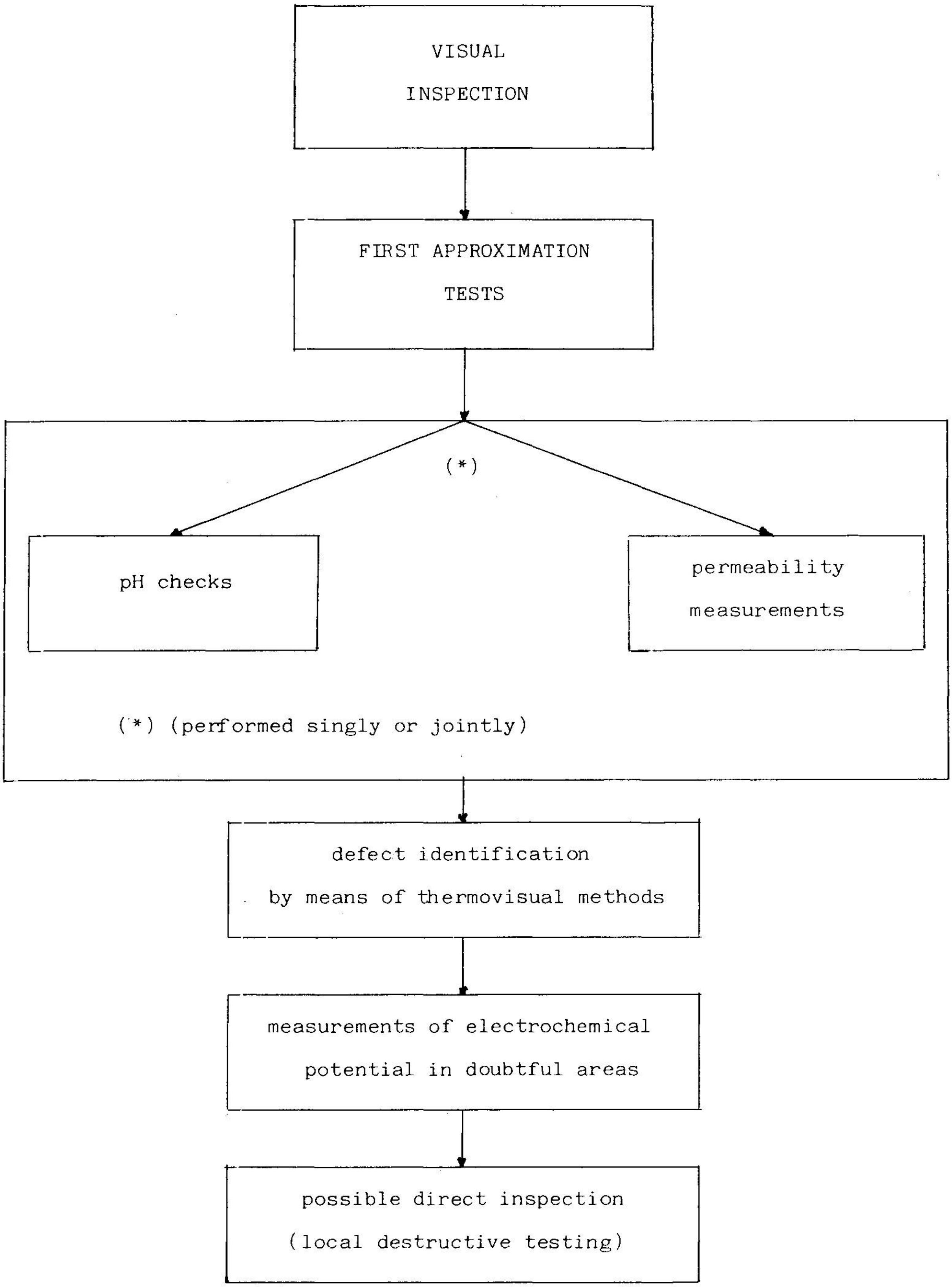
2.2 Special investigation techniques for embedded steel

The non destructive techniques used to assess reinforcement conditions are aimed at determining the geometrical characteristics of the reinforcement and whether corrosion processes are under way.
The determination of reinforcement position, diameter, cover thickness and the presence of joints, is of the utmost importance both when checking the quality of newly built structures and in the assessment of older structures for which no past records are available.
To this end, magnetic detectors can be used (pachometer) but the results can be depended on only when concrete cover is not too thick.
More conclusive results can be obtained by means of thermography, although this requires the prior heating of the reinforcement, either by Joule effect or by means of electromagnetic induction. This technique results in a very clear picture of the way the reinforcement is arranged, and photographic evidence can also be provided.
The corrosion of reinforcing steel can be identified by means of electrochemical potential measurements (3) performed on the concrete surface. This non destructive testing method in based on the consideration that corroded iron portions differ from passivated one because of their different electrochemical potential.
Thus, the measurements are taken by means of a sample electrode of Copper (Cu) in a copper sulfape saturated solution ($CuSO_4$). The electrode's potential, referred to hydrogen, is + 316 mV.
The interpretation scheme for these measurements is given in figure 4.
However, it is important to point out that this method provides insight only on on-going corrosion processes and does not detect older phenomena which may be temporarily inactive: hence, before the tests, the conditions fostering the development of corrosion processes should be restored.

3. Summary considerations on working procedures to be applied in the assessment of existing structures.

In chart 1, reproduced below, we have summed up the logical sequence of steps to be followed when performing the investigations illustrated in the foregoing paragraphs.
It should be kept in mind that no single non destructive test applied in isolation can be regarded as decisive.
As rule, it is adviseable to try to correlate the various experimental approaches, comparing the results in order to enhance the reliability of the assessment.

Chart 1 : Sequence of investigation steps



4. TESTING THE STRUCTURES

The main problem encountered in the interpretation of tests carried out on real structures lies in having to draw conclusions concerning structural behaviour at all limit states, including ultimate LS, through the interpretation of checks performed under serviceability conditions.

The difficulties involved in extrapolating valid data from tests performed under service conditions is clearly borne out by the tests carried out on an overpass that was being torn down (9).

12 beams prestressed by means of sliding tendons were tested in the laboratory till failure. In 6 cases behaviour proved satisfactory and when the beam was demolished, after the tests, corrosion was found to be modest (fig. 5)

In the other 6 cases, instead, failure occurred under applied moments lower than expected and when the beams were taken apart tendon corrosion was found to be exremely severe, with residual sections of less than 50% (ifg; 6).

Yet, from an examination of the moment vs. displacement diagrams in figs. 5 and 6, it can be seen that in terms of dead loads all beams displayed the same stiffness values; hence, any non destructive test based on the assessment of stiffness with no loading would not have revealed the extent of the damages. Furthermore, under service loads, only the beams in the worst conditions of repair manifested cracks and hence service load tests would have detected only the severest cases of deterioration.

Although with the above mentioned reservations, it can be stated that the observation of structural behaviour under service conditions helps to integrate the findings of non destructive tests; especially valuable is the repetition at periodical intervals (e.g., every 5 to 10 years) of tests (such as topographic measurements of displacement, load tests, etc.) aimed at identifying changes in structural response.

Needless to say, a loading test performed till failure on a structural element suitably isolated from the adjacent ones, if practical, would provide much more conclusive data which could then be extented to similar elements by means of non destructive methods. A typical instance of this consists of testing till failure a number of joists belonging to a floor slab made up of a large number of identical elements after having removed the continuous cross members.

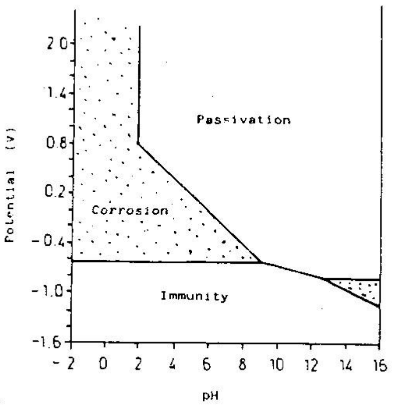
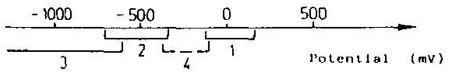


Fig. 1 - Steel corrosion vs. pH value and potential.



1) Perfectly passive steel embedded in aerated concrete

2) Attack through formation of couples

3) Generalized corrosion in carbonated concrete

4) Doubtful case

Fig. 4 - Interpretation scheme for potential measurements.

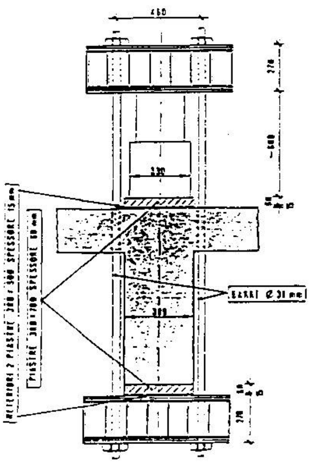


Fig. 2 - Arrangement of on-site compression tests.

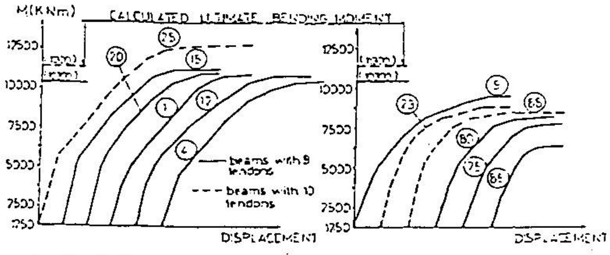
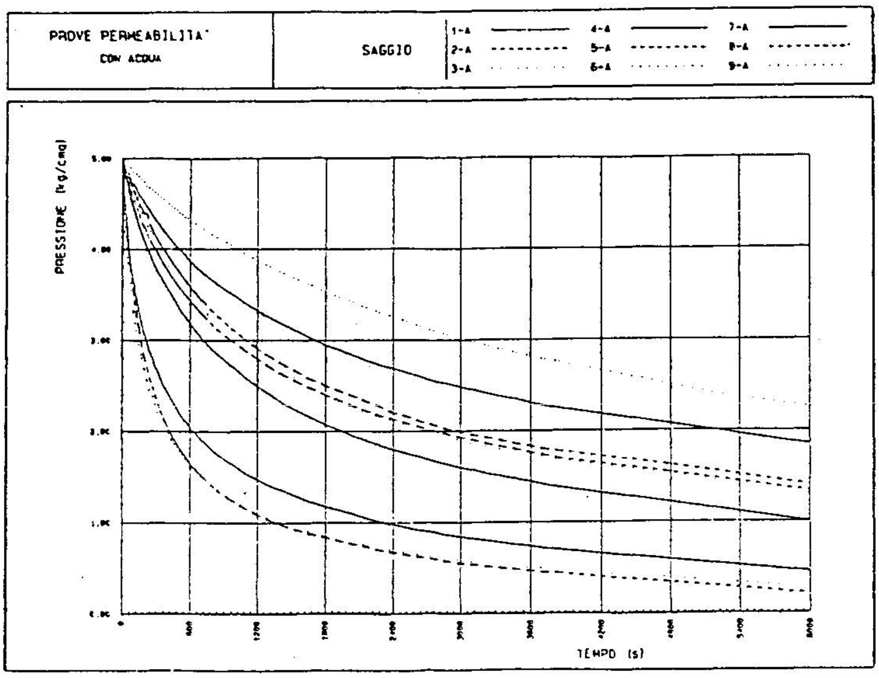
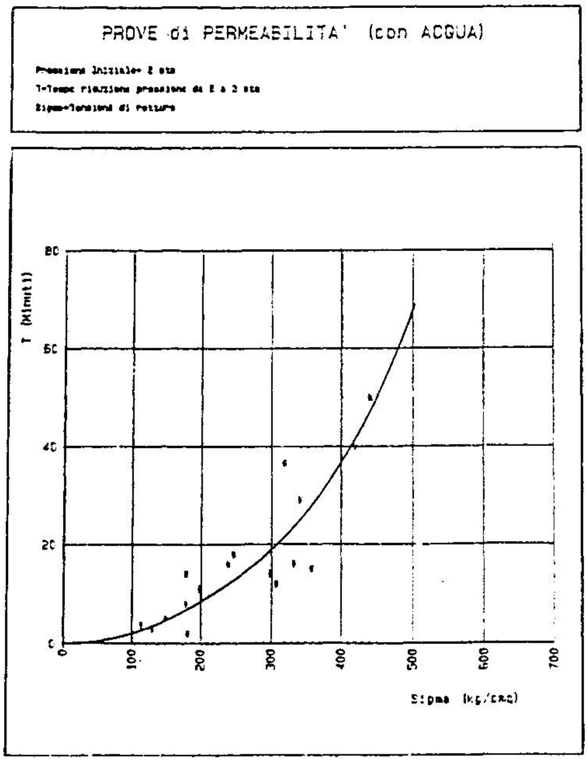


Fig. 5 - Fig. 6: Moment versus displacements diagram of tested beams



(a)



(b)

Fig. 3 - On-site permeability tests:
a) pressure drop times for concretes of different permeability, and b) correlation with strength.

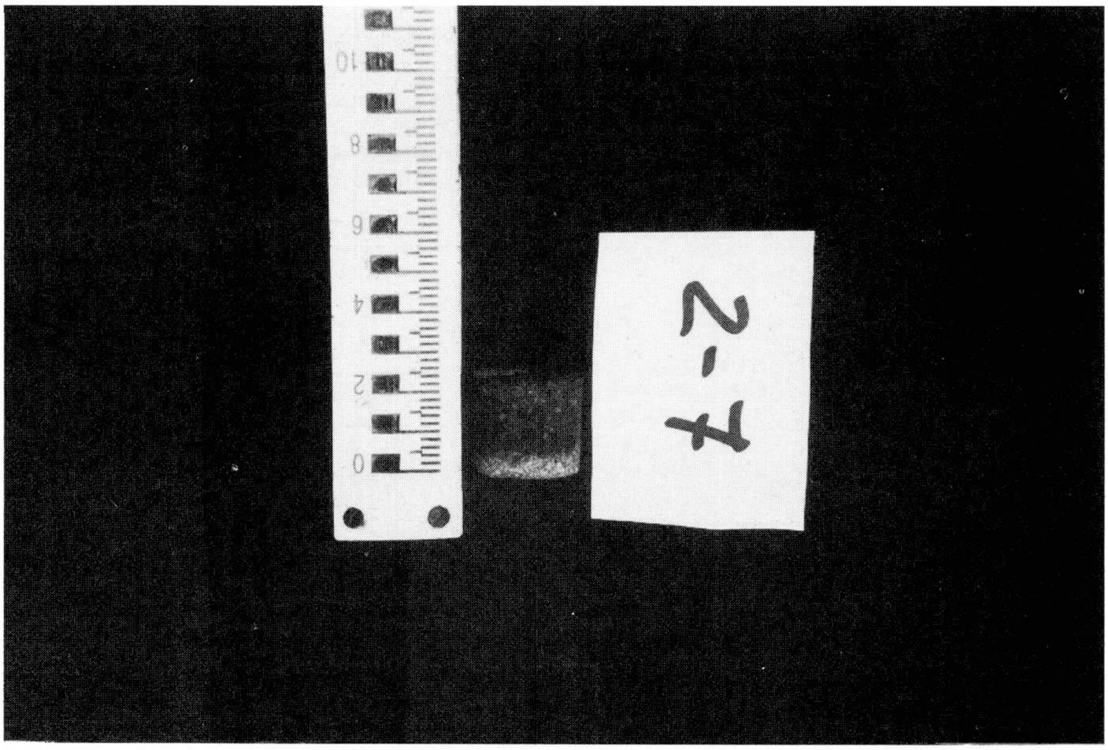


PHOTO 1 - Reaction to phenolphthalein on a microcore: in the colourless area pH $<$ 9.8

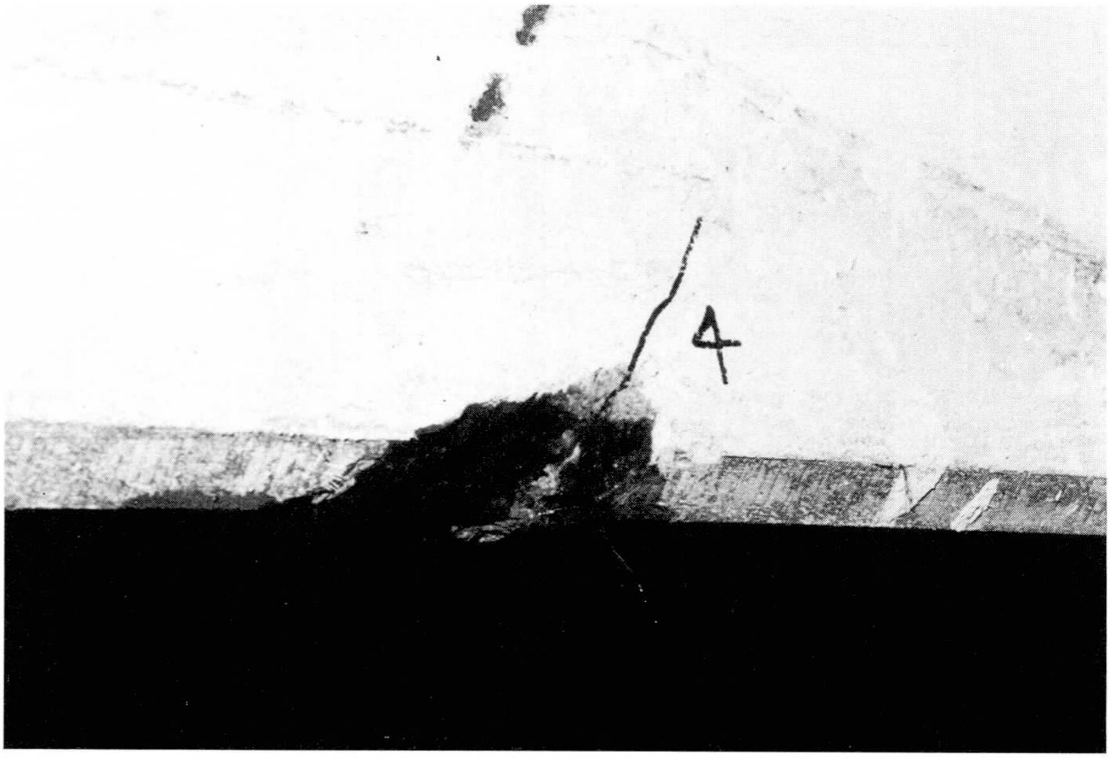


PHOTO 2 - On-site phenolphthalein reaction: the influence of cracks can be observed

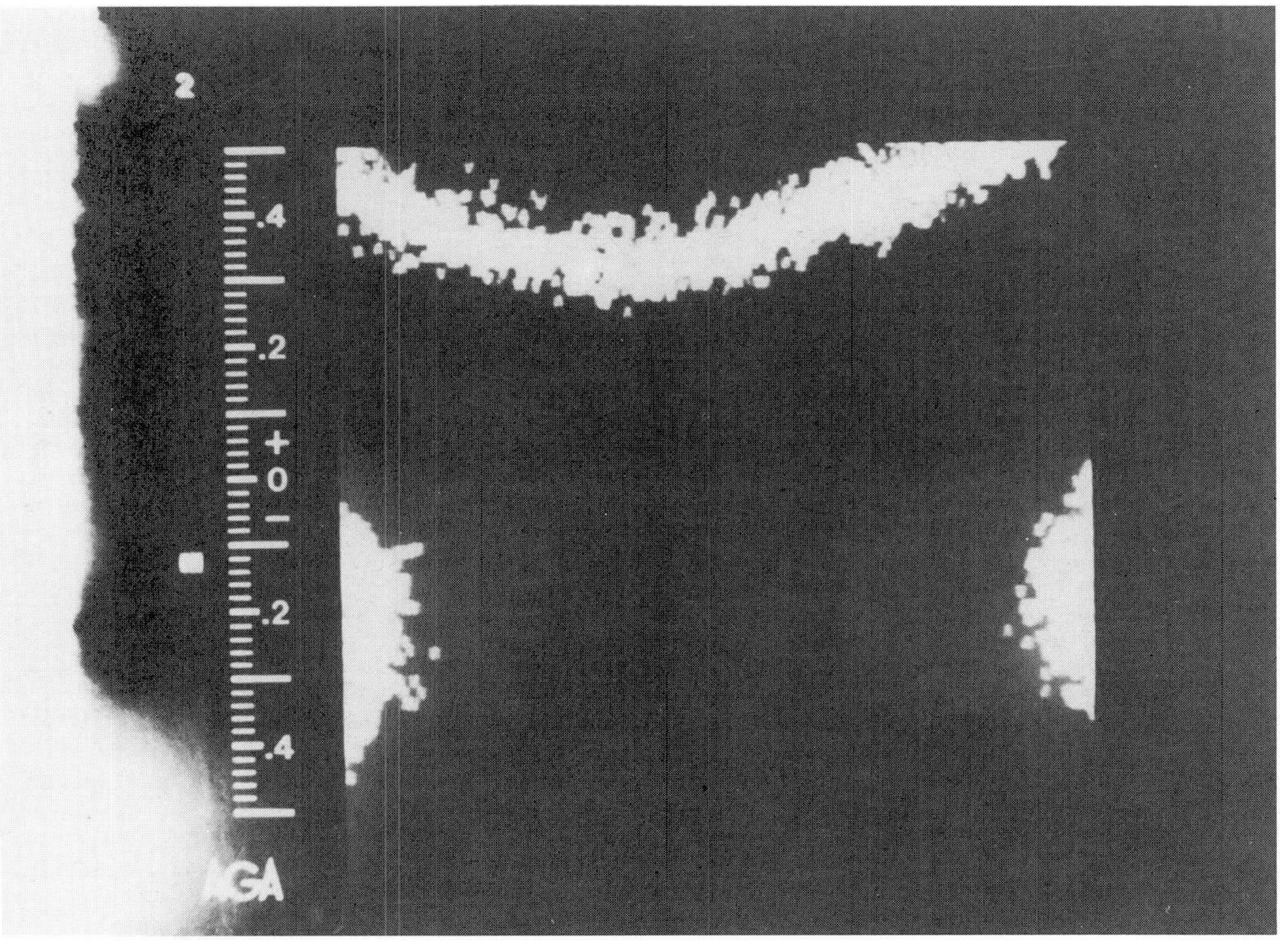


PHOTO 3 - Thermo-visual inspection on a sheath enclosing a pre-heated prestressing tendon: the dark area at centre reveals a grouting defect.

References

(1) Bollettino CEB n° 166 - "Draft Ceb - guide to durable concrete structures" - Contribution to the 24 th Plenary Session of C.E.B. Rotterdam - June 1985

(2) F. Massazza - "Durabilità del calcestruzzo" Aitec - Corso di aggiornamento sul calcestruzzo - Torino Maggio 1986

(3) Levi, Mancini, Napoli - "Structural damages due to alkali - silica reaction (ASR)" L'industria delle costruzioni n° 169 Novembre 1985

(4) F. Finzi - "Speranza di vita dei vari calcestruzzi strutturali" - Convegno C.T.E. 23.01.86 Milano

(5) P. Pedeferri - "Corrosione delle armature nel calcestruzzo" - Atti dell'isti tuto di Meccanica Teorica ed Applicata dell'Università di Udine (1985).

(6) R. Cather, J. W. Figg, A.F. Marsden, T.P. O'Brien - "Improvements to the Figg Method for determining the air permeability of concrete" - Magazine of concrete Research: Vol. 36 n° 129 : Dec. 1984.

(7) M. Seracini, L. Massotti, G. Clementi - "Alcuni risultati preliminari sull'im piego dei raggi infrarossi nello studio di Strutture Murarie" - Bollettino degli Ingegneri, 7 (1977), p. 12.

(8) Norma ASTM C 876 - 80

(9) L. Malisardi, P. Marro -Prestressed concrete motorway bridges - 25 years of observations - Proc. of AICAP Meeting - Riva del Garda - 1985.

(10) P.G. De Bernardi - "Determinazione in situ di caratteristiche meccaniche di vecchie murature" (being printed).

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Non-Destructive Condition Assessment of Concrete

Méthodes non-destructives d'évaluation de l'état du béton

Zerstörungsfreie Verfahren zur Zustandsbeurteilung von Beton

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SUMMARY
Under the auspices of the Swedish National Road Administration research on different non-destructive methods for condition assessment of concrete has been carried out comprising both laboratory and in situ tests on thermography, impulse radar, utrasonic testing, electropotential mapping and chloride content determination.

RESUME
La Direction Nationale des Routes de Suède a commandité des recherches portant sur différentes méthodes non destructives d'évaluation de l'état du béton. Les essais, en laboratoire et sur le terrain, ont fait appel à des techniques telles que la thermographie, le radar à impulsion, les tests ultrasoniques, les relevés potentiométriques et la détermination de la teneur en chlorure.

ZUSAMMENFASSUNG
Unter der Schirmherrschaft des Schwedisch Reichsamtes für Straßenwesen wurden ein Reihe von Forschungsprojekten im Hinblick auf zerstörungsfreie Verfahren zur Zustandsbeurteilung von Beton sowohl mit, Laborversuchen als auch entsprechenden Tests in situ, mit Thermographie, Impuls-Radar, Ultraschall, Elektropotential- Kartierung und Chloridgehalt-Bestimmung, durchgeführt.

1. BACKGROUND

Virtually all material is broken down with the passage of time, sooner or later, owing to the influence of the ambient environment or on account of wear, load etc. This is also true of concrete including reinforcement steel. Bearing structures of reinforced concrete therefore always have a limited service life, i.e. a time during which the bearing function of the structure is satisfactory. The service life can vary considerably, depending on design, material qualities, construction, protective treatment, environmental impact and loads. These matters are more comprehensively reviewed in [1].

Standards and other collections of codes include regulations in the respects that affect the structure as above. This knowledge, expression of which is given in the standards, is steadily increasing, and consequently structures designed and constructed in accordance with these standards acquire ever better durability. In many respects, however, our knowledge of resistance to environmental impact is fairly new and consequently older structures, even if they were perfectly made in accordance with the standards of their time, can be in very great need of maintenance and repair in order to "be kept alive".

As emphasized in the Swedish state-of-the-art report on operation and maintenance of bridges and other bearing structures [2], repair of concrete structures involves removing damaged concrete, and possibly also reinforcement bars, and replacing it with new material. Repairs of this nature involve three important tasks, namely:

- deciding which concrete is damaged and has to be removed
- removing damaged concrete with a suitable method, which leaves the remaining sound concrete and reinforcement undamaged and with a surface capable of bonding to the actual repair material
- supplying the repair material and possible new reinforcement in a manner giving a strong and lasting repair.

In the same way as stated in [3], criteria for removal of damaged concrete can be listed as follows. As a guideline the concrete must be removed if any of the following conditions are met:

- the concrete is found to be delaminated when sounding with a hammer or chain-drag
- the chloride ion content exceeds 0.3 per cent relative to the weight of the cement. To ensure the durability of the repair, every effort should be made to remove concrete containing more than 0.1 per cent chloride ions
- the concrete is not frost-resistant in pure water (this can be applied to a bridge deck slab from underneath through absorption of humidity, despite waterproofing on the upper side)

- the compressive strength upon testing of drilled-out cylinders is lower than is assumed in design, or the splitting strength is lower than seven per cent of the measured compressive strength

- microcracks are found on a chipped surface when it dries after-being blown clean and wetted

- the concrete is carbonated all the way into the reinforcement or so far that the residual service life is judged to be too short.

If the requirements of concrete removal listed above cannot be met, some other protective measure is necessary, such as the use of a proper coating or a cathodic protection system. Similar criteria as above are listed in [4]. These also include assessment and repair of structural effects on concrete of alkaline-silica reactions.

The criteria listed above can be looked upon as the basis for a proper condition assessment of concrete.

2. DESTRUCTIVE CONDITION ASSESSMENT OF CONCRETE

The methods used in inspection to assess the condition of a concrete structure may vary from simple visual inspection to sampling and measurement. This chapter is mainly concerned with various kinds of destructive methods.

The following investigations may enter the picture.

- Visual inspection of free and bared concrete surfaces.
- Carbonation check.
- Visual inspection of drilled-out specimens.
- Determination of chloride content.
- Determination of resistance to frost and salt.
- Strength testing.

2.1 <u>Visual inspection</u>

Visual inspection reveals obvious deterioration such as frost scaling, severe reinforcement corrosion, wide cracks etc. Hidden deterioration and that in course of development may also be revealed through colour changes, leaching of lime etc.

In the experience of the Swedish National Road Administration free or bared concrete can be more closely assessed by attacking it with a chisel and small sledge hammer, in which case the following conclusions may be drawn by observation of rupture surfaces:

- If both cement paste and aggregate appear as ruptural surfaces then the concrete is "sound" and the chloride content is probably less than about 0.1 per cent.

- If ruptural surfaces are formed between paste and aggregate the bond is poor, which may indicate that the chloride content is higher than 0.3 per cent. White sediments are also a sign of a high chloride content.

In addition to this, the inspector has to note the depth to which the concrete is deteriorated, if the reinforcement has started to rust etc.

2.2 Carbonation check

Carbonation should be looked for in the first instance on concrete surfaces that are exposed to air. The carbonation depth is easily examined by spraying an indicator fluid consisting of phenolphthalein dissolved in ethanol onto a dry, fresh ruptural surface through the concrete. This surface may either be a drilled-out core which has been split or a surface that has been produced in the concrete by chipping. If the pH is 9 or more, the concrete is coloured red, whereas uncoloured concrete is considered to be carbonated. When the carbonated area extends as far as the reinforcement this may begin to rust.

2.3 Drilled-out cores

Drilled-out cores can be examined in many ways. Visual inspection, keeping a watch-out for porosity, surface structure, delaminations, cracks, reinforcement, etc. should always be carried out as it affords an important complement to measurements and tests.

2.3.1 Chloride content

The chloride content is determined through chemical analysis of disks sawn from the drilled-out core. These are taken at different levels, enabling a chloride profile to be drawn. The chloride content commonly refers to the total content of free chloride ions in relation to the weight of the cement. A Swedish standard, SS 137235, is available for determination of chloride content. This is similar to British Standard BS 1881, Part 6.

2.3.2 Frost resistance

Frost resistance can be measured in various ways. A common method is for a disk sawn from a drilled-out core, moistened with or submersed in water or salt solution, to be exposed to repeated cycles of freezing and thawing. The loss in weight due to frost scaling as a function of the number of cycles is then a measure of the frost resistance. In order for different tests to be comparable a high level of standardization is essential with regard to salt content, pre-storage time, freezing velocity, duration of freezing cycles etc. See [5].

2.3.3 Strength

The strength can be determined by compressive or splitting tests (tensile strength) of drilled-out cores. International standards are available for both these tests, namely ISO 4012 and ISO 4108 respectively.

Rules for assessment of strength in a finished structure are given in [6]. The implication of these rules is that the compressive strength measured in the structure corresponds to roughly 0.8 times the cube strength in accordance with the standard test procedure. This matter has thoroughly been investigated. See, for example, [7] and [8].

In the experience of the Swedish National Road Administration the relation between splitting (tensile) and compression strength is an important indication of the condition of the concrete. The splitting strength thus appears to be reduced more quickly than the compression strength if the concrete has been deteriorated by frost and salt. A low splitting strength in relation to the compression strength can therefore be sign of deteriorated concrete, even if the compression strength is satisfactory.

2.3.4 Composition

In addition to the investigations mentioned above, the composition of the concrete and its other properties can be determined in detail on drilled-out cores by means of the thin section technique. This involves looking at a concrete section with a thickness of 0.02 mm made by sawing and gradual grinding down of an epoxy-impregnated concrete specimen. A fluorescent substance is added to the epoxy and by studying the sample in a microscope with fluorescent or polarized light information can be obtained on the following:

- cement type
- water cement ratio (W/C)
- mixing (varying W/C)
- air pore system (size, shape, distribution)
- recrystallization
- bond between cement paste and aggregate
- carbonation
- cracks (micro and macro)
- freezing (in the construction stage or later)
- concrete age
- shape of sand grains
- possible alkaline-silica or alkaline-carbonate reactions
- petrographic composition of aggregate.

3. NON-DESTRUCTIVE CONDITION ASSESSMENT OF CONCRETE

Beside indirect strength measurements, condition assessment of concrete by non-destructive methods such as determination of concrete cover and potential mapping is a very useful complement to the destructive assessment techniques dealt with in chapter 2. Under the auspices of the Swedish National Road Administration research on different non-destructive methods for condition assessment of concrete has been carried out in collaboration with the Royal Institute of Technology, the Swedish National Testing Institue, the Lund University of Technology and the Swedish Cement and Concrete Research Institute. Results gained from both laboratory and in situ tests on thermography, impulse radar, ultrasonic testing, electropotential mapping and chloride content determination are so far promising. When using each of these techniques in its proper context, detection of frost scaling, severe reinforcement corrosion, wide cracks, voids, laminations and evaluation of strength is possible.

3.1 Indirect strength determination

Indirect, more or less non-destructive methods can be used to determine strength. In this case compressive strength is not measured and attention is instead concentrated on tensile strength, modulus of elasticity, hardness or some other property which is assumed to have a certain relationship to the compressive strength. These relationships are never unambiguous and consequently the methods will be more or less unreliable in comparison with direct pressure testing of drilled-out cores. In this context it should nevertheless be remembered that the standardized testing of cubes is not particularly reliable either when it comes to the strength of the finished structure. According to [9], different testing methods can be ranked in the following manner in view of accuracy, the figure in brackets indicating the standard deviation in MPa:

1. Testing of drilled-out cylinders	(1.6)
2. Pull-out testing	(3.3)
3. Combined NDT measurement	(3.9)
4. Rebound hammer measurement	(4.5)
5. Testing of standardized cubes	(5.7)
6. Ultrasonic measurement	(8.0)

3.1.1 Pull-out testing

Pull-out testing according to [10] and [11] implies that a cone is pulled out of the concrete by means of an embedded (Lok-test) or subsequently drilled-in bolt (Capo-test), whereupon the pull-out force largely depends on the tensile and indirectly the compressive strength of the concrete.

3.1.2 Combined NDT measurement

Combined NDT measurement (Non-Destructive Testing), usually implies a combination of ultrasonic measurement and rebound hammer testing. See paragraphs 3.1.3 and 3.1.4 below.

3.1.3 Rebound hammer testing

Rebound hammer testing according to [12] involves measuring the rebound of a weight which is thrown against the concrete surface with a certain energy. The rebound is a measure of the surface hardness of the concrete. The results may, however, be difficult to interpret since the rebound value will be too high if the concrete is deeply carbonated or if aggregate close to the surface is struck. See [13].

When determining a relationship between the rebound value and the strength of the concrete in question calibration is essential, either against cast test cubes or against the structure at a point where cylinders are drilled out for compression testing. This method is described in Swedish standard SS 137250.

3.1.4 Ultrasonic testing

Ultrasonic testing involves measurement of the velocity of sound propagation in the concrete, which in turn depends on the modulus of elasticity. See [14] and [15]. In combination with rebound hammer testing according to [9] reasonable accuracy can be obtained. For estimates of the compressive strength on the basis of rebound value and sound velocity a Swedish standard, SS 137252, is available. Concerning detection of deteriorated concrete by ultrasonic testing, see sub-chapter 3.7 below.

3.1.5 Other methods

Other methods for indirect determination of strength exist, such as the American Windsor probe, and the Norwegian TNS method. See [16] and [17], respectively.

3.2 Concrete cover determination

Covering concrete layer can be determined in a non-destructive manner with the aid of electro-magnetic detectors of which several different kinds are available on the market. These devices are known as pachometers. For location of reinforcement bars a simple metal detector will often suffice.

The pachometers do not always give satisfactory results in rather heavily reinforced members, such as bridges, as the effect of deeper steel cannot be eliminated. According to [18] parallell bars also influence the meter reading if their spacing is less than two or three times the depth of cover .

3.3 Potential mapping

When steel corrodes in concrete, a potential difference exists between the anodic half-cell areas and the cathodic half-cell areas on the steel. The potential of the corrosion half-cells can be measured by comparison with a standard reference cell, which has a known, constant value. A copper-copper sulphate cell (CSE) is normally used in field work because it is rugged, inexpensive, and reliable. The potential difference between the steel reinforcement and the reference cell is compared by connecting the two through a high-impedance voltmeter.

This is done by connecting one lead of the voltmeter to the reinforcing steel. The other lead is connected to the reference cell, enabling electrode potentials to be measured at any desired location by moving the half-cell over the concrete surface in an orderly manner. The cell can be used vertically downwards, horizontally, or vertically upwards provided that the copper sulphate solution is in contact with the porous plug and the copper rod in the cell at all times.

A full description of the equipment and test procedures has been published by the American Society for Testing and Materials (ASTM C 876). The interpretation of the potential measurements has been as follows:

- less negative than -0.20 V (CSE): greater than 90 % probability of no corrosion
- between -0.20 and -0.35 V (CSE): corrosion activity is uncertain
- more negative than -0.35 V (CSE): greater than 90 % probability that corrosion is occurring.

Measurements in both the field and in the laboratory have indicated that corrosion can occur at a potential of about -0.2 V (CSE).

An alternative approach to interpreting the data is to examine the potential gradients on a structure. While criteria have not been established, it is generally agreed that differences in potential of more than 50 mV are significant and differences of 100 mV are indicative of active corrosion.

It should be recognized, especially when working on prestressed structures, that if surface measurements are made, it is the potential of the mild reinforcement which is measured. If the potential of pretensioning strand is required, wells must be drilled so that the half-cell can be placed in close proximity to the strand.

Potential measurements cannot be made through post-tensioning ducts. If the potential of posttensioning steel is to be measured, the duct must be opened and the cell placed on the grout adjacent to the post-tensioning steel.

Results obtained in Sweden from performed potential mapping of bridge piers in a marine environment show, according to [19], that there seems to be an almost straight-linear relationship between the measured chloride ion content and the half-cell potential observed. See Fig. 1. The conclusion may perhaps be that potential mapping can be used as a method to determine the chloride ion content in situ. See sub-chapter 3.4 below.

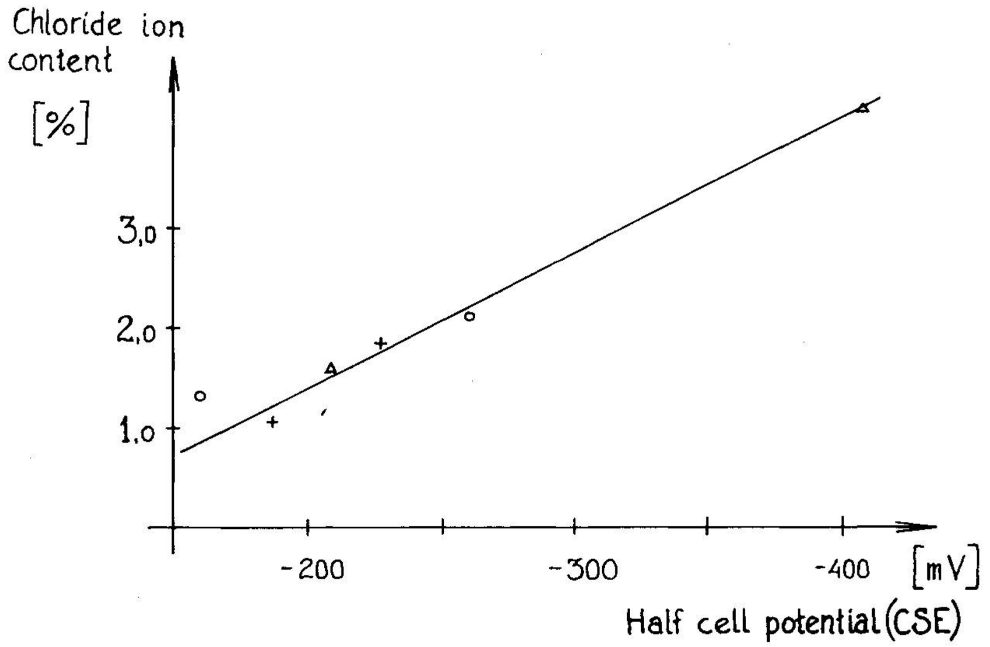


Fig.1 Observed half-cell potential as a function of the actual chloride ion content [19].

3.4 Chloride ion content tests

The chloride-ion content of concrete is usually measured in the laboratory using a wet chemical method of analysis. See paragraph 2.3.1 above. However, the method of sampling affects in situ operations. On behalf of the Swedish National Road Administration a literature survey has been done at the Swedish Cement and Concrete Research Institute [20]. This survey deals mainly with the possibilities of making chloride ion content determinations in situ. From this it can be concluded that there are two promising methods for this purpose. One is based on the concrete surface being sprayed with different salts, among them silver nitrate and potassium chromate. If there are no chloride ions present the brown silver chromate will be the result. The Danish "RCT Rapid Chloride Test" is based on this principle. The other promising method is based on a portable X-ray analyser for fluorescence measurements. Equipment named ASFX, developed on the basis of the latter technique mentioned, is now available in Sweden. The next research step of the Swedish National Road Administration will be to use both the RCT Rapid Chloride Test and the ASFX equipment in parallel for chloride content measurements in salt-contaminated bridges. In Finland there is also X-ray equipment called X-Met 840, which is similar to the ASFX equipment.

3.5 Impulse Radar

Investigations into the use of ground-penetrating radar for detecting deterioration in pavements and concrete bridge decks began in the United States and Canada in the mid 1970s. See [21] and [22]. The pulses are of extremely short duration, approximately one nanosecond, which is the technique known as impulse radar.

An impulse radar equipment normally consists of a monostatic antenna, transmitter, receiver and oscilloscope. Pulses of radio frequency energy are directed into the concrete, a portion is reflected from any interface, and the output is displayed on an oscilloscope. An interface is any discontinuity or differing dielectric, such as air to asphalt, asphalt to concrete, or cracks in concrete. See [23].

A number of American and Canadian studies have been carried out on both bare concrete and asphalt-covered bridge decks, see for instance [24] and [25]. In all cases the radar was found to be capable of identifying anomalous areas on the concrete surface. The practical problem was analyzing the large amount of data that were collected and relating the different radar signatures to physical distress. The use of impulse radar therefore has considerable potential for the automatic processing of the output signal to produce a site plan of a structure identifying the extent and type of deterioration. An important advantage of radar is that it is almost independent of weather conditions. The only environmental influence is that radar does not work effectively if the concrete surface is wet or if there is significant moisture in a bituminous or other surfacing because of attenuation of the signal.

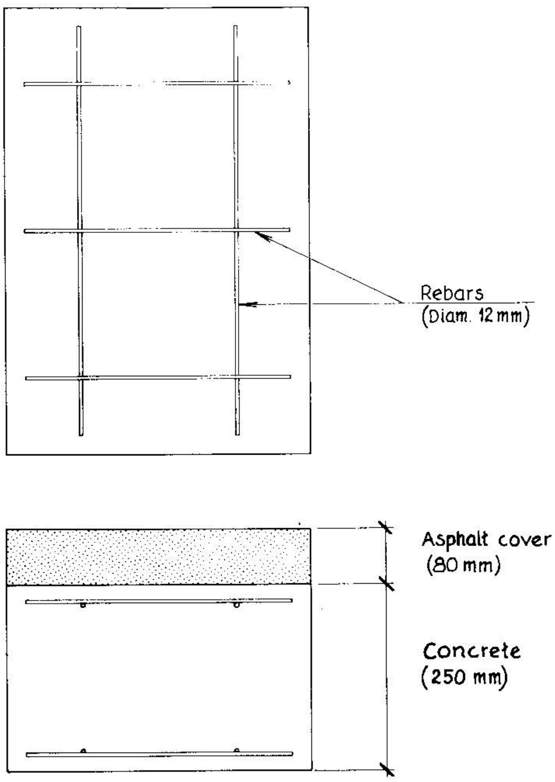


Fig.2 Test specimen with different chloride ion contents 0, 0.3, 0.6 and 1.2 %, respectively [26].

The research efforts on impulse radar in Sweden has been focussed on the detection of whether the concrete bridge deck underneath the asphalt cover is salt-contaminated or not causing different dielectric properties. On behalf of the Swedish National Road Administration tests have been carried out [26] on specimens, shown in Fig.2, with different salt contents. These tests showed that there was a significantly increased attenuation with increasing chloride content. The laboratory investigation was followed by in situ tests on two bridges, which confirmed that salt-contaminated concrete can be detected using the reflectivity of an impulse radar signal as a parameter.

3.6 Thermography

According to [27] and [28] infrared thermography has been found to be capable of detecting delaminations in concrete. The method works on the principle that as the concrete heats and cools, there is a substantial thermal gradient within the concrete because concrete is a poor conductor of heat. Any discontinuity, such as a delamination parallel to the surface, interrupts the heat transfer through the concrete. The differences in surface temperature can be measured using sensitive infrared detection systems. The essential components of such a system are an infrared scanner, control unit, battery pack and display screen.

The images can be recorded on photographic plates or videotape. Either colour or black-and-white images can be used. Ideal conditions are summer sun with no cloud or wind as the temperature differential is reduced by wind, cloud cover and high humidity.

The test specimens used at the impulse radar tests [26], which are shown in Fig.2 above, were also investigated on behalf of the Swedish National Road Administration by using thermography at the Swedish National Testing Institute [29]. It was then discovered that the different chloride contents could be recognized by the thermography technique when the temperature of the surrounding air dropped. This result shows that salt contamination causes a different emissivity of the concrete.

3.7 Ultrasonic Technique

3.7.1 State-of-the-art

Studies of literature dealing with ultrasonic technique reveal that testing of concrete has hitherto mostly been performed on the basis of determination of the time it takes for a sound pulse to pass through the material in direct or semidirect transmission. The various types of equipment available on the market are also designed in the first instance for this application. The transit time is commonly used to calculate the sound velocity in the concrete, as described in paragraph 3.1.4 above.

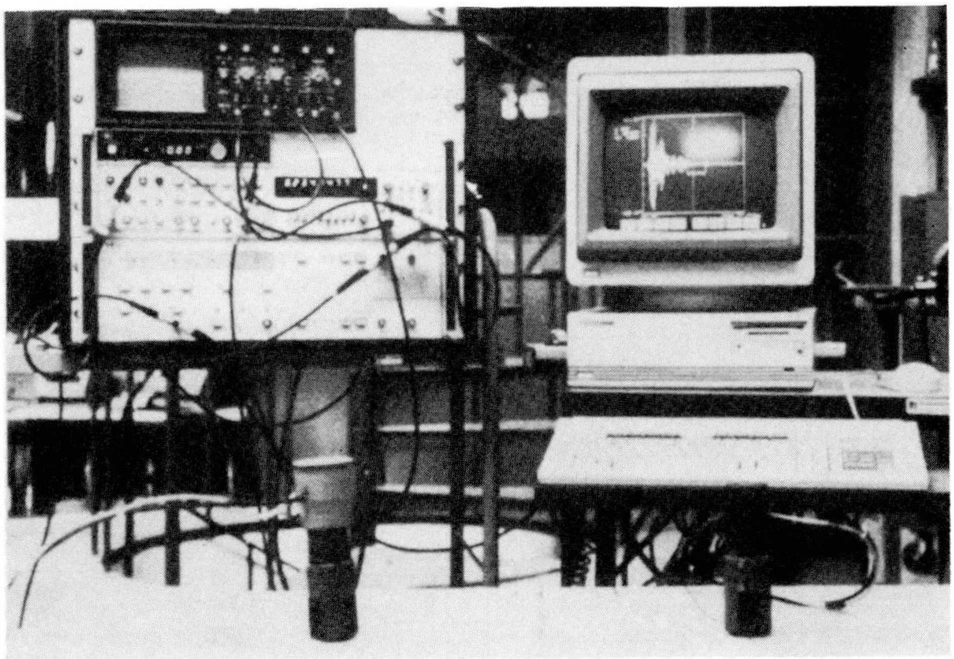
Applications for surveying of cracking and deterioration of concrete are also often described in user descriptions for different equipment and articles. On indirect transmission measurement along different distances, a difference in velocity can reveal the depth of a surface crack in the way of the pulse through purely mathematical calculations. Similarly, the thickness of a layer of poor concrete on sound concrete can be determined theoretically. These applications, in which only the transit time is considered, presuppose similar conditions throughout the entire measurement distance and thus give an average strength over the transit distance. Measurements of this type will also be influenced by embedded reinforcement bars if the pulse is permitted to pass in a direction parallel to a bar within the zone where the steel exerts effect. A correction factor must then be added to the measured sound velocity, as proposed in [30]. Measurement perpendicular to the reinforcement will also be influenced to some extent and consequently the velocity will also have to be slightly adjusted.

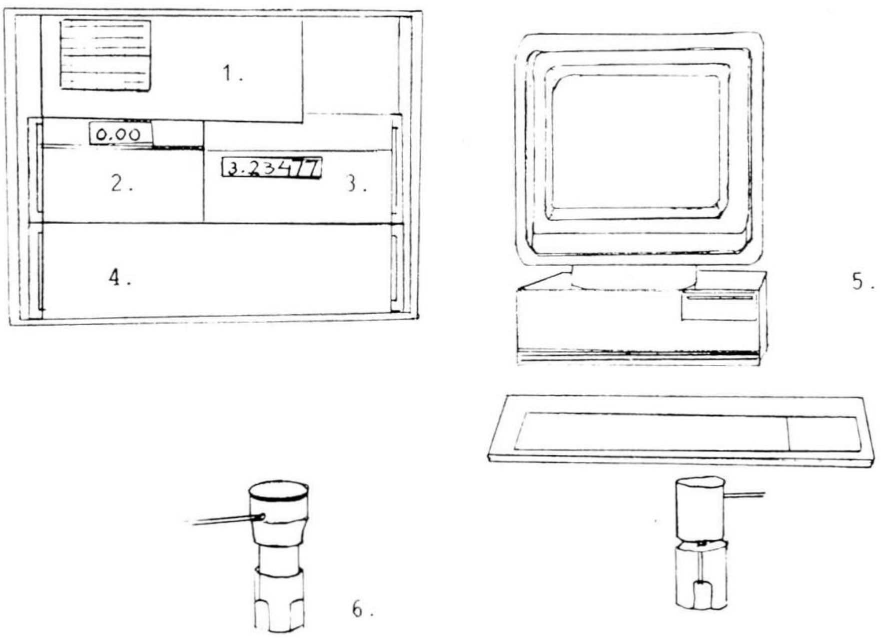
In recent years, new types of applications of ultrasonic testing of concrete have been studied and the findings presented in research reports. At the University of Braunschweig a doctoral thesis [31] has been presented, which deals with the possibilities of developing the thechnology in terms of both new measuring methods, such as intensity measurements, impulse echo measurements and ultrasonic spectroscopy and as regards the development of measuring equipment. At the University of London a thesis has been presented [32], in which the attenuation of the sound signal has been studied, in addition to velocity measurement, as a parameter for determination of strength.

It should be noted that measurements of pulse velocity through concrete are affected by the smoothness of the concrete surface, concrete temperature, moisture content, mix proportion, age of the concrete, and presence of reinforcing steel. Temperatures within the range of 5 to 30°C do not significantly affect pulse-velocity measurements. Outside this range, corrections can be applied according to [33].

3.7.2 Research and development

At the Royal Institute of Techology, Dep. of Structural Engineering, in Stockholm, Sweden, a research project was started in 1986, which is planned to be completed in 1988. This project is intended to further investigate and develop the ultrasonic technology in order to extend the possibilities of detecting deterioration of concrete. The ultrasonic technique will be used to detect damage due to frost attacks, such as laminations, cracks, etc. by determining not only the sound velocity but also





1. OSCILLOSCOPE - continuously displaying signals
2. FUNCTION GENERATER - generating various types of input signals
3. COUNTER - measuring transit time
4. AD-CONVERTER - digitizing received signal
5. COMPUTER - controlling and analysing measurements
6. PROBES - transmitting and receiving signals

Fig.3 The ultrasonic equipment used at the Royal Institute of Technology in Stockholm.

the attenuation of the signal. By exploiting modern digital technology, excellent opportunities are afforded for analysis of the received signal. In this research project the newly designed equipment shown in Fig.3 will be used.

As the goal is to establish relations between measured ultrasonic velocity and loss of energy, respectively, and the properties of the concrete being examined (strength, extent of damage etc.), certain fundamental parameters for the materials included in the concrete must be determined. The ultrasonic velocities and the energy absorptions of the constituent materials and their influence on the sound wave as reflecting and dispersing surfaces will be examined.

With the aim of determining the fundamental relations governing how ultrasonic velocity and attenuation are influenced by combined deterioration due to environmetal and functional effects, tests are performed on salt and frost damaged concrete cylinders. These experiments based on direct transmission are subsequently intended to be used to evaluate experiments with direct and indirect transmission on specimens of severely frost-damaged concrete.

The possibility of detecting laminations in concrete will be investigated by scanning of a boundary surface parallel and perpendicular to the detector surfaces of the transducers for different lamination thicknesses.

4. CONCLUDING REMARKS

In order to assess the condition of concrete structures, several methods are available, both destructive and non-destructive. Each of them have both advantages and disadvantages, as reviewed above in chapters 2 and 3. They are, however, complementary rather than competitive options, which by judicious selection offer rather good condition assessment opportunities for concrete structures.

REFERENCES

1. Durability of concrete structures. CEB-Rilem International Workshop, 18-20 May 1983, Department of Structural Engineering, Technical University of Denmark, Copenhagen.

2. INGVARSSON H., WESTERBERG B., 1985, Operation and Maintenance of Bridges and Other Bearing Structures. Publ. No. 42 from the Swedish Transport Research Board, Stockholm, Sweden, ISSN 0282-8022.

3. Swedish National Road Administration, 1985, Repair of Concrete Bridges. Publ. No. TB 151. (English version), Borlänge, Sweden.

4. Department of Transport, 1986, The Investigation and Repair of Concrete Highway Structures. Departmental Advice Note BA 23/86, London.

5. RILEM, 1977, Method of Carrying out and Reporting Freeze/thaw Tests of Concrete. Materials and Structures No 10.

6. National Swedish Committee on Concrete, 1979, Swedish Regulations for Concrete Structures (BBK 79), Svensk Byggtjänst, Stockholm.

7. LEWANDOWSKI R., 1971, Beurteilung von Bauwerksfestigkeiten an hand von Betongütewürfeln und Betonbohrproben, Schriftenreihe der Institut für Konstruktiven Ingenieurbau der TU Braunschweig, Heft 3, WernerVerlag, Düsseldorf.

8. PETERSONS N., 1973, Bedömning av betongs kvalitet i färdiga konstruktioner - några praktiska fall. The Swedish Cement and Concrete Research Institute (CBI), Utredningar nr 10, Stockholm.

9. BELLANDER U., 1977, Hållfasthet i färdig konstruktion. Del 3. Oförstörande metoder. Laboratorie- och fältförsök. The Swedish Cement and Concrete Research Institute (CBI), Forskning/research 3:77, Stockholm.

10. KRENCHEL H., 1970, Lokstyrkeprøvning af betong. Structural Research Laboratory. Technical University of Denmark, Sagsrapport nr S 3/69, 21 pp. Copenhagen.

11. KIERKEGAARD-HANSEN P., 1975, Lokstrength. Nordisk Betong no. 1975:3, pp 19-28. (Journal of the Nordic Concrete Federation).

12. GAEDE K. & SCHMIDT E., 1964, Rückprallprüfung von Beton mit dichten Gefüge. Deutscher Ausschuss für Stahlbeton, Heft 158, Berlin.

13. INGVARSSON H., 1979, Concrete Strength of a Slipform Concreted Structure, Contribution to the RILEM Symposium on Quality Control of Concrete Structures, Stockholm, June 17-21, 1979. Preprints Vol. 1, p 55-62.

14. EVANS E., 1960, The Effects of Curing Conditions on the Physical Properties of Concrete. PhD Thesis. University of London.

15. HANSEN T., 1960, Creep and Stress Relaxation of Concrete. Swedish Cement and Concrete Research Institute (CBI). Proceedings No. 31, Stockholm.

16. MALHOTRA V.M., 1970, Preliminary Evaluation of Windsor Probe Equipment for Estimating the Compressive Strength of Concrete. Mines Branch investigation report 1 R 71-1, 33 pp.

17. JOHANSEN R., 1977, En praktisk prøvningsmetod for in situ bestemmelse av byggverksfasthet. Nordisk Betong 1977:4. (Journal of the Nordic Concrete Federation).

18. MALHOTRA V.M., 1976, Testing Hardened Concrete - Nondestructive Methods, American Concrete Institute, Monograph No. 9, 188 pp.

19. INGVARSSON H., 1986, Svenska Vägverkets brotekniska forskning och utveckling rörande tillståndsbedömning, reparation och underhåll. Proc. from VTT Symposium 66 on Repair of Concrete Structures, 1986, Espoo, Finland.

20. ROMBEN L., 1986, Bestämning av klorid i betong med fältmetoder (Interimsrapport 1986-04-15), Swedish Cement and Concrete Research Institute (CBI), Stockholm.

21. CANTOR T. & KNEETER C., 1978, Radar and Acoustic Emission Applied to Study of Bridge Decks, Suspension Cables and Masonry Tunnel, Transportation Research Record No. 676, pp 27-32.

22. STEINWAY W.J., ECHARD J.D., LUKE C.M., 1981, Locating Voids Beneath Pavement Using Pulsed Electromagnetic Waves, NCHRP Report 237, Washington.

23. ALONGI A.V., CANTOR T.R., KNEETER C.P., ALONGI A. Jr., 1982, Concrete Evaluation by Radar, Theoretical Analysis, Transportation Research Record 853, pp 31-37.

24. MANNING D.G., HOLT F.B., 1983, Detecting Deterioration in Asphalt-Covered Bridge Decks, Transportation Research Record No. 899, pp 10-20.

25. CLEMENA G.G., 1983, Nondestructive Inspection of Overlaid Bridge Decks With Ground Penetrating Radar, Transportation Research Record No. 899, pp 21-32.

26. ULRIKSEN P., 1982, Application of Impulse Radar to Civil Engineering, Dep. of Engineering Geology, Lund University of Technology (Doctoral Thesis), Lund, Sweden.

27. CLEMENA G.G., McKEEL W.T. Jr., 1977, The Application of Infrared Thermography in the Detection of Delamination in Bridge Decks, Rep. No. VFHTRC-78-R27, Virginia Highway and Transportation Research Council, 35 pp.

28. MANNING D.G., HOLT F.B., 1980, Detecting Delamination in Concrete Bridge Decks, Concrete International, Vol.2, No. 11, pp 34-41.

29. BLOMQUIST N., NILSSON I., 1983, Användning av värmekamera för detektering av saltbemängd betong. Statens Provningsanstalt, laboratoriet för byggnadsfysik, rapport 8311:203 (Swedish National Testing Institute), Borås, Sweden.

30. CHUNG H.W., 1978, Effects of embedded steel bars upon ultrasonic testing of concrete. Magazine of concrete research, Vol 30, No. 102, March 1978.

31. HILLGER W., 1983, Verbesserungen und Erweiteriungen von Ultraschallpruferverfahren zur zerstörungsfreien Fehlstelle- und Qualitätskontrolle von Betongbauteilen. Dissertation TU Braunschweig.

32. CHEUNG L.C.F., 1977, Examination of Concrete by Ultrasonic pulse Attenuation and Velocity Measurements, Thesis, University of London.

33. JONES R., FACAOARU I., 1969, Recommendations for Testing Concrete by the Ultrasonic Pulse Method, Materials and Structures/Research and Testing, Vol. 2, No. 10, pp 275-284, Paris.

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Aspects of the Application of Structural Identification in Damage Evaluation

Problèmes d'application de l'identification structurelle dans l'évaluation des dommages

Ueber die Anwendung der strukturellen Identifikation für die Beurteilung von Strukturschäden

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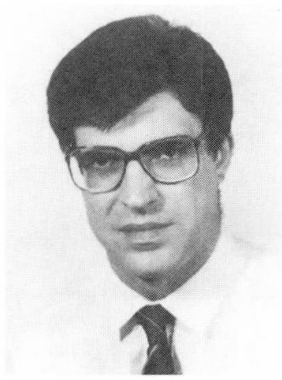

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SUMMARY
Many important features of the actual behaviour of a large civil engineering system can be revealed by experimental tests. In this paper the fundamental role of structural identification in processing the measured data to obtain a maximum amount of information about the state of a structure is discussed. The attention is focused on the aim of using response measurements to evaluate structural damage; changes of modal quantities are taken into account along with parametric physical model.

RESUME
De nombreuses caractéristiques du comportement réel d'un ouvrage important peuvent être connues au moyen d'essais. Dans ce mémoire on discute le rôle fondamental de l'identification structurelle dans l'utilisation des données expérimentales pour obtenir le maximum d'informations sur l'état de la structure. En particulier on se réfère au problème de l'évaluation du dommage structural et de sa distribution.

ZUSAMMENFASSUNG
Wichtige Eigenschaften des wirklichen Verhalten grösserer Strukturen können durch experimentelle Versuche ermittelt werden. Die vorliegende Arbeit erörtert die fundamentale Rolle, welche der strukturellen Identifikation bei der Auswertung von Messdaten im Hinblick auf die höchstmögliche Information über den Zustand der Strukturen zukommt. Besondere Beachtung findet ferner die Benutzung experimenteller Messungen zur Beurteilung von Strukturschäden.

1. INTRODUCTION

In recent times great advances have been made in the technology and the design procedures of structural systems; very important, complex structures have been built and many others are now under costruction.

Notwithstanding the great improvement in analytical techniques for predicting the response of structures and for assessing their safety, the need is increasingly felt for experimental knowledge of the real response of the structure, even limited to particular external conditions.

The results of experimental analysis can be utilized directly to compare specific response quantities - static or dynamic - with those ones analitically predicted and in some instances it is possible to assess the extent to which the real structure reflects the designed structure. But, in the case of large systems mainly, the conditions which can be analyzed are quite particular and limited in number.

Better use can be made of the experimental results by referring to a suitable interpretative model: system identification is the most correct and convenient way of relating experimental and analytical results [1, 2]. It is only quite recently that system identification has been applied to structural engineering. The aims of structural identification are several; models and techniques thus differ depending on the aims, though these have mainly been:

a) to obtain a mathematical model derived solely from experimental results [3];

b) to improve a structured mathematical model by adjustement of the prior values of its parameters [4].

Of late the role of structural identification has been extended to furnish information on the damage state of the structure. This new objective has emerged as a result of general interest now prevalent in evaluating the safety of existing buildings and in developing of a correct policy of periodical checks on structural integrity [5, 7].

Damage accumulates continuously in structural systems during their life under service loads and environmental conditions. In order to assure system safety and serviceability it is possible to plan for periodical experimental tests or continuous monitoring of meaningful dynamic response quantities so as to reveal the possible occurrence of damage and to quantify the extent.

The employment of a systematic and rational approach for processing experimental results is always recognized to be very useful, as in the following two extreme case:

a) when structures and/or environmental conditions are very complex and any damage must be detected indirectly, since a complete care inspection of all members cannot possibly be performed

b) when the loss of integrity of the structural elements is so evident that is can be ascertained by visual inspection and local damage can be described also quantitatively, but correlation with the global state of the structure is desirable and an updated model consistent with the new state has to be produced in order to redevelop the calculations under the design loadings.

This paper examines the methods for using experimentally measured data typically referred to in structural identification with a view to their possible application in assessing the damage. Various conditions are considered and attention is centred around methologies for evaluating structural damage by analyzing modal quantities as functions of time. The use of a parametric physical model is proposed as an interpretative 'robust' model to register changes in frequencies and eigenvectors; related quantities are selected to identify the site of damage in the structure and the extent thereof.

2. RESPONSE QUANTITIES AS DAMAGE INDICATORS

Various conditions can occur in which it is necessary to evaluate structural damage. In certain cases, in the absence of experimental information and visual inspection, knowing the intensity of external loading, the decrease in load carrying capacity can be predicted analytically by means of mechanical models derived from the theory of structures or by expert systems based on knowlegde of the behaviour of the class of structure concerned.

In other cases the damage is detected by visual inspection and it is possible to infer a measure of its extent by adopting different available tecniques which mainly provide information on the amount of repair work to restore the original integrity rather than furnish an evaluation of the loss of the load carrying capability as a property on its own.

As regards indicators for measuring damage if experimental results are available, the main important difference is bound up with whether these results are relevant to the phenomenon at the time when the damage occurs or whether they are relevant to periodical tests with damage occurring when the structure was not being monitored.

The first case is the typical when the structure is subjected to rare exceptional loading of intensity greater than the mean value expected, for example strong ground motion, which results in several members being stressed beyond the elastic limit. From the recorded acceleration responses it is possible to obtain reliable displacement histories and identify the hysteretic behaviour of a group of generalized one-dimensional systems. Knowledge of these response quantities permits quantification of the extent of the damage revealed by inelastic behaviour, if a measure criterium is established.

By referring to a hysteretic response, several damage functions have been introduced; these are based on maximum normalized deformation, stiffness degradation, cumulative inelastic deformation and dissipated energy. Most of these indices were selected mainly on the base of test results of isolated members or simple assemblages. Hence there are many uncertainties involved when they are applied to complete structural system. This is also due to the circumstance that only very limited data are available on entire structures which have experienced inelastic response under controlled conditions, such as monitored real structures subjected to violent external loadings, experiments on full-size structural systems in the real world or on structural models of various scales in the laboratory [8].

Different damage parameters and different approach must be used when there are no histories available for the time during which the damage occur or when the damage is associated with accumulated phenomena over the course of time - such as local overstress, materials decay, fatigue effects, modifications of boundary conditions - rather than with a particularly strong excitation. Information on the overall state of a structure can be obtained by using the change in modal quantities as the damage index. The effectiveness of such indexes has been discussed by many authors and it is now under study [8 - 12].

By comparing low-level vibration tests before and after the damaging event marked correlations has been observed between the change of structure period and the overall damage suffered during an earthquake [12]. Similar correlations are being sought for other structures, isolated elements, buildings, bridge and monuments as a result of various different damaging events. The correlation is less clear with respect to the earthquake case since damage is not distributed according to the loading pattern but is frequently restricted to just a few parts of the structure and more accurate experimentation is needed to detect it.

Other quantities to be considered in damage detection are the natural modes, which generally appear to be less effective than frequencies, as it is more expensive to measure them. However, by appropriate manipulation the modifications in the mechanical characteristics of the structure can be located and - to some extent - quantified. This is discussed in greater detail ahead.

The change in damping too can provide information on structural damage, but though a possible relationship between the latter and the observed index modifications has been sought the results have been unsatisfactory. This is due not least to the fact that for undamaged structures and low-level vibrations the damping factor varies with amplitude; moreover its evident deviation seems to be affected only by marked modifications in the structure.

The feeling today is that the mere use of modal parameters - mainly frequencies and modes - can furnish only general information on the damage state of the structure; the deviation indicate that damage has occurred but not its local extent or underlying causes thereof. Effectiveness could be improved through combined use of a careful mathematical approach and sophisticated experimental techniques that can reveal even small changes in modal parameters. Moreover the measurement of modal quantities - which has been developed considerably in mechanical and aeronautical engineering applications - can now be more easily performed for civil structures too because improved experimental equipment enables good results to be obtained even throug very low-level vibrations, as for example the forced response to environmental forces.

3. IDENTIFICATION OF PHYSICAL MODELS

A closer examination is now made of the more general case related to evaluation of the damage state of a structure by analysing changes in dynamic response; though the procedure is clear from a conceptual point of view several operative aspects need to be dealt with in greater detail.
Since comparison of the response of damaged and undamaged structure has to be developed through a mathematical model, the first step in the procedure is to establish a model of structural system in the virgin state. For civil system, particularly for large structures, the measured data are quite limited since the response in all the principal degrees-of-freeedom cannot be recorded; the same is true for the external forces applied. It would thus appear advisable to adopt a physical interpretative model, for instance a finite element model, which makes use of all prior information on the mechanical behaviour of the system, while uncertainties on some assumptions in the model description concern only the values of a number of physical parameters [13, 14]. The latter are determined in such a way as to minimize the difference between measured and predicted modal quantities.
Let h(x) be the function which relates the vector (nX1)x of parameters with the observed response quantities z. The following relation between z and x is assumed:

$$z = h(x) + n \tag{1}$$

where n n is vector noise, assumed to be stocastic gaussian with x_0 the mean value and covariance Ξ_n independent of x,
Due to the presence of errors both in the mathematical model and in the experimental data, there is little sense in forcing the model to match the data. It is more correct in this context to follow a Bayesian approach according to which the best estimate $\hat{x}$ of parameters x is that which maximizes the probability of occurrence of x given measured quantities $\bar{z}$. The value of $\hat{x}$ is furnished by the minimum of the function::

$$l(x) = [\bar{z}-h(x)]^T \Xi_n^{-1} [\bar{z}-h(x)] + (x-x_0)^T \Xi_n^{-1}(x-x_0) \tag{2}$$

which plays the role of the objective function in the problem. It is made up of two terms, the first takes into account the difference between measured z and predicted h(x) quantities weighted by the inverse of the covariance matrices Ξ_h^{-1}.

Since the response quantities depend nonlinearly on the parameters, the minimization of l(x) is sought by a numerical iterative procedure. For large structures this is not a simple task, because at every iteration step the direct eigenvalues problem has to be solved.

To reduce the amount of computational effort two different techniques have been developed to obtain an approximate relationship between modal quantities and the assumed parameters around the reference solution corresponding to the base values x^0 of the parameters.

The first technique is based on achieving an approximate solution by means of an interpolation quadratic function. Each component $h_i(x)$ is approximated by:

$$h_i(x)=h_i(x^0)+\sum_1^n k\ a_{ik}(x_k-x_k^0)+b_{ik}(x_k-x_k^0)^2 \qquad i = 1,2,\ldots, m \tag{3}$$

where n is the number of parameters and m the dimension of z. The coefficients a_{ik} and b_{ik} are determined by imposing the passage through three points, the solution furnished by the model for three values of the parameters, base x_0, upper x_u and lower x_l values [15].

The second technique is based on an asymptotic expansion of the solution in the Taylor series up to the second order; each component $h_i(x)$ is expressed as:

$$h_i(x) = h_i(x_0)+ \Xi\ \varepsilon_j\ h_{ij} + \Xi_j\ \Xi_l\ \varepsilon_j\ \varepsilon_l\ h_{ijl} + 0(\varepsilon^3) \tag{4}$$

where ε_j is the increment of the parameter x_j and the unknown coefficients h_{ij}, h_{ijl} are obtained by solving successive linear systems derived according to a perturbative scheme up to second order [16].

In both cases an approximated model is obtained, reliable in the description of the relation between response and parameters in the neighbourhood of assumed value x_0. The relation is expressed in closed-form which allows x to be minimized by making use of efficient minimization algorithms that require knowledge of the value of the objective function and of derivatives.

By substituding eq. (3) or (4) into eq. (2) to express h(x) and by denoting with

$$H_i = \left(\frac{\partial h(x)}{\partial x}\right)_{x=x_i} \tag{5}$$

the sensitivity matrix of the problem, the minimization of l(x) is obtained by a recoursive formula:

$$\hat{x}_{i+1} = \hat{x}_i+(\Xi_x^{-1} + H_i^T\ \Xi_n^{-1}\ H_i)^{-1}\ \{H_i^T\ \Xi_z^{-1}[z-h(\hat{x}_i)]+ \Xi_x^{-1}(x_0-\hat{x}_i)\} \tag{6}$$

derived according to a modified Gauss-Newton procedure.

4. USE OF PARAMETRIC MODELS IN DAMAGE EVALUATION

The parametric physical model, which has been identified to describe the behaviour of the structure in its original undamaged state, is taken as a reference to evaluate the modifications of structural properties, mainly, through an identification process following periodical tests [6].

In principle changes in all three modal quantities, frequencies, mode shapes and damping ratio can provide information on damage but actually only the first two appear really suitable for the purpose. If reliable data on changes in frequencies and in eigenvectors too are available, they can be useful for localizing the damage; it can be shown that for each mode u_r the distribution of elastic energy V can be written:

$$V_r = \frac{1}{2} u_t^T K u_r = \frac{1}{2} \sum_i \left(\sum_j u_{jr} K_{ji}\right) u_{ir} = \frac{1}{2} \sum_i \left(\sum_j k_{ij} u_{jr}\right) u_i = \frac{1}{2} \sum_i f_{ir} u_{ir} \quad (7)$$

where the contribution at each node i is indicated. Since the mass is assumed to be costant and it is frequently diagonal, any deviation in the elastic energy can be more conveniently ascertained from the kinetic energy, since $V_r = T_r$; it follows:

$$T_r = \frac{1}{2} \omega_r^2 u_r^T M_r u_r = \frac{1}{2} \omega_r^2 \Xi_i m_{ii} u_{ir}^2 \quad (8)$$

where, unlike eq.(7), the only unknown quantities are the measured data ω_r and u_{ir}. It can be argued from eq. (8) that the analysis of the variation of the kinetic energy distribution among different degrees-of-freedom will indicate where the stiffness decrease is concentrated. Theoretical and numerical applications would enlight the limits and effectiveness of this procedure.

Another problem is evaluation of the amount of damage; in this case it is necessary calculate the quantities which are assumed to be related to the damage. They could be individual coefficients of the stiffness matrix, various coefficients which affect the stiffness matrix of subsystems of the structure, and also some physical parameters which descrive the behaviour of a certain number of 'critical' elements.

This is a typical complex inverse problem which does not always have a unique solution; a priori information based on experience is necessary to provide some constraints for the problem. Within this context it is important to make a preliminary analytical study on the sensitivity analysis of meaningful parameters with respect to certain vibration properties which change significantly with damage. The approximate techniques previously outlined are very useful for performing a simple economic sensitivity analysis.

5. OPTIMAL CHOICE OF OBSERVED QUANTITIES

Structural identification of a structure is based on a number of observed response quantities which are selected by the investigator. The number is usually limited to minimize the cost of instrumentation, but for given figure the problem is that of fixing the optimal location for sensors so as to obtain the best estimate of the unknown parameters [17, 18].

It would be possible to solve this problem by trial and error techniques using different locations for sensor and adopting as the optimal solution that which gives the best parameters estimate. This procedure is quite time-consuming and the results are unsatisfactory in the absence of an efficient choice criterium.
Some of these difficulties can be overcome by adopting a method which does not call for the perform of any structural identification.

Let the measurable quantities of a structure be m, but only p<m are the observed quantities collected in the vector z. The relation between z and the parameters x is then written as generalized expression of eq. (1)

$$z = S[h(x)+n]$$

where mX1 vector h(x) furnishes all the measurable quantities predicted by the model while S is a (pXm) matrix which selects the p quantities z measured among the m ones observable. In the Bayesian approach the a-posteriory probability of x, given z, is:

$$p(x\ z)=\text{cost}\ e^{-\{[z-Sh(x)]^T S\ \Xi_n^{-1} S^T[z-Sh(x)]+(x-x_0)^T\ \Xi_x^{-1}(x-x_0)\}} \quad (9)$$

The estimated values $\hat{x}$ are such as to maximize the probability function (9) which, in the neighbourhood of $\hat{x}$, has a gaussian distribution with mean value $\hat{x}$ and covariance matrix:

$$\Xi = [H^T S^T (S\ \Xi_n\ S^T)^{-1} SH + \Xi_x^{-1}]^{-1} \quad (10)$$

where H is the sensitivity matrix defined by eq. (5) evaluated in $x=x_0$.
The choise of measured quantities is optimal when a suitable norm of Ξ is minimum. Since Ξ can be ill-conditioned, several numerical difficulties are avoided by considerering Ξ^{-1}. Therefore the solution is achieved by the condition:

$$\max_S \{H^T S^T (S\ \Xi_n\ S^T)^{-1} SH + \Xi_x^{-1}\} \quad (11)$$

By assuming the trace as a norm and Ξ_n as diagonal, then

$$\max_{g_i} \sum_{i=1}^{m} g_i \sum_{j=1}^{n} \left(\frac{\partial h_i}{\partial x_j}\right)^2 + \text{tr}(\Xi_x^{-1}) \quad (12)$$

where $\mathrm{diag}\,\{g_i\}=S^T(S\Xi_nS^T)^{-1}S$. (13)

The use of eq.(12) is very simple; each contribution of the m observable quantities to Ξ^{-1} is calculated and the set of p greatest elements is selected.

Since h(x) is nonlinear, the technique outlined above is only approximate and depends on the value ascribed to x for evaluating H according to eq.(5). It was observed that the results of the optimal choice are less influenced by x; in any case the method can be employed to select a few different solutions which are very useful to the investigator.

6. CONCLUSION

In order to have fairly reliable knowledge on the real behaviour of large structural systems it is suggested that experimental tests be performed, some just after the construction is finished and others during the service life. The purpose of the former is to check the validity of the mathematical model adopted in the design and to update this model for further investigations, while that of the latter is to detect degradation of mechanical characteristics of the structure. Within this general context the very important role of system identification is stressed in this paper. In particular attention is focused mainly on the use of parametric physical models as suitable filters for obtaining more specific information on the state of damage from the changes that occur in some quantities of the dynamic response.

Deviations in frequencies and modes are certainly the consequence of damage that has occurred elsewhere in the structure. A strongly interpretative model to a certain extent permits damage to be localized and the amount evaluated by analysing in which way deviations in response quantities can be explained by deviations in selected characteristics of the model. Two different methods of performing a simple, approximate sensitivity analysis are illustrated; these can be conveniently used to ascertained the response quantities - or the combination thereof - which are the most affected by damage. Finally, a priori criterium to define an optimal choise of the quantities to be measured - and as a consequence the location of sensors in the structure - is discussed.

ACKNOWLEDGEMENTS

This paper is part of a research on dynamic behaviour of monitored structures, supported in part by ENEA (Nuclear and Alternative Energies Committee) under Contract No.87/871

REFERENCE

1. EYKHOFF P., *System identification,* John Wiley & Sons, New York, 1974.

2. HART G.C., YAO J.P.T., System Identification in Structural Dynamics, Journal of the Engineering Mechanics Division, December 1977, pp. 1089-1103, vol. 103.
3. *International Modal Analysis Conference Proceedings,* 1982, 1984 and 1985, Union College, New York.
4. *Identification of Vibrating Structures,* edited by H.G. Natke, Springer-Verlag, Udine, 1982.
5. YAO J.P.T., Damage Assessment and Reliability Evaluation of Existing Structures, Eng. Struct., Vol.1, October 1979.
6. NATKE H.G., YAO J.P.T., System Identification Approach in Structural Damage Evaluation, Tech. Rep. No. CE-STR-86-21, School of Engineering, Purdue University.
7. NATKE H.G., YAO J.P.T., Research Topics in Structural Identification, Dynamic Response of Structures, A.S.C.E., March 31 April 2, 1986, PP 542-550.
8. STEPHENS J.E., YAO J.P.T., Survey of Available Structural Response Data for Damage Assesment, Tech. Rep. No. CE-STR-83-23, School of Engineering, Purdue University.
9. CASIRATI M., CASTOLDI A., PEZZOLI P., Dynamics Test for Diagnosis of Damage Caused by an Earthquake. IABSE Symp. Strengthening of Building Structures - Diagnosis and Therapy, Venice, September 1983.
10. FOUTCH D.A., HOUSNER G.W., Observed Changes in the Natural Periods of Vibration of a Nine Story Steel Frame Building. Proceedings 6th World Conference on Earthquake Engineering, New Delhi, India, January, 1977, Vol. 3, pp. 2698-2704
11. CHEN C.K., et al., Vibration Test of a 4-Story Concrete Structure, Proceedings 6th World Conference on Earthquake Engineering, New Delhi, India, January, 1977, Vol. 3, pp. 2753-2758.
12. OGAWA J., and ABE, Y., Structural Damage and Stiffness Degradation of Building Caused by Severe Earthquakes. Proceedings 7th World Conference on Earthquake Engineering, Instanbul, Turkey, September, 1980, Vol. 7, pp. 527-534.
13. NATKE H..G., COTTIN N., Updating Mathematical Models on the Basis of Vibration and Modal Test Results - A Review of Experience, 2nd International Symposium on Aeroelasticity and Structural Dynamics, Aachen 1985, DGLR-Bericht 85-02, 625-631.

14. CAPECCHI D., VESTRONI F., Problemi di Identificazione Parametrica nella Dinamica delle Strutture. Pubbl. No. 96, Ist. Scienza delle Costruzioni, L'Aquila, Febbraio 1986.
15. VESTRONI F., CAPECCHI D., Problemi di Identificazione nell'Analisi Dinamica di una Tubazione. Atti del 1° Congresso Ass. Ing. Offshore e Marina, Venezia, giugno 1986.
16. BENEDETTINI F., CAPECCHI D., A perturbation Technique in Sensitivity analysis of ,Elastic Structures. 1st Italian Conference on Computation Mechanics, Milano, 1985.
17. SHAH P.C., UDWADIA F.E., A Methodology for Optimal Sensor Locations for Identification of Dynamic Systems. ASME Journal of Applied Mechanics, Vol. 45, March 1978, pp. 188-196.
18. Sprandel J.K., Structural Parameter Identification of Member Characteristics in a Finite-Element Model; Ph.D. Thesis, School of Engineering, Purdue University, December 1979.

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Monitoring of Masonry Bridge Abutments

Contrôle de butées en maçonnerie des ponts

Überwachung von Flügelmauern bei Mauerwerksbrücken

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SUMMARY
The paper describes the application of time and frequency domain techniques to the monitoring and assessment of the quality of masonry bridge abutments. Time delay techniques using digital instrumentation are outlined. The application of these techniques to the assessment of masonry quality and void detection are described. Case studies are used to illustrate the techniques.

RESUME
L'article décrit l'application des techniques en fonction du temps et de la fréquence pour contrôler et évaluer la qualité des butées en maçonnerie des ponts. Les techniques de retardement employant une instrumentation numérique sont exposées à grands traits. L'application de ces techniques à l'évaluation de la qualité de maçonnerie et à la détection de vides est décrite. Des exemples concrets ont été choisis pour illustrer ces techniques.

ZUSAMMENFASSUNG
Der Artikel beschreibt die Anwendung von Methoden im Zeit- und Frequenzbe- reich, um die Qualität von Flügelmauern bei Mauerwerksbrücken zu überwachen und zu beurteilen. Zeitverzögerungsmethoden mit Digitalinstrumentation werden vorgestellt. Die Anwendung dieser Methoden auf die Beurteilung der Mauerwerksqualität und auf die Entdeckung von Hohlräumen werden beschrieben. Fallbeispiele dienen der weiteren Erklärung der angewendeten Techniken.

1. INTRODUCTION

In many parts of Europe up to 50% of a country's transportation infrastructure can comprise stone masonry bridges. In the U.K examples have been given [1] of regional authorities where in excess of 50% of the road bridges are stone masonry of approximately 100 years age. From this background it became clear to the NDT Bridge Research Group at Edinburgh University that cost effective investigation techniques are required prior to planning remedial measures.

The traditional method of investigating stone masonry bridges has been inspection by two means - visual inspection by a technician/engineer and coring by rotary core means. It is clear that rotary core drilling may permanently scar the structure and prove expensive due to the fact that scaffolding is almost inevitably required. In these circumstances the potential cost advantage of non-destructive testing is very substantial. The basic procedure would be that the structure could be rapidly assessed over its entirety prior to selective calibration of the quality of the structure by the conventional coring techniques.

When evaluating masonry bridges one of the key components of the structure is the abutments. Early papers [2,3] by the Edinburgh University NDT Research Group outlined a sonic transmission technique which was effectively used to obtain overall transmission velocities and to identify significant voids in masonry structures. More recently [4], work upon transient shock testing of masonry bridge abutments and piers has been reported.

The objective of the work reported herein is to present the findings of full-scale research upon masonry bridge abutments and other relevant structures using the techniques of time domain and frequency domain.

2. TIME DELAY TECHNIQUE

The scientific principles behind the sonic time delay technique were reported earlier [2]. The technique involves measuring the time from the initiation of a sonic pulse on one side of the structure to the transmission and reception of this pulse on the other side of the structure. The commencement of the propagation was measured by an accelerometer adjacent to the point of impact on one side of the structure. The reception of the signal on the other side of the structure was identified by another accelerometer at the other side - Fig. 1.

It was shown earlier [4] that the interpretation of results from such an investigation is basically that:

a) if no propagation is observed then voidage may well be present, or alternatively the structure may suffer leaf separation or discontinuity from a crack - assuming sufficient energy has been input to the structure to overcome signal attenuation/damping.

b) where propagation does exist the greater the velocity the higher the quality of the material and conversely the lower the velocity the poorer the quality of the material.

2.1 Developments in signal Processing: Time Domain

The early research [1,2 & 3] reported above employed an analogue storage oscilloscope with excitation using a conventional steel tipped hammer.

Permanent records were taken using a Polaroid oscilloscope camera. Whilst relatively effective, the technique is limited in terms of the potential for additional signal processing

The more recent work [4] has employed digital signal processing. Data was captured using a Nicolet 4094 digital oscilloscope with 12 bit A/D convertor and twin 360K disk drives. The analog signal recorded on the accelerometers following the hammer blow was digitised using the 12 bit convertor and the digitised signal then stored on a 5.25 inch floppy disk in one of the twin disk drives. The digital data was then available for subsequent detailed analysis in the laboratory. The Nicolet digital oscilloscope has a memory of 16K points and contains a 16 bit processor. The digital oscilloscope also features the powerful analytical tool of non destructive zoom of the trace.

In addition to using digital systems for data capture and analysis, a more sophisticated system of excitation has been used. An instrumented hammer containing a 2.5 tonne load cell with a frequency tuned plastic tip was employed.

3. TIME DOMAIN CASE STUDY: BARGOWER BRIDGE, SCOTLAND

In order to illustrate the power of the technique a full-scale investigation of Bargower Bridge, Ayrshire, Scotland is reported below.

The structure comprised a single 11 metre span masonry arch, which was to be load tested to failure by the Edinburgh University research group as part of the full-scale testing programme of the Transport and Road Research Laboratory, England.

The downstream elevation of the structure was marked out in a 1-metre grid and tested by the time delay technique as described in Section 2. above. The results of the transmission tests are summarised in Fig. 2, using the following coding - Table 2:

CODING	VELOCITY metre/sec	DESCRIPTION
A	>2000	VERY GOOD
B	1500 - 2000	GOOD
C	1000 - 1500	FAIR
D	500 - 1000	POOR
E	<500	VERY POOR
N	NO TRANSMISSION	

Table 1

The time delay technique can clearly be used to identify the shape of masonry abutments and springers by using the summary of data culled from a large number studies - see Section 4. below.

4. SUMMARY OF TRANSMISSION TEST RESULTS

From transmission test data compiled from extensive laboratory and full-scale testing the following table has been compiled based upon average velocities - Table 2.

MATERIAL	AVERAGE SONIC VELOCITY metres/sec	SOURCE	REFERENCE
Good Brickwork	3100	Birjandi et al, 1984	[3]
Poor Brickwork	2500 - 2700	Birjandi, 1986	[8]
Uncracked Reinforced Cavity	3500	Birjandi et al, 1984	[3]
Cracked Reinforced Cavity	2700 - 3000	Birjandi et al, 1984	[3]
Structural Concrete	>4500	Neville, 1975	[6]
Granite Masonry Pier No. 1	3450	Birjandi, 1986	[8]
Granite Masonry Pier No. 2	3370	Birjandi, 1986	[8]
Red Sandstone Masonry Pier	1970	Birjandi, 1986	[8]
Yellow Sandstone Masonry Pier	2040	Birjandi, 1986	[8]
Whinstone Masonry Pier	2500	Birjandi, 1986	[8]
White Sandstone Pier	1700	Birjandi, 1986	[8]
Steel - Rod	5100	Catchpool et al, 1949	[9]
Steel - Bulk	6100	Catchpool et al, 1949	[9]
Dry Sandy Top Soil	200 - 300	Clayton et al, 1982	[5]
Dry Sandy Clay	400 - 600	Clayton et al, 1982	[5]
Saturated Sandy Clay	1300 - 2400	Clayton et al, 1982	[5]
Water	1430 - 1680	Clayton et al, 1982	[5]
Limestone & Dolemite	4000 - 6000	Clayton et al, 1982	[5]

TABLE 2

5. FREQUENCY DOMAIN TECHNIQUE

This technique was used to test the face of the masonry abutments. The method of data collection involved mounting an accelerometer with built in charge amplifier on to the vertical surface of the abutment exposed above water level, followed by excitation of the structure using an instrumented hammer with a 2.5 tonne load cell. The consequent dynamic response of the abutment was then monitored by the accelerometer and transmitted to an FM high frequency tape recorder where the analog signal was recorded for subsequent analysis. The analysis of the recorded signals back in the laboratory involved playing the tape signals into a Bruel & Kjaer 2034 two channel dynamic signal analyser. The procedure involved converting the signal from analog to digital using a twelve bit A/D converter.

Basically two different analyses were undertaken upon the signals. The first analysis involved investigating the longitudinal vibrational response of the abutments (This analysis would have been undertaken until recently by mounting an electro dynamic shaker on the vertical surface and sweeping through a range of discrete frequencies using an exciter.) The analog signal recorded on the high frequency FM tape recorder was converted from the time domain to the frequency domain by undertaking a Fast Fourier Transform algorithm. The principle behind the analyses is that the intact thickness of the abutment is given by the expression below, based upon the physics of rods [7,8]

Intact thickness of abutment $L = Vm/(2x\Delta F)$.

Where L = intact thickness of abutment

Vm = velocity through the material

ΔF = the interval between resonant frequencies as indicated on the FFT frequency domain plot, after Davis and Dunn [7].

6. FREQUENCY DOMAIN CASE STUDY: HIGH BRIDGE, STRUIE, SCOTLAND

High Bridge, Struie, Highland Region, Scotland, comprised a single span stone masonry arch bridge.

The test procedure involved mounting an accelerometer on the surface of the abutment face and then exciting the structure adjacent to the accelerometer using an instrumented hammer with a built in 2.5 tonne load cell. The locations for the tests are given in Fig 3.

The method of assessing the data obtained from the West abutment, comprised analysing the signals recorded on the analog tape recorder. The force input from the hammer, and the response from the accelerometer were analysed using Fast Fourier Transform Algorithms on the 2 channel Bruel & Kjaer signal analyser with subsequent downloading onto a Hewlett-Packard 9816s micro computer for plotting on an ink spray graphics printer.

Before the results are summarised, consider the method of analysis outlined above, in relation to the West Abutment of High Bridge - Fig 4. In this case the fundamental frequency was identified as 220 Hz (i.e. cycles/second). The frequency interval F between resonances can be read off as 400 Hz. Thus assuming L = 3m, then:

L = 3 = Vm/(2 x ΔF) = Vm/(2 x 400)

therefore Vm = 3 x (2 x 400) = 2400 m/s.

The next step in the interpretation relates to value of the fundamental frequency ("A" in Fig. 4) in relation to the interval between harmonics (ΔF in Fig. 4):

If "A" = ΔF, then the structure is "FREE ENDED"

If "A" = ΔF/2, then the structure is "FIXED ENDED"

Thus from Fig. 4, A = 220 and ΔF = 400

i.e. A = ΔF/2 approx. Therefore at this location the abutment is fixed against the fill/rock of the valley sides.

In order to obtain an overall engineering interpretation of the data from the many sets of points taken on the West Abutment face, the data was averaged over the number of blows per point times the number of blows per horizontal row on the abutment face. Thus on Fig 3 assuming a sonic velocity of 2000 metres per second it will be seen that the thicknesses of the masonry analysed using the transient shock vibration method were as follows:

LEVEL	THICKNESS OF MASONRY (metres)	FIXITY
L1	0.85	Free
L2	3.00	Fixed
L3	3.40	Fixed
L4	2.70	Fixed
L5	2.00	Fixed
L6	1.90	Fixed
L7	2.20	Fixed

Table 3

7. CONCLUSIONS

Two non-destructive investigation techniques have been described for the investigation of masonry bridge abutments - time domain and frequency domain.

In the time domain, the time delay technique has been shown to be a powerful tool to rank velocities through masonry abutments. It was stated that this could be further used to distinguish the shape of masonry abutments, differentiating between continuous masonry and soil fill.

Where only one face of an abutment was available, the transient shock (or frequency domain method) was shown to be capable of determining the thickness and fixity of an abutment.

8. ACKNOWLEDGEMENTS

The authors acknowledge the provision of facilities and the ongoing encouragement of Professor A.W. Hendry, Department of Civil Engineering and Building Science, University of Edinburgh. Valuable discussions with Mr J Page, TRRL and Mr. A. Kirk of Blyth and Blyth, Inverness are gratefully acknowledged. The work has been undertaken with financial support from SERC grant number GR/D/09361.

REFERENCES

1. FORDE M.C. and BATCHELOR A.J., Low Frequency NDT Testing of Historic Structures, Proc. 3rd European Conference on NDT Testing, October 1984, Florence, Volume II, pp. 316-324.

2. FORDE M.C., KOMEYLI-BIRJANDI F. and A.J. BATCHELOR, Fault Detection in Stone Masonry Bridges by Non-Destructive Testing, Proc. 2nd International Conference "Structural Faults and Repair", I.C.E., London, May 1985, pp. 373-379.

3. KOMEYLI-BIRJANDI F, FORDE M C and H W WHITTINGTON, Sonic Investigation of Shear Failed Reinforced Brick Masonry, Masonry International, No. 3, November 1984

4. KOMEYLI-BIRJANDI F, FORDE M C and A J BATCHELOR, Sonic Analysis of Masonry Bridges, Proc STRUCTURAL FAULTS & REPAIR-87, University of London, 7-9 July 1987, pp 343-355.

5. CLAYTON C R I, SIMONS N E and M C MATTHEWS, Site Investigation, Granada, 1982

6. NEVILLE A M, Properties of Concrete, 2nd Ed.,Pitman, London, 1975,

7. DAVIS A.G. and C S DUNN, From Theory to Field Experience with the Non-Destructive Vibration Testing of Piles, Proc. I.C.E., Part 2, 57, 1974 pp. 571-593.

8. KOMEYLI-BIRJANDI F., Sonic Investigation of Masonry Structures, PhD Thesis, University of Edinburgh, 1986.

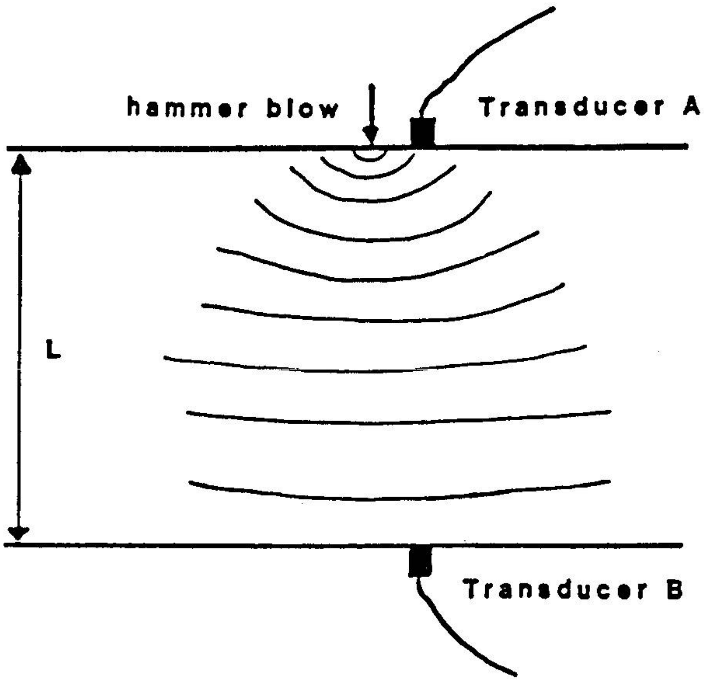

Fig. 1 Time Delay Technique

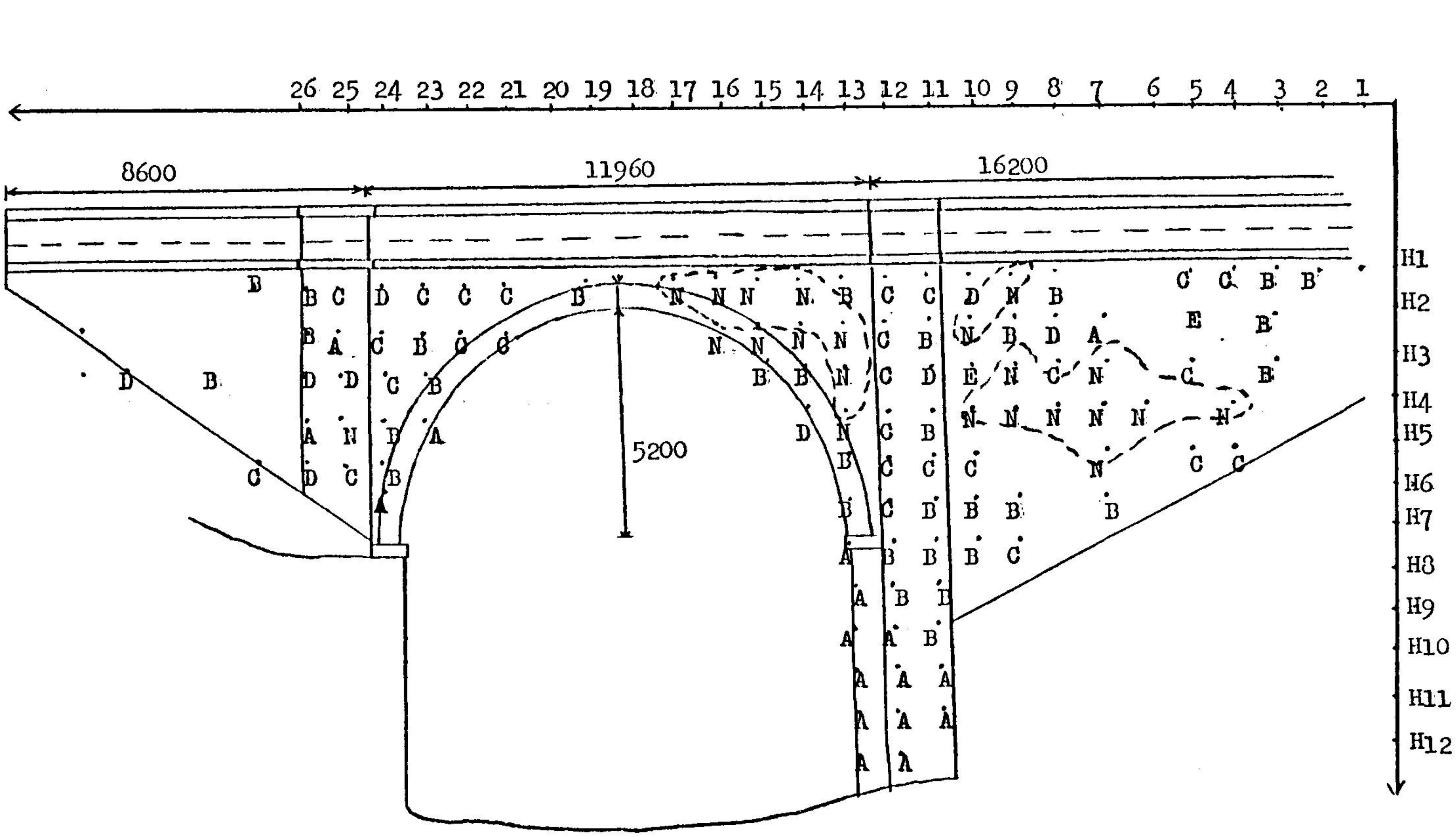

Fig. 2 Bargower Bridge, downstream face, transmission test results.

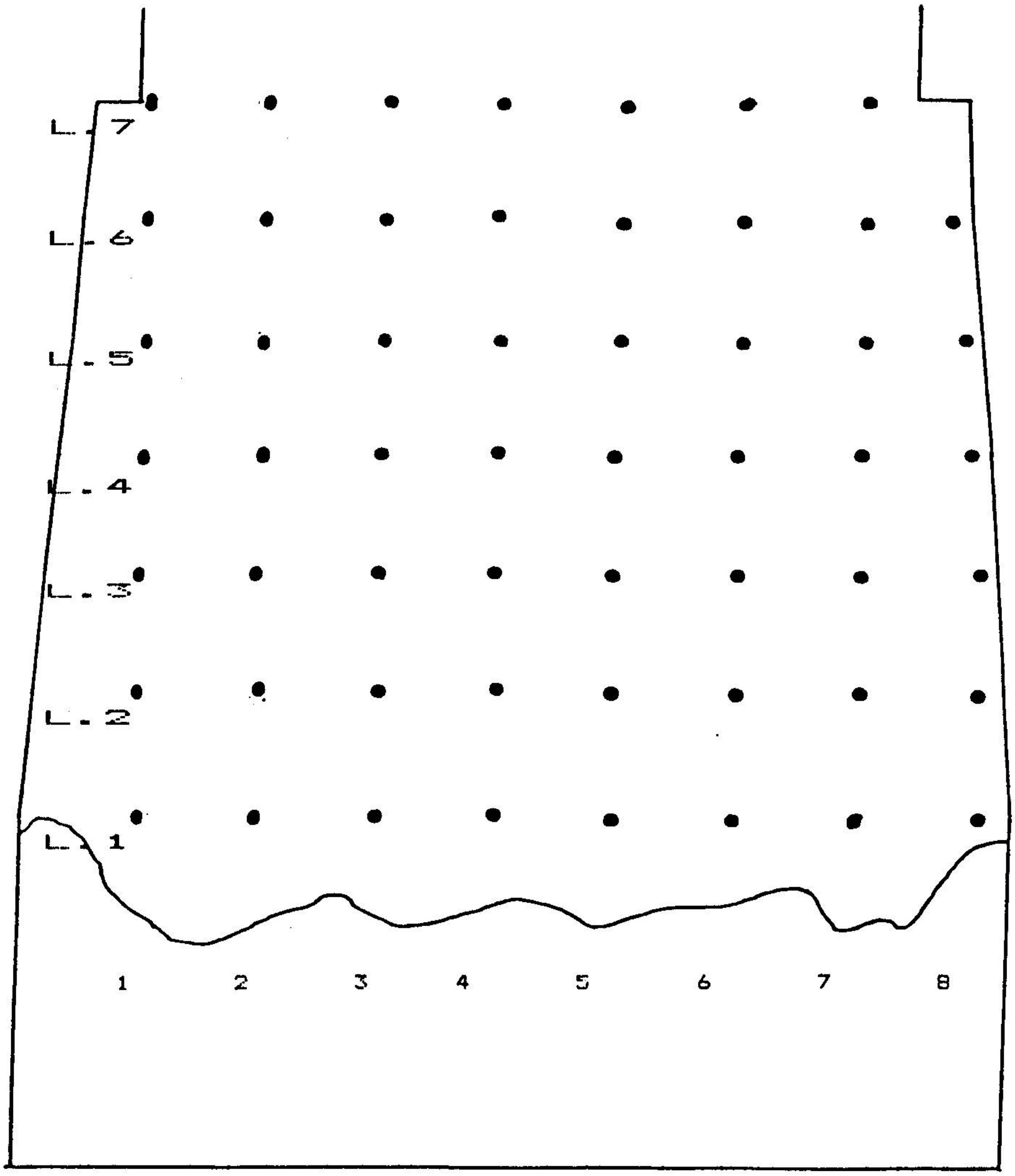

Fig. 3 High Bridge Struie, West Abutment - test locations.

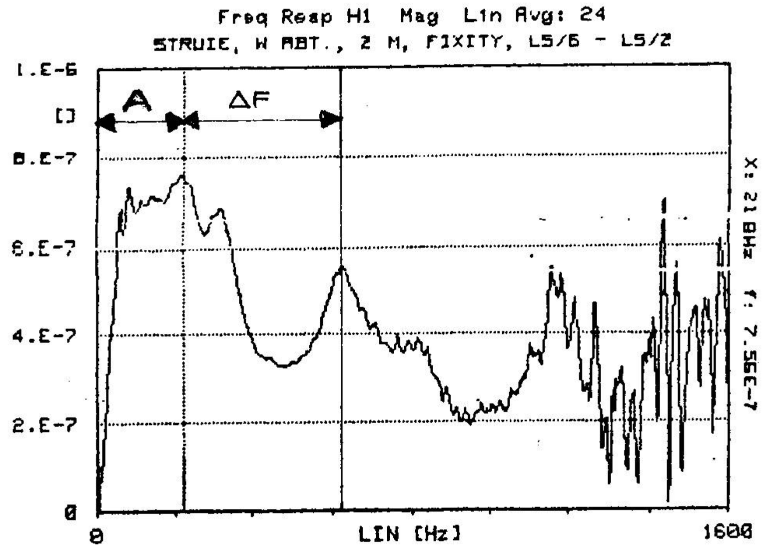

Fig. 4 High Bridge Struie, West Abutment - typical result.

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In-Service Inspections of Deep Water Structures

Inspection en service des structures en eau profonde

Inbetriebinspektionen von Hochseebauwerke

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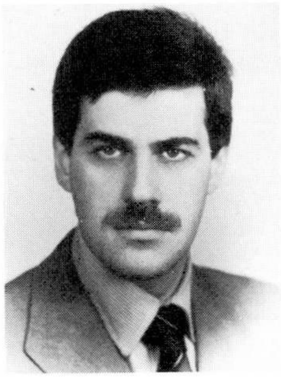

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SUMMARY
This paper deals with the analysis of the main problems which arise in the definition and management of the in-service inspection program for deep water structures. Present underwater inspection tools and techniques are briefly discussed and some applications to existing structures are reported.

RESUME
Ce mémoire traite les principaux problèms qui apparaissent lors de la définition du plan d'inspection pendant la vie des structures en eau profonde. On débat les techniques et les appareils d'inspection en eau profonde et on présente aussi des applications à des structures existantes.

ZUSAMMENFASSUNG
Dieser Vortrag behandelt die Analyse der Hauptprobleme, die bei der Definition und der Handhabung der Inbetriebinspektionspläne von Hochseebauwerken entstehen. Dazu werden heutige Unterwasserinspektionsgeräte und -techniken kurz erörtert und einige Anwendungen bei bestehenden Bauwerken beschrieben.

1. FOREWORD

The definition of an accurate and efficient in-service inspection program is an essential tool for the assessment of the structural integrity of an offshore installation and for assuring its reliable and cost effective operation at the highest possible level of safety to personnel and environment.

Since fixed offshore structures are designed for specific environmental and operational parameters, the inspection program is normally defined on a case by case basis, in order to comply with the specific requirements of governmental regulations and Classification Society guidelines.

The main philosophical aspects of these requirements are thoroughly discussed and reported in technical literature (/1/ ÷ /7/), but the actual trend to design structures for more severe environmental conditions and the recent development of inspection techniques require high priority, beginning with the description of the inspection program so as to include, in addition to sound engineering practise and judgement, all information derived from the design, fabrication and installation phases.

The need to incorporate these findings has necessitated the implementation and, if necessary, revision of the formulation of the R.I.NA. approach to in-service inspection in order to develop a more suitable and comprehensive program. The main philosophy of this new methodology can be summarised as follows:

1) the definition of the inspection plan should start at an early design stage. This leads to incorporation of the design parameters (stress level, fatigue life, stress concentrations, etc.) and selection of the critical items (most stressed areas, essential details, main mechanical components, if any) for inclusion in the inspection plan;

2) all "as built" drawings, survey reports, quality control documentation should be reviewed in order to define key items to be included in the inspection plan;

3) the scope of work of the inspection plan, its limits and applications should be highlighted before starting;

4) all NDT procedures and operators which are deemed suitable for the captioned structure should be previously defined; they should be qualified and certified for the specific tasks;

5) updating and re-evaluation of the inspection plan should be periodically carried out on the basis of the results of the previous inspections.

The main considerations and criteria from the design, fabrication, installation and in-service inspection phases, which affect the definition of a suitable and reliable in-service inspection program as well as the criteria for recording and evaluating the actual conditions of the structure at each stage of its life are discussed in this paper.

Present underwater inspection tools and techniques are also briefly discussed and some applications to existing structures are mentioned.

2. IN-SERVICE INSPECTION PROGRAM CONSIDERATIONS AND CRITERIA

In addition to the usual quality control and cost-effectiveness factors, the definition of a suitable in-service inspection program necessitates carefully taking into account at a conceptual design phase all the design, fabrication and installation considerations as well as findings from in-service inspections.

During the design activities, it is essential to thoroughly analyse all the design parameters, i.e. design load definition, material characteristic assessment, behaviour of the soil/structure interaction, efficiency of the cathodic protection system in order to identify the most significant areas and matters of the structure. This analysis results in:

a) the definition of the most critical nodes of the jacket and deck structure with respect to yielding, stability, punching and fatigue life;

b) the identification of zones affected by welding of padeyes, piping supports, seafastening, anodes and all other items which could reduce the fatigue life of the structure;

c) the identification of elements representing the foundations of cranes, drilling or production packages, bearings and hinges, if any.

In addition to the previous activities, during the design phase it is also advisable to establish a firm interaction between designers and operational personnel in order to develop the design, taking into careful consideration all the problems relevant to the inspectability of the structure.

This interaction is very important to enhance the reliability of both the structure and the inspection results and leads to a significant reduction of cost, since further costly procedures to correct abnormalities can be avoided.

The assessment of all fabrication procedures is another main influential factor in developing the in-service inspection plan. Since an early detection of significant defects is imperative to fabricate an adequate structure and considerably reduces the probability of failures during the life of the structure, quality control procedures are to be complied with in order to check material quality, weldings, tolerances and alignment of members.

During this "as built" phases, scantlings and dimensions, misalignment, welding discontinuities and any other deviation from specification are to be recorded in the inspection records. These records should be compared with the ones derived from the design phase in order to check possible overlaps of critical items in design and fabrication of the structure. This comparison is essential to enhance the selection process and to easure that all critical areas and items are included in the inspection program.

With regard to the installation phases, all the operations, such as load out, seafastening, towing, launching and final hook up operations are to be reviewed in order to ascertain that allowable limits are not exceeded.

During these phases suspected overstressing of members and possible damage which may influence the structural capability of the structure are to be noted in order to modify the frequency of inspections or to include, if necessary, other items in the inspection program.

3. SCOPE OF WORK OF THE INSPECTION PROGRAM

As already mentioned, the inspection program is based on information developed from the design, fabrication and installation phases and may vary from case to case, depending on many factors, such as type of structure, criteria and frequency of inspections, continuous monitoring, maintenance activities and so on. In general, the inspection program consists of the following main points:

a) specification of inspection items and extent;

b) definition of inspection types for each inspection item;

c) means, methods and equipment for each inspection item;

d) frequency and time schedule of inspection;

e) definition of actions to be taken in the case of significant findings or deviations from the inspection program;

f) procedures for reporting, filing and updating.

The definition of items and the relevant type, extent and programs of inspections is derived from the considerations discussed in the previous chapter. In particular, on the basis of different criteria (yield, fatigue, corrosion, consequences of failure), the structural elements (joints, members, components) are to be divided into different classes. For each class, the frequency, extent and inspection procedure and methods are defined in a rational way. This criteria leads to frequent and close inspection by means of one NDT examination after cleaning highly stressed members or members whose failure directly leads to loss of life and/or structure. For less stressed members, whose failure leads to slight local or secondary damage, only a visual inspection is foreseen.

In any case, the task of these structural surveys is to check for possible major damage (bent, severed or missing members) as well as to detect fatigue cracks at member-ends by a close inspection of weld and parent metal areas.

In addition, thickness measurements should also be carried out at selected points in order to check for any corrosion and pitting.

Reports of this inspection should define the location of any damage and the relevant extent, deflection and out-of-roundness of tubular member and any other significant concern. Photography video documentation is advisable for analysis and further review.

With regards to means, methods and equipment, it is to be noted that inspection of underwater structures is to be carried out by qualified personnel in order to comply with a standard similar to that required for above water inspections. The use of telecameras, photocameras and remotely operated vehicles is normally accepted on a case by case basis according to procedures previously agreed upon. The following fire underwater inspection techniques are generally used:

1) visual inspection (obvious damage detection)

2) magnetic particle inspection (surface crack and pit detection)

3) ultrasonic inspection (thickness and flaw detection)

4) radiography (internal defect detection)

5) corrosion-potential measurements.

Variations of these techniques include a magnetographic method of crack measurement and an açoustic holographic technique for internal flaw detection.

The application range of these methods and the relevant advantages and limitations are discussed below. It is only to be noted that, combining the quality assurance procedures for personnel and equipment, the validity and reliability of the results of the inspections are well within the acceptable range.

4. VALIDITY OF UNDERWATER INSPECTION TOOLS

As previously outlined, testing of offshore structures is carried out by means of NDT methods which are normally used to test land structures (/8/÷/10).

As in only a few cases could the particular working conditions be taken into account by making minor modifications or adjustments, completely new equipment was developed and manufactured for underwater applications. The aim was to allow testing to be performed without the need of skilled diver, as the satisfactory completion of the inspection was dependent on his ability.

One of the main aspects of in-service NDT is to compare results from previous inspections or from construction, therefore all NDT are to be performed in accordance with a detailed procedure which defines the requirements to be met with regard to:

a) degree of surface cleaning before NDT;

b) type and characteristics of equipment to be used;

c) diver NDT qualification for the specific task;

d) sequence of all operations;

e) acceptance criteria of findings;

f) report form to be filled in.

An outline of various NDT methods and their relevant advantages and limitations is presented below.

4.1 Visual inspection

The visual inspection is the most popular NDT method and is generally performed by divers using TV cameras or, where possible, Remote Operated Vehicle (ROVs).

The choice between divers and automated vehicles is mainly economical and also depends on the water depth and the extent of the survey. Photographic surveys, using 70 mm film, are required when a high resolution degree is required. When good lighting conditions can be expected, colour films are preferred, otherwise b/w films are used.

For detailed dimensional measurements, photogrammetry is used.

4.2 Magnetic particle inspection

Magnetic particle inspection is frequently used for the detection of major external defects, such as fatigue or stress corrosion cracks /11/.

Various methods of magnetization may be used, generally fluorescent powder on liquid suspension and wood lamp. Particular attention is to be paid to the choice of TV cameras and to the suitable filters to be used for recording purposes.

Recently, a lot of semi-automatic equipment has been made available. Much of it was tested by the authors and the results were particurlaly satisfactory when the true dimensions of the detected defects were recorded on magnetic tapes.

4.3 Ultrasonic testing

Ultrasonic testing is mainly used for thickness measurements and corrosion detection. Ultrasoning testing is widely employed for the detection of internal defects, particularly the ones initiating inside the structure.

For thickness measurements, both manual and automatic equipment is used.

Manual equipment, generally of the digital reading type, is available with built-in memories up to 1000 readings.

A probe in contact with the structure, can be kept on board the supply vessel. It is mounted on a track and moves according to prefixed positions. Its readings are sent to a computer which plots, in real time, the measured thickness.

Detection of internal defects, particularly for structural nodes, is mainly carried out by manual equipment.

Many solutions were studied allow the survey to be carried out from a supply vessel, giving the experts on board the chance to verify the actual results.

Generally, this equipment relies on the principle that the information on the diver's screen is exactly the same as that on the supply vessel.

Ultrasonic testing is appropriate for the detection of water level inside a structure suspected of through cracks.

4.4 X and γ Rays

This type of testing is not commonly used, being generally performed on double wall technique. When possible, X Ray Ir 192 is preferred for safety reasons.

The source is put in watertight envelopes and is kept at any suitable distance from the structure to be inspected, avoiding any contact with water by using gas or air chambers.

4.5 Acoustic emission testing

This type of testing is very promising as it enables the propagation of possible existing defects to be detected. Its main limitation is the high number of probes needed to keep a complete structure under survey.

4.6 Impressed vibrations

This type of testing is only applicable when cracks in some components of the structure are so important as to give rise to characteristic vibrations, easily detectable by a number of accelerometers suitably placed in the structure.

5. APPLICATION TO EXISTING STRUCTURES

As previously mentioned, R.I.NA. has tested many types of NDT methods for underwater applications.

In many cases, NDT techniques were developed together with the manufacturers of the NDT equipment and the results satisfy the technical need.

It is interesting to note that several of the NDT methods already discussed are generally applied to in-service inspection of offshore structures.

Structures in water depths ranging from a few meters to 100 meters were inspected and from the experience gained, no difficulties are expected in the case of deeper water.

Moreover, taking into account that the reliability of divers decreases when NDT is carried out in increasing water depth under saturated conditions, the tendency for future developments can be summarized as follows:

- visual examination: manual submersibles are to be used as far as possible, with the assistance of divers for specific surveys limited in time and extent;
- magnetic particle inspection: is preferable as it gives the chance to use divers who are not necessarily MPI experts as well as evaluation and referring equipment placed on the surface;
- ultrasonic inspection: computer assisted automated equipment of the C or P scan type, is to be used as far as possible, as this solution does not require particularly expert NDT divers;
- Eddy current testing: at the moment, multifrequency equipment which has particular probes is being studied.

6. CONCLUDING REMARKS

The development of an inspection program that yields a reliable assessment of the integrity of an offshore structure requires that:

a) the development should start at the design stage;

b) all the survey report of each phase should be reviewed for inclusion in the inspection plan;

c) the scope of work of the inspection plan should be carefully defined at an early stage;

d) all NDT procedures and operators should be qualified for the designated tasks;

e) re-evaluation of the inspection plan should be periodically carried out on the basis of previous inspection findings.

The initiation of these items should satisfactorily achieve the required structural integrity and assure a reliable and cost effective operation of the structure.

The used underwater NDT methods do not greatly differ from the ones applied to land installations.

The reliability of NDT methods depends on the degree of clearness of surfaces to be inspected and on human performance, which turns out to be strictly depending on environmental conditions. This results in a tendency to study and develop automatic equipment, which allows results to be more reliable and repeatable, and at the end, less expensive.

REFERENCES

1. BALESTRINO P.L., SEMBER W.J., WITLOCK L.T., In-service Surveys of Offshore Installations. 1st AIOM Congress, Venice, June 1986.
2. SOLLIE T., Safety and Quality Assurance of Fixed Offshore Installations in-service - Experiences and Prospects after some 10 years of Involvement. 6th European Maintenance Congress, Oslo, June 1982.
3. MUIR G., Certification for In-water Survey and Inspections. British Journal of NDT, September 1979.
4. VADUS J.R., Underwater Inspection for Deep Ocean Platforms. Ocean Science and Engineering, 9(3), 253-263 (1984).
5. HUGHES D.M., BECKSTED J., HESS T., Underwater Inspection and Repair of Offshore Structures. Paper No. 2378, OTC Conference 1975, Houston, May 1975.
6. FORLI O., Methods for Inspection and Monitoring - Offshore In-service Inspection and Monitoring - A Basis for Efficient Operation and Maintenance. Veritas Conference, November 1979.
7. GUIDUGLI G., PAPPONETTI M., LO SAVIO F., SONSON E., In-service Inspection by Ultrasonics in Deep Waters of Butt and Fillet Welds. 6th European

Maintenance Congress, Oslo, June 1982.

8. ANONYMOUS, Underwater NDT Equipment Evaluation, Offshore Research Focus, N. 26 August 1982.

9. PAPPONETTI M., NDT on Offshore Structures Particularly on Deep Waters, XIII National Congress on NDT organized by the Italian Association of Metallurgy, Milan, May 1983.

10. PAPPONETTI M., NDT of Offshore Structures During the Fabrication and Service, IV National Congress of the Italian Society of NDT, Bari, October 1985.

11. LUMB R.F., WINSMIP P., Magnetic Particle Crack Detection Metal Construction. Vol. 9, July and August 1979.

12. BEGG R.D., MACKENZIE A.C., Monitoring of Offshore Structures Using Vibration Analysis, International Symposium on Offshore Structures, Glasgow, April 1978.

Acquisition of Data from Single Layer Armour Units in Breakwaters Using Radio Telemetry

Acquisition des données expérimentales par radio-télémesures provenant d'unités de blindage a simple couche de brise-lames

Messdatenfernerwerbung von einschichtigen Wellenbrecherblendungseinheiten

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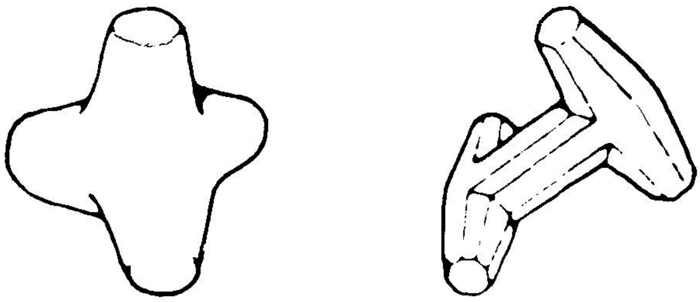
SUMMARY
Armouring units are used to face rubble mound breakwaters and to dissipate the energy of incoming waves. Special types of single layer units have been developed which derive their strength from being placed tightly together in a regular matrix. A major research project has been established to assess the strength of these units in service and to develop design methods for improving their hydraulic and structural performance. A trial installation is described which was used to assess the feasibility of transmitting data both by cable and by radio telemetry through the spray environment.

RESUME
Des unités de blindage sont utilisées en face de brise-lames à blocs pour dissiper l'energie des vagues arrivantes. On a mis au point des unités spéciales à simple couche et de grande résistence grâce à un assemblage très serré en matrice régulière. Un important project de recherches a été défini pour évaluer la résistance en service de ces unités et pour établir des méthodes de dimensionement pour améliorer leurs performances hydraulique et structurale. On décrit une installation d'essai qui a été utilisée pour apprécier la practicabilité de la transmission des mesures soit par câble, soit par radio-télémesures à travers l'environnement des jets d'eau.

ZUSAMMENFASSUNG
Blendugseinheiten werden gegenüber von Klotzwellen brechern verwendet um die Wellenkraft zu gestreuen.
Besondere einschichtige Blendugseinheiten wurden entworfen, deren Festigkeit durch eine enge matritzenregelmässige Zusammenstellung geleistet ist.
Es wurde ein erheblicher Forschungsplan aufgestellt, um die Gebrauchs wiederstandsfähigkeit dieser Einheiten zu schätzen und Bemessungsverfahren zum Verbessern deren Fluss- und Strukturleistungen zu entwerfen. Diese Artikle beschreibt eine Probeeinrichtung und die Messdatenübertragungsfähigkeit entweder mittels Kabeln oder radiofernmessung zu bewerten.

1. INTRODUCTION

Rubble mound breakwaters and similar coastal structures are normally protected from the worst effects of wave action by rock or concrete armour. Rock armour is used where available in sufficient sizes, quantity and quality. Conventional concrete armour units include the Tetrapode, Dolos, Stabit, Fig. 1, although many other types have been suggested and sometimes used. They are generally laid randomly or irregularly in two layers, with variable attitude and interlock between the units, giving a mean armour layer porosity of 30-45%. Since the units rely on self weight for their stability, they are generally very large varying in size from 5 to 90 tonnes.



(a) Tetrapode (b) Dolos (c) Stabit

Fig. 1 Various conventional armour units.

Recent failures of a number of major breakwaters and subsequent research has identified problems in the design and use of irregularly-placed concrete armour units of these types. Under wave action, small rocking movements occur which lead to collisions between adjacent units. If the consequent impact and bending stresses exceed the tensile strength of the concrete, cracking occurs which may lead eventually to complete fracture.

Recent developments in the UK have produced a family of single layer armour units which derive their strength and stability from being placed tightly together in a regular matrix. Such units, which are much smaller and lighter than conventional units, employ porosity within rather than between them to dissipate wave energy. The first single layer armour unit to be developed was the Cob, Fig. 2(a), designed by Coode and Partners. The Cob unit is essentially cubic, with square apertures in each side meeting in a cubic central void. It provides a layer porosity of 55-60%. Between 1973 and 1975 approximately 8700 2t Cobs were deployed on the new La Collette breakwater at St. Helier, Jersey [1,2]. Further Cobs were used on Das Island in the Arabian Gulf.

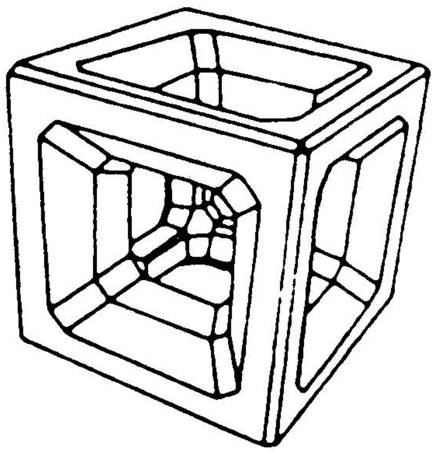


Fig. 2(a) Cob armour units.

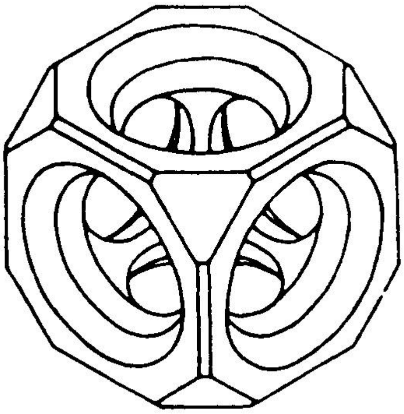


Fig. 2(b) SHED armour units.

In 1981, Shephard Hill Ltd. developed an alternative hollow-cube unit, the SHED, Fig. 2(b), with a similar level of mean layer porosity. This unit embodies a spherical central void within an essentially cubic external form. Approximately 4200 2t SHEDS were positioned on the new Albert Pier sea wall in Jersey during 1981-83. Some 2700 similar SHEDS, were also used in the new breakwater at Bangor, N. Ireland, and a further 3700 in a new breakwater near Limassol, Cyprus. They have also been used in a sea wall in Oman.

Single layer armour units of these types display considerable advantages over conventional, two layer, randomly-interlocked units, such as Dolos or Tetrapode. Their good hydraulic performance, high resistance to movement under wave action, coupled with low unit weight, afford very considerable cost savings. The world-wide market for advanced coastal engineering techniques is enormous. Potential savings to owners and contractors from the use of hollow cube armour units on breakwaters, sea walls, etc. are thus likely to be significant. However, a number of the early installations employing these units are already showing distress. Clients must, therefore, be satisfied that these much smaller units are sufficiently robust for the purpose for which they are intended.

Design procedures for assessing the hydraulic performance and survivability of such units are based almost universally upon the use of scale model tests. However, it is only very recently that any of these tests have used armour units of correctly scaled strength, and even now this is only possible in very special circumstances. Current design manuals are all based on such testing, few giving any guidance on loads and stresses induced and, hence, the strength required for reliable service over the design life of the structure. The major problem then for the designer is that of assessing loads, and hence the stresses induced, in the generally unreinforced units. This is an area of active research world-wide. It should be noted that all such research is presently concentrated upon the conventional two layer units. The problem of assessing in-service stresses is particularly acute for the single layer hollow cube units due to their comparative fragility.

2. OVERALL PROJECT OBJECTIVES

The overall aim of this research project is to develop a design method to determine the size, shape and strength of single layer armour units. This will enable engineers to design and construct armoured rubble structures using regularly-placed single layer units at significant savings over conventionally armoured structures. The overall project objectives are being investigated by a widely-based multi-disciplinary team comprising specialists in hydraulics, concrete structures, materials, stress analysis, structural models and field instrumentation. The main areas of investigation may be considered under three headings:

(i) assessment of hydrodynamic flow conditions within the units;
(ii) assessment of loading and structural performance; and
(iii) stress analyses of single units and entire breakwaters.

This Paper relates to some initial field work under heading (ii) above only.

3. ASSESSMENT OF LOADING AND STRUCTURAL PERFORMANCE

The assessment of wave- and settlement-induced loadings will require work in both the hydraulics and concrete laboratories, and in the field. Measurements of wave-induced loads are to be made in model tests using suitably strain-gauged model armour units. Such measurements will, however, require very careful calibration with strain measurements from full-scale units loaded under controlled conditions in the laboratory. The assessment of actual loads caused by the effects of construction, transportation, installation, settlement and wave loading, determined from field trials, will also be necessary.

3.1 Field measurements

The intention is to instrument fully six typical armour units for installation in a prototype breakwater in areas subjected to significant wave loading. Two possible structures have already been identified. One of these is La Collette Breakwater, Jersey; the other is a second breakwater currently under construction at Bangor, N. Ireland. The latter structure would offer some advantages in that typical transportation and installation stresses, together with the overall breakwater settlement effects, could be monitored from the start of construction. The wave climate, however, is marginally more severe at Jersey where there is a higher incidence of storms. A final decision on the most appropriate structure to be monitored fully will be made in the near future.

3.2 Laboratory studies

The first priority is to develop casting and external vibration techniques to permit the embedment of intrumentation within the units without damage. Due to the urgency of installing units in the field to obtain data from two successive winters during the duration of the project, the six instrumented units designated for installation will be cast at the earliest opportunity.

It is then anticipated that a number of additional units will be cast for a programme of laboratory testing. The controlled conditions in the laboratory will permit more extensive instrumentation of the test units with surface-mounted electrical resistance strain gauges than those to be installed in the field. The exact loading to be applied to these units has yet to be determined but will include quasi-static, dynamic and impact loading as indicated by the early results from the field trials. This phase of the research will be complemented by some limited finite element analysis of single units. This will enable the stress distributions due to the applied loads to be verified. It will also permit an extension of the theoretical assessment of unit strength to a variety of other load cases and unit shapes.

4. ACQUISITON AND HANDLING OF DATA FROM FIELD TRIALS

4.1 Data required

The overall aim of the field trial is to provide data which will ultimately assist the engineer in the design of economic but safe breakwaters. The problem with such field trials is that the loadings are random in nature and site specific. However, although the loadings are random, they are the random output of a determinimistic system with a random input, i.e. the wave climate. In other words, for the data to have significance for future designs, it needs to be normalised with respect to something readily measurable at future sites. Given that the data can be normalised in some way with respect to a vague input wave climate, there are two other important factors which make the data site specific. One is the detailed modification of the waves as they approach the breakwater due to local sea bed topography; the other is the way in which the units are constrained within the matrix, including the effects of static loading distribution due to self weight or settlement. The former can only be handled realistically by using probabilistic or other uncertainty techniques. The latter may be overcome, to some extent, by determining the initial points of contact between adjacent units.

Another important factor to be considered is the relative importance of the various types of loading data. One of the purposes of combined field and full scale laboratory tests will be to show whether it is satisfactory to design on the basis of eqivalent static loads or whether impact loading and load spectra must also be considered in the generally unreinforced units. In addition to self weight loading, it is intended that the field trials should also measure the transportation, placement and settlement loads on the units as these may contribute significantly to eventual failure. The instrumented units should therefore be capable of being handled in exactly the same way as normal units and should look identical as far as possible. A remote telemetry system is clearly essential for retrieving data during the transportation and placement phases.

4.2 Instrumentation

Instrumentation in each of the six units is to be of three types:

(a) Vibrating wire strain gauges embedded in the concrete of each leg of the units, to provide a strain history of shrinkage and creep in the concrete, and to determine the effects of gravity and settlement loading as a result of the position of the unit in the matrix;

(b) Electrical resistance or solid piezo crystal strain gauges and accelerometers in three-axis arrays, for the measurement of dynamic loading during significant events (e.g. impact during installation, storm loading, etc.).

(c) Pressure cells to measure wave pressures and contact pressures between units directly.

Type (a) instrumentation is fundamental for an understanding of the long-term performance of armour units. It will provide invaluable data for the subsequent global stress analysis of rubble mound breakwaters. Instrumentation Types (b) and (c) are essential for the assessment of dynamic and impact loading effects and general structural performance. However, whereas results from Type (a) instrumentation may be obtained at regular intervals by access to the units at low water, the remaining instrumentation yields transient signals. These must be either stored for later retrieval or transmitted directly by cable or radio to an adjacent land station for subsequent processing.

The various sensors together with the pre-processing and transmission hardware must be embedded in the units in such a way that the structural and hydraulic performance matches that of a standard unit as closely as possible. The aim then is to keep everything within the concrete section with only a cable connector and a flush mounted antenna being visible on the surface of each unit.

It is envisaged that a fully instrumented unit will require two 3-axis accelerometers and 6 strain gauge channels. However, since the strain gauges themselves are reasonably cheap, more gauges may be installed with a voting decision system on which elements to use at any particular time. Thus the transmission system is being designed to output a maximum of 12 channels of data per unit.

4.3 Data handling

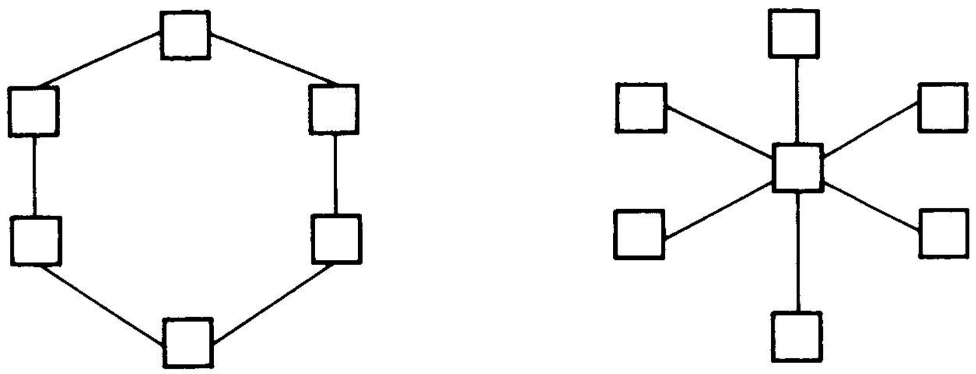
The general wave drag load and wave input load have very different characteristics and need to be sampled at about 25 Hz and 400 Hz respectively. A continuous sampling rate of 400 Hz would produce an unmanageable amount of data very quickly. A system is therefore proposed which can switch between sample rates with a rolling memory in such a way that the lead up to a triggering event can also be captured (pre-trigger operation).

If a base wave period of 8 seconds is assumed, i.e. one main impact lasting 2 seconds (say) every 8 seconds, this produces an average data rate from each unit of 3 kbytes/sec or 25 Mbytes/day of unprocessed data. This can be reduced (by approximately 25%) since any unit will spend at least 6 hours either out of the water completely or well submerged out of the main inpact region. However, this is still a restrictive amount of data.

To deal sensibly with a scheme for the collection of raw data, it is necessary to reconsider what data is really needed. It may be argued that by taking just one channel of raw data from each type of sensor and only recording the statistical variation of the others, the signals associated with these remaining channels could be effectively reconstructed if desired. The philosophy to be adopted then will be to keep raw data storage and transmission to a minimum. Each unit must therefore have sufficient computational power to provide running averages of maximum, minimum, mean and standard deviation of each channel. In addition, raw data and a running average spectrum will be kept for a few selected channels only.

5. TELEMETRY AND TELECOMMAND SUBSYSTEM

The arrangement so far described comprises several identical intelligent armour units each capable of originating data. As such they constitute a distributed instrumentation system. In this context it is possible to consider the instrumented units as a local network and hence analyse the topologies appropriate to such an arrangement. The basic alternative schemes [3] are illustrated in Fig. 3 where the ring and star networks form the basis of two arrangements providing distinct performance regimes.



(a) Ring topology (b) Star topology

Fig. 3 Alternative network topologies

The philosophy of the ring and star topologies are based upon cable transmission systems. The cabled network, in this case a shared resource and units in the network, can gain access via an asynchronous demand assignment arrangement or a synchronous polled system from the network coordinator system. A cabled network essentially needs a signal carrier which, in its simplest form, is the twisted pair. However, one such cable serving multiple units clearly has capacity problems in that it has a fixed bandwidth (typically data rates in excess of 1200 bands are unreliable). So far the system envisaged is incapable of supporting the higher sampling rates of 400 Hz; to go beyond this a balanced cable network would be needed to handle higher data rates. A networked intelligent instrumentation system has been investigated by Barton et al [4] where the network is built around the Ethernet system. Again the nodes or instruments share the network and the transmission capability is assigned to the data gathering instruments on demand. Other systems based on packet switching [5] are also applicable in which the data is actively retransmitted by several nodes, different routes for the data path being selected as demand requires.

An alternative for the cabled network is the radio network. Here there are many schemes, some analogous to the cabled network. The schemes most relevant to consider in this context are bandwidth efficient arrangements which include both access protocol and modulation techniques.

A system worthy of consideration is the radio pocket network described by Davies [6] which, like the cable packet switched network, employs nodes which receive and retransmit data packets independantly through the system in the presence of severe interference. The arrangement is illustrated in Fig. 4 in which the data packets have block identities and originator identities. In the generalised transmit/receive system there must also be a destination identity which indicates to a repeating station the destination of the data packet.

	Destination	Originator	Block No.	Data

Preamble/sync.

Fig. 4 General data packet format.

The attractions of such an arrangement to the current application are clear. A packet network offers flexibility and tolerance to congestion in both a cable and radio system. However, such a network requires several alternative interconnect paths which are not supported in the simple star or ring topologies. In the case of a radio network, more than one radio channel needs to be supported. This would occupy 12 channels for the 12 devices in a single unit per channel assignment. However, time division multiple access schemes allow channel sharing to enable variable data rate transmissions to achieve high bandwidth utilisation.

5.1 Modulation scheme

For a radio network, the environment is of paramount importance in controlling performance. In the sea shore environment it is anticipated that spray and foam will cause severe attenuation in both short and longer term events. It can be shown that attenuation by airated water forms a considerable problem. However, several transmission techniques exist which can cater for the deep fading environment. McGeehan et al [7] have described a scheme which provides a linear modulation channel having good performance in a very narrow bandwidth in a severely fading environment.

5.2 Proposed system

The proposed telemetry and telecommand system is depicted in Fig. 3. Here the network is cabled as a ring but with additional star network capability being provided by a radio system. In this configuration the network can support two cable breaks per unit with the telecommand radio channel supporting data transmission in this contingency mode. An optimal degradation strategy will be developed to cope with radio channel allocation. The radio system will be implemented with narrow band linear modulation techniques. This will provide bandwidth efficiency and good performance during fades. When some or all the units are submerged, telecommand processes will cease or be diverted to the cabled network as required.

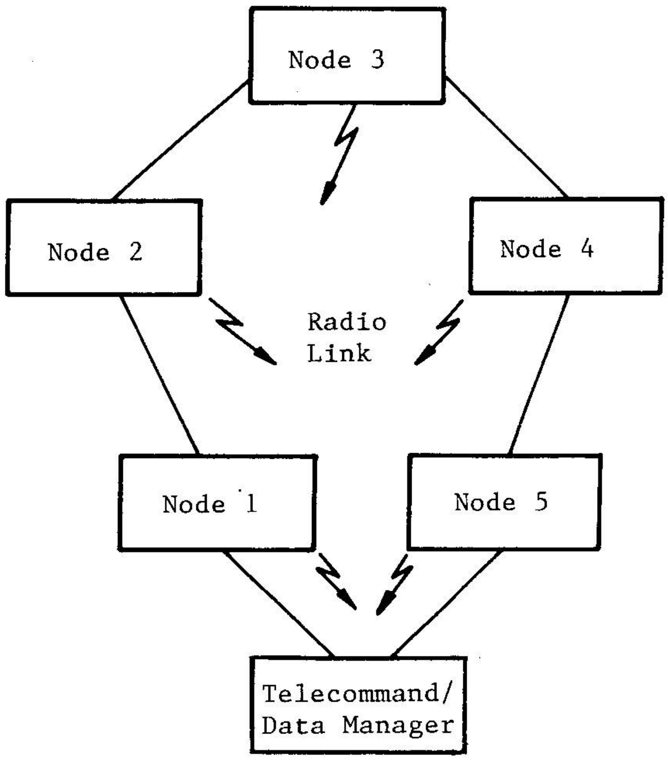


Fig. 5 Proposed network.

6. PRELIMINARY FIELD TRIALS

The project is still in its infancy and many major decisions are yet to be taken which will determine its eventual direction. However, the central feature of the project is the provision of real data from field trials for the necessary calibration of subsequent laboratory and analytical studies. Before embarking on extensive field trials, answers to a number of important questions are needed. These uncertainties include:

(i) the feasibility of using cables or radio telemetry for data transmission;
(ii) the survivability of instrumentation and surface-mounted cabling;
(iii) levels of strain and acceleration likely to be encountered during storms;
(iv) problems associated with installation and prevailing site conditions.

A very limited preliminary field trial was therefore proposed to answer some of these questions. It would also provide experience of the difficult working conditions on site and yield information for the design of equipment for deployment in subsequent field studies. The initial proposal was to test cable survivability during the winter storms of 1986-87 on La Collette Breakwater, Jersey, Figs. 6 and 7. This limited trial was later extended to include the installation of some simple surface-mounted sensors on the Cob units together with an instrumentation canister containing amplifiers and radio telemetry equipment. The data signals from the instrumentation could then be sent simultaneously up the cable and by radio telemetry to a land station close by (Figs. 8 and 9).

6.1 Cable installation

The preliminary trials were conducted with a very flexible 10 mm diameter, 14 core cable with a central cord of high tensile Kevlar and an outer polyurethane

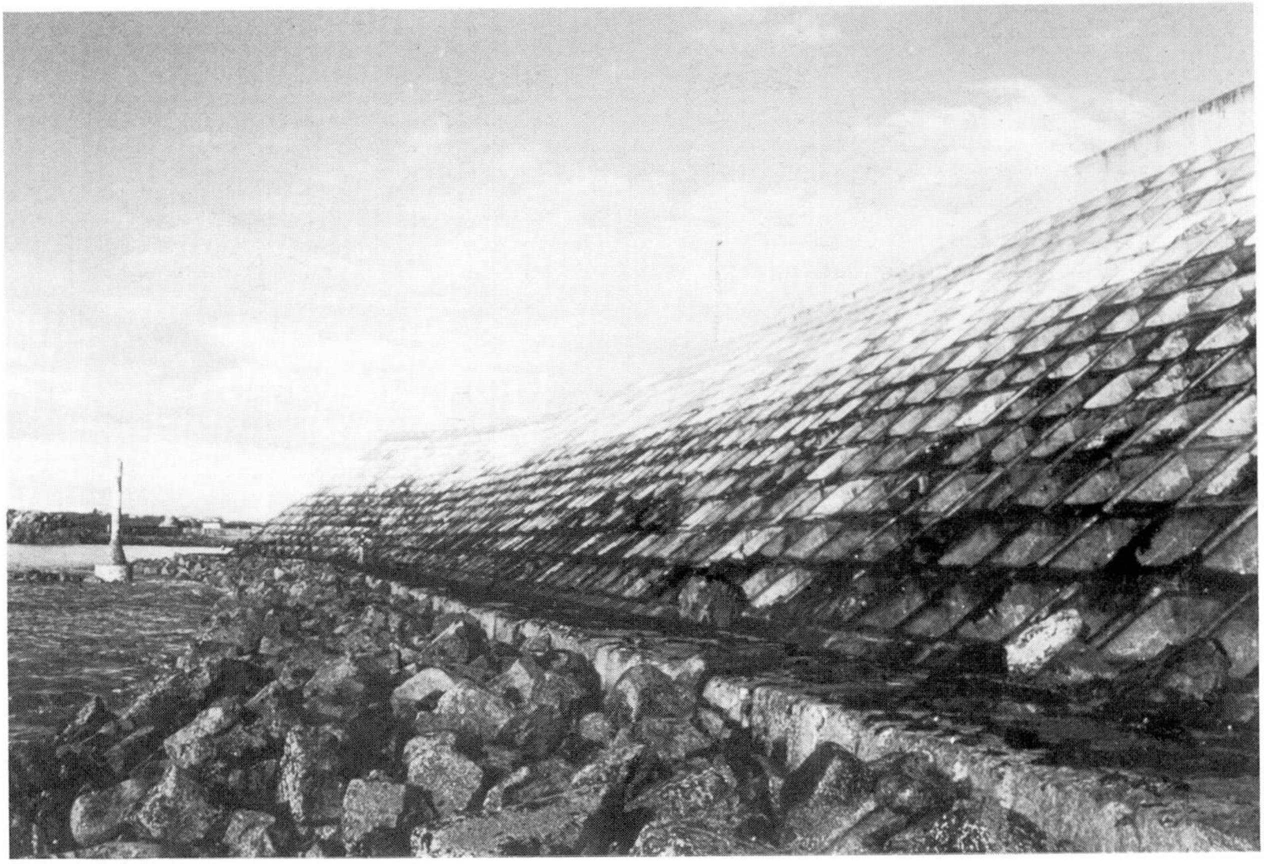


Fig. 6 La Collette as seen from the toe berm.

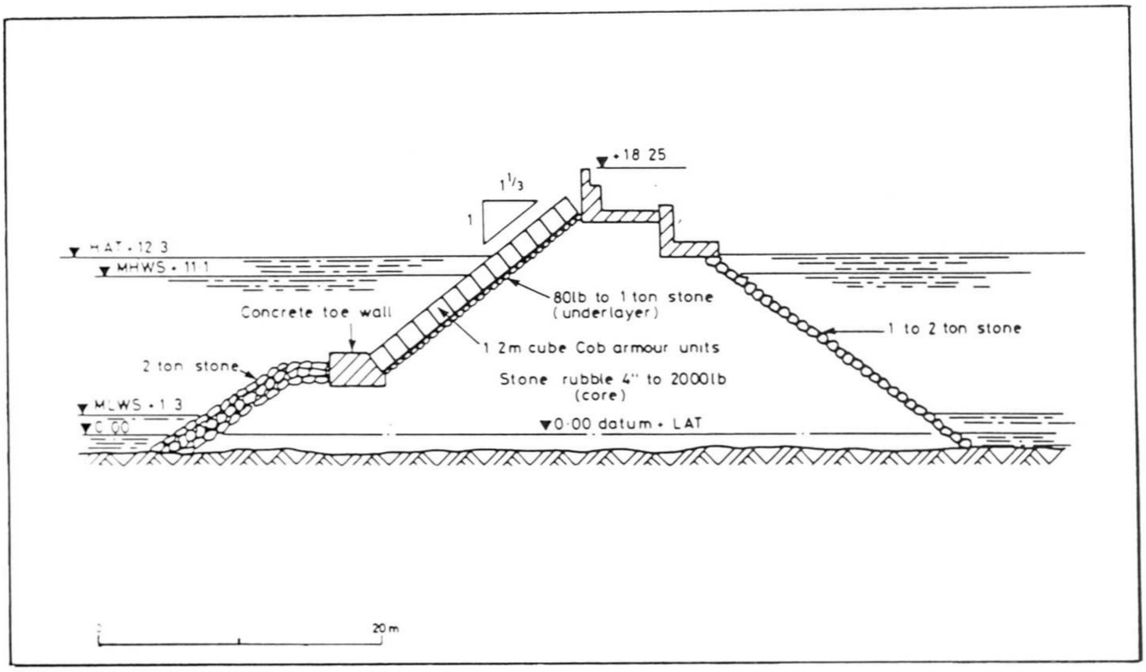


Fig. 7 Typical cross-section of La Collette breakwater.

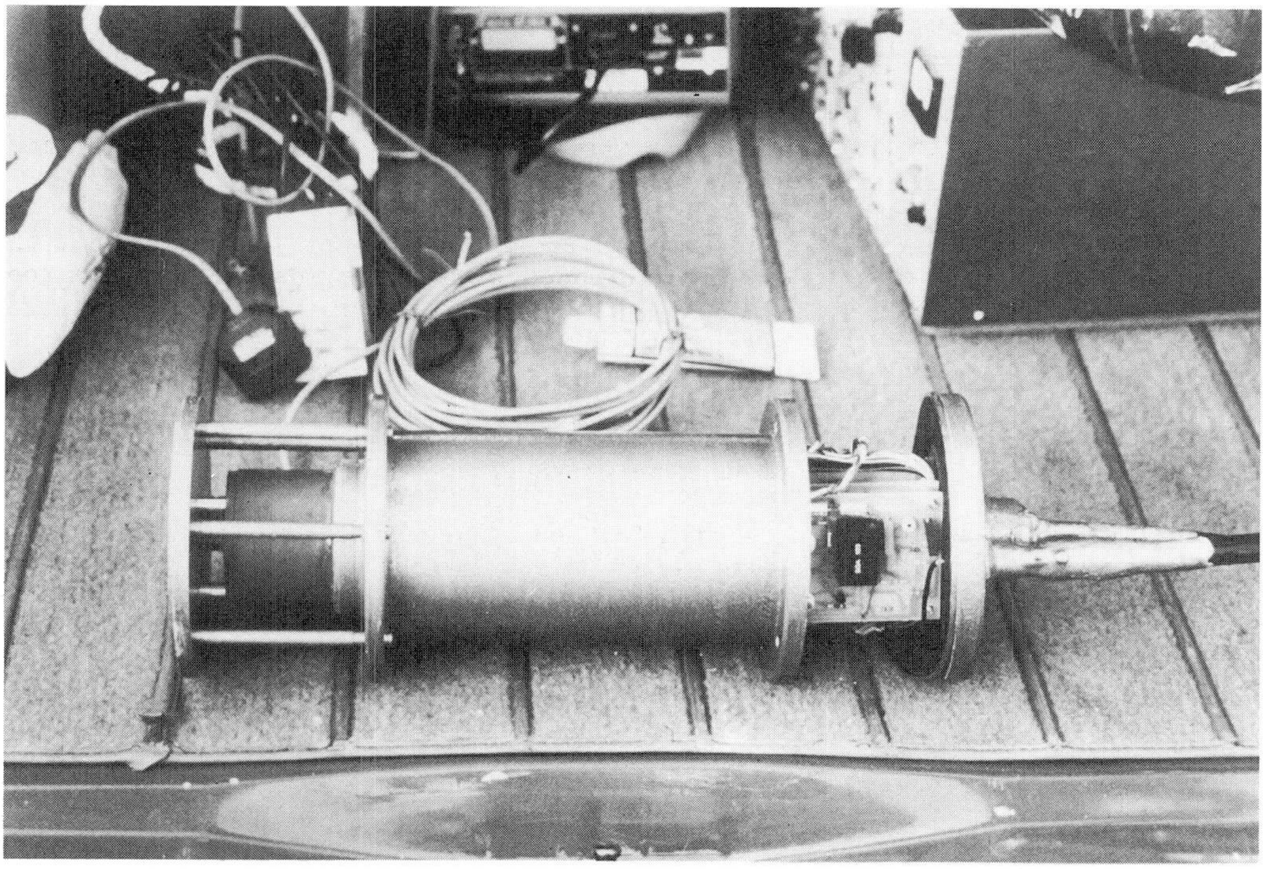


Fig. 8 Instrumentation canister with capping plate open.

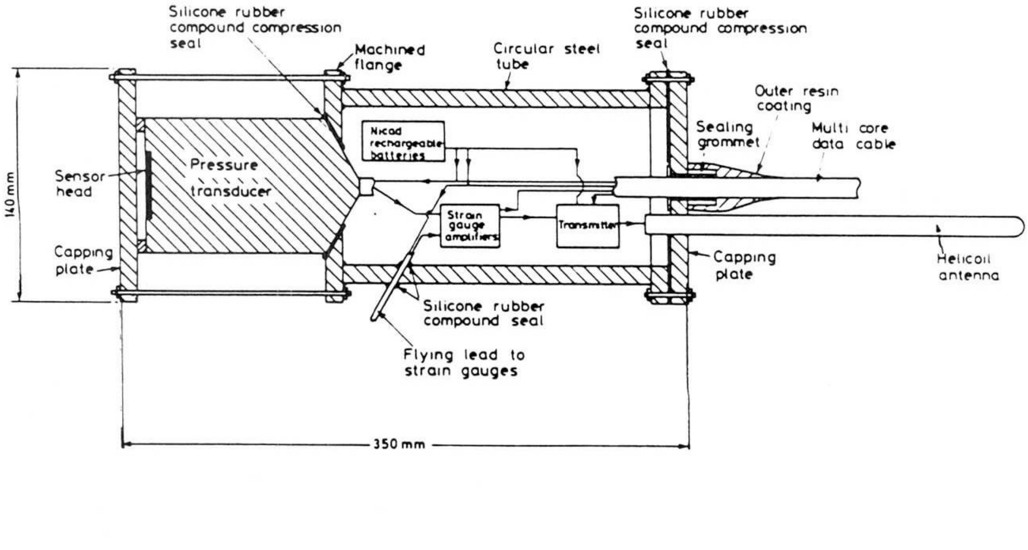


Fig. 9 Schematic section through instrumentation canister.

sleeve. There was some doubt as to whether this cable would be sufficiently robust to span between points of attachment. Moreover, if the data cable were to be tightly stretched and clamped between units, it might either restrain unit movement or become damaged if any significant movement occurred. For these various reasons it was decided to protect the data cable with an outer load-bearing sheath for the length of the armour unit slope. The sheathing selected was 20 mm diameter fibre-reinforced clear PVC tube.

A cable route was selected on the breakwater and deployment began as the tide ebbed below mid-water level. Kemfix threaded studs were inserted in the top and bottom corners of the seaward face of each Cob along the selected cable path. Having placed all the studs, the cable was positioned down the face of the breakwater, Fig. 10. It was then tensioned and clamped with substantial clamps which had been prefabricated from 6mm stainless steel. These were hooked over the sheathed cable at each stud position and secured using stainless steel nuts and washers. Total deployment time was approximately 3 hours.

Weather conditions remained quite calm for several weeks after deployment. The first severe storm occurred during late March 1987. On March 26 the maximum wind speed recorded at Jersey Airport Meteorological Station was 31 knots (Beaufort force 7), direction south south west. Conditions worsened the following day with a peak wind speed of 48 knots (Beaufort force 10) from west south west veering to the north west as the depression passed through. This would suggest that the data cable was subjected to direct incident waves of considerable but unmeasured magnitude. Inspection of the cable directly after the storm and on a subsequent site visit indicated that the cable was completely intact with no apparent abrasion or damage to the sheathing.

6.2 Instrumentation

During the first phase of the trial, an attempt was made to measure two parameters, namely wave-induced water pressure and concrete strain. The basic design philosophy was to produce a waterproof housing on the bottom end of the data cable to act both as a junction box and to encase relevant electronic components. Power supply, signal processing and data recording equipment were stored in a secure hut on the breakwater roundhead. The data cable ran from the hut, over the parapet wall and down the armour layer to the instrumented Cob unit. From there it passed into the instrumentation canister which was rigidly secured to the unit.

Conventional foil strain gauges were mounted in the centre of the upper longitudinal limb of the selected Cob. The surface of the concrete was cleaned using an electrical grinder and then thoroughly dried using an electric hot air blower. The cleaned areas were coated with rapid hardening Araldite resin into which the strain gauge elements were embedded. In the same way, a cable path was cleaned between the instrument canister and strain gauges for attaching the flying lead to the concrete with resin. In general, the resin bonding worked very well given the short time available for preparation and curing.

It became apparent after the first tide that water had leaked into the instrumentation canister through a seal damaged during installation. All the strain gauge amplifiers and the radio transmitter were thus destroyed. As a result it was not possible to test the telemetry system or to receive signals from the sensors via the cable link. A second phase deployment was therefore planned in which the instrumentation canister was replaced by a 'Kulite' soil pressure cell, Fig. 11. The sensing element of the cell consisted of a silicone semiconductor plate covered by an oil bath. On top of the oil bath was a force collecting plate covered by a protective layer of silicon rubber. The frequency response of the cell was approximately 2 kHz.

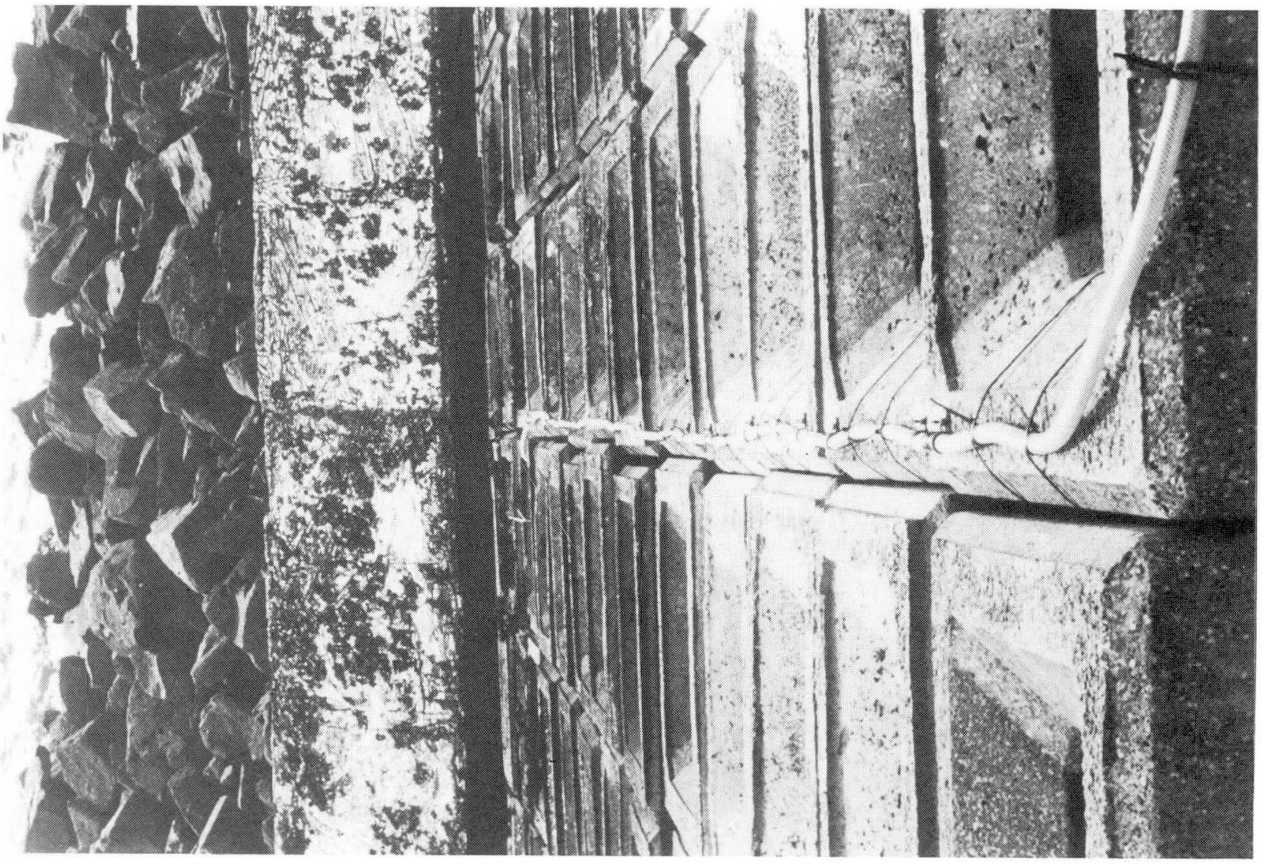


Fig. 10 Data cable attached to face of Cob armour units.

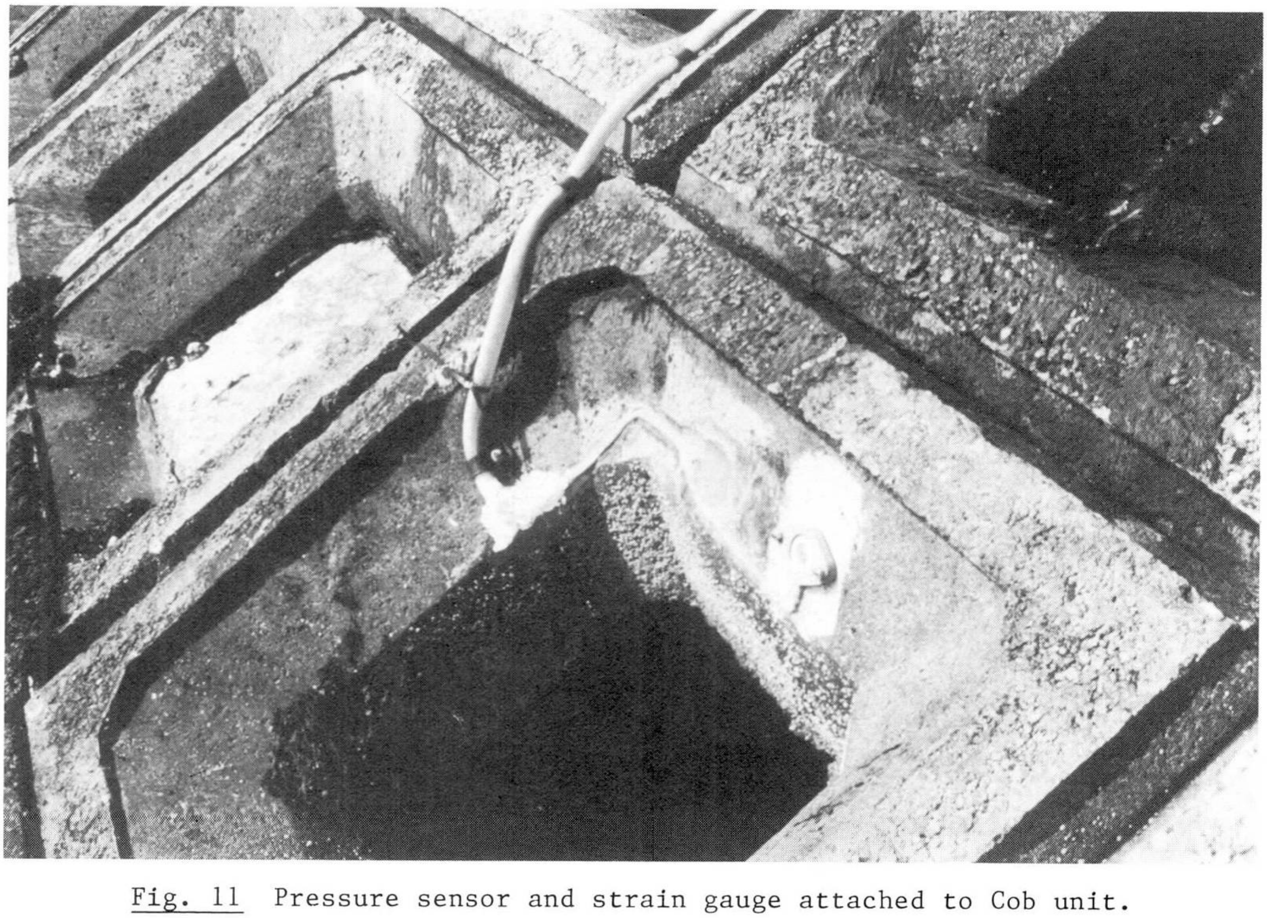


Fig. 11 Pressure sensor and strain gauge attached to Cob unit.

The signal conditioning and recording system was the same as that used for Phase I except that signal amplification was now remote from the sensors and there was no telemetry link. The sensors were activated with a 15V DC supply. Successful results were achieved from the pressure transducer. The conditioned signal had a low signal to noise ratio such that waves of very small amplitude were clearly discernible on the chart record. Some typical traces are shown in Fig. 12.

6.3 Conclusions

Knowledge acquired from the design, construction and deployment of the instrumentation was considerable. The sheathed cable system adopted worked well in terms of ease of deployment and physical survivability. It appeared to be necessary to provide adequate restraint to cable flexure under severe wave attack which might otherwise lead to abrasion and eventual failure. The outer sheathing did not need to be under such tension as to restrain movement of the armour units. The environmental protection provided by the sheathing allows the use of the most suitable data cable which need not itself be heavily armoured.

ACKNOWLEDGEMENTS

The field trial described in this Paper was coordinated by the Single Layer Armour Units Research Club. The project is being suppported by the Department of Environment (UK), Science and Engineering Research Council (UK) and by industrial contributions. Without the considerable support of the staff of Shephard Hill & Co. Ltd. and Coode and Partners on site in Jersey the trial installation would not have been possible.

REFERENCES

1. WILKINSON, A.R., ST. Helier. Consulting Engineer, Oct. 1978, 22-27.

2. WILKINSON, A.R and ALLSOP, N.W.H., Hollow Block Breakwater Armour Units. Proc. Coastal Structures Conf., Am. Soc. of Civil Engrs.,Virginia, 1983.

3. BARNEY, G.C., Intelligent Instrumentation. Prentice-Hall, 1985.

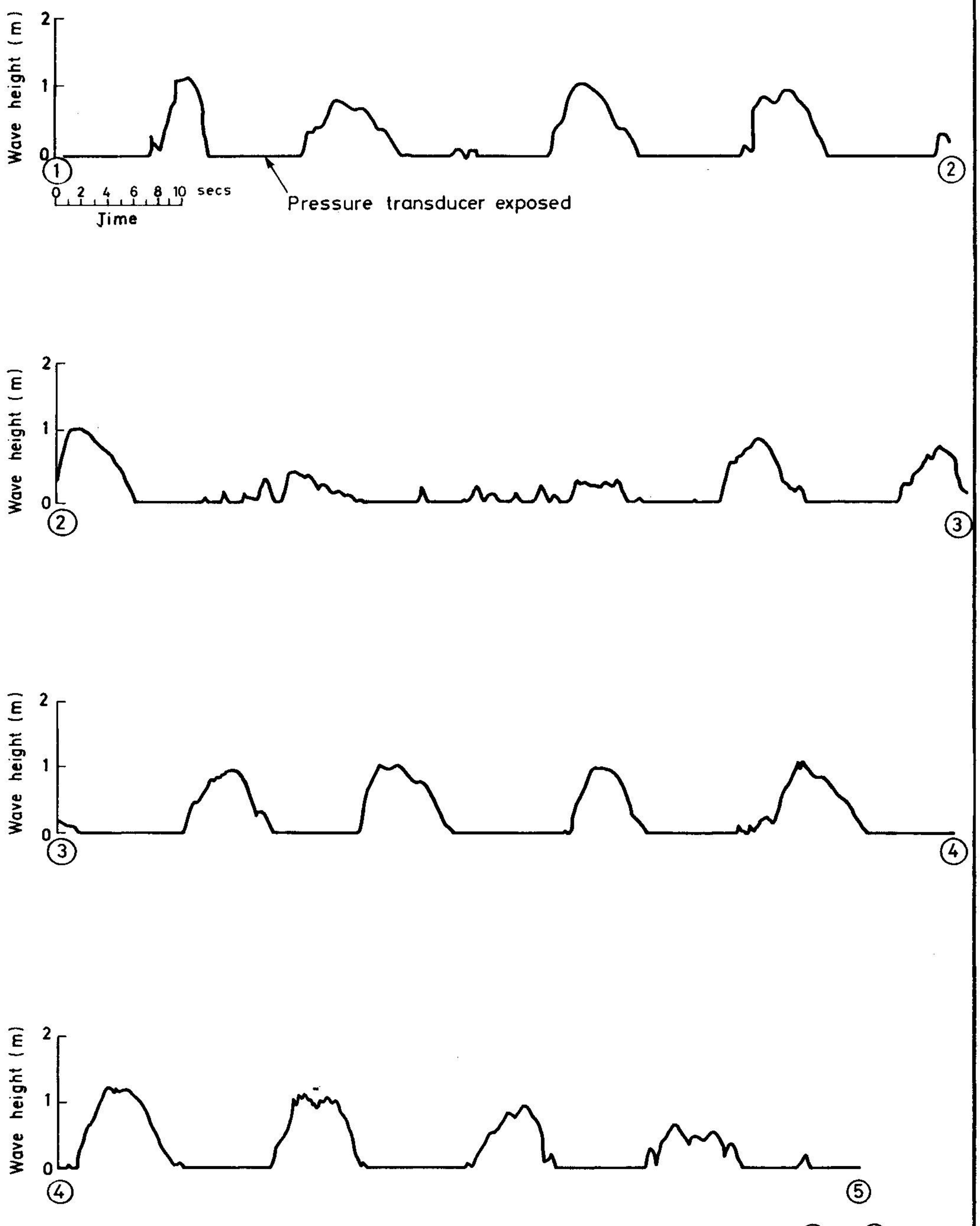
4. BARTON, M.H., Networked Intelligent Instrumentation. To be published.

5. KELLY, P.T.F., Packet Switched Data Communication Networks. British Post Office, Elec. Eng. Journal, Vol. 73, Pt. 3, 1981, 216.

6. DAVIES, T.R., Synchronisation in a Packet Radio Network. Inst. of Elect. Engrs., Colloquium on Clock Recovery and Synchronisation in Digital Mobile Radio, IEE Digest 1986/106, Oct. 1986.

7. McGEEHAN, J.P. and BATEMAN, A.J., Data Transmission over U.H.F. Fading Mobile Radio Channels, Proc. IEEE, Vol. 131, Pt. F, No. 4, 1984.

8. STEPHENS, R.V. and DAVIS, J.P., Single Layer Armour Unit Research - Preliminary Field Studies and Instrumentation Trials, Hydraulics Research, Wallingford, UK, Report No. IT311, May 1987.



NOTE: The four segments shown above represent one continuous trace ① - ⑤

Fig.12 Sample record from pressure transducer

In-Situ Dynamic Tests on Ancient Monuments

Essais Dynamiques In-Situ pour des Structures Monumentales

Dynamische In-Situ Prüfungen an Denkmalsbauten

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SUMMARY
In-situ dynamic tests can be performed from time to time on structures of particular interest, with the purpose of ascertaining their dynamic response. Then the variations observed in the dynamic characteristics from one test to the next may by taken as an index that something has changed in the structure itself. Lastly, attempts can be made to diagnose the cause and type of modifications undergone by the structure. The paper relates three initial experiences pertaining to the Brunelleschi Dome, the remains of the temple of «Ares the Avenger» in the Augustean Forum in Rome and the Arnolfo Tower in the Palazzo Vecchio in Florence.

RESUME
Pour des structures d'intérêt particulier il est possible d'exécuter de temps en temps des essais in situ pour la détermination de leur comportement dynamique. D'éventuelles variations de caractéristiques dynamiques observées d'un essai à l'autre que quelque chose changé dans la structure examinée. Enfin il est possible de déterminer les types de modifications arrivées dans la structure et leurs causes. Le rapport décrit trois expériences sur la Coupole de Brunelleschi à Florence, sur les restes du Temple de Mars Ultor à Rome et sur la Tour d'Arnolfo à Palazzo Vecchio à Florence.

ZUSAMMENFASSUNG
An Bauten von besonderem Interesse ist es möglich, in entsprechenden Zeitabständen Prüfungen am Ort durchzuführen, um das dynamische Verhalten zu bestimmen. Aus den beobachteten Veränderungen der dynamischen Eigenschaften zwischen den Prüfungen ist es möglich, Hinweise auf Veränderungen zu erhalten. Schliesslich kann durch eine Diagnose der eingetretenen Veränderungen versucht werden, deren Ursache zu bestimmen. Der Bericht beschreibt drei Erfahrungsbeispiele: die Kuppel der Brunelleschi in Florenz, die Reste des Tempels des Mars Ultrix im Forum Augustus und den Turm des Arnulf im Palazzo Vecchio in Florenz.

FOREWORD

In the last 20 years ENEL - through its R & D Dept. and in particular through CRIS, in co-operation with ISMES - has developped, and largely applied, experimental and numerical techniques aiming at the dynamic characterization of the big structures pertaining to ENEL installations (dams, chimneys, cooling towers). This characterization was deemed necessary in view of a seismic safety assessment.
The positive experience gained in this industrial framework has prompted attempts in order to transfer these same techniques and know-how to the study, and more specifically to the seismic risk assessment, of monumental structures.
These attempts originated, in recent years, a certain number of in-situ surveys as well as interpretations of results by means of mathematical models. The methodology used in these endeavours is of a general type; it can be broken up into different phases (or types of activity), which we intend to describe in the first paragraphs of the present paper.
In the last-but-one paragraph we illustrate the actual experiences so far carried out, referring to the Arnolfo Tower and to the Brunelleschi Dome in Florence, on one hand; to the remains of the temple of "Mars Ultor" in the Augustean Forum in Rome, on the other hand.

1. DESCRIPTION OF IN-SITU TESTS

According to the type of monument and its environmental situation, the planning of an in-situ test campaign can be oriented either toward a "natural excitation", i.e. taking advantage of environmental dynamic "noise" (wind, traffic, etc.), or toward artificial excitation by means of shakers imparting a sine or a random force to the structure.
In case natural excitation is chosen, since this is usually of slight intensity, it is advisable to install not only the usual contact instruments (seismometers, accelerometers), but also laser interferometric instruments (e.g. "LADIR") which can operate at a distance (up to 200-300 m) with a high sensitivity (down to 0.1 μm). The latter are very convenient for those monuments where it is not possible, or advisable, to get direct access or to execute rigid fixings for "traditional" electro-mechanical instruments.
In case artificial excitation is chosen, it remains to decide whether to install mechanical-type vibrodynes (rotating masses), which can provide only sine excitation, or a different type of exciter (hydraulic or electro-mechanical actuators), which, if driven by a suitable control unit, can provide either sine-wave or non-sinusoidal forces; in the limit this latter type of exciter can impart fully random excitation with a pre-established power spectrum.
The choice between the two possibilities can be taken considering the opportunity of exciting the different modes separately (one by one) or simultaneously. In the former case a sine wave sweep is used; the interpretation of results is easier, but the duration of the test is greater.
In both cases the excitation level is somehow controllable so as

to respond to the necessities of the particular test in hand.
The excitation units have to be placed on the structure or on its foundations so as to excite all interesting "natural vibration modes". This can entail the necessity of moving the exciters (if it is only one) or of installing more than one exciter (multipoint excitation); in fact, an excitation applied in a point of a nodal line of a mode cannot excite that particular mode. As one can see, it is advisable to get a preliminary knowledge (as accurate as possible) of the modal shapes; this can be achieved by means of mathematical models (see further on).
It is good to keep in mind that usually only the first few modes (e.g. 3 to 6) of the structure are of interest.
In a like manner, turning now our attention to the sensors intended to measure the structural response to the excitation, those have to be distributed in such numbers and positions as to pick up the maximum amplitudes of the sought-after modes, i.e. along the antinodal lines of the particular mode of interest. In every point one can choose to place one, two or three-directional sensors according to whether only some particular components of motion, or the whole vectorial information, are desired. As to the nature of the physical quantity being measured, seismometers and accelerometers pick up respectively the local velocities and the accelerations of the movement, and they need to be rigidly joined to the point of the structure whose motion is investigated; the laser interferometer is able to measure the displacement component along the sighting line, and - as already stated - the measurement takes place without material contact and up to distances of 200 ÷ 300 m. It requires, however, a rigid base for the laser emitter.
All the data collected during the test are to be acquired and stored on magnetic support, if possible already in digital form for later processing.
More detailed considerations about the mini- or micro-computer to be used, to the connecting cables, to amplifiers, analog-to-digital converters, signal conditioning etc. can be found in the vast specialized literature concerning data acquisition systems.
The off-line data processing include: computation of transfer functions (excitation vs. response), Fourier Analysis (FFT), coherence analysis, cross-correlations etc. - many of these procedures convert the responses from the time-domain (where they are acquired) to the so-called frequency domain; afterward these frequency responses can be treated with well-known techniques in order to separate from the total response the responses of the single modes. This separation can be controlled (validated) a posteriori; if need be, the modal responses thus identified can be compared with the homologous modal responses as derived from a mathematical model of the structure. From the processing of experimental data - and only in this way - it is also possible to derive the damping factors of each individual mode.
It is to be noted that - in the present stat of the art - all these procedures are based on the hypothesis of linearity of the system: this has to be taken into account, above all as far as the damping factors are concerned: in fact, the damping degree that can be derived from the low-intensity excitation tests may differ quite considerably from the damping degree at high levels of excitation, such as occur e.g. during a strong earthquake.

2. DESCRIPTION OF THE MATHEMATICAL MODELS

The mathematical models that can be used in order to simulate the dynamic behaviour of a monumental structure can span the gamut from the simpler (plane models with 1-D beam elements) to the more complex ones (3-D, F.E. models) through all the intermediate steps (2-D models, in a plane or with a rotational symmetry), according to the geometrical features of the structure to be modeled and to the degree of accuracy which is reasonable to adopt in relation to the available informations.

Obviously, to more complex models there correspond greater computational times and costs, so that also economical considerations, or the degree of urgency of the study, can affect the choice.

Whatever the final choice, hence apart from the degree of sophistication of the numerical model, one can detect at least four different uses of these models (often referring to different phases of a same study):

- A preliminary use in order to plan the in-situ tests, namely to identify the more suitable positions for the sensors and the excitation units; to determine the probable frequency range of the sought-after modes; to estimate the more suitable excitation level, so as to assure both an admissible level for the structural effects induced by the excitation (against the strength of the structural parts) and a sufficient amplitude of the response signals (against the instruments sensitivity).
- A joint utilization of the mathematical model and the test results so as to get a mutual validation. We intend by this that a comparison with the mathematical model can indicate whether the data collected during the tests are reliable and consistent; on the other hand, a comparison between modal shapes and frequencies obtained by the two approaches can give the mathematical model a credibility that nothing guarantees "a priori". Besides, such a comparison allows an "a posteriori" calibration of the numerical value of some physical constants which often are not known with sufficient accuracy for the structure as a whole (e.g. the Young moduli of the materials). This calibration entails a "identification" process which can be made more or less sophisticated according to circumstances.
- use of the mathematical model "backward" in time ("hindcasting") in order to estimate, through additional hypotheses if need be, what has been the structure behaviour, in its pristine state or under various stages of deterioration, under dynamic excitations which happened in the past.
- A like use of the mathematical model, but "forward" in time ("forecasting") in order to assess, again with the help of additional hypotheses where needed, what will be the structure behaviour in its present state, or after alternative interventions, under dynamic excitations which can foreseeably happen in the future. Of course this last use entails the problem of defining an accelerogram, or at least a power spectrum of the earthquake, that should be significant and credible for the monument site.

3. INFORMATIZATION OF TEST RESULTS AND OF NUMERICAL ASSESSMENTS

For monumental structures of a certain importance it appears feasible - as well as desirable - to create, on a suitable informatic support (HW/SW) a structural data-base as complete and continuously updated as possible.
In it one should include not only the knowledge about the "state" of the monument (historical and architectural informations, present configuration, interventions etc.), but also all the numerical data concerning the results obtained successively from in-situ tests, from "continuous" monitoring, from the mathematical-numerical models with their main conclusions.
This data-base would provide a precious lore of historical-technical documentation permanently available to those people and organizations that are responsible toward the safety and preservation of the monument. It would also be of great help to obtain without delay diagnostic interpretations (if feasible) after exceptional events; besides, it could be used to get a preliminary evaluation of the effects of any proposed alteration or reinforcement.
Such an "informatization" would indeed make easier the systematic use of dynamic tests (i.e. their regular and scheduled repetition in time) in order to reveal, through any alteration of the vibrational "signature" of the monument, the tendencies toward structural degradation, or progressive evolution, if any. The diagnostic use of in-situ dynamic tests could further be enhanced by repeating the tests after the site has undergone some exceptional event (e.g. a strong earthquake) that, albeit without apparent damage, may induce some suspicion about possible detrimental effects on the structural qualities of the monument.

4. DESCRIPTION OF SOME TESTS AND ANALYSES CARRIED OUT IN THE PAST

The studies carried out so far with the above-illustrated methodology - albeit not completely followed in each particular case - concern three important monumental complexes, two of them situated in Florence and one in Rome:
- The Arnolfo Tower of the Palazzo della Signoria (Palazzo Vecchio) in Florence.
- The Brunelleschi Dome, part of the Cathedral of Santa Maria del Fiore, in Florence.
- The remains of the Temple of Mars Ultor in the Augustean Forum in Rome.

In the following we propose to illustrate briefly the activities carried out, as well as the main results obtained, for each one of the three monuments.

4.1 The Arnolfo Tower

This well-known monument is part of the Palazzo della Signoria (Palazzo Vecchio) in Florence.
It is a stone-masonry tower, built-in into the structure of the palace, from which it stems at a height of 38.5 m. The

free-standing height of the Tower is about 43 m.
The cross-section is rectangular, with sides of 6.00 m and 8.00 m. The structural scheme is that of a hollow rectangular cylinder, with wall thickness of about 1.50 m.
The structure presents a slight overhanging (1 m) with respect to the façade of the Palace. It was built around 1300 on a design by Arnolfo di Cambio, whence its name.
The in-situ tests were carried out by means of a rotating-vector vibrodyne installed in the so-called "Albergaccio del Savonarola", a small jail situated at an elevation just below the lookout walkway around the Tower.
The response acquisition was effected through 29 seismometers of the type GEOTECH TELEDYNE S13, positioned all along the height of the Tower and in correspondence of the two orthogonal axes of the cross-section. On the side facing Piazza della Signoria the number of measurement points was increased by using two velocity transducers of the laser-interferometric type, installed in the square on the side opposite to the Tower. This allowed to detect the response at some points in which it would have been impossible to locate inertial-type transducers. A cable network for a total length of some 5000 m connected the vibrodyne and all the transducers to a recording station from which the excitation and the data acquisition was controlled.
Environmental circumstances during the tests were particularly favorable, inasmuch as it was also possible to record the Tower response to a series of strong wind gusts.
This will allow, after data analysis on the recorded signals, an interesting comparison between the two types of excitation.
On this monument so far no attempt was made to set up a numerical model with which to interpret the results; however, such a model (e.g. F.E. one) would not appear particularly difficult and it is intended to work on it in due time.
The frequency of the first mode has been preliminarly identified at 0.49 Hz.

4.2 The Brunelleschi dome

For this structure - so well known that we deem superflous to give even a brief description - we carried out, so far, many numerical analyses with increasingly sophisticated F.E. models, under different hypotheses of static loads and of uncracked or cracked state. Also, numerical determinations of modal shapes and corresponding frequencies were carried out, again for the uncracked and cracked structure.
Finally, a preliminary attempt was made to detect and record naturally-induced vibrations, due either to vehicular traffic nearby or to wind gusts.

4.2.1 Dynamical analyses by F.E.M.

The F.E. mesh discretizes one fourth of the Dome (with the Skylight reduced to a rigid body having the right mass but not the real geometry), of the underlying Drum and of the Pillars. It is not attempted to represent the structural interactions with the three Chapels (North, East, i.e. on the back side, South), nor with the main Nave (West side of the Dome) see Fig. 1.

This one-fourth model can represent the whole Dome symmetric or antisymmetric dynamic behaviour thanks to suitable symmetry or antisymmetry conditions on the mutually orthogonal vertical planes (geometric symmetry planes) that bound the model. In this way the two halves of pillars included in the model have been made geometrically symmetrical, which does not correspond with the actual situation [unless the ¼ model is assumed to represent the back (East) ¼ of the Dome].
These are limitative assumptions, which it will not be impossible to eliminate in further developments, provided one accepts the obvious implication of a larger mesh, entailing a much greater computational onus.
Under the above assumptions we determined the first six modal shapes (and relevant frequencies) both for the uncracked dome and for the dome with cracks representing - albeit with considerable simplifications - the present situation. In each case two modes are of the symmetrical type, two are anti-symmetrical and two are torsional ones.
The results are synthetically shown, as far as theoretical frequencies are concerned, in the following table:

		UNCRACKED DOME	CRACKED DOME
MATHEMATICAL MODEL C.R.I.S.	SYMMETRICAL MODES	2.14 4.08	1.86 1.95
	ANTI-SYMMETRICAL MODES	1.03 3.03	0.89 2.25
	TORSIONAL MODES	1.65 2.70	1.60 2.16

As for modal shapes, it has been deemed that the most suitable form of graphic representation is a software-generated animation film; this has in fact been produced with satisfactory results.
The above-presented frequency table shows that, for some modal shapes, a direct pairing is possible between the vibrational modes of the uncracked Dome and those of the cracked structure: obviously in this latter case the frequencies are appreciably decreased, this being interpreted as a quantitative index of the degradation of the "modal stiffness" brought about by the cracking. In fact, the modal stiffness of similar modal shapes are roughly proportional to the square of the respective frequencies, so that the expression:

$$\frac{f_0^2 - f^2}{f_0^2} = 1 - \left(\frac{f}{f_0}\right)^2,$$

where f = frequency of a mode of the cracked structure, f_0 = frequency of the like mode of the uncracked structure, can be taken as an index of the loss of strength resources due to the cracking.
For other modal shapes, however, it is not possible to establish a direct pairing between "sound" and "cracked" model, because the

cracks alter in a deep way not only the partial and global stiffnesses, but also the very geometry (or better the topological internal connections) of the structure, so that the introduction of new internal degrees of freedom brings about some completely "new" modal shapes.

It is also to be remarked that the above computational frequencies - even apart from the error introduced by neglecting the constraints provided by the Chapels and the Nave - have to be regarded as a relative, and not an absolute, succession of values, inasmuch the mechanical properties assumed in the computations are purely hypothetical (they are: $E = 5000$ MPa, $\nu = 0.1$, $\rho = 1800$ Kg m^{-3} etc).

Moreover, it has to be stressed that in the present state of the art the mathematical model cannot yield any information about the level of energy dissipation, in other words about the "damping factors" associated with the different modal shapes.

All that the mathematical model can yield with good reliability (apart from the approximations introduced by the adopted discretization, i.e. the fourfold symmetry and the absence of Chapels and Nave) are the modal shapes.

The next steps of the dynamic investigation of the Dome can be foreseen as follows:

- Increased sophistication of the mathematical model, including in it the Chapels (a fourfold symmetry may be preserved by admitting that the constraint introduced by the Nave is rougly similar to that posed by a Chapel, and by neglecting the different cross-section of West pillars with reference to the East ones). With this refined mathematical model one should compute again the modal shapes and relevant frequencies.
- Validation and calibration of the mathematical model including the cracks, through in-situ determination of some eigenfrequencies and of the corresponding modal shapes. The in-situ tests would also yield directly, as already stated, the damping factor of each mode.
- At last, as a conclusive and crowning step of all the investigation, computation - via the validated and calibrated mathematical model - of the structural response to one or more "design earthquake", obviously either in the present, cracked, conditions or under various hypotheses of reinforcement. This last phase on one hand will allow to get insights on the seismic safety conditions of the Dome as well as on the effectiveness of different proposals for intervention, but on the other hand it will pose delicate, complex problems in order to define one or more earthquakes that can be considered as credible, significant and consistent with the site seismo-tectonic and seismographic profile.

4.2.2 In-situ dynamic surveys: a first attempt

On the Brunelleschi Dome one has carried out - apart from the sporadic measurements of traffic-induced vibrations made many years ago and not well documented - a simple survey of wind-induced vibrations over two out of the eight "sails" (curved panels) forming up the structure. The input (wind gusts) has been of moderate intensity. The response was measured by 8 seismometers placed on the two "sails" at three different levels, spanning the elevations from the "serraglio" (the summit ring on which the

Skylight rests) to the boundary between the masonry Dome and the underlying stone Drum.
In order to detect the vertical component (if any) of the displacements at the elevation of the "serraglio" also two vertical-component seismometers were installed, one for every monitored sail. The transducer signals were recorded on magnetic tape as acquired; the recording covers a time span of about 36 hours.
During selected intervals of time the signals were fed also to a spectrum analyzer; this allowed us to reveal both the frequencies preferentially excited by wind gusts and the phase and amplitude relationships (transfer functions) between the different points being surveyed.
The selectively predominant frequency turned up to be 1.86 Hz for the North "sail" and 1.77 Hz for the East "sail": this difference points out to structural asymmetries that it will be necessary to investigate in depth.
A fuller analysis of the recorded data is pending; we hope that the results will yield indications toward a preliminary calibration of the mathematical model, so that the latter can be used to plan, if necessary, the operational procedures to follow for an effective forced-vibration test.

4.3 Remains of the "Mars Ultor" Temple in the Augustean Forum in Rome

This monument is reduced nowadays to three Corynthian columns (in Carrara marble), 17 m high, connected among themselves as well as to a back wall (in "peperino", a tufaceous conglomerate) by a summit horizontal structure. The whole is tied to a transversal wall, to which a flat lesena similar to the three columns is attached (see Fig. 2).

4.3.1 Dynamic numerical (F.E.) analysis

These remains have been discretized by a F.E. mathematical model made up by 1434 elements (of the isoparametric, 2nd order tupe) adding up to about 29000 degrees of freedom (of which about 1600 blocked by boundary constraints) (see Fig. 3).
The physical properties of the different materials and the structural constraints were assumed on the basis of some trial computations so as to obtain, from the mathematical model, a frequency response sufficiently like the one that was recorded during the in-situ dynamic tests (see § 4.3.2).
After this rough "calibration" of the numerical model we carried out a numerical simulation of the dynamic excitation of the first mode as carried out in the field.
The comparison between the amplitude level of the vibrations (computed vs. measured values) turned out to be quite good, thus confirming that the "calibrated" mathematical model is adequate to represent this geometrically and physically complex structure.
It will now be possible to obtain numerically an answer to several questions concerning the monument, first of all e.g. to evaluate the effectiveness of possible interventions aiming at achieving a better seismic safety of these archeologically important remains.

4.3.2 In-situ dynamic surveys

The in-situ tests were carried out taking avail both of natural-excitation sources (wind gusts, vehicular traffic in the nearby urban streets) and of an artificial-excitation device (vibrodyne) with a controllable sine-wave output (amplitude and frequency of the imparted force, which was in fact a rotating vector, could be varied in a controlled way). The movements of the several points under observation were measured by the laser-interferometric, contactless system (LADIR) already mentioned under § 4.1. Besides, the movements of some more easily accessible points (especially along the foundations) were monitored with electro-mechanical seismometers of the classic type (see Fig. 4 and 5).
By effecting a sine sweep (with the vibrodyne) or by analyzing the response spectra (with the natural excitation) it has been possible to detect the first mode of vibration, at a frequency of about 2 Hz and with a damping factor of the order of 2% of the critical damping. In the low-intensity input range that was covered by the tests, the structural behaviour is very nearly linear (see Fig. 6).
The low frequency of the first mode, as well as the corresponding modal shape, induce to expect a pronounced deformability of the foundation layers (this is confirmed by the fact that in the calibration of the mathematical model it has been necessary to assign a very low value to the Young modulus of the thin foundation layer represented in the mesh).
The informations thus collected, anyway, have been very useful in order to carry out the calibration of the mathematical model already related under § 4.3.1.
It is good to keep in mind that the campaign of in-situ surveys cannot be considered exhaustive, inasmuch only the first mode has been clearly identified.
If these investigations will have a sequel, it will be therefore necessary to repeat, and enlarge in scope, the in-situ surveys.
It can be added that the mobile equipments developed for excitation, measurements and data processing - already exhaustively tested in the many surveys carried out on the big industrial structures of different ENEL installations - worked in a fully satisfactory way and without any noteworthy difficulty also in this new context.

5. CONCLUSIONS

Although the experiences so far carried out have been partial ones with a demonstrative character, one can already state that the methodologies developed until now, and systematically used for industrial structures, can be used with full confidence and put to good avail also for monumental structures. They can undoubtedly lead to a better knowledge of the dynamic behaviour of such structures, thus opening the way to a quantitative evaluation of their seismic safety with technical-scientific tools that reflect the current "state of the art".
Sooner or later it will be inevitable, we think, to pass from episodic, demonstrative applications to a systematic use of those tools. If and when this decision will be reached - which obviously

entails both a strategic view and the availability of adequate financial funding - an inestimable wealth of knowledge will be within reach. We allude to that vast body of specific, detailed data, informatically organized for the best efficiency of use, that will ensure in a way hitherto impossible the protection of the ingent treasure of architectural/archeologic monuments existing in our Country and exposed to the seismic risk which is so widespread in our territory.
Such a knowledge will have to be considered not as a closed body, gained "una tantum" for each monument, but on the contrary as a peculiar type of "monitoring", to be repeated with a quasi-regular time schedule, albeit at intervals which are by no means short (e.g. some years for those structures which suffer no particular problems). In this way the dynamic "dossier" of each structure can be viewed as an objective documentation undergoing a "continous" evolution and updating, which should follow and accompany the monument in every phase, alteration or transformation of its existence.

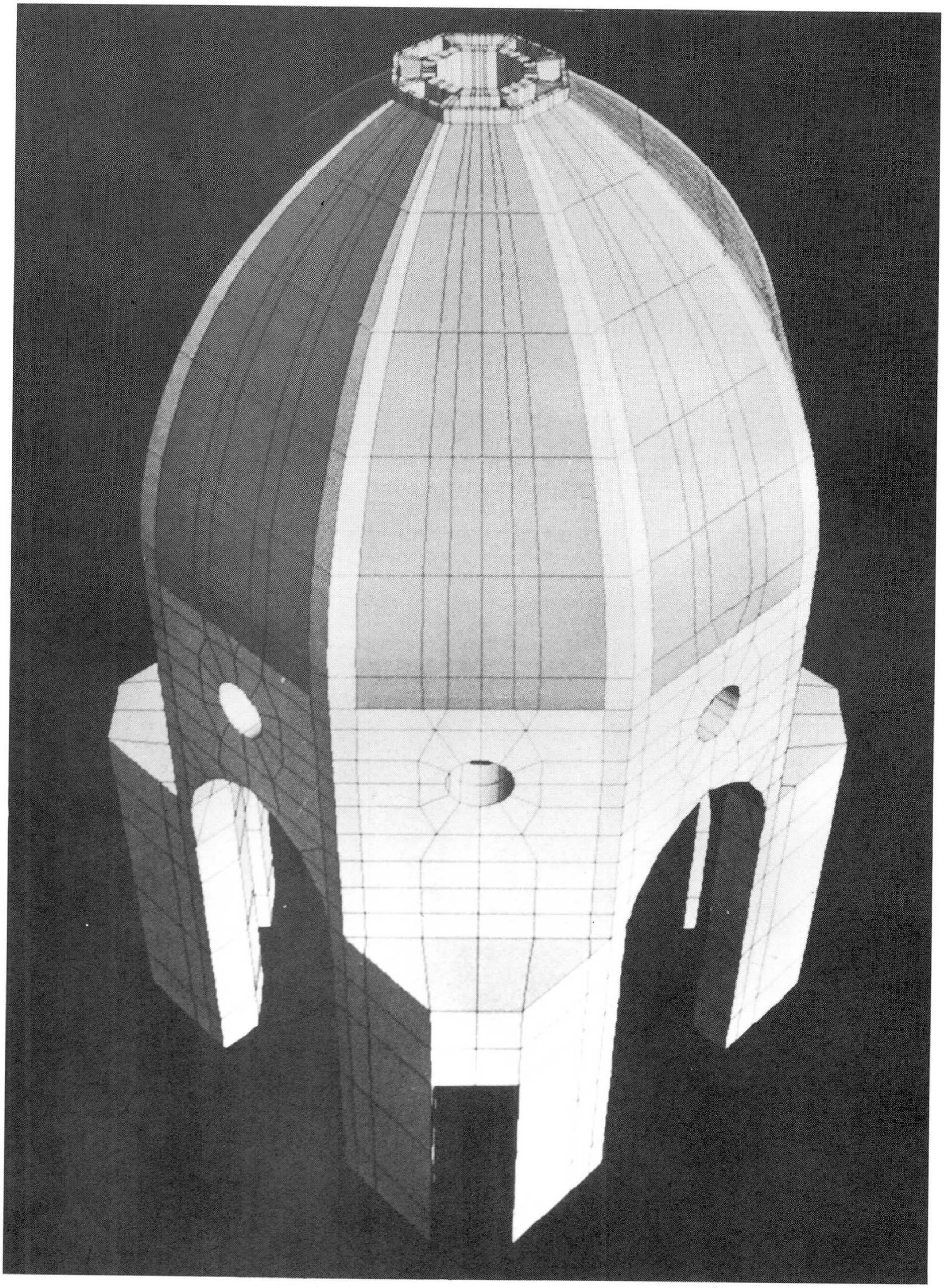

Fig. 1 - 3D F.E. mesh used for dynamical analyses of Brunelleschi Dome.

Fig. 2 - Panoramic view of remains of the "Mars Ultor" Temple in the Augustean Forum in Rome.

Fig. 3 - 3 D F.E. mesh used for numerical simulation of dynamic behaviour for the remains of the Mars Ultor Temple.

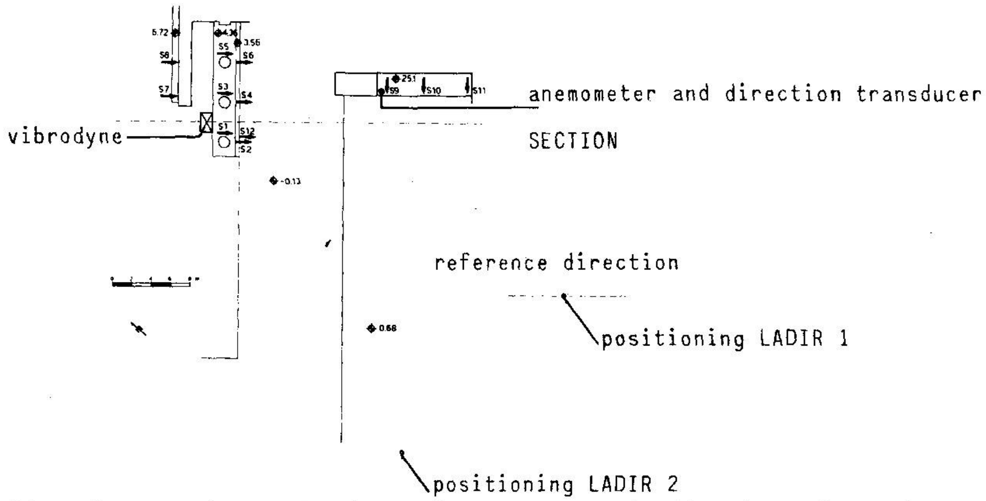

Fig. 4 - Horizontal view of sensors and vibrodyne locations.

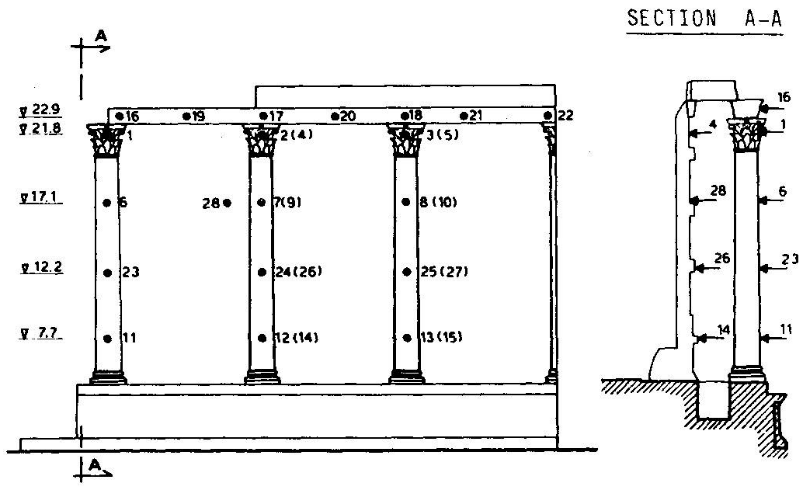

Fig. 5 - Vertical view of sensors and vibrodyne locations.

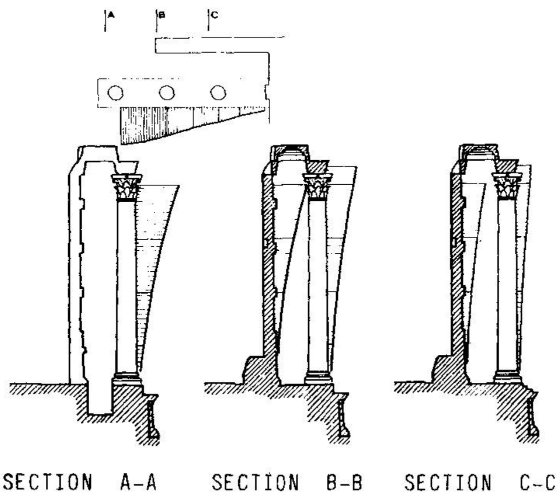

Fig. 6 - Graphic representation of measurement data: first mode.