Session 2

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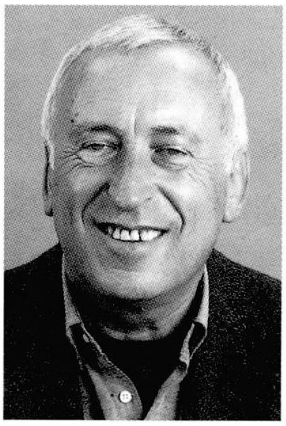
Indirect Methods of Investigation for Evaluating Historic Masonry

Méthodes indirectes d'investigation des constructions anciennes en maçonnerie

Einsatzmöglichkeiten indirekter Erkundungsmethoden an historischem Mauerwerk

Fritz WENZEL
Prof. Dr.
Univ. of Karlsruhe
Karlsruhe, Germany

Martin KAHLE
Civil Eng.
Univ. of Karlsruhe
Karlsruhe, Germany



F. Wenzel is a structural engineer and lectures at the faculty of architecture at Karlsruhe Univ. He is concerned with the diagnosis and therapy of old building structures and participated in the restoration of historic buildings. He has reported on this work iin publications, lectures and seminars.

M. Kahle received his Civil Eng. degree from Karlsruhe Univ. where he became research assistant at the Inst. of Structural Eng. He is working on the investigation of historic masonry by means of non-destructive methods.

SUMMARY

In recent years, the applicability of indirect investigation methods to historic masonry has been assessed and further improved through research projects. On the basis of the experience gained from this research into radar, geoelectric, seismic and ultrasonic testing as well as from its first commercial exploitation, its use can now be recommended. This applies particularly to the evaluation of existing historic buildings and the assessment of their structural condition.

RÉSUMÉ

L'utilisation de méthodes indirectes d'investigation des constructions anciennes en maçonnerie a été évaluée et améliorée ces dernières années. Les expériences réalisées avec les méthodes du radar, de la géoélectrique, de la sismique et des ultrasons ainsi que les premières applications commerciales permettent d'affirmer la fiabilité des investigations indirectes dans le cadre de l'inventaire et de l'analyse de l'état de conservation des constructions anciennes en maçonnerie.

ZUSAMMENFASSUNG

Die Anwendbarkeit indirekter Untersuchungsmethoden an historischem Mauerwerk wurde in den letzten Jahren in Forschungsprojekten erprobt und verbessert. Aus den Erfahrungen bei diesen Untersuchungen mit Radar, Geoelektrik, Seismik oder dem Ultraschallverfahren sowie ersten kommerziellen Einsätzen kann die Empfehlung abgeleitet werden, indirekte Verfahren gezielt im Rahmen der Bestandserkundung und Zustandsanalyse historischen Mauerwerks anzuwenden.

1. INTRODUCTION

In recent years there has been an intensification of research to improve the range and scope for non-destructive and partially destructive methods of investigation and measurement in the field of historic masonry. This has come about by the modification and further development of certain methods, such as ultrasonic, seismic, geoelectric and radar, which are already well established in other areas of science and technology [1]. Interest by the building industry lead to the first commercial exploitation of these methods, and demonstrates their degree of success. Nevertheless, further research and development work is necessary in order to increase the capability of these techniques and thus widen their market potential.

Positive results in the laboratory and field work currently indicate that these techniques can be recommended for the evaluation of existing historic masonry. They allow determination of characteristics and material constants needed for the historic, static and physical analyses. This, in turn, presents important advantages for the architects and engineers involved, such as:

- reasons for existing damage can be detected more accurately, thus facilitating more reliable repair work;
- the stability of constructions can be assessed more precisely;
- measures for the securing, repair, strengthening or alteration of existing historic buildings can be planned earlier and on a reliable planning basis; and
- interventions in the construction can be reduced by the use of less invasive target-oriented methods.

However, these new methods should not be overestimated. They do not offer neat and convenient solutions for each and every problem. In particular, they are no substitute for the intense and indispensible in-situ work at the specific building.

These techniques do not provide stand-alone solutions; they must become part and parcel of systematic, well coordinated evaluations of the existing building. On the other hand, they do supply data needed by the architects and engineers for their analyses and facilitate correct conclusions. As difficulties may arise when the various disciplines have to work together, the paper recommends how to proceed in a sensible and methodical way.

'Non-destructive' is not a neutral term and so it gives rise to problems when it is used, particularly for those involved in the preservation of historic monuments who expect too much of these methods. A 'non-destructive investigation' in absolute terms will be the exception in the future, too, as damage to the construction finally occurs when drilling is carried out in order to countercheck and calibrate the results. Therefore, the advantage of these methods is not that interventions into the construction can be avoided in total, but that better results can be achieved, which in turn facilitates a sensible concept for the necessary repair work with lesser interventions.

Thus the term 'non-destructive', which is quite common in other areas, cannot be defined without difficulty in the sensitive area of investigations of historic buildings. Therefore, the term 'indirect methods' will be used herein as it describes the situation more accurately: physical measurements are being carried out, which only provide an "analogue" of the results and information searched for.

2. EVALUATION OF HISTORIC BUILDINGS, GENERAL APPROACH

Leaving apart the subsoil conditions and urban environment, when planning new buildings, the condition on the site is of lesser importance. The planning can be done in an office. But dealing with historic masonry necessitates intensive in-situ work; the geometry, the load bearing system, the materials and their characteristics are not to be freely choosen by the planner. They are simply given without being obvious in all details; they have to be determined, taken into consideration and should be altered only partially.

Therefore, the basic planning facts are of great importance when dealing with historic buildings. The survey and evaluation of existing buildings are areas which not only need enough time to be dealt with but also adequate financial funds to be made available. "... you will pay to have good investigation, whether you have it or NOT!" (Karl Terzaghi). If this is not taken into account, unpleasant surprises can occur during the work, causing unavoidable changes and alterations, which in turn lead to additional costs, time delays and further destruction of the historic building. Experience from unsuccessful conservation measures have demonstrated quite clearly the importance of detailed surveys and evaluations of existing structures.

When dealing with complex buildings, the appraisal and the assessment of their structural condition should be done step-by-step in a well coordinated way (fig. 1). The indirect investigation methods must be integrated into the overall concept, since the necessary documentation should be available for their use. Also, the investigation aims must be determined beforehand.

In the ideal situation the evaluation of the existing building starts with the historical investigation, which means collecting documents, plans, drawings, photographs, etc. from archives and other sources. In situ, the building researcher depicts the findings which can be derived from the surfaces in writing or by means of drawings or photographies. In the following step, investigations from the field of restoration and building archaeology may be necessary, supported by specific techniques (for example dendrochronoly). The acquired data gives information about the historic and technical composition of the construction. With this knowledge the historical value of the building can be deducted.

The survey of the structure supports the historical investigation as well as all other investigations and analyses. It provides the means to define the geometry of the building and draw the corresponding plans. Measurement by hand, geodetic methods, computer-aided processes and photogrammetry supply different degrees of accuracy. Additional information is included in the plans or documented by photographs.

The survey of damage includes the mapping of visible damage to the structure, for example deformations, cracks, disintegrated areas as well as specific surface characteristics, for example the detachment of flakes, sanding, discolouration and efflorence. Need may arise to trace the development of the damages in the course of time by continuous or periodic monitoring.

Structural investigations are directed at the composition and structure of the parts. Very often the visual inspection of the surface informs about the materials used and the binding. In order to gain information about the interior, i. e. possible voids, multiple leafs or discontinuities, interventions into the building are necessary, e. g. drillings. In this context, indirect methods offer a non-destructive or only partially destructive way to gain data covering huge areas.

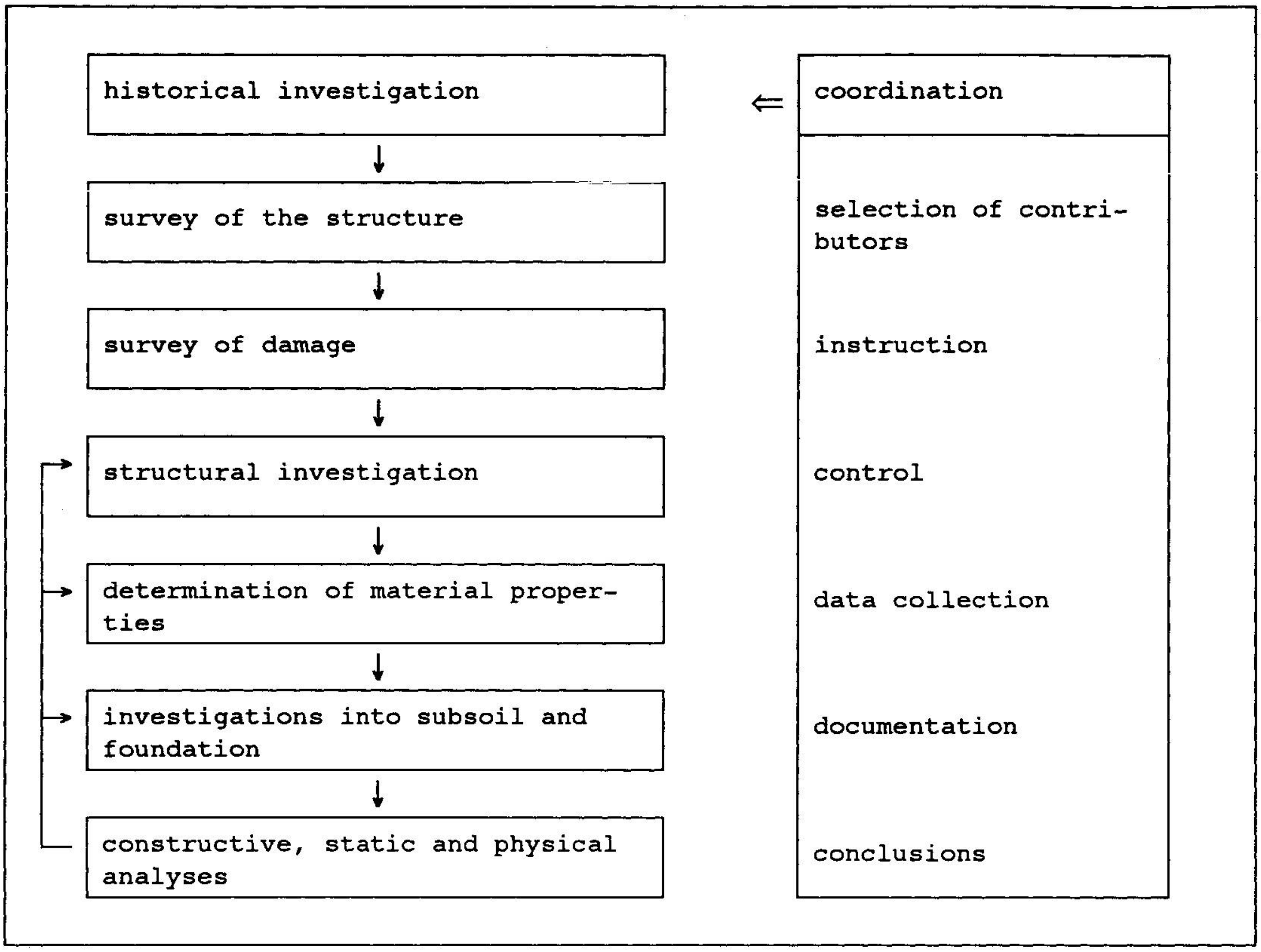


Fig. 1: Survey and evaluation of historic buildings

In order to determine material constants, e. g. strength and deformation parameters, it is necessary to take samples, usually by drilling, from the surface or the interior of the component. In addition to the mineralogy, it may be necessary to determine the concentrations of detrimental salts, and also the hygrical characteristics, e. g. equilibrium and saturation moisture, as well as physical and mechanical constants.

The investigations into subsoil and foundation are used to establish the nature, geometry, structure and condition of the total foundation system, including the ground water regime, together with the relevant properties.

The data obtained serve as the basis for constructive, static or physical analyses. They aim to determine the reasons for damage, describe the bearing system, assess the structural strength as well as to investigate into the moisture and heat balance of the building. In addition, the repercussions of the planned measures, which might bring about alterations in the structure, applied loads or environmental influences, have to be checked. When the required data are not available, further in-situ investigations of a more precise and more extensive nature have to be carried out.

The coordination of the investigations should be done at a central place. It is also there that the conclusions are drawn from the evaluation of the building, in order to formulate a rational concept for the securing, repair, strengthening or alteration of the historic building.

3. UTILISATION OF INDIRECT METHODS OF INVESTIGATION

3.1 Preconditions

Indirect methods can be used for investigations into the building structure, the determination of material constants as well as for historical surveys and probe the subsoil and foundation. When applying them, exact surveys of the parts should be available as they are needed for the preparation, execution, evaluation and documentation of the investigation. In addition, it should be distinctively clear what the aims of the investigation are, i. e. investigation purpose and measurement area must be defined exactly. Taking into account the sequence of surveys, this may only be possible at a later stage, indeed, it may be after the constructive, static and physical analyses have shown, that it was not possible to obtain the necessary data by conventional methods.

A further precondition for the utilisation of indirect methods must be that free access to the area concerned be granted and that impediments due to other investigations or construction work be avoided. In addition, the weather conditions have to be taken into account; in particular the radar investigations are improved by dry weather conditions, as the quality of the data increases with reduced absorption.

3.2 Users of indirect methods

In general, indirect methods will be applied by specialized technical firms or institutions. One reason for this is the necessary equipment, involving high investment, another is the know-how which is necessary for the operation of the equipment and the evaluation of the data collected.

This has lead to a situation in which one more specialism enters the field of investigations into historic buildings while at the same time the other parties involved in the evaluation of the construction are unfamiliar with its possibilities and limits. Those who apply the indirect methods mostly come from other backgrounds (e. g. geophysics) and thus some difficulties of communication are almost inevitable.

The success of investigations carried out by means of indirect methods mainly depends on the equipment available to the operators, but also on their experience with the equipment in the context of historic buildings. These are the criteria that should be considered foremost when comparing alternative offers with possible price differences of a lesser importance.

3.3 The investigation concept

The expectations from the use of indirect methods have to be clearly defined, i. e. the engineer or architect responsible has to state exactly which information he needs at a specific point.

The user has to evaluate whether the data in fact can be obtained, which conventional or indirect method should be employed and estimate the cost of the investigation. Experience has shown that it is quite difficult to describe the problems and the surrounding aspects by verbal means or through plans or photographs. It is always recommended to make an on-the-spot inspection of the building.

The specialist operator and the engineer or architect respectively need to cooperate in formulating a concept for the investigation, which has to be agreed to by those responsible for the coordination of the project.

In many cases it will be possible to limit investigations to smaller areas thus also limiting the investigation costs. Here considerations are necessary as to what extent any findings can be transferred to other areas. If investigations into larger areas are necessary, a two-step approach is being recommended. A typical sample area is chosen in order to test the applicability of the procedure, to optimize the measuring programme, to calibrate the data and to check the results. The parameters derived serve the investigation into larger areas in an efficient way.

3.4 Preparation of the measurements

After the order has been received, the operator converts the investigation concept into a measurement programme which determines the most important parameters, i. e. the measurement arrangement, the direction of profiling or the measurement grid. In addition, it has to be decided whether the investigation will be carried out using a scaffold, a ladder or a lifting platform and what might be a suitable point in time for the measurements.

3.5 Performance of the investigation

Additional measurement parameters are being determined at the building itself. The first data acquired should be examined thoroughly and a preliminary evaluation of their reliability should be carried out on the spot in order to alter the measurement arrangement when necessary. These checks should be carried out in between the measurements in order to detect changes in the measurement conditions in an early state.

The measurement points or profiles must be related to a system of coordinates, generally, with one axis horizontal and one vertical. They in turn must be referenced to geodetic fix points or distinctive points of the building noted in the plans, as this is the only way, that the information can be reconstructed by anybody concerned.

Very often supplementary work is necessary for the data evaluation. It is important to note down characteristics that can be directly detected from the surface of the parts, so as to correlate them with possible measurement anomalies. If the existing plans are not accurate enough, it may be necessary to measure the width of certain parts in order to calculate the velocity of mechanic or electromagnetic waves.

3.6 Evaluation of the data obtained

In general, the evaluation of the data obtained is carried out in the office and takes two or three times the time necessary for the measurements themselves. It comprises the data processing, the presentation of the data in diagramms or tables, the data evaluation and the interpretation of the results.

An investigation report should be drawn up containing the most important basic findings. Such a report comprises a short description of the methods used, a list of the measurement and evaluation parameters, a presentation of the measurement data (preferrably in the annexes) as well as a verbal and graphic presentation of the overall results. It should be clearly stated on what basis the interpretation of the data was carried out and to what extent the statements made in the report are reliable. The most important results and conclusions should then be summed up in a clear and understable form for all those concerned, particularly the non-specialists.

3.7 Significance of the results

By definition, indirect methods do not bring about the results needed directly, but only physical data that can be related to the parameters sought. However, in most cases there is a dependency between the measurement data obtained and several parameters, and this is the reason that unmistakable statements are rarely possible. The situation can be improved by using several indirect methods in parallel and/or by drilling to control and calibrate the results.

If the user of indirect methods have broad experience with the investigations of historic masonry, they will base their work on a sensible investigation concept and carry it out with the necessary care. In this case, the results will be reliable. But first of all, the concept has to be agreed to, i. e. the necessary funds have to be allocated, the investigation should be carried out without constraints and the necessary samples have to be taken.

4. INDIRECT METHODS FOR EVALUATING HISTORIC MASONRY

4.1 Preliminary remarks

Radar, geoelectric, magnetic, ultrasonic and seismic testing methods along with infrared thermography and various moisture meters are among the available tools for which there is a stack of experimental as well as practical experience in the field of historic masonry. However, further research work is necessary in order to solve still existing problems and develop equipment and methods. Therefore, the following findings and statements are not conclusive.

4.2 Radar method

Radar is a powerful and versatile method of evaluating existing masonry in an efficient and comprehensive way [2, 3, 4, 5]. Pulses of electromagnetic waves in the frequency range of 100 to 1000 MHz are being directed into the structural part via a transmitter which is located at the surface of the part. The waves propagate through the part with a specific velocity which is determined by the material. At interfaces of different electrical properties, some waves are reflected and the remainder is transmitted. The signals are picked up by a receiver, amplified depending on the delay time, transmitted to the microcomputer and finally displayed in radargrams on the screen. The reflection arrangement is used in order to investigate the structure of a part (fig. 2). Multiple leaf masonry or facing walls are being detected either by the reflections due to leaf boundaries or by the different reflection patterns of the leafs. The reflections from leaf boundaries, which have separated, are characterized by significantly higher amplitudes. Voids can be detected as long as their dimensions are of structural importance. In case of discontinuities due to enclosed metals the resolution of radar is even higher, but often not sufficient to detect modern metal ties in the middle of a wall. Disintegrated, porous or sagged zones can be detected because of their increased reflectivity.

The moisture content of masonry materials is determined on the basis of their relative permittivity which in turn is calculated from the velocity of the electromagnetic waves when penetrating the part. In general, these calculations necessitate a transmission arrangement consisting of transmitter and receiver, which means that both surfaces of the structural part must be accessible. Because of the high measurement volume, smooth lateral moisture profiles do result that present the lateral moisture distribution in a realistic way.

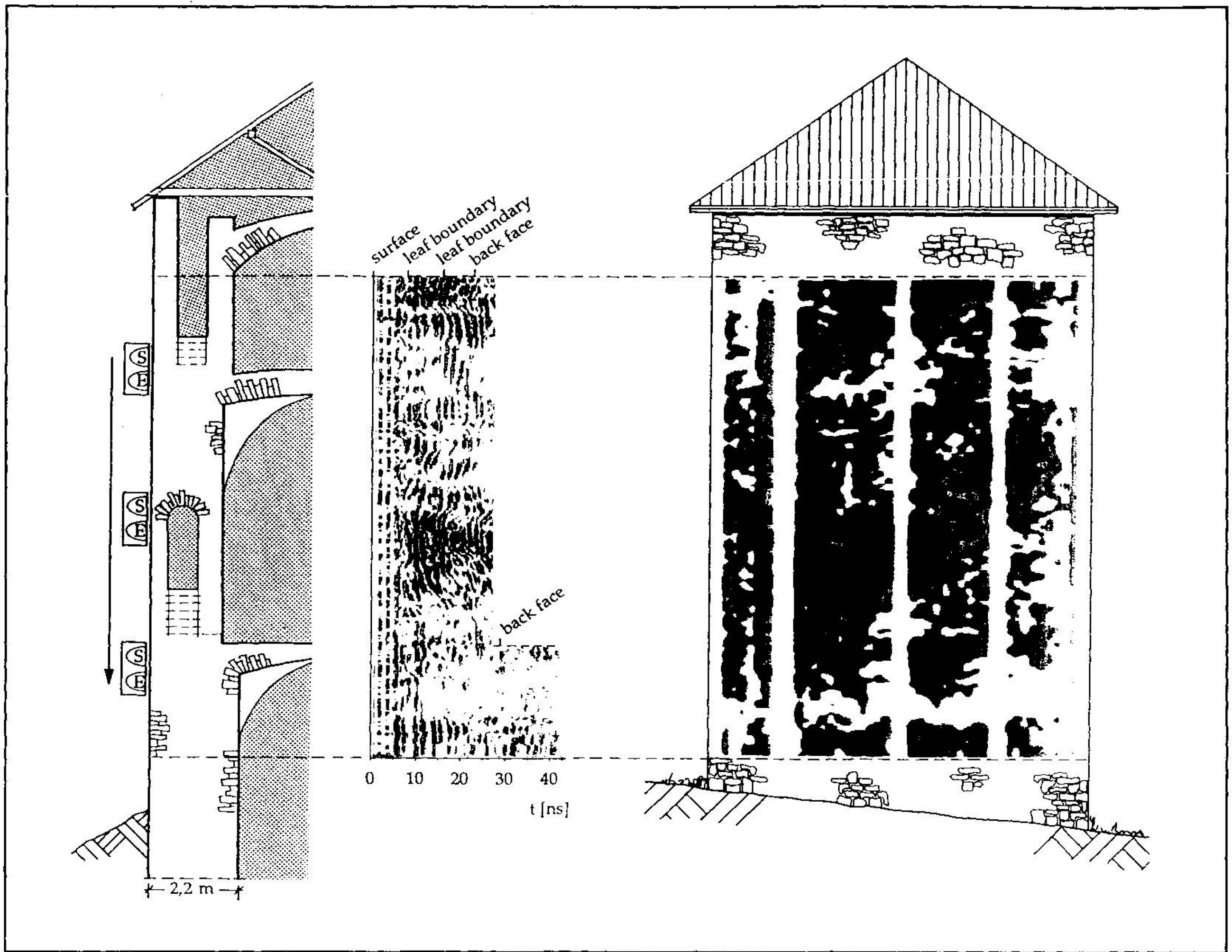


Fig. 2: Radar investigation on the wall of a medieval tower; left: cross section and radargram, right: view of the wall with radar data

4.3 Geoelectric method

In the geoelectric investigation method, an electrical direct-current field is induced in the test zone by means of two or more electrodes. The current within the circuit of the equipment, the voltage between the two probes (which pick up the potential difference at the surface) and a specific factor, which is derived from the configuration used, are the parameters from which the specific electrical resistivity of the material is calculated. The value depends mainly on the two parameters 'moisture content' and 'concentration of dissolved salts' [4, 6, 7, 8].

Therefore, it is not acceptable to state quantitative moisture contents directly, even if this is done in the case of moisture meters (see 4.8). However, it is possible to detect moist zones by mapping the specific electric resistivities, e. g. by using the Wenner arrangement as shown in figures 3 and 4.

With the geoelectric method it is also possible to investigate into the structure of a building, above all with regard to multiple leafs and voids. However, the resolution is quite low compared to radar.

A special variant of the geoelectric technique, the self-potential method, is based on the principle that iron corrosion generates an electric field that is picked up by two probes [9]. This method is very suitable for the detection of active corrosion processes. However, precondition is that the location of enclosed metals is known and that one of the probes is placed directly at it.

Fig. 3: Investigation with the geoelectric method

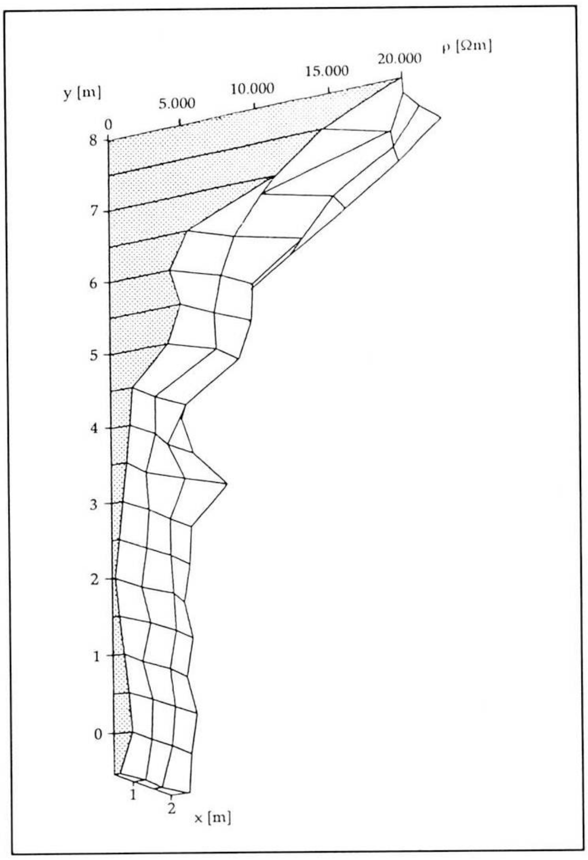


Fig. 4: The distribution of the electric resistivity ρ

4.4 Magnetic method

Magnetic methods in various forms can also be applied to historic masonry in order to detect the presence of enclosed metals. A simpler equipment used to detect the location and diameters of steel bars in reinforced concrete structures work on the basis of permanent magnetic or inductive methods. The pulse induction method is used for the detection of metals enclosed in greater depth. With this method, a pulsed electromagnetic field induces an electric current in metals and the mutual induction caused by this current is picked up by a receiving coil.

The geomagnetic methods used in geophysics measure the earth's magnetic field which is locally distorted by masses that can be magnetized. This is a possibility to detect enclosures that are near to the surface but not for detecting longitudinal ties in the middle of thick rubble walls (fig. 6).

4.5 Infrared thermography

With the infrared thermography, the heat radiation emitted by the surface of the building - the intensity of which is dependent on the temperature - is being registered. Thus anomalies with regard to the temperature can be detected and presented fast and without even touching the masonry, i. e. without the need of a scaffold etc. A precondition is, that there is a heat flow through the part. This method has been successful in those cases where there are differences in material obscured by plaster, e. g. closed openings, joints, carcassing timber. In addition, humid areas can be located [10, 11].

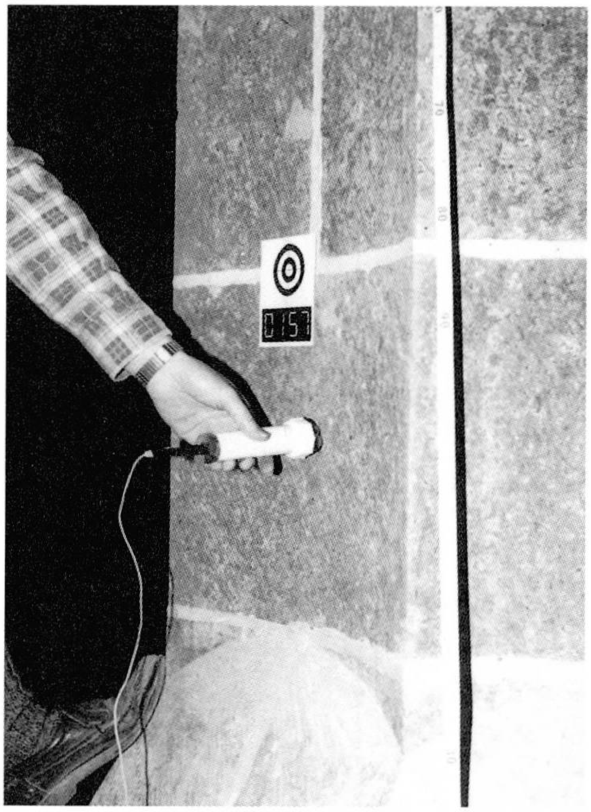


Fig. 5: Investigation with the self-potential method

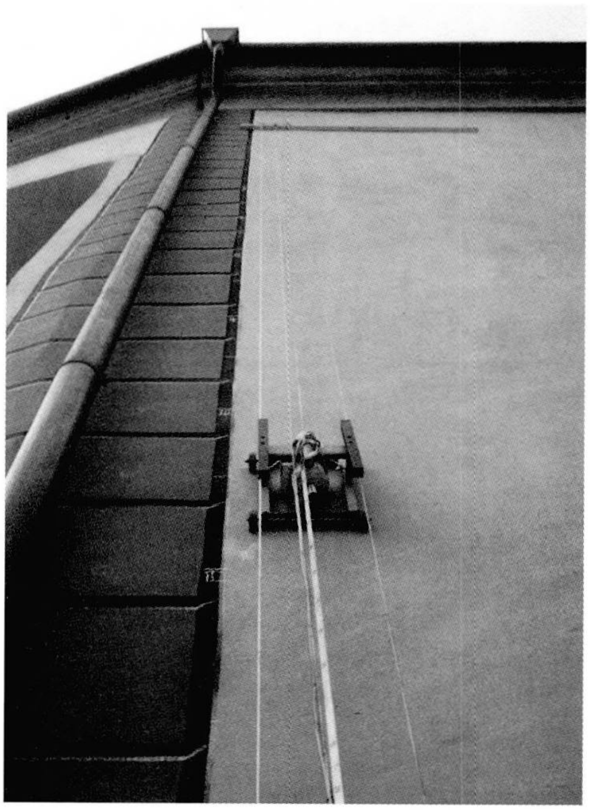


Fig. 6: Investigation with the geomagnetic method

4.6 Ultrasonic method

With the ultrasonic method, mechanic waves with frequencies higher than 20 kHz are generated by means of piezoelectric or magneto-strictive transducers, and picked up after passing through the part that is to be measured. However, only masonry up to a thickness of 1 m can be inspected with this method, limiting structural investigations to thin or monolithic parts [6] (fig.7). In all other cases, the seismic method is used as the pulses propagate far enough because of the higher signal intensity and the lower frequencies, causing less absorption.

Further research work is necessary to increase the knowledge as to whether measuring the velocity of ultrasonic waves allows statements about the strength of stones or masonry (fig. 8). While the relation between the velocity of elastic waves and material properties has been proven physically for homogenous materials, the transformation into a measurement method that can be used under real conditions has not yet taken place [12, 13, 14, 15, 16].

4.7 Seismic method

With the seismic method, mechanic waves in the frequency range between 300 Hz and 3 kHz are generated at the surface of the structural part, preferrably with a hammer. They propagate through the part and are picked up by one or more receivers. The development of reflection methods is still being worked on, therefore only the transmission arrangement will be discussed here. With this arrangement, the propagation time of the longitudinal wave, which is the fastest of the various kinds of waves, is measured between the transmitter and the receiver(s). The wave velocity calculated from this value depends on the one hand on the material properties, on the other hand also on voids, for example open or unfilled cross joints. Such anomalies cause deviations of the wave propagation and therefore a seemingly lower velocity. The seismic method permits a differentiation between areas of different materials or structures, e. g. multiple leaf masonry, filled openings, joints. As the measurement results are related to the mechanic properties, this method is often of great help. In addition, large voids can also be detected [6, 17]. The seismic has also been used for checking the effectiveness of injections aimed at the improvement of damaged masonry [18].

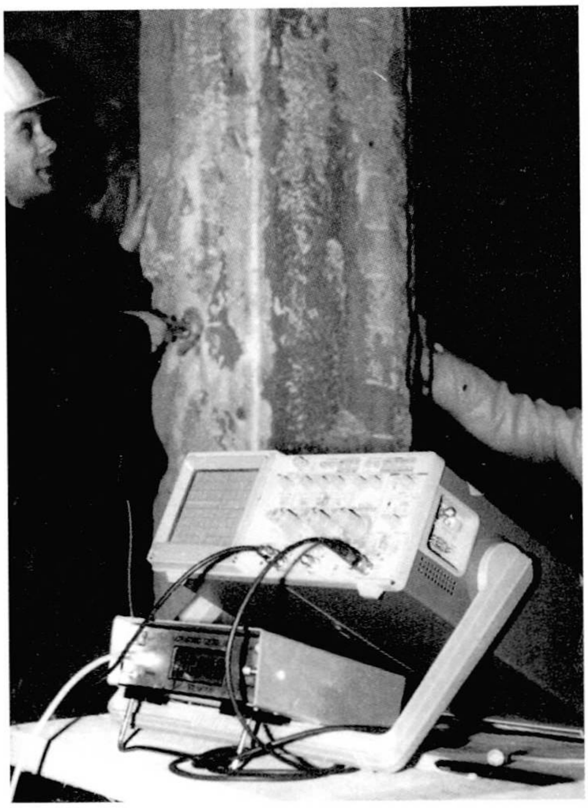


Fig. 7: Investigation of a sandstone pilar with the ultrasonic method

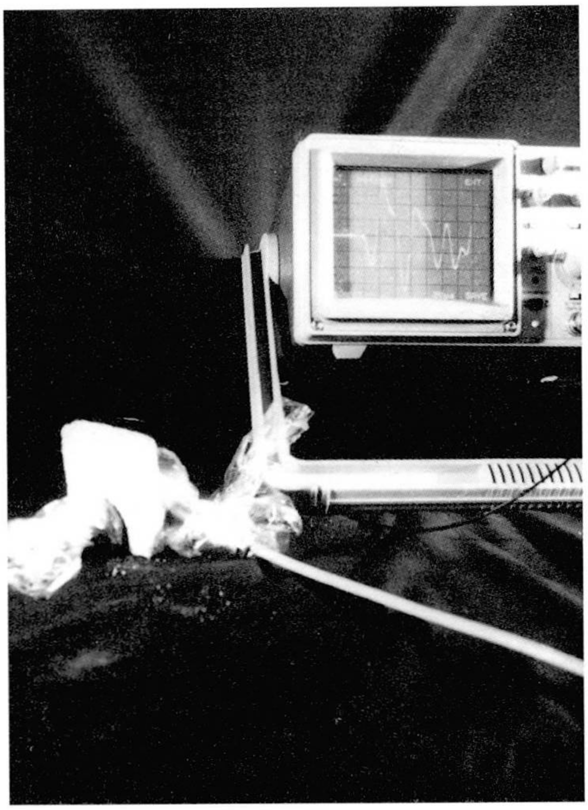


Fig. 8: Ultrasonic measurement to determine the mechanic properties of mortar

4.8 Moisture meters

Various non-destructive methods for measuring from the surface of masonry the moisture contents of materials are offered on the market and being used.

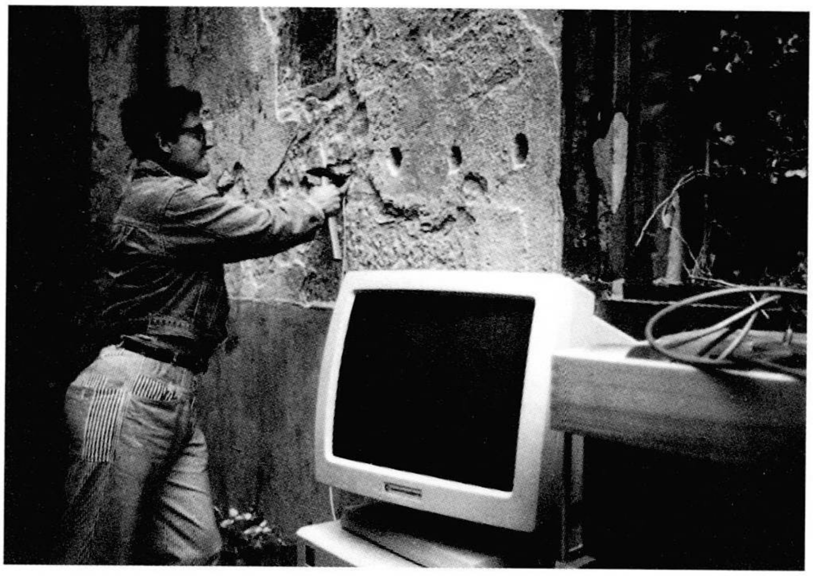


Fig. 9: Investigation of a wall with the seismic method

In contrast to the geoelectric method with four probes, the conductivity meters comprise only two electrodes which generate an electric field within the part and also pick up the potential difference. Therefore, there are two reasons why this equipment is not suitable for historic masonry: on the one hand, the high transition resistances between the electrode and the part itself falsify the measurement values to quite an extent; on the other hand, the specific electric resistivity does not only depend on the moisture content, but also on the concentration of dissolved salts [4].

With the capacitive technique, the relative permittivity of the material is determined using stray field capacitors. If the measurement frequencies are sufficiently high, the permittivity mainly depends on the moisture content. However, the units on the market only offer a penetration depth of a few centimeters, which is too shallow. In addition, the electrodes must be placed on the surface with very high precision in order to avoid measurement mistakes.

Neutron meters emit fast neutrons which are slowed down mainly by collisions with hydrogen atoms. The slow (thermic) neutrons are counted with counter tubes and the detection rate serves to derive the moisture content [19, 20]. Up to now, the applicability of neutron meters to masonry has not been investigated in a systematic way but experience indicates that it can be used. Disadvantages are the use of a radioactive radiation source (which has to meet legal requirements), the influence of chemically bound water on the measuring results and the relatively shallow penetration depth of approximately 10 cm.

4.9 Combinations of indirect methods

In many cases, it will be reasonable to combine indirect methods in order to get more reliable results. When doing so, the additional expense is not very high and can well be justified, if significantly better results are expected.

The combination of radar with other methods can be recommended in many cases. Radar is used for the complete area as it is very versatile, fast in picking up data and of high resolution. For checking and improving the interpretation of the data, an additional method, possibly with only few measurement points, can be used. The following gives three examples for such a combination.

The radar method is well suited for the detection of leaf boundaries, but it offers no statement concerning the characteristics of an inner core in the case of multiple leaf masonry. This can be achieved in the following way: By means of the seismic method, the propagation velocity of the elastic waves in the masonry is measured, while the velocity in the stone is measured at the surface or on drill cores using the ultrasonic technique. If the two values differ significantly, this serves as a clear indicator for the existence of a rubble infill.

Quite often, anomalies that were detected with radar cannot be classified. Magnetic methods, in some cases also the geoelectric method, can be used for checking as to whether there are enclosed metals. In addition, the self-potential method can be used to detect active corrosion.

If radar is applied for the determination of the moisture contents, the geoelectric method can be used to measure the specific electric resistivities, thus allowing statements about the concentration and distribution of dissolved salts. However, the use of two indirect methods influences the accuracy of the results, which should then be checked by sampling.

4.10 Taking samples

In general, samples have to be taken in order to control and calibrate the results obtained via indirect methods. If necessary, they must be inspected and further examined. The extraction procedure is dependent on the required form and dimensions of the sample, the necessary depth of the intervention and other factors. In most cases the taking of drill cores will be chosen, but for the determination of the moisture content of masonry it may be sufficient to take drilling debris gained with a spiral drill. In very few cases individual stones or mortar taken from the masonry serve as samples.

The samples are visually inspected and, if necessary, tested in the laboratory in order to determine ultrasonic velocities, mechanical properties, natural or saturation moisture content, hygroscopic moisture, salt concentrations, etc.

In addition, the drilling holes can be inspected with an endoscope, in particular, if the information required cannot be gained from the cores.

5. THE USE OF INDIRECT METHODS - COSTS AND BENEFITS

5.1. Costs

The time required for the measurements depends on several factors. Roughly estimated, an area of approximately 50 sqm can be covered with one method in the course of one day; with radar it can be twice the size. The evaluation and documentation of the investigation, which in general are carried out in an office, take two to four times the time needed for the measurements.

The prime costs of the investigation derive from the time needed for the preparation, performance, evaluation and documentation of the measurements. Additional costs arise from travelling, accommodation, scaffolds or lifting platforms, as well as from additional sample taking and examination. Costs for investigations that necessitate one day of measurements amount to DM 3,000 to DM 6,000. In many cases, this will be enough to clarify questions, but it can be well above this sum.

5.2. Benefits

A great advantage of the indirect methods is the possibility of non-destructive measurements. However, interventions into the masonry will be necessary in the future, too, in order to check and calibrate the results. On the other hand, fewer samples are necessary under these circumstances and they can be taken in a very target-oriented way.

The decisive advantage of indirect methods is due to the fact, that relevant information on an extensive area of the building is gained and no important features are missed. This has the following advantages:

- The reasons for existing damage can be determined in a reliable way. Once they have been dealt with, the damaged areas can be repaired. The reoccurence of damage can be avoided.
- Risks due to structural failure or desintegrating ornamental parts can be assessed. Necessary measures can be initiated.
- The structural strength of existing buildings can be assessed in a more reliable way. Strengthening can be limited to the parts where it is unavoidable.
- Construction measures can be initiated early and on a sound planning basis. Thus, the necessary decisions can be taken and mistakes be avoided.
- New target-oriented and less invasive repair methods, for which the knowledge of certain characteristics is needed, can be implemented.
- The success of measures can be checked by later investigations.

The advantages mentioned above are in parallel with reduced interventions into the historic masonry and savings during the works as well as lower follow-up costs due to the employment of inappropriate measures.

5.3 Decision making

In many cases, the costs of an investigation with indirect methods, plus an additional sampling, will be much lower than other savings, achieved due to the elaboration of an optimized working concept for the securing or repair of the historic building. However, very often it is difficult to convey this to the client and others concerned. The reasons for this are, that on the one hand a new thinking must prevail with regard to the necessity of an extensive evaluation, and on the other hand, clients do hesitate less to spent sums of this order of magnitude for apparently necessary construction work than for an investigation, the necessity of which in their understanding is not so obvious.

However, apart from the cost-benefit analysis, there are also other criteria for a decision in favour of the use of indirect methods:

- specifications or requirements imposed by the Monument Protection Authority with regard to interventions into the masonry in the course of the investigation or the carrying out of construction work;
- requirements expressed by architects, structural or proof engineers who need detailed information when evaluating the masonry and planning construction work;
- requirements imposed by official bodies in case of risks due to structural failure or disintegrating ornamental parts.

6. CONCLUSIONS

Indirect investigation methods have proved their applicability to historic masonry in the laboratory as well as in the field. However, further research is necessary. When evaluating existing historic buildings, these methods bring about important results which allow a reliable assessment of structure and condition of the building. Thus, it is possible to determine the cause of existing damage, and sensible concepts for the securing and repair of historic masonry can be developed. The planning of these measures can be initiated early and on a more reliable basis. Fewer interventions into the masonry are necessary. Costs can be cut through sound planning and reduced target-oriented meaures, so that the use of indirect measures becomes an economic option.

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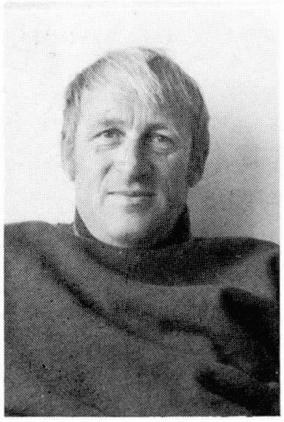
Historic Preservation - High and Low Tech Diagnostic Technology

Conservation de monuments historiques, méthodes d'analyse sophistiquées et simples

Historische Bauwerkserhaltung - hochentwickelte und einfache Diagnoseverfahren

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

Carolyn L. SEARLS
Consultant
Wiss, Janney, Elstner Assoc.
Emeryville, CA, USA



S. Thomasen has a Struct. Eng. degree from the Techn. Univ. of Denmark, while with WJE, he has investigated building failures and material deterioration, taught at California Polytechnic in San Luis Obispo and designed large projects.

C.Searls has a BS in Architectural Eng. from California Polytechnic in San Luis Obispo, a MS in Struct. Eng. from UC Berkely. She has done field investigations and analysis of structural failures, evaluation and rehabilitation of masonry, marble and terra-cotta facades.

SUMMARY

A key to preservation and continued use of buildings is an accurate assessment of the existing structure. Historically, the engineer has relied on traditional investigative procedures, experience and intuition. Lately, sophisticated technology has become available for use in diagnostic examinations, but its use requires understanding its limitations. Best results are often achieved by a synthesis of the art of structural engineering and the science of modern diagnostic and analysis techniques.

RÉSUMÉ

Une solution pour la conservation et l'utilisation permanente d'un bâtiment dépend de l'évaluation précise de l'état de la structure existante. Dès le début, l'ingénieur s'est basé sur des procédures d'études traditionnelles, son expérience et son intuition. Plus récemment, pour l'examen de l'état de structures, une technologie sophistiquée est devenue possible, mais son emploi requiert la compréhension de ses limites. Les meilleurs résultats sont souvent obtenus par la synthèse de l'art de l'ingénieur civil et la science d'un examen méthodologique moderne et des techniques d'analyse.

ZUSAMMENFASSUNG

Einer der Schlüssel zur Erhaltung und weiteren Nutzung von Gebäuden ist die richtige Einschätzung des bestehenden Bauwerks. Von alters her hat sich der Ingenieur auf traditionelle Verfahren, seine Erfahrung und Intuition gestützt.. In neuerer Zeit ist eine hochentwickelte Technologie für diagnostische Untersuchungen verfügbar geworden, aber der Gebrauch dieser Technologie erfordert ein Wissen um deren Grenzen. Die besten Resultate werden oft durch eine Synthese der Ingenieurbaukunst und der modernen wissenschaftlichen Diagnose- und Analysetechniken hervorgebracht.

1. INTRODUCTION

A key element in planning for the preservation of a building is the correct assessment of the existing structure based on results from a diagnostic analysis. To make this assessment, the engineer searches for patterns and measurable conditions. Historically, the engineer has relied on traditional investigative procedures, simple analyses and a mixture of experience and intuition to make the assessment. A structural engineer faced with the task of preserving a historic structure requires qualitative information for the general evaluation of the structure's condition and quantitative data for the engineering assessment. Traditionally, qualitative evaluations have been by visual inspection, supplemented by field and laboratory tests, while quantitative assessment has required destructive testing and removal of material specimens for laboratory analyses. Lately, sophisticated modern technology has become available for diagnostic examination and analysis of historic structures, but the use of such technology has not always improved the quality of the investigation. This article describes a few high tech and low tech systems and when they may be appropriate.

2. SELECTION OF DIAGNOSTIC TECHNIQUES

The condition evaluation and structural assessment of a historic building requires a carefully planned program of exploration and testing. Some of the factors that influence the choice of appropriate diagnostic techniques are:

- The historic value of the structure and the effect of any destructive testing on the historic materials must be considered. Destructive testing may be acceptable in less exposed areas of a structure, such as a closet in a church or the inside face of a spandrel beam at an arch bridge, but for most investigations nondestructive testing is preferred.
- The practical experience and technical expertise of the investigator. High-tech equipment often produces a lot of data, and requires an expert operator to interpret the readings.
- The cost of high-tech diagnostic technology versus the benefit from the investigation. For example, hourly computer readings of a dozen climate controls installed in a historic structure can generate an impressive amount of data in a year, but the benefit might not justify the cost of maintaining the system and sorting the results.

3. VISUAL INSPECTION AND DOCUMENTATION

3.1 Visual Inspection

Inspection and documentation of existing conditions is used in detecting patterns of failures, designing repairs and identifying areas for further close-up examination and testing. A visual survey of the structure, usually from grade using binoculars, is supplemented by close-up examination of selected areas. Close-up inspection can be performed from stationary or from suspended scaffolds, telescopic truck-mounted lifts, and from specially built snooper scaffolds for inspecting the underside of tall bridges. When more ordinary types of scaffolding are inappropriate or too expensive, modern mountaineering techniques can be applied to the inspection of high rise buildings, large domes, and towers.

3.2 Documentation Techniques

Sketches accompanied by photos are often the most effective documentation technique.

Recording existing conditions on the sketches allows the investigator to assimilate information about the structure and can lead to insights about its behavior.

Drawings of small simple structures can be made by hand measuring techniques. Rectified photography, where a metric camera produces an orthophoto, is used to make exact scale drawings. If less precision is acceptable, a hand operated semi-metric camera will deliver good results in much less time.[1] When the drawing does not have to be to an exact scale, a low tech alternative is to enlarge a 35 mm negative to the format of a baseline drawing to indicate existing damage and required repair (Fig. 1).

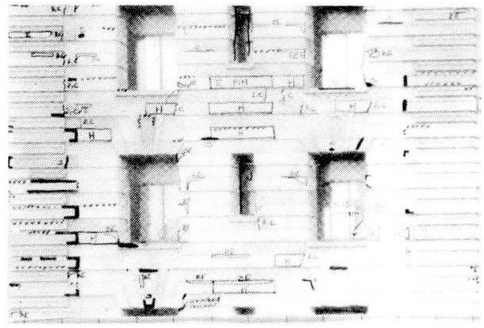


Fig. 1 - Repair drawings made from photo

Macro stereo-photogrammetry has been used to record the weathering of building stone.[2,3] The techniques include depth measurements between weathered and unweathered surfaces, metal plugs set in the stone as a reference surface, the assessment of the depth of inscriptions remaining on tombstones, and comparative thickness measurements made with calipers.[2]

For the investigation at York Minster, the observed cracks were drawn on a plexiglas model of the church, allowing correlation of interior and exterior crack patterns and allowing the cracks to be viewed in three dimensions.[4] A similar high tech technique can be utilized by plotting the crack pattern on a computer model of the structure.

Several nondestructive techniques, described later in the paper, are used for documentation of an historic structure. They include pachometers for locating embedded steel, X-ray and gamma radiography for photographing the interior of a structure, and the use of soniscope and impact echo tests for detecting internal flaws.

3.3 Inspection Openings

Inspection openings are destructive, but they are also the surest, and often the only way, to verify the condition of the internal parts of a structure. Sometimes the inside of a structure can be observed in a less destructive manner by using a fiberoptic borescope. The instrument's light transmitting wand is inserted into a 12 mm diameter drilled hole. The interior can then be viewed through the eyepiece on the end of the wand. The observations can be documented by a camera that attaches to the eyepiece (Fig. 2).

Fig. 2 - Fiberoptic Borescope

A veterinarian's portable x-ray unit, designed for diagnosing cows in the field, was used to document the structure of wooden plank houses. The portable x-ray system provided a fast, nondestructive technique to record the width of the planks, the location of nails, and the details otherwise hidden by finishes.[5]

4. MONITORING STRUCTURAL MOVEMENTS

4.1 General

Structures are monitored, either continuously or intermittently, to detect and quantify structural movements. Monitoring is done when deformations gradually increase with time, and could eventually result in damage or collapse. Monitoring is also used in determining the cause of observed distress and the appropriate intervention.[6]

Before selecting a diagnostic technique, the purpose of the monitoring and the use of the information from the monitoring should be considered. High tech monitoring techniques often create copious amounts of data, and the analysis can be very time consuming unless it is well planned from the start.

4.2 Crack Width

It is often important to determine if the width of a crack is constant or if it has daily, seasonal or long-term changes. In many investigations, a dab of nail polish across the crack is an appropriate technique for finding out if the crack is dormant. For larger cracks, an inexpensive plastic tell-tale can be placed across the crack, or a deformeter can measure deformation between two set points across the crack with an accuracy of 0.0025 mm. Cracks can be measured continuously with a gage that records movement as scratches on a replaceable brass button, which can be read with a calibrated microscope, and electronic or electromechanical gages, connected to a computer on site or in the engineer's office, can be used to trace movement continuously or at preset intervals.

4.3 Total Movement of a Structure

The traditional technique for monitoring total movement of a structure is to periodically survey targets which have been placed on the structure.

The investigation of the Cape Hatteras Lighthouse used an electro-optical device (EOD) to measure horizontal motion of the tower relative to its base. With the EOD installed at the base, the horizontal motion was monitored by tracking a Halogen light source mounted 45 m above the base. The EOD resolved motions into north-south and east-west deflections which were plotted on a strip chart recorder. A low speed strip chart recorded long-term, low frequency response to solar radiation, while a high speed chart, triggered by a pre-selected wind velocity, recorded high frequency responses to wind.[7]

5. CONCRETE CONDITION SURVEYS

5.1 Delamination of Surface Layers

Thermal strains, freeze/thaw cycles, moisture or salt movement within concrete, and corrosion of embedded metal can delaminate large surface areas. The flaws are seldom detected by visual observation, but are easily found by acoustical testing or, as it is commonly called, tapping with a rubber or wood mallet. On large concrete slabs, this low-tech test is done by dragging a steel chain across the slab while listening for hollow sounds. The pulse-echo test, which transmits a stress wave into the material and measures the time of reflection, is useful for mapping surface delaminations in large areas of fairly homogenous materials such as concrete, but is less successful in materials with many internal voids, as is often found in stone and masonry.

5.2 Structural Properties

Quantitative data for evaluation and engineering analysis has traditionally been obtained by laboratory testing of core samples removed from the structure. Many samples are required for a representative picture of the varied conditions found in a historic structure. The procedure is expensive and the visual scars left in the historic fabric are objectionable. However, testing core samples is an established standard procedure, the results are well understood and can be correlated with many different material properties.

Several nondestructive tests of concrete provide broad coverage of large areas for a low cost. These tests require compression testing of a limited number of companion core samples to calibrate the non-destructive tests. The rebound hammer, or Schmidt Impact Hammer, is fast and gives an estimate of the surface hardness, and therefore compressive strength, but the test is most useful for detecting suspect locations that warrant further investigation.

Pulse-velocity measurement with soniscope is used for locating internal defects in concrete and masonry. A signal transducer is placed on one side and a signal-receiving transducer directly opposite on the other side of the test member. Velocity measurements indicate relative strength, modulus of elasticity and presence of cracks and voids. The alignment of the transducers is both critical and time consuming.

In the impact-echo technique, a short stress pulse, introduced into the structure by striking the surface with an impact hammer, is reflected from external boundaries and from internal discontinuities, such as cracks or voids. A transducer, mounted on the same surface as the impact, measures the motion in the time domain and this signal is sorted into frequencies by a Fast-Fourier Transform (FFT) analyzer (Fig. 3). The technique is used for locating internal flaws, measuring thickness of concrete members where only one surface is accessible, and for evaluating concrete strength. The relationship between propagation velocity and concrete strength is influenced by many factors, including mix design, moisture content and age of concrete, and a calibration curve must be established for each tested material. The impact-echo technique was recently used to estimate the concrete strength in the piers of an eleven arch concrete bridge in California. Strength information was required for seismic analysis of the 80 year old bridge. The concrete surface of the massive piers was soft and deteriorated. The impact-echo testing provided many measurements quickly and inexpensively. The results reflected the more useful average strength of the concrete rather than the site specific values obtained from traditional core testing[8].

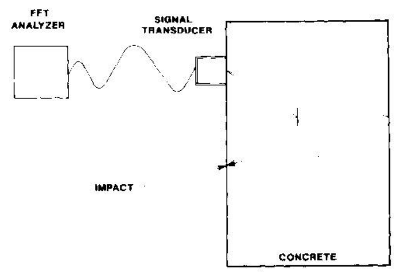


Fig. 3 - Impact Echo Technique

5.3 Locating Embedded Steel

Embedded metal items can be located with a pachometer, a metal detector that measures disturbances in the magnetic field at the surface. The pachometer is calibrated to directly read bar sizes versus distance from the probe. The radar or microwave technique utilizes electromagnetic or radio waves to detect internal discontinuities and to locate reinforcing steel. The pachometer and the radar and microwave techniques are most useful when the steel is not too congested and is located fairly close to the surface.

Gamma radiography and X-ray radiography can be used when the steel is deeply embedded, or where several steel sections are close together. The technique provides a projected picture of the section on a sensitive film, placed on the opposite side from the radiographic source. The systems are portable, but the setup, especially the alignment of the film on the opposite face of a wall from the radiographic unit, can be time consuming. Special safety features are also required to avoid harmful exposure, and in some localities special licenses are required for the transportation and use of the equipment.

5.4 Corrosion of Embedded Steel

The pachometer is used to locate the steel, but it provides no information about the condition of the steel, such as corrosion. The copper-copper sulfate test will detect active corrosion by measuring the electrical resistance and the potentials between the steel and the concrete surface. This isopotential test is performed by connecting one electrical lead of the copper-copper sulfate half cell system to the embedded steel. The presence of active corrosion can now be detected by placing the other lead, which has a porous plug and a copper rod immersed in copper solution, on the surface of the concrete. A large surface area can be tested from one setup, since most structural framing and reinforcing steel is connected into a conductive grid. The readings are fast, and the data acquisition is often computerized with the readout in form of a map showing the contours of the measured electrical resistance and potentials.

The isopotential test is simple and useful, but it only shows active corrosion. In historic structures, corrosion might have been active at one time but it has now stopped. The detection of active or inactive corrosion by the use of X-ray photograms or by gammagrams is sometimes possible when the corrosion is greater than 0.2 mm, but the results are only qualitative. The measurement of steel thickness loss from corrosion can only be established with any degree of accuracy by cutting inspection openings.

6. MASONRY CONDITION SURVEYS

6.1 General

Several previously described diagnostic techniques are available for inspecting and monitoring the effect of environmental degradation on masonry. Other techniques, some of them unique to masonry structures, have been developed for the quantitative testing of material strength.

6.2 Strain Build-Up in Masonry

Built-up stresses in masonry structures without expansion joints results from the long term expansion of most fired clay masonry and certain marbles during thermal and/or wet/dry cycles. The built-up strain can be quantified using a destructive test. Electrical resistance strain gages are attached to the surface of a masonry block and read. The block, with the gages attached, is cut loose and the gages are read again. The change in reading is a measure of the released built-up strain. The stress in the masonry is then found by multiplying the strain by the modulus of elasticity, as determined for the material by a compression test.

The flatjack method is a "semi-destructive" test used to directly measure built-up stress in unit masonry structures (Fig. 4). Targets are placed above and below a horizontal mortar joint and the exact distances are measured. The mortar is cut out, which causes the joint to close by a small amount because of the built-up stresses. A hydraulic flatjack, a flexible steel envelope thin enough to fit within a mortar joint, is inserted into the joint and pressure is applied until the original distance between targets is restored. The method provides for fast measurements of the direct compressive stress in the masonry[9].

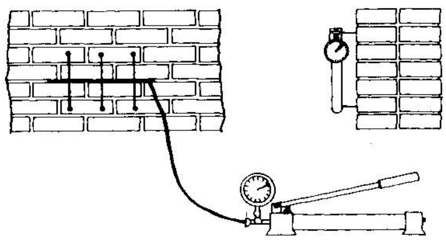


Fig. 4 - Flatjack stress test

6.3 Structural Properties of Masonry

Traditionally, as for quantitative testing of concrete, cores or larger specimens are removed from the masonry structure for destructive laboratory testing. The quality of materials and construction of masonry structures varies widely and for statistical significance a large number of specimens are required. The samples are difficult to transport without damage. For these reasons, in-situ testing of masonry has become the preferred method.

Several previously mentioned nondestructive tests are used on masonry structures. The rebound hammer, which measures surface hardness, has a limited use on thin masonry and stone veneers. While both the pulse-velocity Soniscope and the impact-echo technique have been used successfully to map variations in density and to locate flaws, their effectiveness for evaluating strength and material properties in masonry and stone structures has been limited because of the voids often found in these materials.

The in-place shear test measures the shear strength of horizontal joints in unit masonry, an important value for evaluating seismic capacity. In this test, a masonry unit is removed and the vertical joint on the adjacent test unit is cut. A hydraulic jack applies sufficient force to cause movement of the test unit. The measured shear strength depends on the normal stress at the mortar joints. In a modification of the test, the normal stress on the tested joints can be varied by applying loads to flatjacks inserted in joints above and below the test unit.

Flatjacks can also be used to develop a stress-strain curve for masonry. Two flatjacks are inserted above each other, separated by several courses of masonry. The deformation is measured as the compressive load on the masonry is varied. If flatjacks are not available, the test can be made with ordinary hydraulic jacks, but this requires removal of several masonry units and not just cutting of mortar joints.

7. CONCLUSION

Both high-tech and low-tech diagnostic techniques have appropriate use in evaluating historic structures. The inspection and diagnostic testing of any but the simplest structure is a skilled task and the inspector must be an experienced engineer. He or she must understand that sophisticated techniques are now available to perform many tests better and faster than traditional methods, that these techniques require highly skilled operators both for running the equipment and for interpreting the results, and that, sometimes, experience and intuition are superior to high-tech technology.

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Cycles of Structural Intervention in Historic Buildings

Cycles d'interventions structurales dans les bâtiments historiques

Zyklus von Eingriffen ins Tragwerk historischer Gebäude

Eric P. JOKINEN
P.Eng.
Robert Halsall and Assoc.
Toronto, ON, Canada

E. P. Jokinen, born 1943, received his BSc from Queen's Univ. Kingston, 1966. Design experience in Bermuda and Canada. Involved in building science and structural engineering for historic buildings at Robert Halsall and Assoc. Ltd.

SUMMARY

This presentation looks at a model for estimating the frequency of structural interventions in historic buildings based on the life expectancy of various building components. Experience with historic buildings is compared to related experience with historic bridges.

RÉSUMÉ

Cet article traite d'un modèle pour estimer la fréquence d'interventions structurales dans les bâtiments historiques, basé sur la durée de vie en service de divers composants. Les bâtiments historiques sont comparés aux ponts historiques.

ZUSAMMENFASSUNG

Der Artikel beschreibt ein Modell, welches in Abhängigkeit von der Dauerhaftigkeit der verschiedenen Gebäudekomponenten die Häufigkeit struktureller Eingriffe in historischen Gebäuden abschätzt. Die gewonnenen Erkenntnisse über historische Gebäude werden mit ähnlichen Erfahrungen verglichen, die an historischen Brücken gemacht wurden.

1. INTRODUCTION

The structure and fabric of historic buildings are inseparable. Most of our older historic buildings have load bearing exteriors wherein the architectural expression is also the supporting structure.

The most common agent of deterioration, either direct or indirect, is water. Northern climates accelerate the destructive influence of water through numerous freeze-thaw cycles during a typical year.

During the course of our work with historic buildings, we have noted a pattern of recurring repairs of a similar type in buildings of the same age.

This paper proposes a preliminary method to estimate the frequency of these *cycles of structural intervention.*

2. CONCEPT

The following major factors have been identified as impacting on the service life of historic buildings.

2.1 Service Life of Critical Building Envelope Components (F_{be})

This factor could be called the starting point for the proposed equation. This sets the units (years) for the result, F_i , of the equation. The building envelope (roof , exterior walls , deck waterproofing) is the first line of defense against deterioration of building structures due to atmospheric factors.

The service life of building envelope systems is supposedly indeterminate in historic buildings, if the buildings are to be preserved.

This is fine at the systemic level. However, we must now explore the impact at the component level.

Table 1 illustrates the typical service life for these components in the mid-latitudes of North America.

2.2 Environment (F_{en})

The environmental factor is a non-dimensional multiplier. The influences are numerous and the subject of many papers . In a northern climate there can be many variables. See Weaver [1].

Acid rain pollution is widespread in industrialized areas. Freeze-thaw cycling is very important and can vary widely at a given latitude.

System	Component	Service Life
Roofs	Asphalt shingles	20 years
	Slate shingles	30 years
	Built-up roof	25 years
Masonry Walls	Pointing	20 years
	Sealants	20 years
	Metal ties and supports:	
	Plain steel	10 years
	Painted steel	20 years
	Galvanized steel	30 years
	Stainless steel	indefinite

TABLE 1: Typical Service Life of Building Envelope Components

A maritime environment can also impact on the durability of structures. Seasalt, borne by rain and snow, will infiltrate the building fabric on which it falls.

Climate may also require the use of de-icing chemicals in large quantities. These enter the structure of the building with disastrous results.

Water is the most prevalent agent of deterioration in buildings. Obviously the amount of precipitation at a site impacts directly on this influence. The humidity levels determine the amount of organic growth, another important agent of deterioration for woods and masonry.

Typical values for the environmental multipliers could range from 0.50 for maritime exposure to 1.25 for a cold, dry climate.

If a site is subjected to more than one factor , only the worst case factor would be applied.

Internal environmental factors also impacts the life of the structure. The presence of termites, and high humidity levels which promote fungal growth, are particularly damaging to wooden structures.

2.3 Type of Structure (F_{st})

If a particular type of structure is known to perform poorly in a given environment, then the appropriate multiplier would be applied.

An example would be concrete structures which, when subjected to high numbers of freeze-thaw cycles, carbonation [2] and de-icing salt [3], are particularly susceptible to deterioration of both the concrete and reinforcement.

Values for this factor might range from 0.8 to 1.2 . This range is rather low due to the overlap with the environmental factor.

2.4 Success of Previous Interventions (F_{pr})

The success or failure of previous interventions will impact directly on the period of time before the next major repair is required. A typical case is the repointing of masonry joints. Many repointing campaigns of historic masonry structures have been undertaken in our area without regard to conservation principles. This has caused the intervention cycle to be reduced to 3 to 5 years.

Thus this multiplier might be as low as 0.25.

2.5 Maintenance (F_m)

Maintenance is a very important factor in determining the frequency of major repairs. Buildings under the care of state agencies are typically well cared for and are properly funded. The non-dimensional multiplier in this case could be as high as 2.0

On the other hand historic monuments under the care of volunteer groups, such as churches , do not benefit from professional care and often suffer from inadequate funding. Here the multiplier could be as low as 0.75.

2.6 Socio-economic Factors (F_{se})

The socio-economic factor takes into account the reaction time or delay period in obtaining funding and approvals for major repairs. The state of the economy, the political will, and the heritage status of the site can impact on this. A value of 5 years is recommended as a starting point.

2.7 Heritage Status (F_{hs})

The heritage status of a project affects the success or failure of subsequent interventions. Attar [4] provides a model for assessing the impact of durability versus the authenticity of the intervention.

A defective detail in the original structure may contribute materially to deterioration. However, this detailing may be essential to the heritage character. Maintaining this detail will increase the frequency of repairs. A typical value might be 0.9 for this situation.

The multiplication and addition of these factors yields the estimated frequency , F_i , (in years) of major structural interventions.

$$F_i = F_{be} (F_{en} * F_{st} * F_{pr} * F_m * F_{hs}) + F_{se}$$

3. DATABASE RESEARCH

The formal protection and conservation of historic buildings in Canada has only taken place in the last 20 years. Recording , monitoring and management of these resources is now well documented. Unfortunately, the time frame of 20 years is too narrow to allow statistical confirmation of the proposed factors.

Correspondence with Monumentenwacht Nederland confirmed a similar situation in Holland.

Research of a heritage bridge register will be discussed in the next section and was more fruitful.

The deterioration and repair history of civil engineering structures , such as canals, locks and dams, would be very relevant to this study. These structures are normally owned and maintained by state agencies and are well documented. Research into these types of structures unfortunately must wait for the next paper.

With the emergence of Facilities Management as a new discipline, substantial statistical data will become available to future generations.

4. SAMPLE PROJECTS

In the absence of hard statistical data, one must form an opinion based on observations from daily practice.

We are involved in a number of structures dating from the first era of major development in North America. These structures are now approximately 120 years old and many require major repairs.

Experience on these structures indicates that a minor structural intervention is required every thirty (30) years and a major intervention is required every sixty (60) years. Maintenance appears to be the major determining factor.

A recent project involved the evaluation of two (2) masonry (1860) and concrete (1900) fortifications. The environment is maritime and the concrete structures suffer from the use of seasalt contaminated aggregates. The structural intervention cycle for optimal management of these resources appears to be 15 years.

5. HERITAGE BRIDGES

The Province of Ontario, Canada , currently maintains a register of 85 heritage bridges. The majority are owned and maintained by local municipalities, under the scrutiny of the Ministries of Transportation and Culture and Communications of Ontario. See Reel [5].

Our research looked at the files of 9 of these bridges. The results are presented in Table 2.

Site	Bridge/Location	Built	Type	Maintenance History
3-96	Bank Street, Ottawa	1912	R/C arch	1916 - gunite repairs 1926 - deck waterproofing 1933/34 - gunite repairs 1936-38 - surface repairs 1941/44/45 - surface repairs 1956 - spandrel repairs 1960 - major rehabilitation 1975 - joint repairs 1992 - major rehabilitation
37-1038	Joe Kelley's, Newmarket	1925	Steel pony truss	1985- new deck and stringers recommended
26-79	Inverlea, Peterborough	1910	R/C arch	1967 - major rehabilitation 1985 - major repairs recommended
26-78	Hunter Street , Peterborough	1919	R/C arch	1990 - major rehabilitation
25-251	Church Street, St. Marys	1864	Stone arch	1979 - major restoration
9-2	Caledonia	1928	R/C bowstring	1983 - major rehabilitation
15-14	Pakenham	1901	Stone arch	1967 - masonry repairs 1974 - deck and masonry repairs 1984 - major restoration
19-262	Blackfriars, London	1875	W.I. bowstring wood deck	1950 - major rehabilitation, deck replaced 1964 - minor repairs 1974 - minor repairs 1983 - steel frame repairs 1986 - deck replaced, masonry 1990 - sidewalk repairs
16-47	Lyndhurst	1856	Stone arch	1986 - restored

TABLE 2: Maintenance history of selected Ontario bridges.

As can be seen , only recent maintenance repair history was available for most bridges. However, those that did have complete maintenance histories were very informative.

Sites # 3-96 and # 19-262 had the most complete maintenance histories. These tended to indicate an intervention cycle of approximately 10 years for historic bridges.

It is interesting to note that the early bridge builders in North America placed a building envelope over their wooden bridges. Many of these picturesque "covered bridges" are still in use.

6. THE PAST AND THE FUTURE

As time goes on , the list of historic structures expands. The federal government of Canada assesses the heritage status of all structures over 40 years old.

This threshold is now bringing the Modern era of architecture and sophisticated precast and post-tensioned structures under the purview of conservation regulation. This will create new challenges for the conservation community. We are already observing service life durations of only 20 years in entire **systems** in these contemporary structures, whereas the typical **component** service life in traditional construction often exceeds this number.

7. CONCLUSION

This paper proposes an equation for estimating the frequency of major repairs to the structure of historic buildings. In the absence of sufficient statistical and scientific data, the equation is proposed as a guide for use by heritage resource managers in prioritizing repairs to these structures.

As this process is confirmed and refined, a similar process may evolve that will allow the estimation of the cost of such interventions

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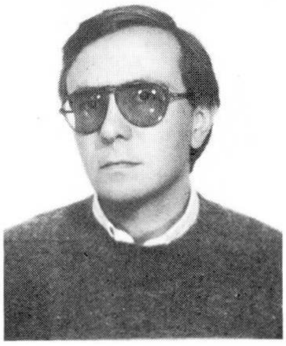
Ambient Vibration Effects on the Colosseum

Effets des vibrations environnantes sur le Colisée

Wirkungen von Umgebungsschwingungen auf das Kolosseum

Paolo CLEMENTE
Researcher
ENEA Casaccia
Rome, Italy

Giovanni BONGIOVANNI
Researcher
ENEA Casaccia
Rome, Italy



P. Clemente, born 1959, got his civil Eng. degree and his PhD in Structural Engineering at the Univ. of Naples. Since 1985 he has been researcher at ENEA Casaccia, and has worked in the field of structural and earthquake engineering.

G. Bongiovanni, born 1954, got his nuclear Eng. degree at the Univ. of Palermo. Since 1984 he has been researcher at ENEA Casaccia, and has been involved in strong motion instrumentation and records analysis.

SUMMARY

This paper describes some results obtained from ambient vibration tests on the Colosseum. Three structural resonances are investigated for the tallest portion of the monument. A troublesome question related to the basement characteristics is addressed for future research. The amplitude of the vibrations was found not to influence the structural state of the monument.

RÉSUMÉ

L'article décrit les résultats d'essais sur les effets des vibrations environnantes sur le Colisée. Trois cas de résonances ont été étudiés pour la partie supérieure du monument. Une question troublante concerne la caractéristique de la fondation, laquelle mérite une recherche ultérieure. Il est montré que l'amplitude des vibrations n'a aucune influence sur l'état structural du monument.

ZUSAMMENFASSUNG

Der Bericht beschreibt die Ergebnisse von Untersuchungen zum Einfluss der Umgebungsschwingungen auf das Kolosseum. Es wurden drei Resonanzfälle für die höchsten Tragwerksteile des Bauwerks identifiziert. Die ungelöste Frage hinsichtlich der Unterbaueigenschaften wird Gegenstand weitergehender Forschung bilden. Es gilt als erwiesen, dass das Ausmass der Schwingungen keinen Einfluss auf den Erhaltungszustand des Monuments ausübt.

1. INTRODUCTION

The present paper is part of a research project organized by ENEA (the italian agency for new tecnologies, energy and environment), solicited by the Archeological Commission of Rome, to investigate the seismic hazard to historical monuments in Rome and to take necessary steps for their preservation.
As a matter of fact the several world-famous historical monuments in Italy have suffered to various degrees during past earthquakes. Although Rome is not classified in the zoning map as seismic region, strong earthquakes have been historically felt in the city: the latest of these has been the Avezzano earthquake of January 13, 1915, whose magnitude was 6.8. Documents describing this earthquake provide detailed informations about damage to residential buildings but not to historical monuments [2].
The symbol of the monumental heritage in the Capital of Italy is, without doubt, the Colosseum. To date Colosseum is not very healthy due to many reasons. Among these are, according to many authors, the vibrations induced by the very caotic traffic of Rome.
The structure has been monitored to study the effects of the traffic induced vibrations, as well as the vibrations from the near underground, and to have a first glance at its dynamic characteristics.
The investigation has regarded the tallest wall of the structure (Figure 1).

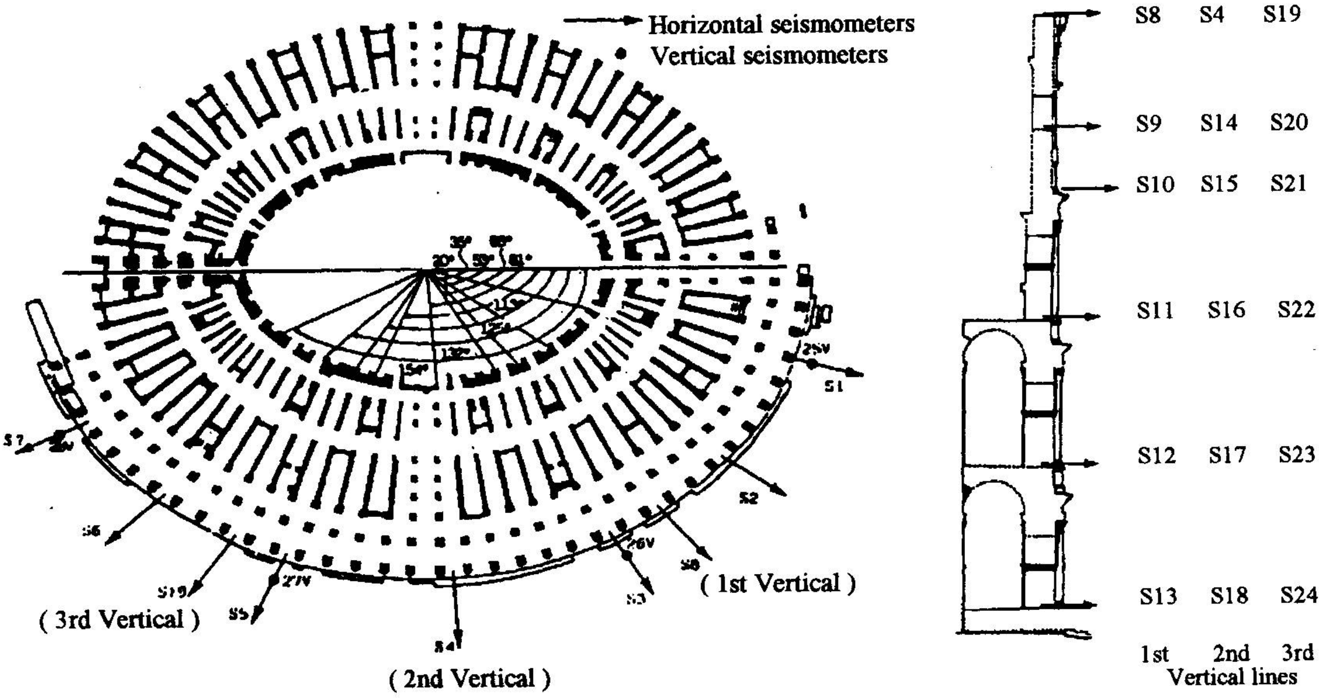


Fig. 1 Plan view and sections of the seismometer locations

The instrumentation has been done by using 13 seismometers deployed in four different configurations.
Data processing consisted of spectral analysis: FFT, power spectral density, cross spectral density, phase and coherence functions.
This analysis resulted in the characterization of the structural resonances and the associated modal shapes. These have not been related to the structure modal shapes due to its complexity.
A statistic analysis have been carried out to determine the effective value of the velocity, which has been compared to the maximum values suggested by the german code (DIN 4150).

2. EXPERIMENTAL ANALYSIS

The measurements have been done by ISMES in May 1985 using 13 seismometers (Teledyne Geotech S13800). The signals have been recorded on magnetic tape by a TEAC SR-50 14 traces analog recorder. The analog signals have been later digitized with a sampling rate of 0.005 sec.
In figure 1 is depicted the layout of the seismometers locations. The locations S1 to S7 are fixed in horizontal radial directions at the top of the wall. Three different vertical lines of measurements have been considered and instrumented with the other six seismometers at separated times. A fourth layout regarded the measurement of the vertical vibrations: four vertical seismometers have been located on the basement (Locations 25V to 28V).
Four registrations, at different hours of the day and in two different days, have been carried out. For each configuration 30 minutes of recording have been performed at hours of very intense traffic.
ISMES analized the whole 30 minutes record in order to extract the 5 minutes interval characterized by the highest energy content. The analysis herein described regards these 5 minutes intervals.

3. SPECTRAL ANALYSIS

As we have already said, four series of measurements have been carried out. The spectral analysis interested all the series of measurements.
A frequency domain analysis has been carried out calculating the power spectral density for each record of each series of measurements and the cross spectral density with phase and coherence functions between a reference record and each of the other ones.
For the analysis the IMSL routine CSSWD have been used with the following parameters [10]:

W (window) = Bartlett-Priestley

M (window parameter) = 5000

The time series have been analyzed for time intervals of 100 seconds, out of the 300 total, in order to discriminate any difference due to the transit of the train in the nearby underground. The records obtained during the transit of the train at the locations S13 and S8 are illustrated in figure 2.

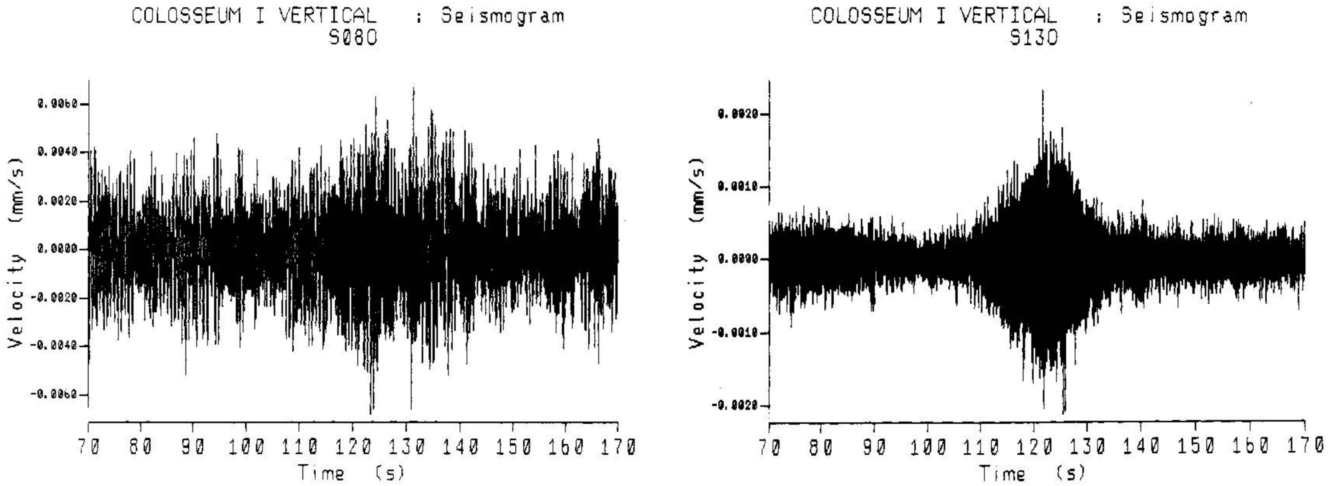


Fig. 2 Seismograms at locations S13 and S8 obtained during the transit of the train of the underground

The transit of the train is apparent only at the S13 location at the basement, moreover there is amplification over the whole time range going from the basement to the top of the wall. The frequency domain analysis proves that there is no significant difference in the behaviour, with and without the transit of the train.
The cross spectral analysis individuates the frequencies that can be considered structural resonances. The corresponding peak amplitudes of the power spectral density give an idea of the associated modal shapes.
The analysis shows several relevant frequencies for each vertical, but only the following:

$$f_1 = 1.46 \text{ Hz} \qquad f_2 = 1.70 \text{ Hz} \qquad f_3 = 2.75 \text{ Hz}$$

are common to all the verticals and present significative coherence and phase relationships so that they are recognized as structural resonances.
In Table 1 phase and coherence of the cross spectra relative to the first vertical line at the above mentioned three frequencies are listed.

1st vertical	Freq.	1.46	1.70	2.75
S13-S12	Phase Coherence	0.00 1.00	0.00 1.00	0.00 0.90
S13-S11	Phase Coherence	0.00 0.98	0.00 1.00	0.00 0.97
S13-S10	Phase Coherence	0.00 0.98	0.00 0.98	0.00 0.92
S13-S9	Phase Coherence	0.00 0.98	0.00 0.98	π 0,90
S13-S8	Phase Coherence	0.00 0.98	0.00 0.98	π 0.80

Table 1 Cross spectral phases and coherences relative to the structural resonances

In figure 3 are illustrated the corresponding modal shapes. Obviously the change in the stiffness at the sensor locations S11, as at sensors S16 and S22 for the second and third verticals respectively, is found in the modal shapes.
The power spectral densities for to records in locations S13 and S8 and the relative cross spectrum are reported in figures 4, 5 and 6 to illustrate the complexity of the response of the structure.

The analysis of the records obtained at the top of the wall shows the following results:
a) the cross spectral analysis identifies the same two frequencies, f_1=1.46 Hz and f_2=1.70 Hz, as the vertical lines;
b) the coherence functions are very good for all the locations relatively to the frequency f_2 and quite poor for the first one at the locations S5, S6, S7 with respect to S1. This result is probably due to the small amplitude of the signals of these measurements as from the power spectral densities. It also could be related to the results from the vertical sensors discussed later.

The seismograms in figure 7 are relative to the vertical sensors 25V, which is closer to the underground, and 26V. As you can see the effects of the train transit fade rapidly. The power spectral densities of the records at 25V to 28V show a very sharp peak at the frequency f_1, the same of the horizontal seismometers. The cross spectral analysis confirms that this is a structural resonance. Figures 8, 9 and 10 show the power spectral densities and the cross spectrum of the records at 25V and 26V.

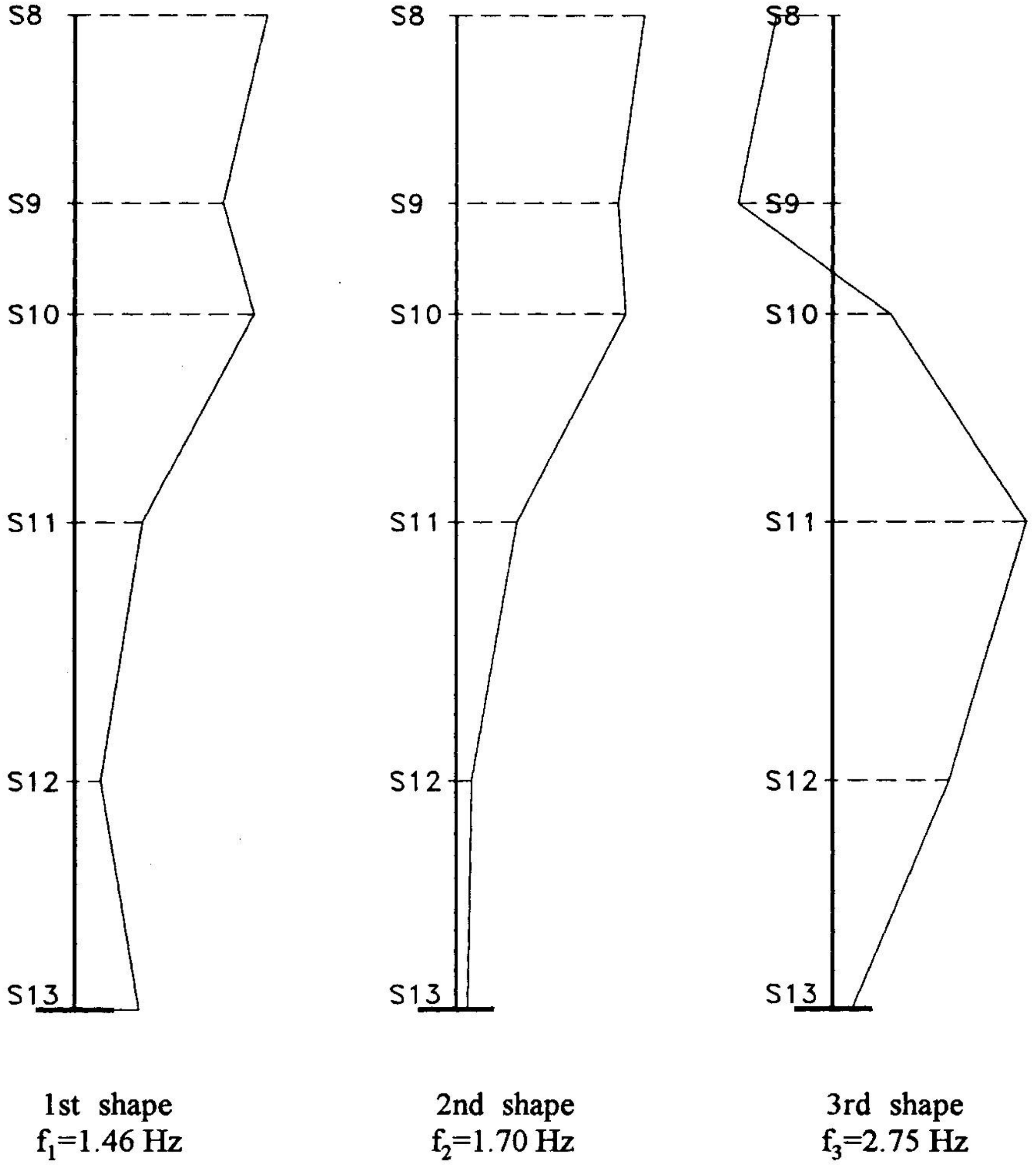


Fig. 3 Modal shapes of the first vertical associated to the structural resonances

Although the number of sensors is limited, it is apparent that the basement of the structure has not a pure rocking movement as from spectral analysis. A lack of structural continuity of the top portion of the basement could, in part, explain this behaviour. The heterogeneity of the soil underlying the monument could play an important role [6, 7, 8 and 9]. In any case further investigations are necessary.

4. OTHER RESULTS

The original signals have been analyzed in order to calculate the velocity effective values and the peak values over successive time intervals lasting 1.28 sec.

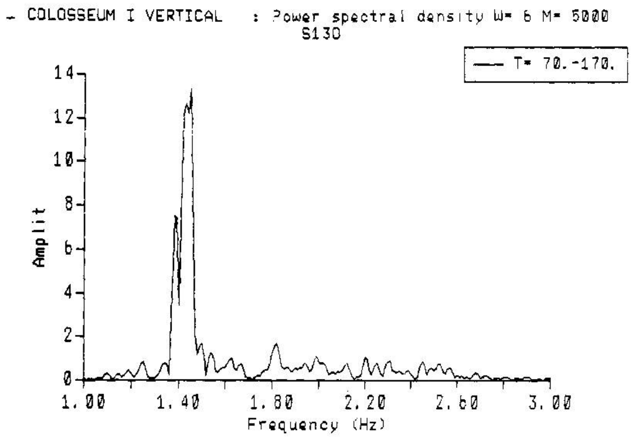


Fig. 4 Power spectral density - S13

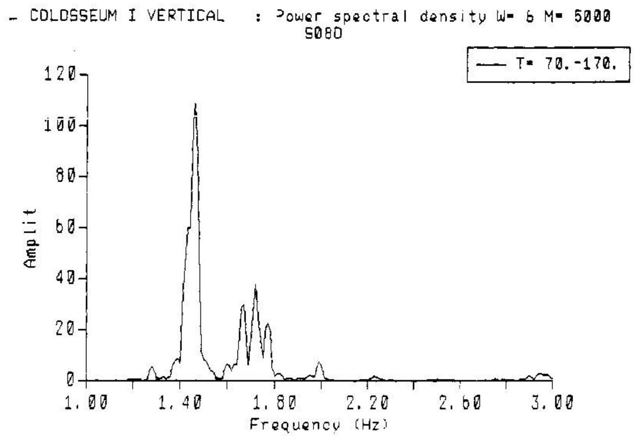


Fig. 5 Power spectral density - S8

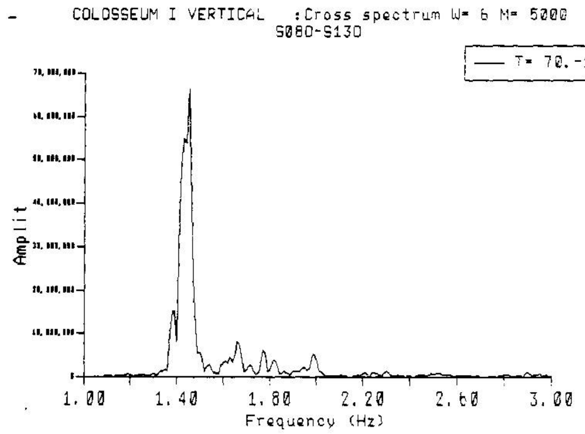


Fig. 6 Cross spectrum S13-S8

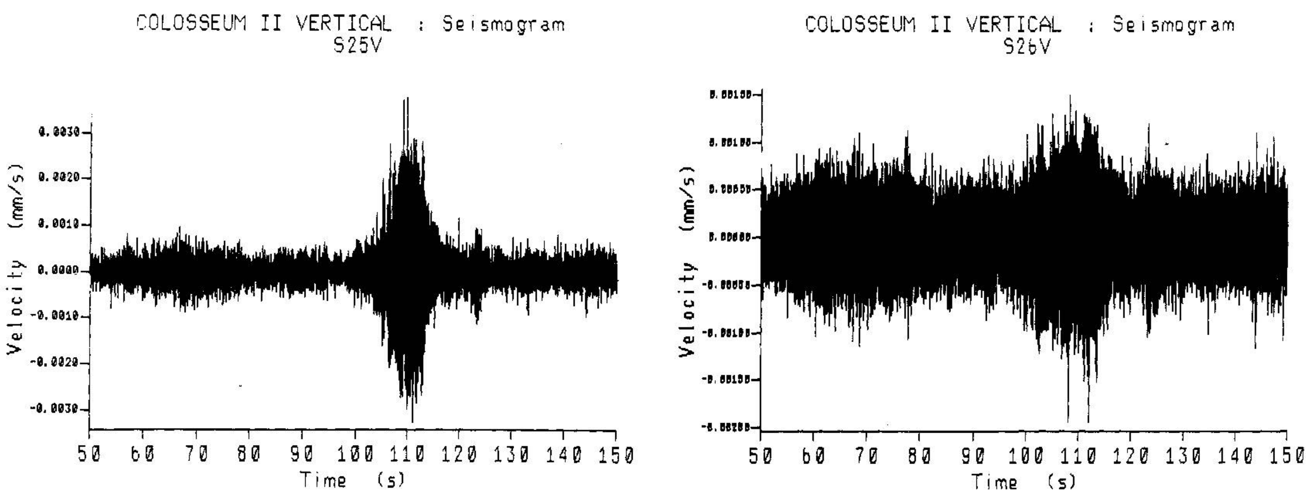


Fig. 7 Seismograms at locations 25V and 26V obtained during the transit of the train of the underground

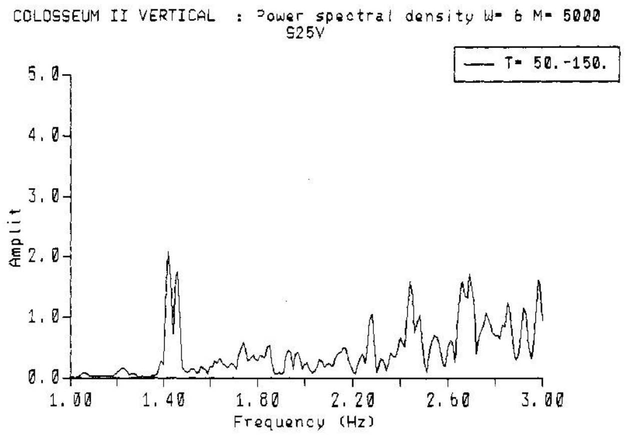


Fig. 8 Power spectral density - 25V

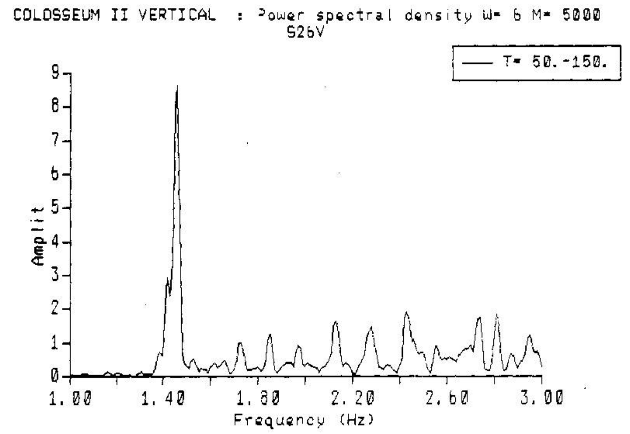


Fig. 9 Power spectral density - 26V

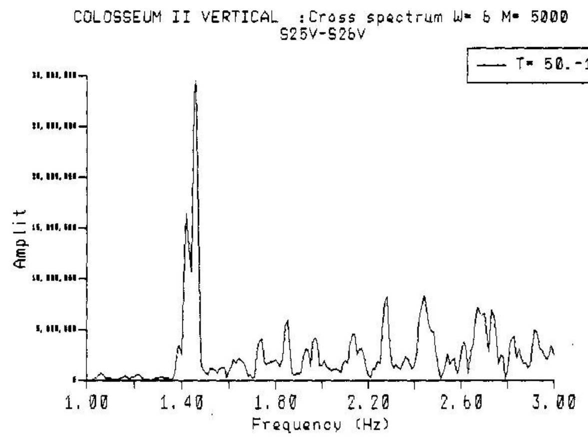


Fig. 10 Cross spectrum 25V-26V

The velocity effective values have been calculated according to the well-known formula:

$$\underline{x} = \sqrt{\frac{\int_{t_1}^{t_n} x^2 dt}{t_n - t_1}}$$

where:

$\underline{x}$ = effective value
x = recorded value
t_n - t_1 = time interval.

The velocity effective values are always < 0.12 mm/sec and the peak values < 0.32 mm/sec.
The comparison with the maximum value at the basement of historical and archeological monuments suggested by the german code DIN 4150 (2-3 mm/sec) pointed out the limited effects of the traffic induced vibrations on the Colosseum. This result, in conjunction with the statements referring to the seismograms at locations S13 and S8, allow us to state that the traffic induced

vibrations could have contributed, over the tenths of years, to the bad health status of the monument, but they do not represent an immediate hazard to it.
It is reasonable that many of the illnesses of the monument are to be found in the weathering of the exposed surfaces as well as in the pollution and aced rains. Nonetheless more detailed monitoring of the structure and geotechnical investigations are advisable in order to define the restoration design.

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Investigations on the Masonry of the Leaning Tower of Pisa

Essais sur la maçonnerie de la Tour Penchée de Pise

Untersuchungen am Mauerwerk des schiefen Turms von Pisa

Raffaello BARTELLETTI
Prof. of Struct. Eng.
Univ. of Pisa
Pisa, Italy

Leo FINZI
Prof. of Theory of Struct.
Politecnico di Milano
Milan, Italy

Luca SANPAOLESI
Prof. of Struct. Eng.
Univ. of Pisa
Pisa, Italy

SUMMARY

An extensive programme of experimental tests was carried out on the masonry of the Leaning Tower of Pisa from 1984 - 87. These experiments were directed towards establishing a design for stabilization. The authors, who developed and carried out the programme, present the results of the individual tests and the conclusions that can be derived from the series of investigations executed during the programme.

RÉSUMÉ

Un programme très complet d'essais expérimentaux a été exécuté sur la maçonnerie de la Tour Penchée de Pise pendant les années de 1984 à 1987. Les essais avaient pour but la définition d'un projet pour la stabilisation du monument. Les auteurs, qui ont formulé et conduit le programme, présentent les résultats des essais et les conclusions du programme.

ZUSAMMENFASSUNG

Ein umfangreiches Programm experimenteller Untersuchungen am tragenden Mauerwerk des Turmes von Pisa ist in den Jahren von 1984 bis 1987 verwirklicht worden. Diese Untersuchungen waren auf die Planung der Stabilisierung des Denkmals ausgerichtet. Die Autoren dieses Beitrages, die das Programm erstellt und durchgeführt haben, berichten über die erzielten Ergebnisse einzelner Untersuchungen und über die Schlussfolgerungen, die aus ihnen gezogen werden können.

1.Foreword

From 1984-87 the authors led an extensive programme of experimental tests to learn the nature of the masonry structure of the Leaning Tower or Pisa. The authors present the investigations and conclusions in this paper.

It must be mentioned that in 1983 the Ministry of Public Works set up a design team composed of structural engineers, geotechnical engineers and an art historian, with the task of establishing a design for the definitive consolidation work to be carried out on the Leaning Tower. The authors of this article were the structural engineers of the design team.

The first job of the design team was to organize and study the existing documentation on the state and characteristics both of the elevated section and the foundation soil of the Tower.

The next task was the study and outlining of an extensive programme of experimental investigations on the Tower and its soil. A programme directed at the collection of specific information to allow for consolidation design. This programme, approved by the Ministry of Public Works in 1984 was executed in 1985-86 by ISMES (Institute for the testing of models and structures) and RODIO s.p.a., both completely reliable and competent bodies well suited to the delicate work in question.

It should be stated why it was necessary to perform experimental investigations on the Tower. The Tower in fact is well known in the scientific community for its inclination, related historical aspects and safety problems; such that it might appear that the critical point of the Tower lies in the foundation soil and that the possibility of crisis and collapse is to be found only here.

In reality this is not the case. The structure itself also shows critical aspects comparable to those existing in the foundation, fully justifying the attention given to learning the nature of the Tower's structure.

To close this foreword, although not entering deeply into the subject, the authors would like to point out that with the extensive knowledge obtained from the wide testing programme the design team was requested to stop its work, delivering the final plan for the consolidation work of the Tower to the Ministry of Public Works for a definitive and complete resolution of the problem, as foreseen by the original objectives.

2.Introduction

The programme of experimental tests - finalized as already described, not only to expand scientific knowledge but directed at the collection of data to permit the planning of consolidation work - was designed to supply information relating to:

- the structural conformation and characteristics of the Tower,
- the stress states existing in the most important areas,
- the mechanical characteristics of the materials.

To this end the following tests, briefly described in subsequent headings, were performed:

1. Sounding of the masonry.

2. Imaging of the walls' internal structure using television waves.

3. Core boring of the walls with the samples subsequently undergoing compression tests.

4. Testing with flat jack to determine the state of stress of the freestone.

5. Deformation tests performed inside perforations by means of a dilatometer.

6. Measurement of stress states using the doorstopper decompression technique.

7. Acoustic investigation.

8. Impregnation tests for the walls.

On the basis of the first three tests in the list - and in particular the tests with television waves - it was possible to better define the internal structure of the Tower's masonry already outlined and here briefly re-stated.

Essentially the elevated structure is formed by a hollow cylinder and in the foundation by a circular mass as the base. The elevated section is formed both on the external and internal face by two walls of squared blocks having a depth of about 40cm filled with mixed masonry of cobbles very good hydraulic lime and mortar which, however, contains numerous cavities and has far lower mechanical characteristics (see fig.1).

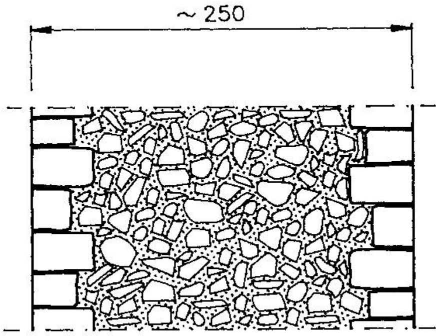

Fig. 1

The foundation is made of stone masonry and hydraulic lime mortar, consolidated in 1935, by Rodio, using cement grouting.

3. Mechanical tests on wall samples

The masonry of the monument was tested through 19 perforations. The total length of all these perforations was 96.02m. The boring was performed in the foundation, 1st and 2nd tiers. These perforations were made dry so as not to impregnate the permeable masonry and by aspiration so as not to induce dangerous stresses in the water table mortar and walls, using a diamond bit of 76/53mm diameter.

A high percentage of the boring core was obtained often reaching 100% of the bore length. Thus permitting a satisfactory knowledge of the Tower's masonry composition. 36 significant specimens were taken from the core samples and these were tested, some at ISMES in Bergamo and some at the Laboratory of the Istituto di Scienza delle Costruzioni di Pisa.

The samples were classified according to the area of extraction and composition in the categories: freestone, mortar, mixed masonry.

3.1 Freestone

The first group of samples numbers 12 test cylinders taken from squared stone blocks of calcareous stone in the Tower's external face. The dimensions range from 5.35 to 5.38cm in diameter and from a minimum height of 5.54 to a maximum of 10.60cm The unit mass is of 2.66 to 2.72 g/cm^3, with an average value of 2.69 g/cm^3 and standard deviation of 0.0176 g/cm^3 (0.65%).

In an attempt to normalise the compression test results, the varying height of the samples was taken into consideration using an appropriate correlation between strength versus the ratio of diameter to height.

The cylindrical strength (f) was between 69.7 and 152.8 N/mm^2 with the following average values:

148.3 N/mm^2 for samples taken from the foot of the first tier

91.8 N/mm^2 for samples taken from mid way up the first tier

89.1 N/mm^2 for samples taken from mid way up the second tier

The Young's modulus (E) was far more uniform than the strength, varying from a minimum of 71300 N/mm^2 and a maximum of 95300 N/mm^2. The average value is of 83636 N/mm^2 with a standard deviation of 7155 N/mm^2 (8.6%).

The samples pertaining to the external face also permitted the measurement of the thickness of the face itself, thereby obtaining the following average values:

lower part of the first tier (4 measurements) 40cm

mid section of the first tier (4 measurements) 32cm

mid section of the second tier (3 measurements) 23cm

While for the internal face there were no direct measurements available in as much as the core boring was stopped a few centimetres before it completely penetrated the wall itself. We can, however, assume that the thicknesses are of the same order of magnitude.

3.2 Mortar

The second group of samples consists of a series of eight results obtained from core samples taken exclusively from the mortar. Disregarding the anomalous result obtained from one test (0.80 g/cm^3), the average value for unit mass of the remaining seven samples was 1.54g/cm^3 with a standard deviation of 0.08g/cm^3 (5%).

The sample with a unit mass of 0.80g/cm^3 so showed a notably different value for cylindrical strength (22 N/mm^2) from the other samples, while for three samples it was not possible to determine the mechanical characteristic. The remaining four samples had diameters ranging from 5.27 to 5.32cm and height varying from 5.85 to 9.62cm. Again in order to permit a comparison of the results the compressive strengths were normalized to the cylindrical strength. Instead of using the above mentioned correlation, which is suitable for stone material, a correlation more appropriate to this type of material was adopted. The cylindrical strengths obtained vary from 10.2 to 13.3 N/mm^2, with an average value of 11.5 N/mm^2 and standard deviation of 1.29 N/mm^2 (11%).

3.3 Mixed masonry

The remaining 16 samples, obtained from the infill material contained between the faces, consisted of mortar comprising large pieces of stone material of extremely variable quantity. This is demonstrated by the unit mass which ranges from a minimum of 1.55 g/cm^3, virtually the same value as that obtained from the mortar to a maximum of 2.3 g/cm^3 which is quite close to the value obtained from the for the stone. The average result was of 1.90 g/cm3, with a standard deviation of 0.26 g/cm^3 (14%).

The number of samples which underwent compression tests was 15. These having diameters between 5.30 and 6.16 cm and varying in height from 7.5 to 14.9 cm. Once again, in order to obtain the cylindrical strength an appropriate correlation was applied.

The average cylindrical strength varying between 7.3 and 18.8

N/mm^2, was 10.7 N/mm^2 with standard deviation 3.5 N/mm^2 (33%).

The results for the tests of Young's modulus carried out on these samples were of little significance due to their varied nature, including the presence of cavities and the varying sizes of the stone material. Indeed the values go from a minimum of 2840 N/mm^2 to a maximum of 39600 N/mm^2 , without the possibility of forming a useful correlation for unit mass. This is obviously also due to the presence of cavities.

4. MEASUREMENTS OBTAINED WITH FLAT JACK

For the experimental evaluation of the states of stress in the Tower's faces a total of 32 tests were performed with the flat jack technique. Jacks in the shape of a circular segment 4 mm thick were used, inserted into milled cuts along the horizontal face joints. Ten of these tests concerned the internal face in sections 1 and 3B.

The tests were performed at four tiers, more precisely indicated in fig.2.

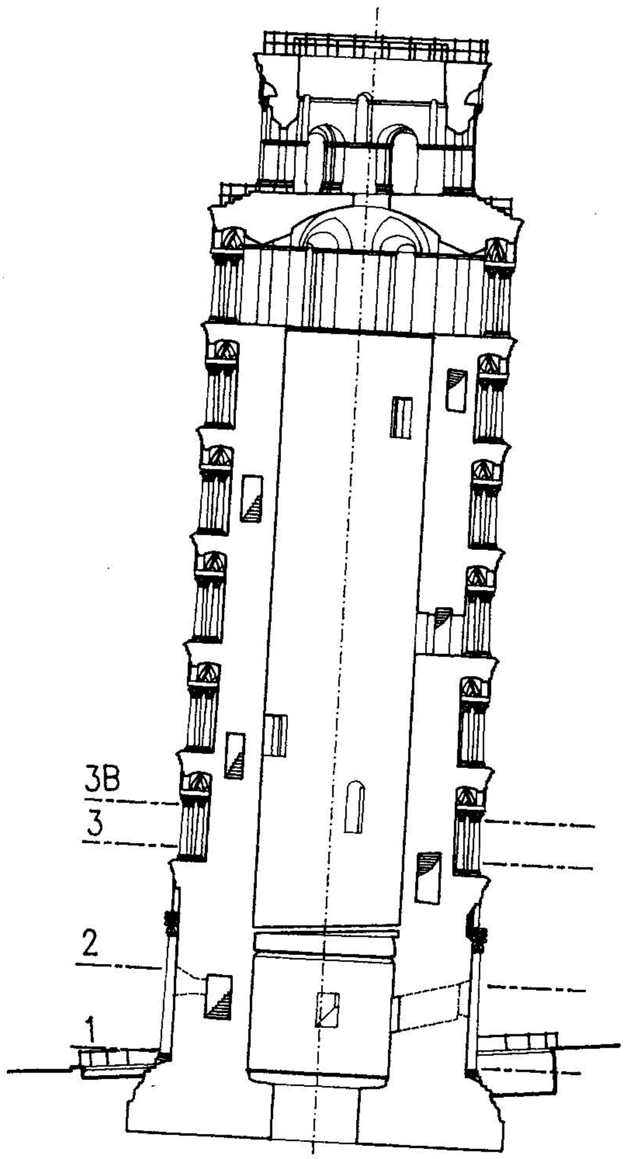

Fig. 2

These enabled the on site formulation of Young's modulus for the freestone face.

4.1 Stresses in the facing walls

The 32 values of stresses measured, with the exception of 4 results disregarded due to evident anomalies due to particular local conditions, proved to be, for each of the 4 horizontal sections investigated, virtually on the same plane. Thus permitting the formation of trapezoidal diagrams of the stresses referring to the plane of maximum inclination of the Tower with the following range of values:

Section 1:	σ_{min} = 0.29 N/mm^2 (0.12)	σ_{max} = 6.07 N/mm^2 (6.8)
Section 2:	σ_{min} = 0.33 N/mm^2 (0.75)	σ_{max} = 5.53 N/mm^2 (5.6)
Section 3:	σ_{min} = 0.65 N/mm^2 (0.6)	σ_{max} = 4.2 N/mm^2 (4.24)
Section 3B	σ_{min} = 0.65 N/mm^2 (0.8)	σ_{max} = 6.93 N/mm^2 (6.84)

The diagrams were obtained with the process of linear regression from all the useful stress measurements. The extreme values measured are indicated in brackets.

The apparently anomalous low values of the stresses on section 3 are possibly explicable due to local redistribution phenomena of the stresses which particularly concern the external facing wall, caused by sudden variation in diameter between the second and first tiers of the Tower.

4.2 Young's modulus for the treestone

Young's modulus were obtained from the results of each point measured. These were very disparate due to unavoidable errors in measuring variations where stresses were very small. Therefore, excluding the points where stresses were less than 1 N/mm^2 (11 points), we obtain an average modulus value of 54000 N/mm^2 with a standard deviation of 10280 N/mm^2 (19%).

4.3 Considerations on the flat jack measurements

The maximum and minimum stresses obtained from the tests were about 7 and 0.1 N/mm^2 respectively, while the Young's modulus varies between about 50000 and 66000 N/mm^2 .

Furthermore we can observe that both the stresses and the modulus were determined with reference to the stress states and deformation of a homogeneous half-space in the presence of fissures. In effect the measurement was taken with the flat jack inserted into a gap along a mortar joint to a presumed depth of half the face's thickness. One can therefore, assume a notable influence on the results due to the varied nature of the material (fill material, mortar and stone), tending towards an overestimate of the acting stresses and an underestimate of Young's modulus.

5. DILATOMETER TEST

To obtain information concerning the elasticity of the wall fill 18 dilatometer tests were performed using the bore holes of the core sampling.

Based on the total experimental values obtained, the average value for E is:

E_m = 8758 N/mm^2, st. dev. = 4322 N/mm^2 (49%)

However, some considerations must be made about the validity of these results, because the standard deviation shows that the average is the result of widely scattered values.

The reasons for this dispersion seem essentially to lie in the local conditions of the bore hole test area, influenced by the cavities and composition of the mixed masonry.

In brief the dilatometer tests studied, although supplying less reliable results than those obtained with the flat jack, have enabled the calculation of Young's modulus value for the mixed aggregate wall fill in the order of 12000 N/mm^2 .

6.OTHER TESTS AND INVESTIGATIONS

The last three tests listed in point 2 (doorstopper, acoustic, and impregnation) supply specific data which are less reliable and less significant than those illustrated in the previous points. The decompression test by means of the doorstopper technique directed at determining the main stresses on the fill wall, seem strongly influenced by the presence of micro fissures, by the choice of values for E and the lack of uniformity in the material under study (mortar or stone).

The acoustic investigation supplied a qualitative evaluation of the material examined, including the noting of discontinous surfaces (as for example between foundation wall and soil), while the propagation velocity, linked to Young's modulus E, was strongly influenced by the presence of cavities or fissures in the wall such that it is of no significance reporting the values.

The impregnation tests, carried out by filling the hole bores from the core sampling with a mixture of water, cement and additives, to determine the wall's suitability for low pressure impregnation, have enabled the assumptions to be made concerning the impregnation of the mixed masonry.

7. CONCLUSIONS

From the investigations performed and from an analysis of the results shown above it is possible to conclude as follows:

Nature of the walls

The masonry of the foundation ring, comprising a mixture of lime and sand mortar with calcareous stones and having a thickness of about 4 m is thoroughly cemented and contains no cavities as a result of the cement injections carried out in 1935.

The elevated part of the wall is formed by two freestone faces in calcareous marble enclosing mixed masonry of calcareous stone and mortar and contains many cavities.

Mechanical characteristics of the wall

For the freestone:

$f_{min} = 80$ N/mm^2 $E_{min} = 70000$ N/mm^2

For the inner masonry:

- Mortar $f_{min} = 10$ N/mm^2 $E_{av} = 6000$ N/mm^2
- Mixed $f_{av.} = 10$ N/mm^2 $E_{av} = 12000$ N/mm^2

The maximum stress measured on the outer freestone is 7 N/mm^2.

With reference to the last piece of information, the maximum stresses existing clearly seem high and far above those in similar buildings.

Concerning the stone material and mortar, both seem to have excellent mechanical characteristics with high strength values. The nominal stresses existing in the inner masonry are noticeably lower, in comparison to the values noted above, for the lowest Young's modulus which characterizes this material in comparison to the freestone.

However, there are many other elements which make this picture far from satisfactory. With regard to this we can only mention the followings:

- the existence of mortar joints, occasionally of great thickness in mixed masonry;
- the varied nature of the fill material and its high percentage of cavities;
- the numerous individual areas, first among which is that between the 1st and 2nd tiers where the outer face is supported by the mixed masonry;
- the presence of the stair well which creates discontinuity and forms a further particular area.

In conclusion a thorough evaluation of the problem requires an accurate analysis and modelling of the structure, for which the data resulting from the experiments conducted by the authors, and briefly reported in this paper are a necessary starting point.

Automatic Diagnosis of Anomalous Structural Behaviour

Diagnostic automatique d'un comportement structural anormal

Automatische Diagnose anomalen Tragwerkverhaltens

Stefano LANCINI
Electrotecnician
ISMES
Bergamo, Italy

Roberto MENGA
Civil Eng.
ISMES
Bergamo, Italy

Maurizio VAVASSORI
Electronic Eng.
ISMES
Bergamo, Italy

Francesco ERRICHIELLO
Civil Eng.
Public Works
Milano, Italy

Lorenzo DUICO
Civil Eng.
Univ. of Pavia
Pavia, Italy

SUMMARY

The article describes the features of the installed monitoring system and analyses the different methods employed to validate the measurements carried out and to define the 'confidence thresholds'. Moreover, it presents the 'physiological' behaviour in time of some monuments and the first decisional guidelines for the diagnosis of the risk.

RÉSUMÉ

L'article décrit les caractéristiques d'un système de surveillance installé et analyse les différentes méthodes employées afin de valider les mesures effectuées et de déterminer les seuils de confiance. Il décrit le comportement physiologique de certains monuments et présente des directives pour établir une évaluation du risque.

ZUSAMMENFASSUNG

Nach einer kurzen Beschreibung der Eigenschaften der installierten Kontrollsysteme werden im Artikel die verschiedenen angewandten Methoden dargestellt, um die Messergebnisse zu validieren und Vertrauensbereiche zu definieren. Ausserdem werden die physiologischen Verhaltensweisen einiger Denkmäler im Laufe der Zeit sowie die ersten Entscheidungsrichtlinien für die Diagnose des Risikos beschrieben.

1. FOREWORD

On March 17, 1989 the Civic Tower of Pavia collapsed; this collapse took away one of the most representative monuments of the town. After this event, the Ministry of Civil Defence appointed a technical-scientific Committee to analyze the causes of the tower collapse and to determine the state of other monuments of the town.

The Committee activity, performed on behalf of the Public-Works Office of "Regione Lombardia", includes a wide and complex plan of monitoring surveys and interventions to be carried out on the Cathedral and on six town towers.

This article describes the experience gained in the historical monuments of Pavia; data collected by a complex monitoring system have been systematically processed and critically interpreted in order to define limits and alert thresholds on different measurement channels as to investigate the structure behaviours providing some interpretation criteria in order to realize an automatic system for risk diagnosis.

2. THE AUTOMATIC MONITORING SYSTEM

In order to control the Cathedral and the six Towers, a sophisticated automatic monitoring system based on eight Peripheral Measurement Units and one Data Acquisition Centre have been installed in 1990. The peripheral measurement units, installed inside the monuments to be monitored, are linked via radio with the centre, located at the University of Pavia. Fig.1 shows the block diagram of the system and the list of the instruments installed an each monument [2].

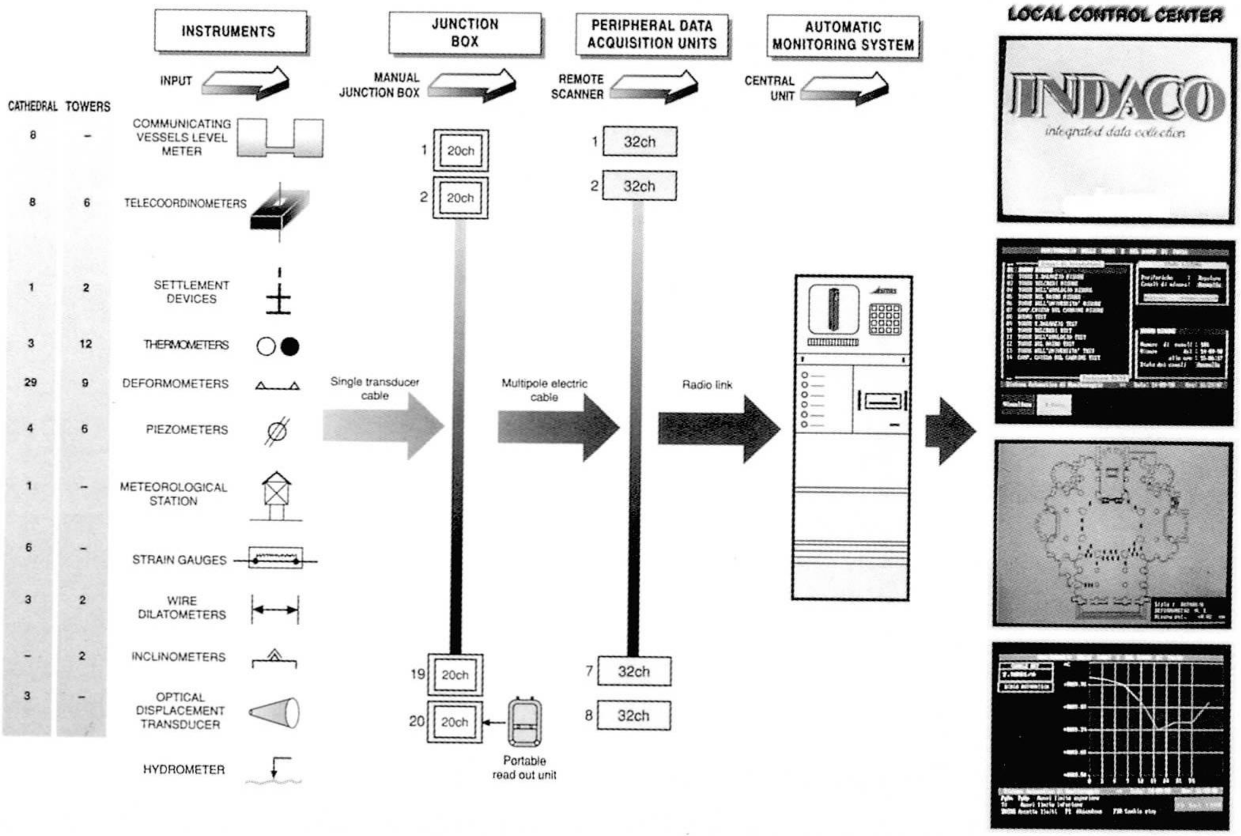

Figure 1: Pavia Towers and Cathedral. Data acquisition system: block diagram

The installed instrumentation allows to acquire the most important "cause" and "effect" variables on each monument. In particular the monitoring activity concerns:

- opening-closure of significant cracks in the Cathedral and on the towers, measured by automatic deformometers;
- global displacements of the structures, considering the displacement of the Cathedral dome and the displacement of the top of the towers, measured by plumb-lines and automatic telecoordinometers;
- planimetric displacements of the Cathedral dome, measured by automatic optical sights;
- foundation settlements of the columns of the Cathedral dome, measured by a level measuring circuit of communicating vessels;
- stress on the chains installed between two pillars of the Cathedral, measured by strain gauges;
- cross strains of some tower walls and of one Cathedral column, measured by wire dilatometers;
- "cause" variables such as air and masonry temperature, solar radiation, wind, groundwater level, measured by means of meteo-units, piezometers, thermometers placed inside the walls.

3. MEASURES MANAGEMENT

Measures automatically gathered by the monitoring system have been periodically transferred into the historical data bank managed by MIDAS code [4], installed on ISMES computers in Bergamo. Measures acquisition, storage and processing, carried out for about three years, have allowed to set-up a data base which can be used for graphic tables representing chronological measurement diagrams and results of processing and correlations among these magnitudes. This data base was also used to analyze the acquired measures and to evaluate the thresholds values for the on-line control.

4. ON-LINE CONTROL

The automatic monitoring system installed on the Cathedral and on the Towers is provided with special package (INDACO), developed by ISMES, able to evaluate the reliability of the acquired measures and to identify any anomaly.

These anomalies can be caused either by instrument failure or by really "anomalous" situations in the structure behaviour; it is therefore important to determine which problem is actually present.

Different criteria have been determined to identify different situations. Each read value is subjected to a validation and control procedure carried out on the automatic monitoring system. The architecture of the control is showed in Fig. 2.

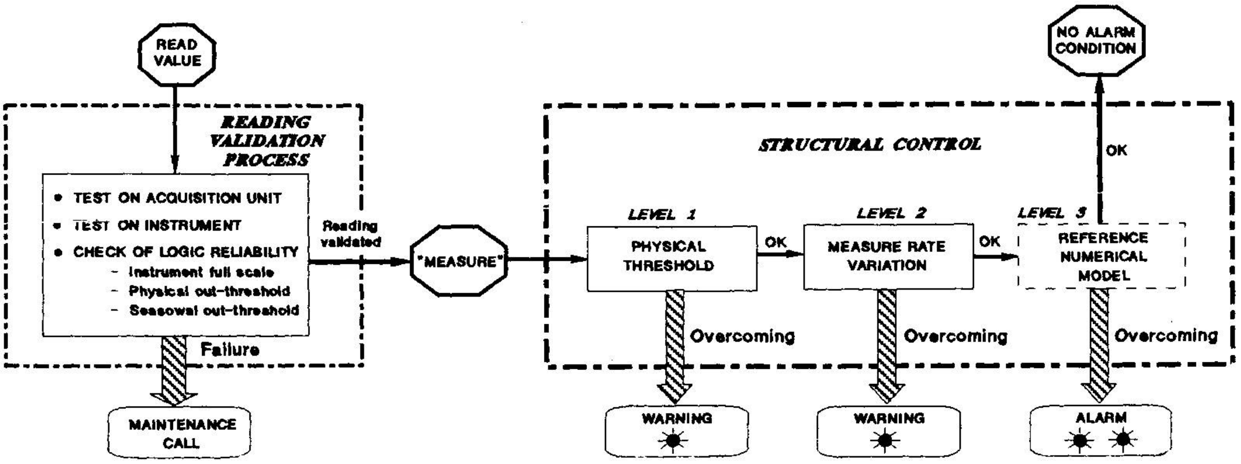

Figure 2: On line control architecture

The instrument measures are deeply investigated and checked, thus excluding any instrument fault: the "reading validation process". After these controls each acquired value is compared with preset and various ranges ("thresholds") for the "structural control". Measures inside these ranges are first of all reliable and then detect a normal behaviour of the structure under control. The measures are validated according to the past behaviour of the structure by means of two different control levels:

- the first level consists in the control of a physical threshold in order to verify that the measure falls within a variation range defined according to the physical characteristics of the quantities under examination or to the past evolution of the measures. This variation interval is determined either through a minimum and a maximum value ("fixed" thresholds) or through a tolerance interval connected with a periodic variation of measures ("periodic" thresholds).

- the second level consists in the control of the variation rate of the measures. This speed shall not overcome a preset threshold, determined, even in this case, according to the values obtained with the measures taken in the past. This control is carried out only after the successful of the first level controls.
  The characteristic parameters of the "fixed" thresholds are minimum and maximum values calculated according to measures average and standard deviation.
  While measures showing periodic variation are interpolated with a "periodic" function through a Fourier series development.

A third level control is the comparison between real measures and measures provided by reference numerical models based on the real behaviour of the structure. This has not been installed on line but it is activated off line by forecasting automatic procedures of the MIDAS code, after the measure has been stored in the historical data base.

The validation criteria are the same for all sensors installed for "cause" and "effect" magnitudes too.

Figure 3a shows the opening-closure history of a crack (solid line), read in the dome of the Cathedral, together with the established threshold bounding values (dotted lines). Figure 3b gives the same information on the opening-closure variation rate: the dotted line is the maximum threshold value.

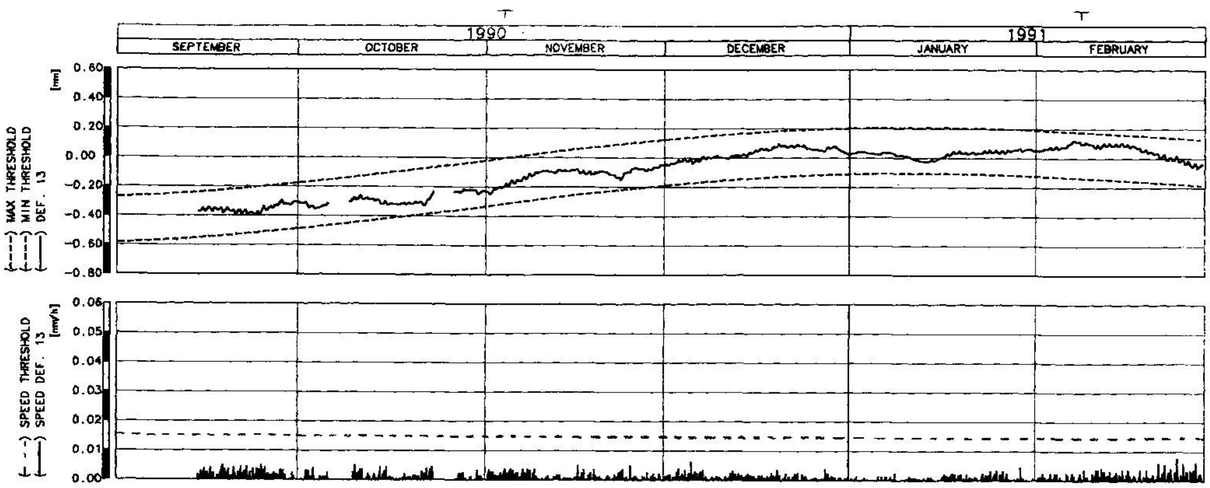

Figure 3: An example of periodic threshold of the measures.

5. EXPERT SYSTEMS AS EVOLUTION OF THE ON LINE CONTROL

The software developed for checking the performance of the Pavia monuments is able to alert in real time on the presence of readings in disagreement with the established thresholds. It should be considered that the information provided consist of different types of signals, which are related to a given time and in principle may not be easily correlated within a consistent physical process.
The risks correlated to such a situation are twofold. On one hand, the surveillance personnel cannot judge about the potential risk associated to the anomalous readings; specifically they cannot differentiate accidental conditions from indicators of a risk event.
On the other hand, the correlation of the measures and the evaluation of the actual impact of the different 'on-line' readings may require delays not compatible with the safety needs. This consideration holds when brittle structural collapse is included in the risk scenarios, as in the case treated in this paper.
The introduction of an expert System, capable of synthetizing the readings received with consistent structural interpretative models is the natural evolution of the on-line monitoring network. Being based on artificial intelligence techniques, the mandatory step towards the development of an expert system is the robust understanding of the structural behaviours possibly occurring in the operational life of the monitoring network. In section 6 the indications emerged from the study of the possible structural behaviours of the Pavia monuments are described. They are used as guiding indicators in the design of the expert system, whose main characteristics are summarized in section 7.

6. BEHAVIOUR ANALYSIS

Measures acquired by the automatic monitoring system allow to analyze the structural behaviour of the monuments and to evaluate their safety state.

The global displacements of the structures and the local deformation phenomena, especially for the Cathedral because of its complexity, were determined by an interpretative analysis of the measures, particularly referring to the most important structural parts (dome, drum and piers), considering horizontal displacements of the dome columns, displacements of the top of the dome, differential settlements of the columns bases and opening-closing cycles of the main cracks.

Planimetric displacements of the dome columns, measured by the telecoordinometers, mainly occur in radial direction with different values from column to column (the magnitude is about one millimeter). A strong correlation with air temperature variations has been evidenced. The measures acquired by the monitoring system have also allowed to point out daily phenomena, of the order of 0.1 millimeter. The different displacement amplitudes, both seasonal and daily, are due to the different structural stiffness and to the different insolation (greater amplitudes for columns placed on the South side of the structure).

Displacements measured at the dome top by optical sights have a components depending on temperature seasonal variations (Figure 4).

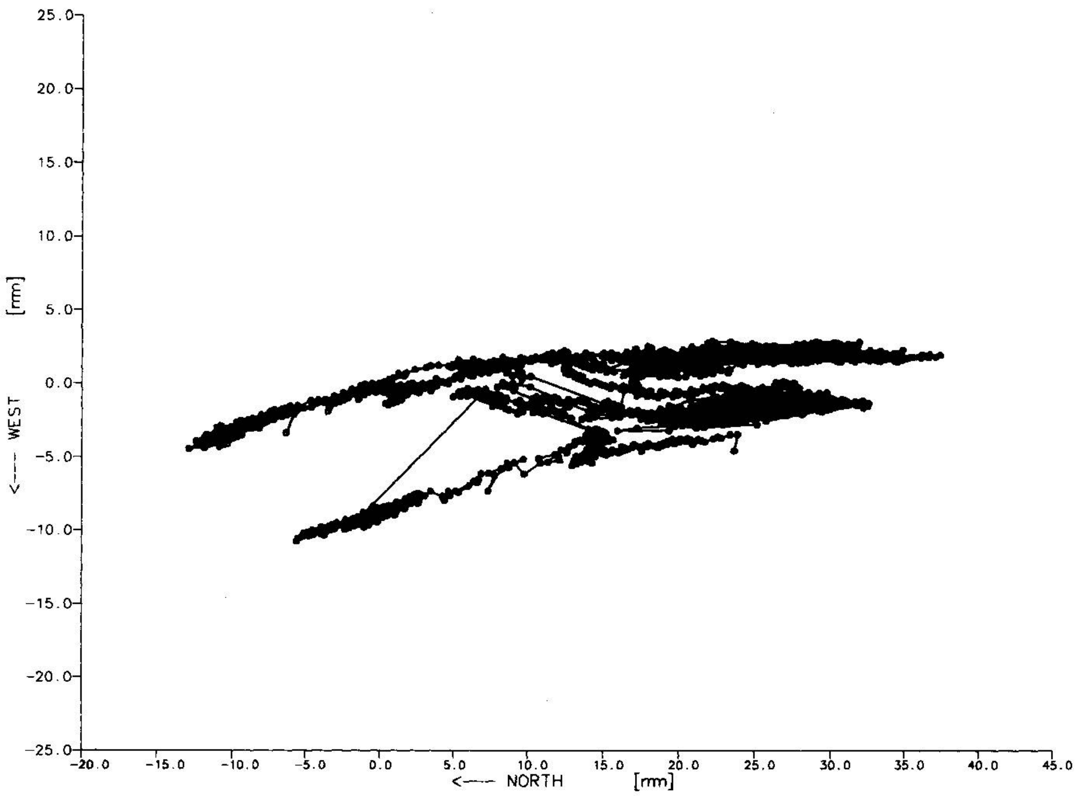

Figure 4: Planimetric displacements of the Cathedral dome

There is a logic correlation between piers horizontal displacements and vertical displacements of the dome top: in summer piers move outwards the octagon while the dome top lowers; on the contrary, in winter piers move inwards the octagon and the dome top lifts.

Cracks show periodic patterns according to the seasonal variation of the temperature, with different amplitudes: greater values are usually recorded by instruments placed on cracks near the nave arch between two dome columns.
Generally deformometers installed on cracks have provided values indicating regular stability, thus excluding drift phenomena.

7. RISK DIAGNOSIS

The behaviour analysis carried out on the Cathedral and on the Towers has pointed out the presence of particular cyclic phenomena with seasonal and daily period mainly linked to temperature variations. This analysis has also allowed to check the logical consistency between the information provided by different instruments (telecoordinometers on piers, optic sights on the dome top, strain gauges on cracks) concerned with the same phenomena.

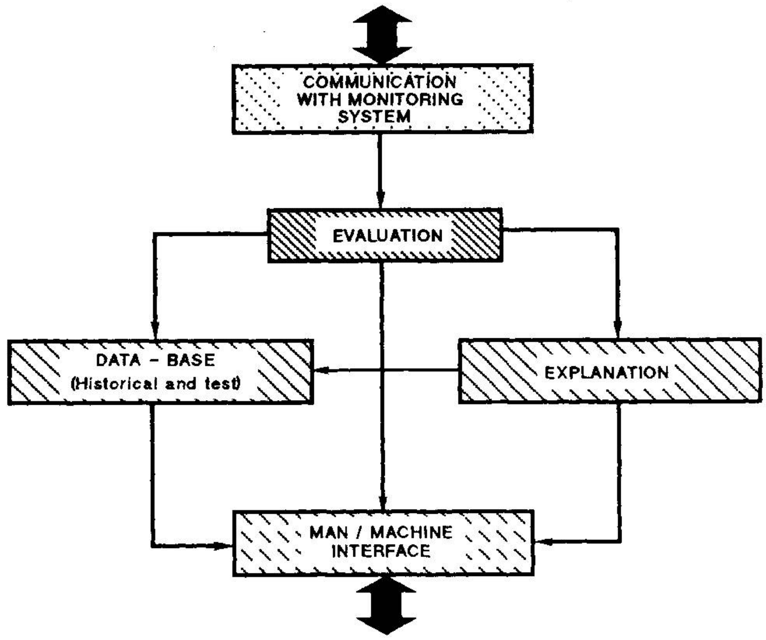

Figure 5: MISTRAL Architecture

These typical connections, both from a qualitative and quantitative point of view, have been introduced in MISTRAL expert system, developed by ISMES to manage and interpret signals coming from automatic monitoring systems installed on important structures. This system is already working for the on line behaviour evaluation of some dams.

Now applied for the first time on a monumental structure, it is experiented on the Pavia Cathedral with off line test. It completes the on line control procedure as showed in Fig. 5.

It allows to filter alarms coming from the automatic monitoring system, by intercepting signals due to accidental overcoming of the control thresholds, providing an explanation to its assumptions and an evaluation of the structural behaviour. This system, therefore, carries out in real time a part of the evaluations of the safety state of the structure, which are usually made off line by experts in safety problems.

From an engineering point of view, the expert system checks homogeneity, priority and congruence of each result of the on-line controls, according to reference structural behaviours. Each control is taken into account also considering the correctness of the installed instruments and their intrinsic and statistic reliability.

According to engineering considerations, it has been possible to determine measures groups concerned with the same process and physical phenomena interesting the structure. The control of the congruence relations allows to

determine the real state of each single measure group under consideration; from the local condition of the different groups it is possible to define the overall structure condition, according to considerations linked to experience and engineering evaluation.
MISTRAL expert system, filtering and classifying anomalies of each elementary control by comparison with reference structural behaviours, can point out the presence of real critical situations and generate alarm signals only for those conditions really interesting Control Experts. This expert system is therefore another instrument for Responsible for "surveillance", helping them to carry out their activity more timely and effectively.

8. CONCLUSION

The engineering knowledge and the learning of the structural behaviour of great modern civil buildings has increased drastically in the last years and has reached a very high level. A similar knowledge must be still developed for the historical monuments in order to define general criteria of the automatic surveillance using on line control systems. The problem involves great and objective difficulties; very soon there aren't geometric drawings, structures are complex and different from each other and building materials are often not known.
ISMES has gatered a great experience about historical monuments, both in the monitoring system installation and managing and in numerical mathematical modelling. Some very important examples are: Brunelleschi Dome in Florence , S. Marco Cathedral in Venice, Milano Cathedral, Loggia della Signoria in Florence, Atri Cathedral, Arezzo Cathedral and Pisa tower (as associated of "Consorzio Progetto Torre di Pisa"). These experiences has allowed to improve the technics and methods used in data analysis. Therefore the use of an expert system for the Pavia cathedral control is the result of an evolutive process.
If this project will provide positive results the expert system could become an important help instrument in solving problems related to preservation of the precious artistic Italian heritage and in preventing catastrophic accidents, such as in Pavia, from taking place.

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Diagnosis of Masonry Towers by Dynamic Identification

Détermination de l'état de tours en maçonnerie par identification dynamique

Diagnose von Mauerwerktürmen mittels dynamischer Identifikation

Michele FANELLI
Professor
ENEL-CRIS
Milan, Italy

Alberto PAVESE
Civil Engineer
Univ. of Pavia
Pavia, Italy

M. Fanelli, born in 1931, received his civil engineering degree at the Univ. of Bologna. He became Professor in Construction Techniques in 1970. He is director of the Centre for hydraulic and structural research of ENEL.

A. Pavese, born 1962, received his PhD from the Politecnico of Milan. His research activities are concentrated on dynamic tests and linear structural identification of masonry structures. He is now at the Univ. of Pavia with post-doctor fellowship.

SUMMARY

The behaviour of monument structures can be assessed by using experimental tests in the dynamic field. Structural characteristics and pathologies can thus be investigated. Two parametric techniques are presented to obtain an optimal numerical model. The first one is an iterative procedure, based on eigenparameter sensitivity detected by parametric studies and model-real behaviour comparison; and secondly, the test modelling is performed through special formulation of the model-real behaviour discrepancy function.

RÉSUMÉ

Le comportement des structures de monuments peut être déterminé par des essais dynamiques, ce qui permet de connaître les caractéristiques et l'état de la construction. Deux techniques paramétriques sont présentées qui permettent d'obtenir des modèles numériques optimaux: Une procédure itérative, basée sur la sensitivité des paramètres propres, détectés par des comparaisons entre des études paramétriques et du modèle réel ainsi qu'un modèle expérimental, réalisé à l'aide d'une formulation spéciale du comportement du modèle réel, basé sur une fonction de discrépances.

ZUSAMMENFASSUNG

Das Verhalten von historischen Bauwerken kann durch experimentelle dynamische Versuche bestimmt werden, die Rückschlüsse auf Eigenschaften und pathologischen Zustand der Gebäudestrukturen erlauben Zwei parametrische Techniken werden vorgestellt, die optimale numerische Modelle ermöglichen: zum einen ein iteratives Verfahren, bei dem für die Abstimmung zwischen Modell- und Prototypverhalten die Empfindlichkeit der Eigenschwingungsparameter aufgrund von Parameterstudien eingesetzt wird; zum anderen eine Optimierung mittels einer speziell formulierten Diskrepanzfunktion.

1. INTRODUCTION

The knowledge of the behaviour of ancient masonry buildings is totally inadequate for attempting to construct any kind of reasonably representative numerical model. The main problems which arise stem from the identification of the composition ratios of the material properties.
The research basically aims at evaluating the possibility of identifying a numerical model of the towers through dynamic experiments, and at studying the sensitivity of the techniques applied in identifying any local defects in order to diagnose the presence of local defects (i.e. voids, cracks, material degradation). The experiments were preceded by numerical studies with simple models aimed at evaluating the degree of significance of the various parameters involved and at defining their probable ranges of variation.
Scope of this paper is to illustrate the results obtained from vibration tests performed on medieval towers and then to propose one method for identifying stiffness matrix based both on numerical-experimental comparison and sentivity analysis. An alternative identification technique named "PLECTRON" is illustrates at the end of the paper and an example of application will be presented in order to demonstrate its limits and capabilities in real cases.

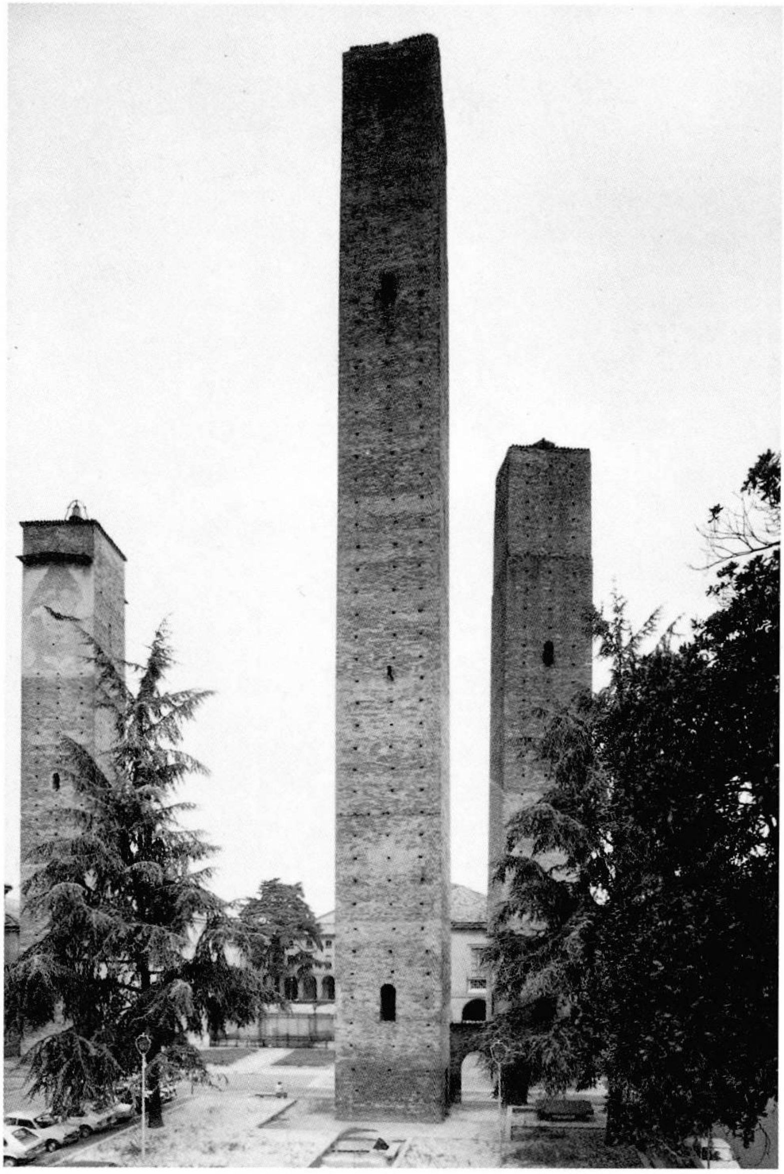

Figure 1 Orologio's, Majno's, and Fraccaro towers

The dynamic identification study presented here refers to University tower in Pavia, also known as the Fraccaro tower (Fig. 1 the right one). Fraccaro tower extend vertically for approximately 40 m above ground, while has openings variously arranged along the height and in plan, and rests on the ground by means of a foundation block whose dimensions are similar to the parts above ground and whose height is approximately 5 m. The masonry of the tower consists of two brickwork faces, about 15 cm thick, the intervening void being filled with mortar mix, bricks and stonework, whose thickness varies according to the height. The composition features of the foundations are similar to elevation walls. As far as the construction date is concerned, some historians date the towers in the early years of our millenium. Laboratory tests performed on specimens taken from blocks of masonry of the Civica tower, which collapsed in March 1989, have provided information on the mechanical properties of the material [1,2]. Main results of these tests are summarized in table 1

f_u	3.1	0.17	4.1	0.08
f_{fessur}	2.5	0.20	3.1	0.13
E_t	3314	0.31	2906	0.09
E_s	2670	0.34	2654	0.22
v_t	0.16	0.27	0.22	0.45
v_s	0.32	0.05	0.35	0.02

Table 1 Results tests performed on specimens taken from blocks of masonry of the Civica tower

Further studies are perfomed on others two towers in figure 1, Majno's tower and Orologio's towers bat the results will be object of a successive work.

2. DYNAMIC MEASUREMENTS IN SITU

For the purposes of comparison on the Fraccaro tower, the techniques of ambient vibration and forced excitation were used in the present study. To this end velocity transducers, seismometers and Ladirs (which are coherent light sensors based on the Michelson's interferometer) are used in order to study oscillation induced by the exciting source.
The signals were stored in a digital memory and processed by a spectrum analyser using windowing tecnique in order to obtain better definition of the spectrum amplitudes (Flat-top window), resonance frequencies and damping ratios (Hanning and uniform windows). Table 2 reports the results of the spectrum analysis of the signals, recorded during two differents tests on Fraccaro tower in terms of resonance frequencies and damping ratios determined by the half-power method.
The forced vibration tests with a vibrodyne are currently one of the most effective methods for dynamically caracterizing a structure. In the case in question a rotating vector generating vibrodyne was cantilever-mounted on the north wall of the Fraccaro Tower at a height of 27.25 m.

ω_i	Ambient tests		Forced tests July 90				Ambient tests		Forced tests July 91			
	July 90 (E-W)		Direct. E-W		Direct. N-S		July 91 (E-W)		Direct. E-W		Direct. N-S	
	Hz	ξ	Hz	ξ	Hz	ξ	Hz	ξ	Hz	ξ	Hz	ξ
1° flex	0.75	2.70	0.73	2.70	0.76	2.60	0.74	2.7	0.72	2.4	0.740	2.30
2° flex	3.56	1.70	3.46	1.70	3.58	1.50	3.54	1.2	3.44	1.0	3.520	1.00
1° tors	4.25	0.80	4.18	1.0	4.18	1.0	4.36	0.7	4.24	1.1	4.240	1.10
1° axial	-	-	-	-	-	-	6.30	5.5	-	-	-	-
3° flex	8.00	1.10	7.86	2.50	7.76	1.50	8.18	2.3	7.84	2.5	7.960	2.70
2° tors	11.44	0.60	-	-	-	-	11.70	0.3	11.60	1.0	11.60	1.00
4° flex	-	-	-	-	-	-	13.90	3.5	13.72	4.0	13.80	3.50

Table 2 Ambient and forced vibrationts tests - Frequencies and damping ratios

The results are also reported in table 2; it is important to note that by comparison of the results obtained by forced tests, only in W-E direction, with the corresponding values obtained through ambient vibration tests it is possible to verify a good correspondence up to the third frequency of vibration (torsional shape); beyond this limits the tower behaviour becomes dependent on the intensity of exciting force. This can determine differente values both in frequencies and damping, probably due to the fact that different mechanisms are involved in different levels of forces. Same conclusion is also possible to do it on damping ratios.

3. NUMERICAL MODELLING

The Fraccaro tower was modelled with linear elastic solid elements. The use of a linear elastic model to represent a structure which by its very nature is non-linear can be justified by the fact that the level of force with which the dynamic tests were performed limited, generally not greater than 5 kN. The mechanical properties of the materials used in modelling are those obtained from the laboratory tests and in situ [1,2]. Table 3 give the results obtained from the numerical analysis compared with corresponding experimental values.

Natural	Direction N-S			Direction E-W		
Frequencies	Experim.	Numerical	Δ	Experim.	Numerical	Δ
	Hz	Hz	%	Hz	Hz	%
1°flex	0.73	0.709	-2.87	0.76	0.705	-7.2
2°flex	3.46	3.834	+10.8	3.58	3.867	+8.0
1°tors	4.18	5.059	+21.0	4.18	5.059	+21.0
3°flex	7.86	9.395	+19.5	7.76	9.406	+21.0

Table 3 experimental and numerical eigenvalues before identification procedure

The error in evaluating resonance frequencies is particularly evident in the torsional mode and in the 3rd flexural mode, in this case with approximately a 20% error is shown. The modal parameters of the structure depend on stiffness, mass distrbution, restraint conditions and dissipative mechanisms characteristics. Further on it is assumed that the terms of the mass matrix are determined with sufficient precision from the volumes calculated and from the specific density and that any cavities are such that they do not cause appreciable variation.

Natural	Direction N-S			Direction E-W		
Frequencies	Experim.	Numerical	Δ	Experim.	Numerical	Δ
	Hz	Hz	%	Hz	Hz	%
1°flex	0.73	0.727	-0.40	0.76	0.741	-2.5
2°flex	3.46	3.621	+4.65	3.58	3.732	+4.24
1°tors	4.18	4.242	+1.48	4.18	4.242	+1.48
3°flex	7.86	8.494	+8.07	7.76	8.550	+10.2

Table 4 Experimental and numerical eigenvalues after identification procedure

The damping ratio being not greater than 2.7% it appears resonable to neglect in the calibration procedure the dissipative behaviour. In the model the tower was divided in 13 zones delimited by orizontal planes and moduli of elasticity of the portions were made to vary within a range of possible values, determining the ensuing variations in frequencies and modal forms. The variations in E are considered on individual layers of the mesh, maintaining the value constant on the rest of the tower and eliminating the combinations of variation in the various layers, which would otherwise increase the number of analyses required Fig. 2 and 3 shows the variations in modal and frequencies domain. It can be seen that the rotations of the torsional mode are very sensitive to the changes in modulus of elasticity over most of the height, whereas in the 2nd and 3rd bending modes the positions of the nodes undergo appreciable changes when the variations in E affect the zones of maximum bending of the modes. On observing the results obtained in the modal analysis of the mesh with uniform E, a tendency to overestimate the frequencies and ratios in relation to the experimental can be noted. By comparing then the experimental shapes, e.g. in direction E-W, is possible to note that position of node and maximum deflection of the modal shapes are localized in different heights with respect

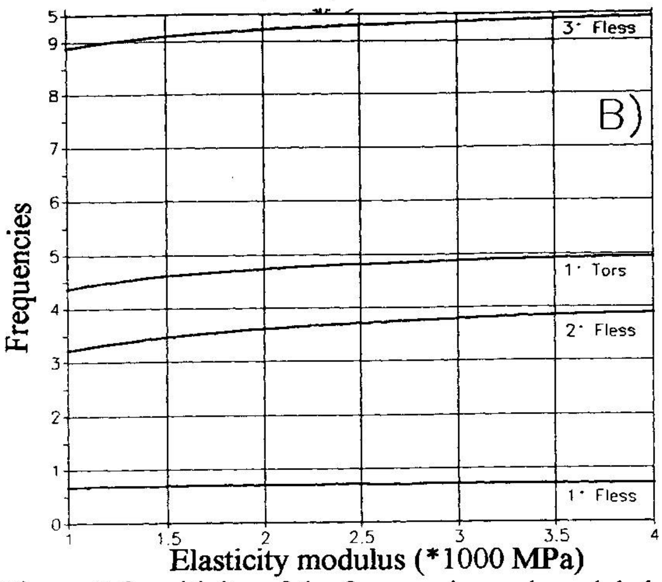

Figure 2 Sensitivity of the frequencies and modal shapes

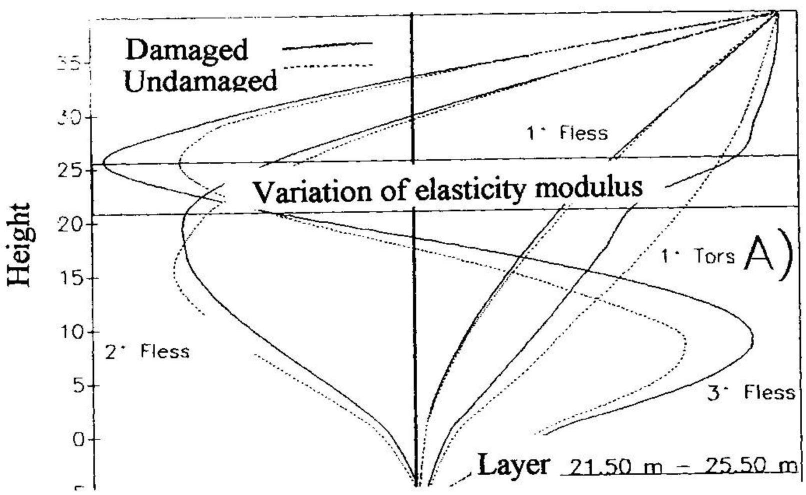

Figure 3 Sensitivity of the frequencies and modal shapes

to experimental observation. By adjusting the values of modulus of elasticity by successive approximations in the zones of maximum bending, a distribution is achieved which provides the frequency response given in Table 3. It can easily be seen that the frequencies have values closer to those of the experimental case. The result was obtained by modifying the modulus of elasticity in the zones of maximum bending of the second and third modal shapes.

5. IDENTIFICATION BY INVERSE PARAMETRIC TECHNIQUE

At the most basic level, it is common practice to "adjust" the modulus of elasticity value in order to bring into line the value of the first natural frequency calculated, by means of a mathematical model of the structure, with that obtained from field. It is however commonly found that, having made this adjustment, there is then no automatic agreement between the "theoretical" and experimental values of the successive frequencies (see previous section for example). It has recently been established that important information is contained not only and not so much in the natural frequency values, but also and possibly above all in the corresponding modal shapes obtained from measurements made on the

actual object. Matching the mathematical model and actual structure through differentiated adjustment by zones of the elastic constants is normally performed by trial and error, aiming at "matching" on the modified mathematical model the greatest number of natural frequencies and corresponding modal shapes. If it is true, as supposed, that the structure provides us -through test results- with a large copy of information revealing its modal shapes, it is worthwhile investigating whether it is possible to exploit this information directly in a zonal parameters identification.
A direct identification procedure of this kind can be produced in theory, apart from practical difficulties of performance and leaving aside momentarily the problems of precision and reliability of results. The last part of the paper intends to analyses the potentialities of this method in view of a future application at a reale structure like a the University's tower.
We shall consider a structure for which the modal shapes of the first natural P modes have been extracted from in situ tests. We then suppose that this structure can be discretized by means of a suitable linear elastic mathematical model; with reference to unforced, undamping vibration problems we can write the system equilibrium equations

$$\underline{\underline{K}}\underline{\Phi}_j = \omega_j^2 \underline{\underline{M}}\underline{\Phi}_j \quad (1)$$

where $\underline{\underline{K}}$ and $\underline{\underline{M}}$ are stiffness and mass matrix of reference structure, ω_j is the jth eigenvalues whereas $\underline{\Phi}_j$ is the corresponding eigenvector obtained from experimental tests.
We can note how the matrix of rigidity K of the mathematical model may be written formally as follows:

$$\underline{\underline{K}} = \varepsilon_1 \underline{\underline{K}}_1 + \varepsilon_2 \underline{\underline{K}}_2 + \cdots + \varepsilon_n \underline{\underline{K}}_n \quad (2)$$

where $\underline{\underline{K}}_i$ are the matrices of rigidity constructed for the entire discretization on the basis of the incidence matrices of the individual zones defined previously to differentiate the elasticity constants, whereas ε_i are the multiplicative coefficients of a reference elasticity modulus.
Similarly we can write for the matrix of mass M:

$$\underline{\underline{M}} = \mu_1 \underline{\underline{K}}_1 + \mu_2 \underline{\underline{M}}_2 + \cdots + \mu_n \underline{\underline{M}}_n \quad (3)$$

where $\underline{\underline{M}}_j$ are the mass matrices constructed for the entire discretization, whereas μ_j are multiplicative coefficients of reference density.
We now wish to establish that the mathematical model with differentiated elasticity and inertia constants matches as perfectly as possible the frequencies and modal shapes determined in the experimental tests. If it were possible to achieve perfect agreement, for each mode j, clearly being ω_j^2 and $\underline{\Phi}_j$ eigenparameters correlated to tested structure and $\underline{\underline{K}}$, $\underline{\underline{M}}$ matricies connected to reference structure wich behaviour we want to superimpose to real one, in general we can write

$$\underline{\underline{K}}\underline{\Phi}_j - \omega_j^2 \underline{\underline{M}}\underline{\Phi}_j = \underline{D}_j \neq \underline{0} \quad (4)$$

Taking into account (2), (3) e (4) are able to construct a global error norm on the p modes:

$$N = \sum_{j=1}^{p} \underline{D}_j^t \underline{D}_j = \sum_{j=1}^{p} (\underline{\Phi}_j^t \underline{\underline{K}}^t - \omega_j^2 \underline{\Phi}_j^t \underline{\underline{M}}^t)(\underline{\underline{K}}\underline{\Phi}_j - \omega_j^2 \underline{\underline{M}}\underline{\Phi}_j) \quad (5)$$

where evidently

$$N = N(\varepsilon_1, \cdots, \varepsilon_n, \mu_1, \cdots, \mu_m) \quad (6)$$

The criterion of maximum likelihood of the mathematical model is now to be expressed in the conditions of minimising the global error norm N, in relation to the adjustment variables ε_i and μ_j:

$$\frac{\partial N}{\partial \varepsilon_1} = 0, \cdots\cdot, \frac{\partial N}{\partial \varepsilon_n} = 0$$
$$\frac{\partial N}{\partial \mu_1} = 0, \cdots\cdot, \frac{\partial N}{\partial \mu_m} = 0 \qquad (7)$$

wich is a linear system in n+m equation; in matrix form we can write

$$\underline{\underline{A}}(\varepsilon_i, \mu_j) = \underline{0} \qquad (8)$$

It is possible to verify that one of the equations in (8) is linearly dependent on the others. Therefore ε_i and μ_j are determined, assuming one of them to be known. In general it will be possible to take one of the densities as known: it can immediately be seen that an error in the estimate of the density supposed to be known, expressed by a factor μ_j relation to the "true" value, involves an equal factor of error for all the densities, and factor likewise equal to μ for all the moduli of elasticity. That procedure directly and simultaneously takes into account experimental information (ω_j and $\underline{\Phi}_j$).
The question naturally arises whether the proposed procedure can actually be performed and whether it is effective. Without prejudice to the fact that only application to sufficiently complex real cases can demonstrate the effective potential and limits of the procedure, the intention was to test it on a very simple case, which we shall now illustrate. With reference to Fig. 4, we have taken the system with 20 degrees of freedom. Experimental test were simulated by a F.E. code.

Real values	Refer. values	ε_i, η_j	Identification results 1° mode		2° mode		3° mode	
5000	1000	5.0	2.7976	4.9745	4.9996	4.9999	5.0004	5.0000
4000	1000	4.0	2.2381	3.9796	3.9996	3.9999	4.0004	4.0000
3500	1000	3.5	1.9583	3.4821	3.4997	3.4999	3.5003	3.5000
3000	1000	3.0	1.6785	2.9847	2.9997	2.9999	3.0003	3.0000
2500	1000	2.5	1.3988	2.4872	2.4998	2.4999	2.5002	2.5000
2000	1000	2.0	1.1190	1.9898	1.9998	1.9999	2.0002	2.0000
2500	1000	2.5	1.3987	2.4872	2.4997	2.4999	2.5002	2.5000
3000	1000	3.0	1.6785	2.9846	2.9997	2.9999	3.0002	3.0000
3500	1000	3.5	1.9582	3.4821	3.4996	3.4999	3.5003	3.5000
4000	1000	4.0	2.2380	3.9816	3.9996	3.9999	4.0003	4.0000
26873	26873	1.0	1.0000	0.9949	1.0000	0.9999	1.0000	1.0000
25420	25420	1.0	0.5595	0.9949	0.9999	0.9999	1.0001	1.0000
29294	29294	1.0	0.5595	0.9949	0.9999	0.9999	1.0000	1.0000
25562	25562	1.0	0.5594	0.9948	0.9999	0.9999	1.0001	1.0000
23000	23000	1.0	0.5595	0.9949	0.9999	0.9999	1.0000	1.0000
17915	17915	1.0	0.5593	0.9946	0.9999	1.0000	1.0001	1.0000
15494	15494	1.0	0.5597	0.9952	0.9988	0.9989	1.0000	1.0000
14525	14525	1.0	0.5595	0.9948	0.9999	0.9989	1.0000	1.0000
14525	14525	1.0	0.5595	0.9946	0.9999	0.9999	1.0000	1.0000
7262	7262	1.0	0.5595	1.0000	0.9999	1.0000	1.0000	1.0000

Table 5 Identification of stiffness and mass constants using flexural shapes

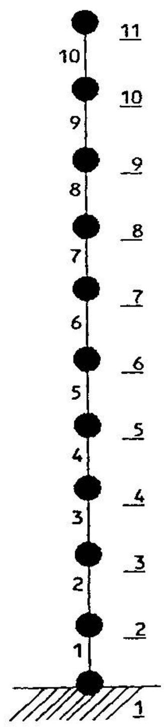

Fig.4 Example

In table 5 are shown the identification results; the first ten values are the coefficients ε_i, whereas the others values are related to mass concentrated to nodal points. Underlined values were imposed in the identification procedure. It should be noted that there is greater precision in the parameters estimation when mass was imposed at the top of the cantilever, according to the fact that in this zone the contribution to total kinetic energy takes on significant value. In general accuracy increase according to modal shapes with a large number of nodes.

4. CONCLUSIONS

This work illustrates two identification methods. The first method, useful in preliminary structural caracterization, is based on an iterative procedure of arrangement of elasticity modulus distribution using sensitivity analysis and numerical-experimental comparison. The second method basically consists of modifying the stiffness matrix of the structure on the basis of the frequencies and of the modal shapes measured experimentally. This method is translated in a calculation program obviously named "PLECTRON". This method have demonstrated good precision and accuracy in a simple case of application bat presently is waiting further applications to check its efficiency in practical problems Finally we will attempt to make some preliminary critical considerations on the method proposed. It is clearly that for an effective application to cases of a complexity corresponding to objects of actual importance, the information on the experimental modal shapes would have to be accurate and complete. Since in practice a completeness with the same detail as the discretization of the mathematical model is not feasible, the experimental information will have to be completed adequately before being able to develop the method at a numerical level. This could be performed for example using cubic splines for those components of the modal vectors some of which are in fact measured (e.g. the orizontal displacements due to the flexural deformations of a tower). The diagnostic aspect of the proposed method would also appear to be of importance, not only in a comparative sense, i.e. to reveal any variations in time between one in-situ test and another, but in an absolute one with regard to the space variation of variables. In concept, the method could alos be extended to achieve a differentiated estimates of damping coefficients of the various zones (e.g. introducing complex rather than real ε_i adjustment factors in the case of Hysteretic damping).

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Diagnose und Ueberwachung des Tragverhaltens von Glockentürmen

Analysis and Monitoring of the Safety of Bell Towers

Analyse et surveillance de la sécurité des clochers

Wolf-Jürgen GERASCH
Civil Eng.
Univ. Hannover
Hannover, Germany

ZUSAMMENFASSUNG

Die Messung von Eigenfrequenzen, Eigenschwingungsformen und Dämpfung bei Glockentürmen ist bei historischen Bauwerken eine Voraussetzung, um die wirklich einwirkenden Kräfte zu ermitteln. Die beim Läuten der Glocken auf die Türme einwirkenden dynamischen Kräfte, können zu Resonanzschwingungen anfachen. Die Berechnung der Eigenfrequenzen ist unpräzise, weil wesentliche Parameter nur ungenau ermittelt werden können. Untersuchungsergebnisse zeigen, dass die Standsicherheit von Türmen über die Messung der Eigenfrequenzen und der Eigenschwingungsformen überwacht werden können. Schäden, welche die Standsicherheit der Türme beeinträchtigen, werden durch diese Methode festgestellt.

SUMMARY

The measurement of natural frequencies, eigenforms and damping of bell towers is a prerequisite for determining the forces effecting historical buildings. The dynamic forces engaged when ringing the bells may lead to resonance vibrations of the structure. The calculation of natural frequencies is not precise, because important parameters are unknown. Test results prove that the stability of towers can be assessed by measuring the natural eigenforms. Cracks, which are a risk for the stability of towers, will be discovered. Damages having an influence on the safety of towers can be identified with this method.

RÉSUMÉ

La mesure des fréquences naturelles et des vibrations et amortissements correspondants de clochers est nécessaire, dans les bâtiments historiques, pour déterminer les forces réellement en présence. Les forces dynamiques agissantes sur les clochers lors de la sonnerie des cloches, peuvent mettre la construction en résonance. Le calcul des fréquences naturelles n'est pas trés précis car des paramètres essentiels ne sont connus que de façon approximative. Les résultats d'étude ont montré que la sécurité des tours peut être contôlée sur la base des mesures de fréquences naturelles et de vibrations correspondantes. Les dégâts qui influencent la résistance des tours peuvent être déterminés à l'aide de cette méthode.

1 Einleitung

Schwingungsmessungen an Kirchtürmen werden zur Zeit vorwiegend aus zwei Gründen durchgeführt. Sind Schäden an einem Turm vorhanden, so wird untersucht, ob die Schäden durch die dynamischen Kräfte, die vom Glockengeläut auf die Konstruktion einwirken, verursacht worden sind. Sollen in einem Turm weitere Glocken eingebaut werden, so ist festzustellen, ob die zusätzlich auftretenden dynamischen Kräfte vom Turm aufgenommen werden können. Es fehlen aber systematische Untersuchungen, z.B. auch an Kirchtürmen, um Veränderungen des globalen Zustandes der Tragkonstruktion zu beschreiben. An einem Beispiel wird gezeigt, daß, trotz Befürchtungen von Statikern, sich das Tragverhalten eines Kirchturmes in einem Zeitraum von 20 Jahren nicht verändert hat. Bei einem zweiten Turm wurden Mauerwerksbögen und ein Teil des Mauerwerks abgebrochen. Die Stabilität des Turmes wurde vor dem Abbruch und danach ermittelt. Dabei wird auch auf die Besonderheit der Glockentürme eingegangen. Die Eigenfrequenzen und Eigenschwingungsformen eines weiteren Kirchturmes wurden gemessen. Es wurde anschließend ein mathematisches Modell (FE-Modell) erstellt, das die gleichen Eigenfrequenzen wie der Glockenturm aufweist. Das Eigenschwingungsverhalten des Modells wird außerdem für Mauerwerk mit durchgehenden Vertikalrissen untersucht.

2 Diagnose

Bevor mit der Sanierung von Schäden an historischen Bauwerken begonnen wird, sollte die Ursache, die für den Schaden verantwortlich ist, gefunden werden. Da Kirchtürme im wesentlichen Wind- und Glockenkräften ausgesetzt sind, ist unter anderem zu untersuchen, ob diese Kräfte die Schäden verursacht haben können. Dort wo starker Straßenverkehr ist, wird auch untersucht, ob die Erschütterungen zu einer Umlagerung des Bodens unterhalb der Fundamente führen konnten, so daß unterschiedliche Setzungen der Konstruktion möglich waren.

Die Beanspruchung der Türme durch die Windbelastung kann vom Statiker abgeschätzt werden. Zur Zeit werden Messungen an einem Kirchturm durchgeführt, um die Windlastannahme noch zu verbessern. Es ist aber in der Regel bei historischen Kirchtürmen nicht möglich die dynamische Belastung aus Glockengeläut genau genug zu ermitteln, da die Steifigkeit, die Masse und der Elastizitätsmodul des Mauerwerks häufig nicht ausreichend genau vorliegen. Die Beanspruchung der Kirchtürme durch das Glockengeläut hängt aber wesentlich vom Verhältnis der Glockenerregerfrequenzen und der wesentlichen Eigenfrequenzen des Turmes ab (Resonanzerregung)[1]. Die Eigenfrequenzen und Dämpfung der Türme lassen sich aus Messungen sehr leicht ermitteln. Sind hochempfindliche Schwinggeschwindigkeits- oder Schwingbeschleunigungsaufnehmer vorhanden, so ist schon eine Windgeschwindigkeit von $v \geq 3\ m/s$ ausreichend, um die Eigenschwingungsgrößen aus Messungen zu ermitteln. Der Winddruck in Windrichtung stellt eine stochastische Erregung dar, bei der der Turm in seinen Eigenfrequenzen und Eigenschwingungsformen antwortet. Die Autokorrelationsfunktion [2]

$$\psi(\tau) = \lim_{T \to \infty} \frac{1}{T} \int_0^T y(t) \cdot y(t+\tau)\, dt \qquad (1)$$

der Antwort eines gedämpften Systems ist bei stochastischer Anregung identisch mit der aus einem Ausschwingvorgang (freie Schwingung) zu gewinnende Abklingkurve.

$$\psi(\tau) = \text{Konst.} \cdot e^{-\delta \cdot f \cdot \tau} \qquad (2)$$

mit: δ: log. Dekrement f: Eigenfrequenz

Somit sind aus den Messungen die wesentlichen Eigenschwingungsgrößen bekannt. Häufig besteht auch die Möglichkeit, mittels einer Impulserregung (Springen im Turm in Horizontalrichtung) aus der freien Schwingung die Eigenfrequenzen, Eigenformen und Dämpfungen zu ermitteln. Besonders geeignet sind Türme mit einer Eigenfrequenz von $f = 0,5\ Hz \div 2,0\ Hz$. Liegen steifere Konstruktionen vor, so reicht der Impuls für eine Anregung nicht aus. Die Bewegungen, auch Rißbewegungen, des Turmes bei Glockengeläut können gemessen werden oder bei Kenntnis der Glocken kann eine theoretische Berechnung durchgeführt werden.

Das Glockengeläut erzeugt dynamische Kräfte mit tiefen Frequenzen $f < 2,0\ Hz$ durch die schwingende Masse der Glocken. Beim Anschlagen des Klöppels an die Glocken wird wegen des kurzen Impulses ein Spektrum bis $f \cong 500\ Hz$ angeregt. Die hochfrequenten Wellen werden über den Glockenstuhl in das Mauerwerk transportiert. Ungünstig sind Stahlglockenstühle, da die hohen Frequenzen ungedämpft in das Mauerwerk übertragen werden. Treten im Mörtel hohe Beschleunigungen auf, so ist nicht auszuschließen, daß er schneller altert. Es können zur Zeit aber noch keine quantitativen Angaben über die Wirkung von Beschleunigungen auf die Alterung des Mörtels vorgelegt werden.

Führen stark befahrene Straßen dicht an einem Kirchturm vorbei, so ist auch zu untersuchen, ob die vom Straßenverkehr im Boden erzeugten Schwingungen zu einer Kornumlagerung führen, so daß es zu unterschiedlichen Setzungen der Konstruktion kommen kann. Diese Gefahr bestand in der Vergangenheit sicherlich wesentlich häufiger, da die Fahrzeuge nicht abgefedert waren und ein unebener Straßenbelag vorlag. In Abhängigkeit vom Boden können Beschleunigungsamplituden von $a \geq \frac{1}{10}\ g$, [g: Erdbeschleunigung] die unterhalb der Fundamente im Boden auftreten, Bodenumlagerungen bewirken. Führen asphaltierte oder verkehrsberuhigte Straßen an Kirchtürmen vorbei, so sind durch die vorhandenen Schwingungen keine Bodenumlagerungen zu erwarten.

3 Überwachung

Auf dem Weg zu einer Diagnose ist es auch erforderlich, daß die dynamischen Eigenschaften des Tragwerks beschrieben werden. Mit Kenntnis dieser Eigenschaften kann über eine Langzeitüberwachung die Standsicherheit eines Turmes beobachtet werden. Eine Schwächung des Mauerwerks, d.h. ein Verlust an Stabilität — durch Vertikalrisse — muß sich, wenn sie für die globale Standsicherheit einen Einfluß hat, in der Veränderung der Eigenfrequenzen und Eigenschwingungsformen wiederfinden.

Zeigen sich bei der Überwachung über Jahre keine Veränderungen in den dynamischen Eigenschaften eines Turmes, so ist auch davon auszugehen, daß bezogen auf das globale Tragverhalten, keine Änderungen eingetreten sind.

Die tiefsten Eigenfrequenzen von Kirchtürmen setzen sich aus der Drehung auf dem Untergrund und der Biegung des Turmschaftes zusammen.

$$f_{ges} = \frac{f_D \cdot f_B}{f_D + f_B} \tag{3}$$

Es müßte noch untersucht werden, ob durch die Veränderung der Drehfrequenz auf Vorgänge im Boden geschlossen werden kann. Die reinen Biegeeigenfrequenzen können aus den Meßsignalen der Dehnungsmeßstreifen berechnet werden. Die Schwinggeschwindigkeits- oder Schwingbeschleunigungsaufnehmer, die im Turm installiert sind, zeigen f_{ges}, so daß f_D berechnet werden kann.

Treten Frequenzänderungen in Kirchtürmen auf, so können diese zu zwei Erscheinungen führen:

1.) Eintreten von Resonanzschwingungen (Glockenerregerfrequenz = Eigenfrequenz des Turmes),
2.) Minderung des globalen Tragverhaltens.

Die Möglichkeit, daß eine Schwächung des Querschnittes zu einer Vergrößerung der dynamischen Lasten führen kann, stellt, bezogen auf historische Bauwerke, sicherlich eine Ausnahme dar, die für diese Tragwerke aber von Bedeutung sein können. Frequenzveränderungen von $\Delta f = 0,01\ Hz$ können dargestellt werden. Somit sind Steifigkeitsveränderungen, die bei 2% liegen, zu beobachten.

4 Beispiele

4.1 Kirchturm A (Langzeitüberwachung)

Ein Kirchturm, der eine Höhe von 50 m aufweist, war 1974 von Prof. Dr. F. Müller auf sein Eigenschwingungsverhalten und das Schwingungsverhalten bei Glockengeläut (5 Glocken) untersucht worden. Nach 18 Jahren sollte festgestellt werden, ob sich das Eigenschwingungsverhalten des Turmes verändert hat, da der subjektive Eindruck bei einigen Beobachtern zu dem Ergebnis geführt hatte, daß sich die Schwingungen bei Glockengeläut in den letzten Jahren vergrößert hätten.

Die Messungen im Jahr 1974 hatten zu folgenden tiefsten Eigenfrequenzen geführt: $f_x = 1,16\ Hz$ (Glockenschwingrichtung) und $f_y = 0,95\ Hz$ Die Dämpfung war zu $\delta = 0,15$ (richtungsunabhängig) berechnet worden.

Dem Spektrum bei Winderregung konnten bei der Messung im Jahr 1992 die Eigenfrequenzen und Dämpfungen entnommen werden. *(Bilder 1 u. 2)*

$$f_x = 1,15 \pm 0,01\ Hz \qquad \delta_x = 0,11$$
$$f_y = 0,93 \pm 0,01\ Hz \qquad \delta_y = 0,14$$

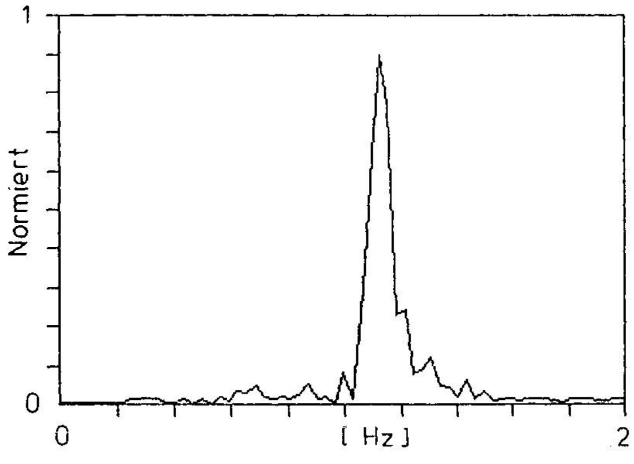

Bild 1: Spektrum bei Winderregung

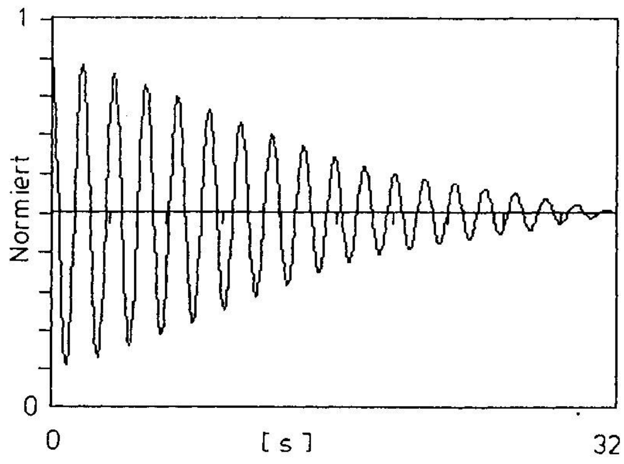

Bild 2: Autokorrelationsfunktion

Die maximalen Amplituden an der Turmspitze bei Glockengeläut ergaben sich zu $s = \pm 5,5\ mm$ (1974) und $s = \pm 4,3\ mm$ (1992). Der geringe Unterschied hat seine Ursache im Einbau eines neuen Glockenstuhls im Jahre 1985, als die Klöppelanschlagzahl der großen Glocke von 46 Schlägen/Min. auf 40 Schläge/Min. reduziert wurde. Die maximalen Dehnungen ergaben sich bei vollem Geläut zu $\varepsilon = 4,0 \cdot 10^{-5}$. Wird für das Mauerwerk ein mittlerer E-Modul von $E = 4,5 \cdot 10^{-3}\ MN/_{m^2}$ eingesetzt, so ergibt sich eine maximale Spannung aus Glockengeläut von $\sigma_{max} = \pm 180\ kN/_{m^2}$. Ist die Schwinggeschwindigkeitsamplitude an der

Spitze des Turmes bekannt, so kann die Spannung im Mauerwerk auch über die Beziehung [3]

$$\sigma_{max} = \sqrt{E \cdot \varrho} \cdot 1,71 \cdot 1,0 \cdot v_{max}$$

berechnet werden. Bekannt sein müssen ebenfalls der E-Modul und außerdem die Dichte des Mauerwerks. Bei einer Schwinggeschwindigkeitsamplitude von $v_{max} = 30\ mm/s$ wird die Materialspannung zu:

$$\sigma_{max} = \sqrt{4,5 \cdot 10^6 \cdot 1,8} \cdot 1,71 \cdot 0,03 = 146\ kN/_{m^2}.$$

Da die auf den beiden unterschiedlichen Wegen ermittelten Spannungen gleich groß sind, kann der E-Modul folgendermaßen berechnet werden:

$$\begin{aligned} \varepsilon \cdot E &= \sqrt{E \cdot \varrho} \cdot 1,7 \cdot 1,0 \cdot v_{max} \\ \wedge \quad E &= \frac{(1,7 \cdot 1,0 \cdot v_{max})^2 \cdot}{\varepsilon^2} \end{aligned} \tag{4}$$

Die Dichte des Mauerwerks muß bekannt sein, um den E-Modul berechnen zu können. Wird der E-Modul über die Laufzeit der Longitudinalwelle ermittelt

$$E = v_L^2 \cdot \varrho \tag{5}$$

so muß ebenfalls die Dichte bekannt sein.

Obwohl beim Glockengeläut sehr stark spürbare Schwingungen im Kirchturm erzeugt werden, sind bisher keine Schäden aufgetreten und die Systemeigenschaften haben sich nur unwesentlich verändert. Der Abstand zwischen der dritten Erregerfrequenz der großen Glocke und der Eigenfrequenz liegt bei

$$\eta = \frac{f_{Err.}}{f_{Eig.}} = \frac{1,0}{1,16} = 0,86$$

Der Vergrößerungsfaktor der Kräfte der Glocke 1 in der dritten Erregerfrequenz liegt bei V = 4,5. Wird der Kirchturm durch Vertikalrisse, d.h. durch die Schwächung des Querschnitts, weicher, so kann sich der Vergrößerungsfaktor auf V = 28,5 erhöhen, was zu Schäden an der Konstruktion führt.

Immer dann, wenn die Eigenfrequenz etwa 10%÷20% über der dritten Erregerfrequenz einer großen Glocke liegt, sollten in Abständen von etwa zehn Jahren erneut Messungen erfolgen. Es gibt zahlreiche Kirchtürme bei denen das Verhältnis $\eta = 0,8 \div 0,95$ beträgt und bei denen eine Veränderung der Steifigkeit zur wesentlichen Erhöhung der Kräfte führt.

Werden Risse beseitigt, so kann sich die Eigenfrequenz, da das Bauwerk eine größere Steifigkeit erhält, erhöhen, und somit von

$$\eta = \frac{f_{Err.}}{f_{Eig.}} > 1,0$$

zu $\eta = 1,0$ streben, wodurch die sanierte Konstruktion stärker beansprucht werden kann als vor der Sanierung.

4.2 **Kirchturm B** (Sanierung)

Das Messen der Eigenfrequenzen vor und nach der Sanierung ist ohne großen Aufwand durchzuführen. Im *Bild 3* ist die freie Schwingung nach einer Impulserregung dargestellt. (Springen gegen eine Außenwand) Bei dem Bauwerk *(Bild 4)* handelt es sich um einen Turm, der eine Höhe von $h = 80\ m$ aufweist. Da in den Bögen und Querwänden Risse aufgetreten sind, sollte ein wesentlicher Teil abgetragen werden. *(Bild 5)* Die Eigenfrequenzen wurden vor dem Abtragen der Konstruktion und danach erfaßt.

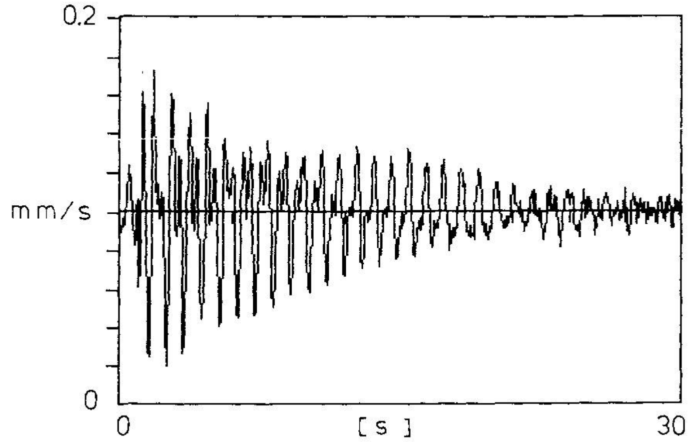

Bild 3: Freie Schwingung nach Impulserregung (Sprung)

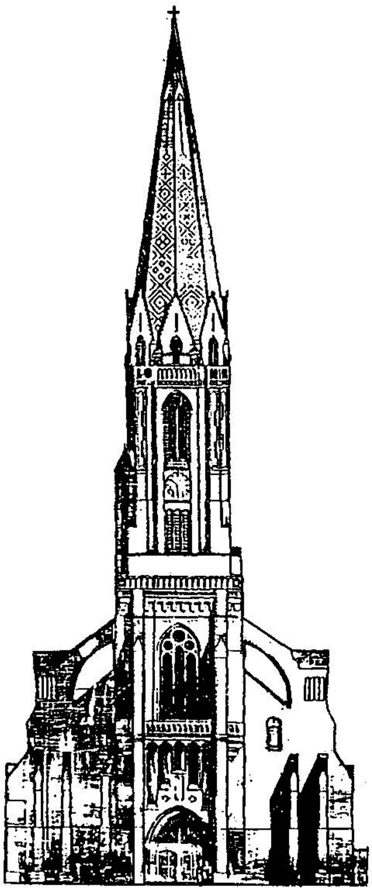

Bild 4: Kirchturm

Es ergaben sich folgende Werte:
Vor dem Umbau:

$$f_x = 1,06\ Hz$$
$$f_y = 1,25\ Hz$$
$$\delta_{x,y} = 0,2$$

Nach dem Abtragen von Wandteilen:

$$f_x = 1,02\ Hz$$
$$f_y = 1,24\ Hz$$

Bild 5: Abriß eines Teils der Bögen und der Querwand

Die wesentliche Erregerfrequenz der großen Glocke liegt bei $f = 1,05\ Hz$. Betrugen die Schwingungsamplituden vor dem Umbau maximal $s_x = \pm 1,8\ mm$, so gingen sie auf $s_x = \pm 0,8\ mm$ nach dem Abtragen der Querwand herunter. Um zu vermeiden, daß nach der Sanierung wieder Resonanzschwingungen auftreten, soll zwischen dem Turm und dem neu aufgemauertem Wandteil ein Luftschlitz gelassen werden.

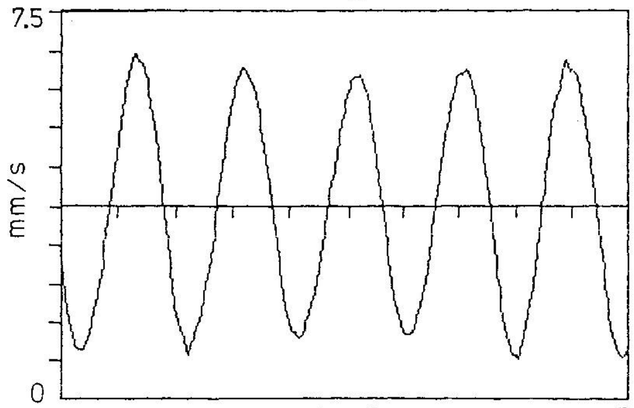

Bild 6: Zeitverlauf der Bogenschwingung (volles Geläut)

Vor dem Umbau führten die Mauerwerksbögen und die Seitenwand Bewegungen beim Glockengeläut von $s = 1,1\ mm$ aus. Die Schwinggeschwindigkeitsamplituden ergaben sich zu $v = 6,9\ mm/s$ *(Bild 6)*. Entsprechend [4] sind bei stationären Vorgängen Schäden an der Konstruktion nicht auszuschließen. Die vor der Sanierung vorhandenen Risse können durch die jahrzehntelange dynamische Beanspruchung entstanden sein.

Nach der Sanierung werden wieder Schwingungsmessungen durchgeführt, um eine Aussage über die Einwirkung der dynamischen Belastung auf die Konstruktion vornehmen zu können.

Außerdem werden zur Zeit Messungen durchgeführt, um die Beanspruchung des Turmes bei Sturm zu erfassen.

4.3 Kirchturm C (Schadensfrüherkennung)

Die Veränderung der Eigenfrequenzen ist ein Indiz dafür, daß sich die Steifigkeit des Systems verändert hat, wenn davon ausgegangen werden kann, daß die Massen gleich geblieben sind. Die exakte Ermittlung der Eigenfrequenzen und Eigenschwingungsformen kann nur über Messungen erfolgen. Dafür ist aber eine Dauerüberwachung nicht erforderlich, da sich Schäden, wenn nicht außergewöhnliche Beanspruchungen vorliegen, langfristig ausbilden. Besonders sinnvoll sind Messungen zur Ermittlung der Eigenfrequenzen nach der umfassenden Sanierung eines Turmes, da dann davon auszugehen ist, daß die Standsicherheit der Konstruktion vorliegt. Bei geschädigten Bauwerken kann durch Messungen, die in kürzeren Abständen erfolgen sollten, festgestellt werden, ob sich die Standsicherheit verändert. Schwierig wird es sein, was eventuell über FE-Berechnungen, die parallel durchgeführt werden können, möglich wird, eine Versagenswahrscheinlichkeit zu benennen.

An einem Beispiel, einem gemauerten Glockenturm, der eine Höhe von $H = 30\ m$ aufweist und etwa 40 Jahre alt ist, soll gezeigt werden, wie sich Risse auf Eigenfrequenzen und Eigenschwingungsformen bemerkbar machen.

Bevor ein mathematisches Modell aufgestellt wurde, erfolgten Schwingungsmessungen am Kirchturm, weil die Befürchtung geäußert worden war, daß beim Läuten der Glocken „große“ Schwingungen auftreten. Die Schwingungsmessungen führten zu Eigenfrequenzen von $f_x = 1{,}29\ Hz$, $f_y = 1{,}89\ Hz$, $\delta_{x,y} = 0{,}09$. Bei vollem Glockengeläut (4 Glocken) traten an der Turmspitze Wegamplituden von $s = \pm 1{,}9\ mm$ auf. Die dritten Erregerfrequenzen liegen etwas unterhalb der Eigenfrequenz von $f = 1{,}89\ Hz$.

Erregerfrequenzen: $f_{3G_1} = 1{,}63\ Hz$, $f_{3G_2} = 1{,}69\ Hz$, $f_{3G_3} = 1{,}75\ Hz$

Die Vergrößerung der statischen Lasten liegen bei dem zur Zeit vorhandenem Frequenzabstand im Mittel bei $V \cong 5{,}0$. Treten Schäden am Mauerwerk auf, (Vertikalrisse) so kann sich die Eigenfrequenz auf die Erregerfrequenzen zubewegen, so daß eine Vergrößerung von $V \cong 21$ eintreten kann. Um dieses zu vermeiden, wurde vorgeschlagen Wiederholungsmessungen in Abständen von $5 \div 10$ Jahren durchzuführen.

Da vom Kirchturm verläßliche Baupläne vorlagen, (Masseverteilung und Trägheitsmomente) wurde ein mathematisches Modell aufgestellt. (FE-Modell *NASTRAN*) Das Ausgangsmodell hatte einen E-Modul von $E = 3{,}3 \cdot 10^3\ MN/_{m^2}$ für das Mauerwerk. Die berechneten Eigenfrequenzen lagen etwa 20% unterhalb der gemessenen Frequenzen. Wird ein E-Modul von $E = 4{,}5 \cdot 10^3\ MN/_{m^2}$ im Modell verwandt, so ergeben sich die Eigenfrequenzen, die auch bei der Messung ermittelt worden waren.

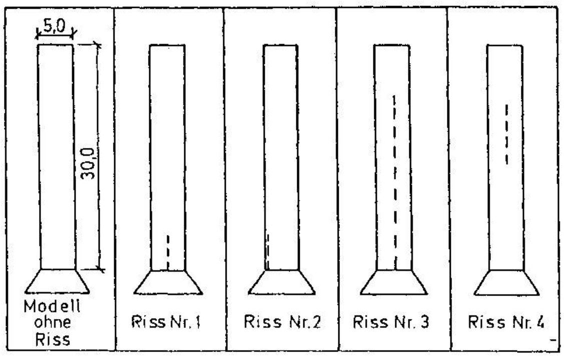

Bild 7: Darstellung der Risse

Eigen-formen	Risse ohne	Riß 1	Riß 2	Riß 3	Riß 4
1	1,34	1,34	1,34	1,34	1,34
2	1,88	1,60	1,87	1,38	1,84
3	6,66	5,70	6,57	4,71	6,52
4	7,21	6,04	7,04	3,60	6,40
5	9,36	7,80	9,21	6,18	9,27
6	14,60	11,66	13,60	8,80	10,10

Tabelle 1: Eigenfrequenzen $[Hz]$ in Abhängigkeit von den Vertikalrissen

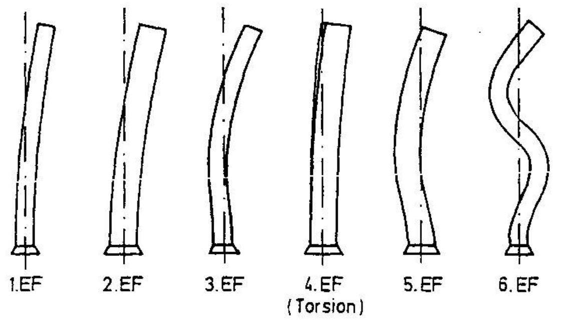

Bild 8: Darstellung der Eigenschwingungsformen (ungerissener Zustand)

In das mathematische Modell werden vier unterschiedliche Risse eingebaut. *(Bild 7)* Die Veränderung der Eigenfrequenzen ist in *Tabelle 1* dargestellt. Die Eigenschwingungsformen für den ungerissenen Zustand sind in *Bild 8* zusammengestellt. Es ist zu erkennen, daß der Riß 1 zu einer Senkung der Eigenfrequenz in der wesentlichen Eigenschwingungsform führt. Tritt in diesem Bereich ein Vertikalriß auf, so kann es zu einer Resonanzerregung mit der dritten Erregerfrequenz einer Glocke kommen und sich somit die Beanspruchung des Turmes wesentlich erhöhen, so daß der Riß vergrößert wird. Die Veränderung der Eigenfrequenzen ist beim Riß 2 gering. Es ist eine Senkung der Eigenfrequenzen in der dritten und vierten Eigenschwingungsform zu erkennnen. Extreme Auswirkungen auf das Eigenschwingungsverhalten des Turmes hat der Riß 3. Es verändern sich nicht nur die Eigenfrequenzen, sondern auch die Eigenformen verschieben sich. Die dritte Eigenschwingungsform des Ausgangssystems wird zur vierten Eigenschwingungsform, da sich die Frequenz der Torsionsschwingung stark verändert. Ein Riß im oberen Bereich des Turmes hat unwesentliche Auswirkungen auf die ersten beiden Eigenfrequenzen. Veränderungen in der dritten und vierten Eigenschwingungsform treten aber deutlich hervor.

Die Untersuchungen zeigen, daß auch bei kleinen Rissen, die die Stabilität der Türme nicht beeinträchtigen, Veränderungen in den Eigenfrequenzen zu beobachten sind. Das vorgestellte Verfahren ist für Türme geeignet, um die Standsicherheit zu überwachen. Wie vorher beschrieben, können die beiden Grundbiegeeigenfrequenzen aus der Antwort der Türme bei Windererregung oder einem Impuls (Springen) meßtechnisch ohne großen Aufwand ermittelt werden. Sollen aber mindestens die ersten sechs Eigenschwingungsformen angeregt werden, so ist eine andere Erregung erforderlich. Die Türme können mittels einer statischen Last ausgelenkt werden. (Ein Seil wird zwischen dem Turm und einem Autokran gespannt) Es erfolgt dann eine plötzliche Entlastung, so daß die Türme frei ausschwingen können. Die Energie ist ausreichend, um auch höhere Eigenformen anzuregen.

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Seismic Capacity and Retrofit of Existing Brick Masonry Building

Résistance sismique et réparation d'un immeuble en maçonnerie

Sismische Tragfähigkeit und Nachrüstung eines bestehenden Ziegelmauerwerkgebäudes

Tsuneo HASUDA
Senior Research Eng.
Railway Techn. Res. Inst.
Tokyo, Japan

Hideo KATSUMATA
Research Eng.
Obayashi Corp.
Tokyo, Japan

Atsuhiko KANOU
Manager
East Japan Railway Co.
Tokyo, Japan

Teruki ISHIBASHI
Senior Research Eng.
Railway Techn. Res. Inst.
Tokyo, Japan

Hiroaki ETO
Chief Research Eng.
Obayashi Corp.
Tokyo, Japan

Hitoshi KUNIHIRO
Structural Eng.
East Japan Railway
Tokyo, Japan

SUMMARY

This paper describes an experimental study on seismic capacity and retrofit of an existing masonry building constructed in 1914. The structural system of this building consists of brick walls, which contain steel elements inside the walls. Brick masonry wall models with or without steel elements are tested for evaluation of seismic capacity of this building. Models retrofitted with reinforced concrete or steel walls are tested. Reinforcing effects of steel elements and retrofit performance are discussed.

RÉSUMÉ

Le rapport porte sur l'étude expérimentale de la résistance sismique et de la réparation d'un immeuble en maçonnerie construit en 1914, dont le système structural consiste en murs de briques contenant des éléments métalliques. Des murs en maçonnerie en briques, avec et sans éléments métalliques, ont été testés en vue de l'évaluation de la résistance sismique du bâtiment. Des prototypes renforcés avec du béton armé ou des parois métalliques ont également été testés. Les effets du renforcement par des éléments métalliques ainsi que l'efficacité de la réparation sont discutés.

ZUSAMMENFASSUNG

In diesem Aufsatz wird eine experimentelle Studie über die seismische Widerstandsfähigkeit und Nachrüstung eines bestehenden, im Jahre 1914 gebauten Gebäudes aus Ziegelsteinmauerwerk beschrieben. Das Tragsystem dieses Gebäudes besteht aus Mauerwerkswänden mit eingelegten Stahlteilen. Wandmodelle mit und ohne Stahlelemente wurden daher geprüft, um die seismische Widerstandsfähigkeit des Gebäudes zu ermitteln. Ausserdem wurden auch mit Stahlbeton- bzw. Stahlwänden nachgerüstete Modelle untersucht. Die verstärkende Wirkung der Stahlelemente und die Wirksamkeit der Nachrüstung werden erörtert.

1. Introduction

Tokyo Station, located near the Imperial Palace, is the central station of Japan. The building of Marunouchi side of this station was constructed in 1914. Japanese people love this historical and Western-styled building because the building symbolizes rapid modernization of Meiji Era. The building is a brick masonry and steel structure, which is 400m long and 2 storied. In the original figure, it was 3 storied, however the top floor was demolished because of heavy damage during World War 2.

Recently, its owner, East Japan Railway Company is planning a redevelopment project of Marunouchi area, including renewal of this station. Considering symbolic existence of this building in Japan, it is strongly hoped to reserve the building in some ways. Therefore, it is needed to investigate the structural performance of this building, especially seismic capacity, and if necessary, to develop retrofitting techniques. For these purposes, the authors carried out the following tests and investigation.

(1) diagonal shear loading test of brick masonry walls;
contribution of steel elements to behavior of walls was investigated.
(2) direct shear loading test of mortar bed joints of the masonry;
influence of normal stress on the shear strength of masonry walls was estimated.
(3) diagonal shear loading test of retrofitted brick masonry walls;
retrofitting techniques for brick masonry walls were discussed.
(4) Proposal of a simplified estimation method for reinforced brick masonry walls:
contribution of various reinforcing elements to shear strength of the wall was determined.

2. Test of brick masonry walls

The structural system of this building consists of the next three components; (a) brick masonry walls, (b) steel frames or elements encased inside the brick masonry walls, and (c) floor slab diaphragms supported by the steel frames. Typical detail of the frame is shown in Fig. 1. The walls reinforced with the encased steel elements is considered to resist earthquake load, therefore testing was carried out to evaluate the seismic performance of this structural wall.

Test specimens were five brick masonry walls and were subjected to diagonal compression shear loading. One specimen, named BW0, was cut off a structural brick masonry wall in this existing building. The other four specimens were newly constructed to investigate reinforcing effects of the steel structural elements which were encased inside the brick walls. The list of the specimens is shown in Table 1. The major test variable was presence of steel structural elements.

Specimens BW0, BW1, and BW2 were pure brick masonry walls. Specimen BWS had steel reinforcing elements inside and outside the brick wall, which were corresponding to web reinforcing bars and main bars of usual reinforced concrete walls, respectively. The dimensions and detailing of the steel structural elements were determined under consideration of correspondence to original ones. Specimen BWSC was provided larger steel columns than BWS in order to represent confinement of adja-

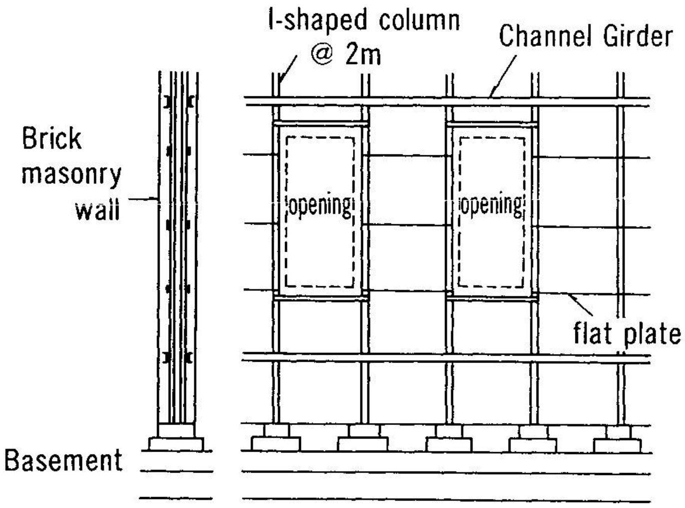


Fig. 1 Typical Steel Frame

Table 1 List of specimens

Specimen	Component
BW0	Brick masonry wall (existing)
BW1	Brick masonry wall (new)
BW2	Brick masonry wall (new)
BWS	Brick masonry wall (new) Steel columns and tie bars
BWSC	Brick masonry wall (new) Steel strong columns and tie bars

Table 2 Material properties of wall specimens

a) Bricks and brick masonry	E	σb
Brick unit (new)	30.0	113.8
Brick unit (existing)	6.0	30.4
Bed joint mortar	3.9	3.77
Brick masonry pile (new)	8.0	30.0
Brick masonry pile (existing)	3.0	14.0

E : Modulus of elstiticity (GPa)
σb: Compressive strength (MPa)

b) Steel elements	E	σy	σt
Tie bar FB-32×2	208	709*	772
Column H-100×50	207	265	420
Column H-100×100	209	327	468

E : Modulus of elsticity (GPa)
σy: Yield strength (MPa) *0.2% off set
σt: Tensile strength (MPa)

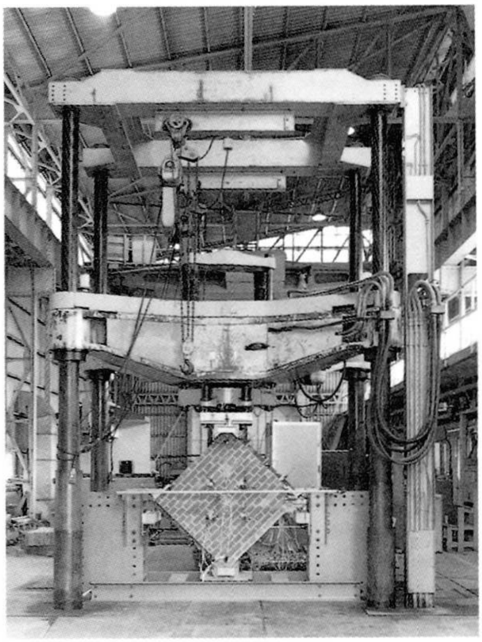


Photo 1 Loading Apparatus

cent walls in a multi-bay wall. Dimensions of the specimens are shown in Fig. 2. Material properties are shown in Table 2.

Static and monotonic diagonal compression load was applied. The shear strain of the walls was calculated from the measured displacements of the two diagonals. The shear force was also calculated from the applied load. The test set-up employed is shown in Photo 1.

Shear force - shear strain relationships of the specimens are shown in Fig. 3. Representative crack pattern after the testing is shown in Fig. 4. The new pure brick masonry walls (Specimens BW1 and BW2) showed very brittle failure. When a diagonal crack appeared in the wall, load was completely lost simultaneously. Most of cracks were observed along joints of the brick masonry. The old pure brick masonry wall (Specimen BW0) showed more ductile manner because the measurement point of

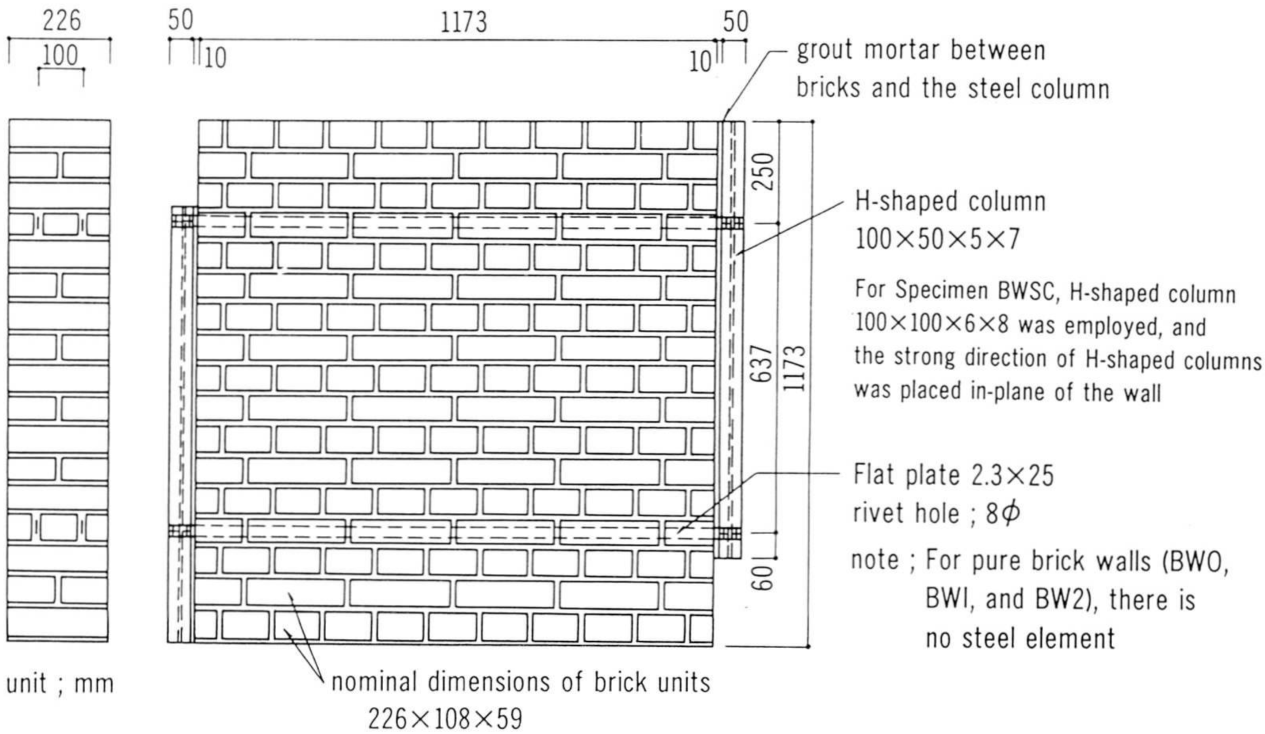


Fig. 2 Dimensions and detail of brick masonry wall specimens

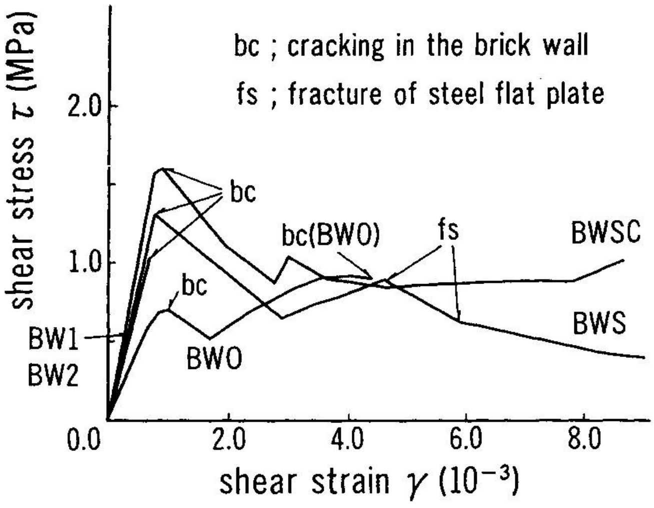


Fig.3 Shear stress and shear strain relationship

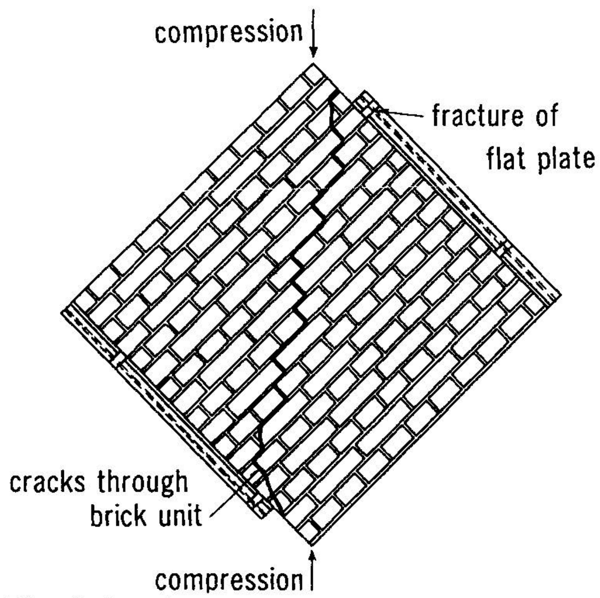


Fig.4 Crack pattern of a wall specimen

the displacement failed locally. This old wall might show the same brittle behavior as the new walls.

On the other hand, Specimen BWS kept up a reduced load after cracking of the masonry wall. This was due to the frictional resistance at the masonry bed joints and the tensile capacity of the steel structural elements inside the masonry wall. For this type of reinforced walls, it is possible to expect such post-cracking strength. At the ultimate stage, the wall steel elements fractured at the connection to the column steel element. This was due to stress concentration at the rivet holes in the connection. Specimen BWSC showed almost same behavior as Specimen BWS.

3. Test of bed joints

Bed joints of brick masonry walls were tested under combination of direct shear stress and normal stress to quantify the influence of normal stress on bed joint sliding shear strength. Major test variables were (1) normal stress level and (2) construction of brick masonry specimens.

The normal stress level of the existing building is approximately 0.5MPa, so 4 stress levels distributing around this value were applied as testing normal stress levels. Two types of specimens were employed. One was cut off the existing brick masonry building, so dimensions of the specimens were slightly distributed. The other was newly constructed with the same materials and methods as the brick masonry wall specimens mentioned before. Dimensions of a typical specimen are shown in Fig. 5. For each test variable, three specimens were tested to grasp scatter of test results.

Loading concept is shown in Fig. 6. A lateral hydraulic jack applied direct shear force to the bed joint. A vertical hydraulic actuator applied constant axial force to the upper side of the masonry specimen. Relative displacement between the upper and lower parts of the specimen was measured as the sliding displacement at the bed joint.

Representative relationships of shear stress - sliding displacement at the bed joint are shown in Fig. 7. It was observed that;
(1) Initial stiffness was very high.

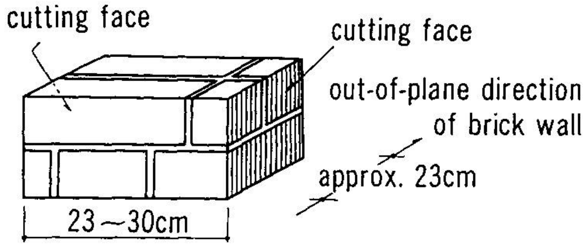


Fig.5 Dimensions of bed joint specimens

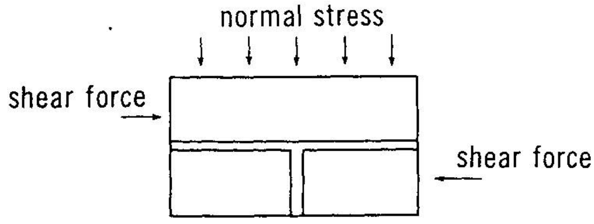


Fig.6 Loading concept

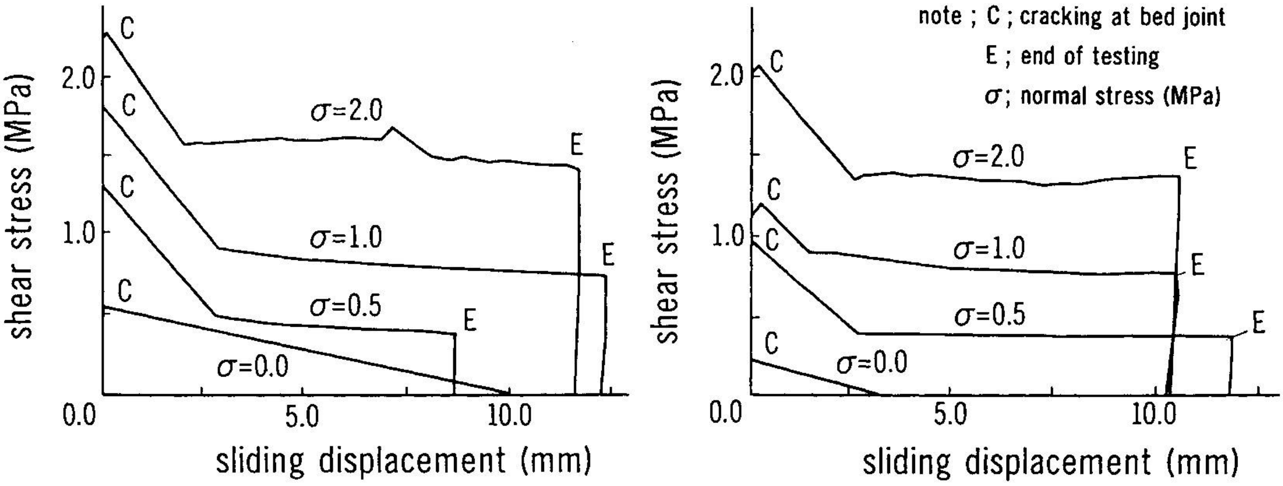


Fig.7 Shear stress and sliding displacement relationship

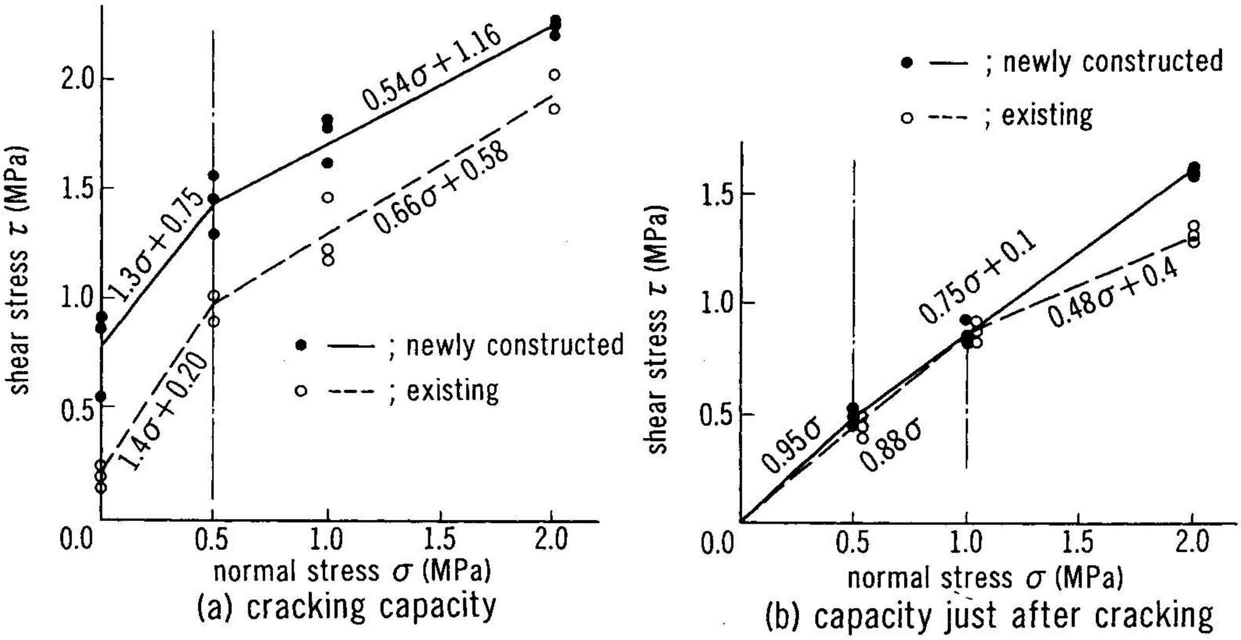


Fig.8 Sliding shear capacities

(2) When cracking occurred at the bed joint, the shear resistance was reduced very rapidly, however, this reduction stopped at a certain force level corresponding to the normal stress level.
(3) After the load reduction, the shear stress level was almost constant while the sliding displacement increased.
This post-cracking behavior is due to friction at the cracked bed joint interface.

Both shear capacities at the maximum load and in the post-cracking stage are shown in Fig. 8. The horizontal axis of this figure indicates the normal stress level. It is evidently understood that both shear capacities linearly increase as the normal stress level grows, where the normal stress is low. However, this increasing rate is reduced where the normal stress is high. Equations for evaluation of these shear capacities can be experimentally established as shown in this figure.

4. Test of retrofitted brick masonry walls

Two retrofitted brick masonry walls and one reinforced concrete wall were tested to verify structural performance of such retrofitting methods. Major dimensions of specimens and testing procedures, loading and measurement, were the same as employed in the previous testing of brick masonry walls.

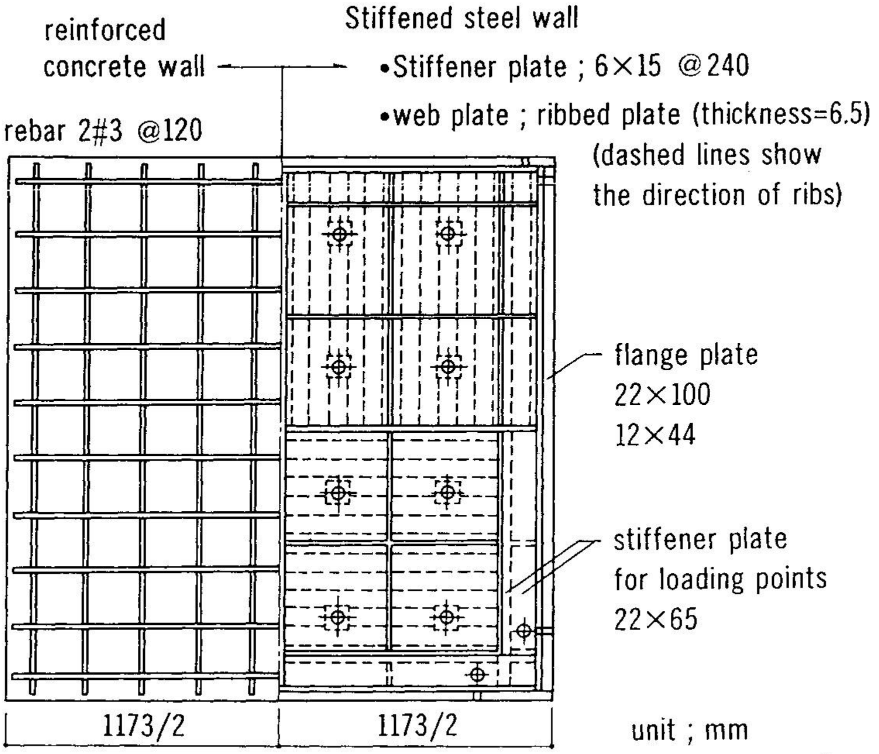
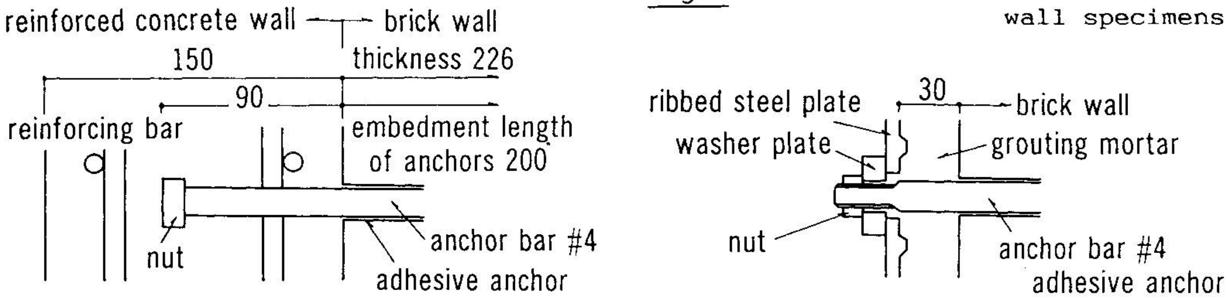


Fig.9 Dimensions of retrofitted wall specimens



(a) retrofitting with reinforced concrete wall (b) retrofitting with steel wall

Fig.10 Detail of connections

Table 3 Material properties of retrofitted wall specimens

a) Concrete and mortar	E	σb
Concrete	22.6	24.7
Grout mortar	24.0	44.8

E : Modulus of elstiticity (GPa)
σb: Compressive strength (MPa)

b) Steel elements	E	σy	σt
Rebar #3	188	335*	557
Anchor bar #4	189	375	543
Ribbed plate t=6.5	210	327	429
Flange FB-22×100	209	292	455

E : Modulus of elsticity (GPa)
σy: Yield strength (MPa) *0.2% off set
σt: Tensile strength (MPa)

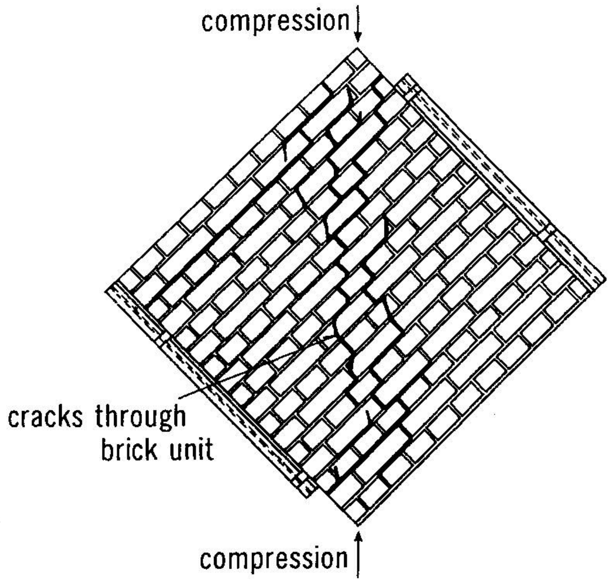
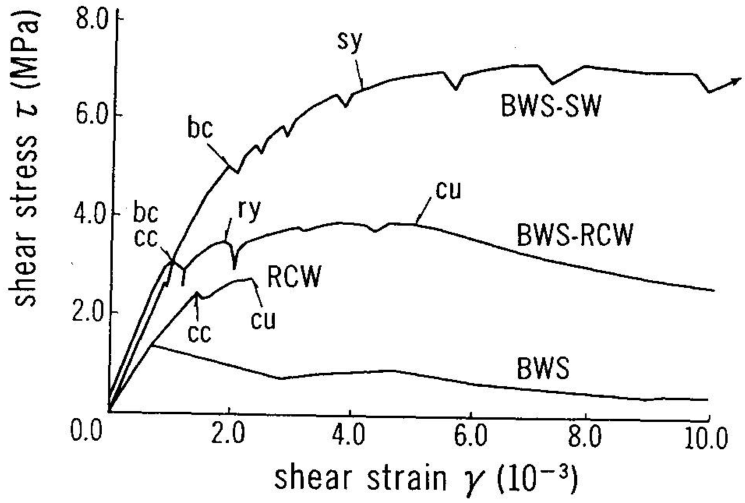


Fig. 11 Crack pattern of a retrofitted wall specimen



bc ; cracking in the brick wall
cc ; cracking in the concrete wall
ry ; yielding of reinforcement bars
sy ; yielding of steel panel
cu ; crushing of concrete

stress was obtained from division of shear force by area of section of brick masonry

Fig. 12 Shear stress and shear strain relationship

The retrofitted prototype was a brick masonry wall reinforced with steel elements, like Specimen BWS.

Dimensions and detailing of the specimens are shown in Fig. 9 and 10, respectively. Material properties are shown in Table 3. One retrofitted specimen, BWS-RCW, was a reinforced brick masonry wall connected to a new reinforced concrete wall. For the connection between the two walls, adhesive anchor bolts were employed. At the first stage of construction, these anchor bolts were embedded into the brick wall, and arrangement of rebars and casting of concrete for the additional wall was carried out. For comparison, a reinforced concrete wall with the same dimensions and detailing, Specimen RCW, was also tested. The other retrofitted specimen, BWS-SW, was a brick masonry wall connected to a stiffened steel wall with grout mortar and adhesive anchors. For the web plate of the steel wall, ribbed steel plates were employed to integrate grout mortar and the steel wall. Construction of the specimen was conducted as the following process: embedding of the adhesive anchors, setting of the steel plate wall, and pouring of grout mortar to the gap between these walls.

Cracks in the brick wall of the retrofitted specimens were more distributed than a non-retrofitted specimen, Specimen BWS, as shown in Fig. 11. Shear force - shear strain relationships of the specimens are shown in Fig. 12, comparing Specimen BWS. The retrofitted specimens showed much higher strength and deformation capacity than the non-retrofitted specimen. From these results, it is concluded that these two techniques are available for retrofitting of steel-reinforced brick masonry walls.

5. Proposal of a simplified evaluation method for brick masonry wall strength

One common design criterion for this type of brick masonry buildings is "not to allow cracking of brick walls." However, this criterion is too severe and not realistic for Tokyo station building, considering very large earthquake load which is regulated in the Japanese building code. From our testing, it can be predicted that brick masonry walls crack approximately at 0.1% of shear strain, and that the earthquake response of the walls of the station building is larger than this critical strain level.

The other criterion is "to allow cracking of brick walls but to avoid heavy damage, such as collapsing, etc." Fortunately, the walls of this station building are reinforced. It can be expected that in the post-cracking stage, friction at the cracked interfaces can transfer earthquake loads as long as reinforcing steel elements do not fracture. The authors carried out testing of the connections of the steel structural elements which were cut off the existing station building. Test results showed that yielding of the steel elements occurred before fracturing at the rivet bolt holes and that elongation of the steel elements at the fracturing was larger than 1%. That is, some plastic deformation capacity after cracking may be expected. It can be concluded that the design criterion, to allow cracking, is available. On the basis of this discussion, the authors propose a simplified design strength evaluation as follows.

Contribution of brick masonry walls is defined as a function of axial stress σ_L, or long-term axial stress. In actual, shear stress transfer is influenced by aspect ratio of the wall, strengths of materials and so on. These influences, however, are very complicated. It is judged that the following equation is adequate on the point of view of simplicity. The constant, 0.8, is determined from the tests of bed joints.

$\tau=0.8\sigma_L$ Eq.1

Contribution of reinforcing steel elements is evaluated as the following equation, based on the testing of reinforced brick masonry walls.

$\tau=p_S\sigma_S$ Eq.2

where, p_S: ratio of horizontal reinforcing steel elements inside the brick wall panels, σ_S: yield stress of these steel elements.

From the test results of the retrofitted specimens and usual design assumptions in Japan, contributions of retrofit walls are estimated as follows.

$\tau=F_c/10*(t_{RC}/t_b)$ for retrofitting reinforced concrete walls Eq.3

$\tau=\tau_y*(t_s/t_b)$ for retrofitting steel walls Eq.4

where, F_c: concrete compressive strength, τ_y: shear yielding stress of steel, t_{RC}: thickness of the reinforced concrete wall, t_s: thickness of the steel wall, t_b: thickness of the brick wall.

Total shear strength is summation of these contributions.

Table 4 shows comparison between test results and the estimated values by this method. However, for application to the testing mentioned before, slight modification was needed as follows; (a) axial stress was equal to shear stress due to loading condition, (b) flat plate did not show yielding up to fracturing at the connection due to high material strength, so the calculation of Eq. 2 was carried out employing the predicted stress which might occur at fracturing of the connection, and (c) for the retrofitted specimens, it was impossible to distinguish the bearing axial stress of the brick masonry from that of the retrofitting element, thus capacity was calculated by summation of the brick masonry part, namely BWS, and the retrofitted part. The evaluation is higher than the test results for brick masonry walls, however lower for the retrofitted specimens. Further study is necessary to verify the evaluating method for contribution of brick masonry walls.

Table 4 Estimation of shear strength of reinforced brick walls

Specimen	Test*	Estimate
BWS	8.13	11.06
BWSC	8.64	11.47
BWS-RCW	38.5	27.4
BWS-SW	66.6	65.4

*: capacity at 0.4% of shear strain

6. Conclusion

Testings of brick masonry walls were carried out for evaluation of seismic performance and establishment of seismic retrofitting methods of a historical brick masonry building reinforced with steel elements. From the testing of five brick masonry wall specimens, it was found that some bearing capacity in the post-cracking stage can be expected due to friction of cracked interface caused by reinforcement with steel structural elements. Contribution of the friction was determined from the testing of mortar bed joints of brick masonry. Seismic retrofitting methods, addition of reinforced concrete walls or that of steel walls, was tested. A simplified evaluation method of the bearing capacity of such brick masonry walls in the post-cracking stage and that for retrofitted walls was proposed, as shown eqs. (1) to (4). In this method, shear strength of the wall is evaluated as summation of the next three components; that is, (a) friction, which is a simple function of axial stress, (b) tensile resistance of steel elements, which is corresponding to yield force of the elements, and (c) contribution of retrofitting walls, if any.

Acknowledgment

This research project was carried out under supervising by Dr. T. Okada and Dr. K. Takanashi, Institute of Industrial Science, the University of Tokyo, Dr. M. Hirosawa, Kogakuin University, and Dr. T. Kubo, Nagoya Institute of Technology. The authors also express acknowledgment for advice and assistance by Ministry of Construction, Tokyo Metropolitan Government, East Japan Railway Company, Railway Technical Research Institute and Obayashi Corporation.

Durability Assessment of the Mortars of Ganga Canal System

Evaluation de la durabilité des mortiers du Canal du Ganges

Bestimmung der Dauerhaftigkeit der Mörtel im Ganges-Kanalsystem

S. K. MALHOTRA
Scientist
Central Build. Res. Inst.
Roorkee, India

S.K. Malhotra got his MSc degree in inorganic chemistry at Univ. of Roorkee. For the past 26 years, he has been working at DBRI, Roorkee. His research interests are lime and lime-based materials and cementious binders from agro-industrial wastes.

N.G. DAVE
Dr.
Central Build. Res. Inst.
Roorkee, India

N.G. Dave, Ex-Deputy Director at CBRI, Roorkee, had education in India and USA. After ten years of teaching, he joined DBRI and has been engaged in research on lime pozzolana, cementious binders, process technology and energy conservation.

SUMMARY

Investigations on the durability assessment of lime based mortars and plasters of bridges and related structures of the Ganga Canal system after a span of about 150 years show that the mortars possess good compressive strength. Since there is no evidence of the presence of calcium hydroxide, indications are that the mortars are heavily carbonated and almost all of the calcium hydroxide has been converted into the strength giving mineral 'Calcite'. The studies thus reveal that the examined mortars are in a sound state.

RÉSUMÉ

Des études pour l'évaluation de la durabilité des mortiers et plâtres des ponts et des structures annexes du Canal du Ganges, qui a plus de cent cinquante ans, ont été réalisées. Elles montrent que les mortiers présentent une bonne résistance à la compression. Comme il n'y a pas d'évidente présence d'hydroxide de calcium, cela indique que les mortiers sont très fortement carbonatés et que presque tout l'hydroxide de calcium a été converti par compression en un minéral calcite. Les études révèlent ainsi que les mortiers du Canal du Ganges qui ont été examinés, sont en bon état.

ZUSAMMENFASSUNG

Die Studien dienten der Zustandsbeurteilung der bei Brücken und angrenzenden Bauwerken im Ganges-Kanalsystem vor rund 150 Jahren verwendeten Kalkmörtel und Verputz und zeigen, dass der Mörtel eine gute Druckfestigkeit aufweist. Nachdem keine Hinweise auf das Vorhandensein von Kalziumhydroxid hindeuten, kann abgeleitet werden, dass der Mörtel stark karbonisiert ist und das Kalziumhydroxid praktisch vollständig in Calzit umgewandelt wurde. Die Studien belegen also den guten Zustand der im Ganges-Kanalsystem untersuchten Mörtel.

1. INTRODUCTION

In a structure, the selection of building materials is made to ensure its future performance i.e. durability. Durability is usually considered as a property of resistance to a slow rate of detriortion to environmental factors. A more durable structure lasts for a longer period. Selection of any building material for use under any given situation is made with the knowledge of the performance of the material under the required circumstances. Several sophisticated and accelerated methods are available now a days for assessing the durability of a material. But in olden days, the long term performance of a material was the only synonym considered for assessing its durability. Thus, on the basis of the materials evaluation, much could be known about the performance and durability of a structure.

Lime has been used as a material of construction since the dawn of civilization. In india definite evidences of its use have been available in the remains of the Indus valley civilization at Harappa, Lothal, Mohanjodaro and other places as far back as at least 5000 years ago. Lime mortar [1] was used in the construction of pyramids of Egypt. Greeks used mortars based on lime to cover the walls made up of unburnt bricks. Romans perfected the use of lime mortar by adding pozzolanic material. Beside historic buildings and monuments [2] various hydraulic structures, such as dams, canals embankments etc. have also been made in lime mortars and concrete.

However, use of lime has its own merits in construction. Lime provides better workability, greater water tightness, high plasticity, better volume stability, autogenous healing capacity, high water retention value, good adhesion and is itself a durable material.

2. GANGA CANAL

Ganga canal [3] is one of the oldest water carrying system in India, streching over 563 km and carrying 6750 cusec of water, was constructed during 1839-1858 A.D. is a unique example of ninteenth century achievements. In all the works of canal, lime and "surkhi" (Burnt clay pozzolana) as a binder was used in mortars and plasters. Lime was obtained through calcination of highly calcareous lime stones from the quarries of Dehradun (Northern India) or collected from the basins of the rivers in the area, it generally is fat. Downstream, however, limestones characterized by the presence of earthy materials were used. Enhancement in hydraulic property of fat lime was achieved by the incorporation of "surkhi" obtained by grinding overburnt bricks to a fine powder. The mortar was further fortified by the addition of traditional materials like jute fibre, ground lentils, geleteneous wild fruits or jaggery. All the constituents were thoroughly wet ground together to a fine paste before use.

The composition [4] of the mortars used under various situations are given in Table 1.

3. COLLECTION OF SAMPLES

A number of lime mortar samples for examination were collected from different situations. First set of samples were collected

TABLE - 1

MORTARS AND PLASTERS USED IN GANGA CANAL SYSTEM

SITUATION	COMPOSITION		
	LIME	SURKHI	SAND
Inlet and dam rivers	1S	2	-
Arches of the bridges	1R	1	1
Bridges (Class I)*	1R	1	-
Bridges (Class II)*	1K + 0.05 S	1	-
Bridges (Class III & IV)*	2K	1	-
Inspection House	1R	1	-
Chokies	1R	1	-
Finishing works	7S	1	-

S - Stone lime; K - Kankar lime; R - River lime

*Classification of bridges are based on the span and breadth.

from Dhanauri bridge which is a class I bridge and is situated at 23 kilometer downstream from the origin of the canal at Hardwar at a very stratigiic situation, where Rutmov river level crosses the Ganga canal.

Second set of mortar samples were collected from the inspection house at Pathri, which is situated 11 kilometer downwards Hardwar. This is also very interesting situation, as the canal has to cross the voluminous monsoon river Pathri, which is flowing at a higher level. The canal, therefore, has to pass under the river.

Several chokies (security posts) were constructed along the length of the canal. Third set of mortar samples were collected from these chokies.

In all the cases, while collecting the samples, effort was made to take out the entire mortar. Attempts were also made to collect the samples from the situations as distant from each other as was possible under the circumstances, so that the statistical variations could be accommodated.

4. EVALUATION OF MORTAR SAMPLES

The samples collected from different situations of the Ganga Canal system were evaluated for various properties i.e. compressive strength, free lime content and pH values. For chemical characterisation the samples were subjected to thermal and X-ray diffraction analyses.

4.1 Compressive strength

The strength under compression is the primary function of any structure and therefore, is the most important property of the materials. In addition, the compressive strength is also a good index of many other engineering properties

Lime based mortars gain strength predominantly by carbonation process [5] by the absorption of atmospheric carbon dioxide (to convert lime into calcium carbonate) which continues over a considerable period of time. Further development of strength can take place by the reaction between lime and the oxides of silicon and aluminium added in the form of "Surkhi" [6] to generate calcium silicates and aluminates.

For measurment of compresive strength, cubes were cut from the lime mortar samples. These were of 40 mm of size. Some of them were some what smaller. Six cubes were tested on a compression testing machine of two tonne capacity. The average compressive strength values of the tests are reported in Table-2.

TABLE - 2

COMPRESSIVE STRENGTH OF MORTAR CUBES

SITUATION	Average Density (Kg/m^3)	Average Compressive Strength (MPa)
Dhanauri bridge	1520	5.76
Inspection House	1850	8.60
Choki (security post)	1715	5.24

4.2 Free lime content

The strength of lime based mortars mainly depend on the amount of lime present. Therefore, the estimation of the amount of lime present in the set mortar samples become important. Amount of lime present in free or uncombined state was determined by the modified Frankie method [7]. Approximately 1 g of the dried sample was taken in an Erlenmayer flask together with 10 ml of acetoacetic ester and 60 ml of isobutyl alcohol. The mixture was refluxed over a water bath for 2 hr. After cooling, the contents were filtered under vacuum and the residue washed with isobutyl alcohol was titrated against a standard perchloric acid solution using thymol blue as indicator. The results of three typical samples are given in Table - 3.

4.3 pH Value

The presence of free lime content in the mortar samples were further confirmed by the pH determinations. The pH values of the aquous extracts were determined with the help of a Phillips precision pH meter model PR 9405 M. Twentyfive grammes of powdered sample was taken and mechanically shaken for two hours with 100 ml double distilled water and allowed to stand for 22 hrs. These were then filtered and pH values determined. The results of three typical samples are given in Table 3.

TABLE - 3

FREE LIME AND "pH" VALUES OF MORTAR SAMPLES

SAMPLE	FREE LIME (%)	pH VALUE
M-1	0.09	8.0
M-2	0.11	7.4
M-3	0.12	7.6

4.4 Thermal Analysis

For further information, the mortar samples were subjected to thermogravimetry (Fig.1), differential thermogravimetry (Fig.2) and differential thermal analysis (Fig.3) with the help of Simultaneous Thermal Analyser, model STA-1500 with Trace-II system (PL Thermal Sciences Limited, U.K.). The rate of heating were maintained at 10°C per minute and the temperatures were measured with a plati- num-platinum - rhodium thermocouple. A sample of alumina was used as a reference material.

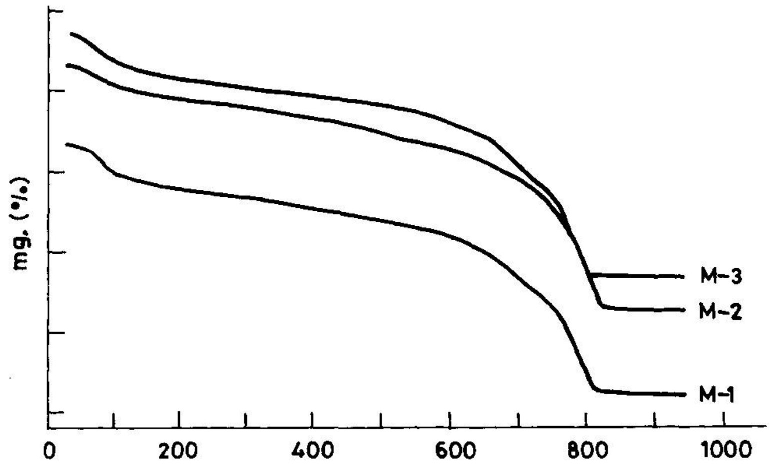


FIG. 1. THERMOGRAVIMETRY CURVES.

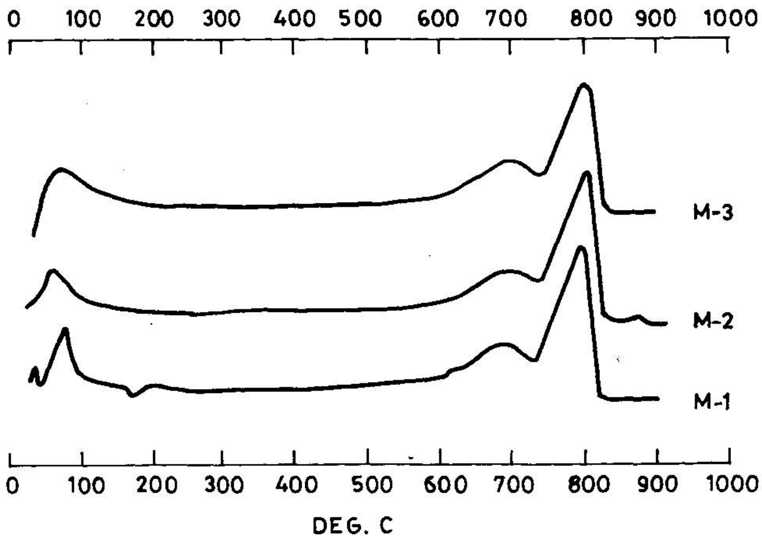


FIG. 2. DERIVATIVE THERMOGRAVIMETRIC CURVES.

The most significant observations are the strong endothermal effects between 734°C and 832°C, due to the presence of calcium carbonate decomposition and are supported by DTG curves. These changes can be assigned to the presence of the mineral calcite [8,9]. Broad endothermal effets in DTA curves between 660°C to 760°C and suspported by corresponding weight loss in DTG curves, appear to be due to the presence of amorphous or poorly crystalline calcium carbonate [10]. A weak but sharp endotherm around 575°C (not shown by DTG) is due to the transformation of mineral quartz. The exothermic effect in DTA at 174°C (only in sample M-1) is also accompanied by a loss in

DTG curve, can be due to the presence of cellulosic material; which, as evinced due to the incorporation of some local material [4] during the preparation of the mortars.

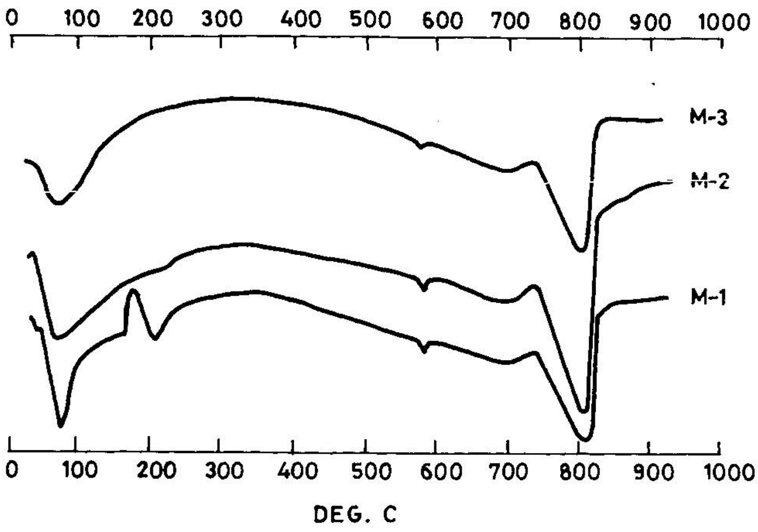


FIG. 3. DIFFERENTIAL THERMAL ANALYSIS CURVES

4.5 X-ray diffraction

The X-ray diffraction (XRD) patterns (Fig. 4) were also obtained by means of Phillips X-ray diffractometer, Model PW 1760, using Ni filtered CuKα radiations. The powder specimens were placed in a recess in a plastic plate, compacted under just sufficient pressure to cause cohesion without the use of a binder. The results obtained were compared with standard data from ASTM powder diffraction file.

The X-ray diffraction patterns obtained in all the three mortar samples strongly support the presence of minerals calite, quartz and magnesite [11].

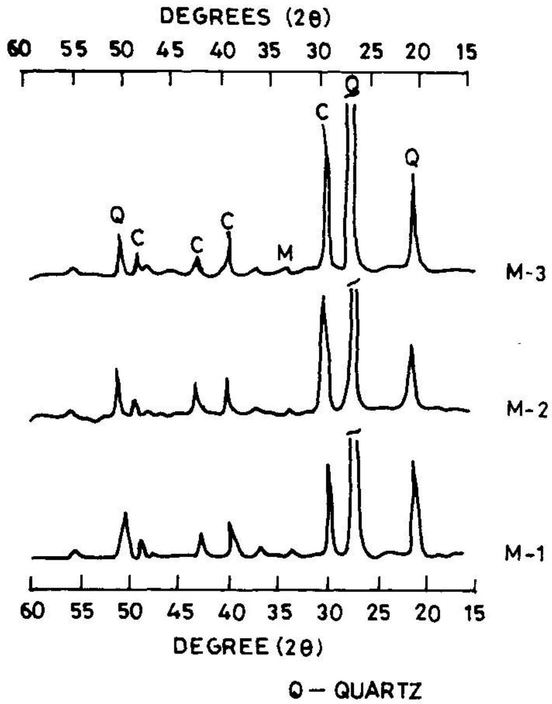


FIG.4. X-RAY DIFFRACTION (XRD)

5. CONCLUSION

The results indicate the presence of very little amount of calcium hydroxide, which was confirmed by the "free lime" and pH determinations. From thermal and X-ray diffraction experiments the most prominent observation is the presence of calcium carbonate, which indicate that the calcium hydroxide added to the mortars has been converted almost completely into strength giving mineral calcite. This is also evident from the fairly good amount of compressive strength values of the mortars.

The presence of poorly crystalline or amorphous calcium carbonate is also envisaged due to the process of dissolution of CO_2 through water into lime and "Surkhi" paste and this process continues over a long period through capillaries even after the paste has set. The hydraulic products of lime and silica and/or lime and alumina reactions may not have been formed or if had formed, might have also undergone carbonation.

The observations, therefore, reveal that the mortars, of Ganga canal system possess good compressive strength even after a long period of about 150 years. These structures, therefore, are in sound state and likely to remain serviceable for many more years.

6. ACKNOWLEDGEMENT

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Dynamic Characteristics of a Chinese Monument

Caractéristiques dynamiques d'un monument chinois

Dynamische Eigenschaften eines chinesischen Baudenkmals

Dongping FANG
Lecturer
Xian Jiaotong Univ.
Xian, China

Yutaka MIYAMOTO
Prof. Dr.
Iwate Univ.
Morioka, Japan

Shoji IWASAKI
Assist. Prof
Iwate Univ.
Morioka, Japan

Hideaki DETO
Assist. Prof
Iwate Univ.
Morioka Japan

Hiroshi HIKOSAKA
Prof. Dr.
Kyushu Univ.
Fukuoka, Japan

Maohong YU
Prof.
Xian Jiaotong Univ.
Xian, China

SUMMARY

The dynamic characteristics of a Chinese monument and the properties of its connectors, three-dimensional finite element methods, were studied. The wooden structure was analysed by a program, the connectors were modeled as flexible elements. Based on the structure's natural frequencies, the coefficients of the flexible elements were determined and the structure's natural frequencies and vibration mode were clarified. The structure's semi-rigid connectors were proven to be advantageous in their aseismic aspects when comparing the structure's earthquake responses with its fixed connectors.

RÉSUMÉ

Les caractéristiques dynamiques d'un monument chinois et les propriétés de ses connecteurs ont été étudiés. La structure en bois a été analysée, les connecteurs étant considérés flexibles dans le modèle. Basé sur la structure des fréquences naturelles, le coefficient de la flexibilité des connecteurs est déterminé, la structure des fréquences naturelles et le mode de vibration ont été clarifiés. La structure semi-rigide des connecteurs offre un avantage antisismique, en comparaison de celles des connecteurs fixes.

ZUSAMMENFASSUNG

Von einem historischen hölzernen Baudenkmal wurden die dynamischen Eigenschaften mittels der Finiten-Elemente-Methode untersucht. Dabei wurde besonderes Augenmerk auf die Modellierung der nachgiebigen Holzverbindungen gelegt. Anhand gemessener Eigenfrequenzen des Gebäudes konnten die Steifigkeiten der Verbindungen kalibriert und so die Schwingungsformen bestimmt werden. Dabei zeigte es sich, dass die nachgiebigen Verbindungen im Vergleich zu steifen Anschlüssen Vorteile bezüglich des Erdbebenwiderstandes aufweisen.

1. INTRODUCTION

The 12-meter high Xian City Wall is one of the largest and best-preserved defence architectures built during medieval times in China. The national relic—the Front Tower (Fig.1) over the North Gate of Xian City Wall—was built in the 1370's (Ming Dynasty in Chinese history). This valuable architectural structure is composed of over 6000 wooden components and supported mainly by 36 columns each with an average diameter of 0.55 meter (Fig.2 and Fig.3).

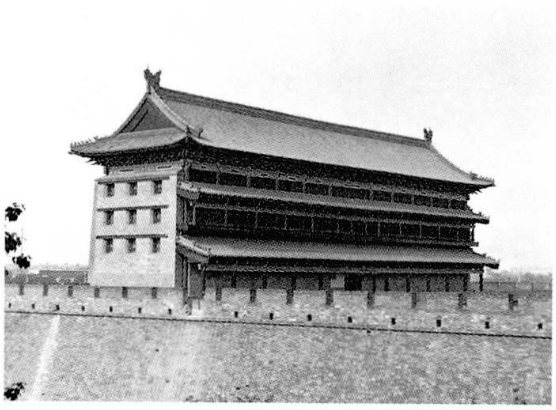


Fig.1 The Front Tower

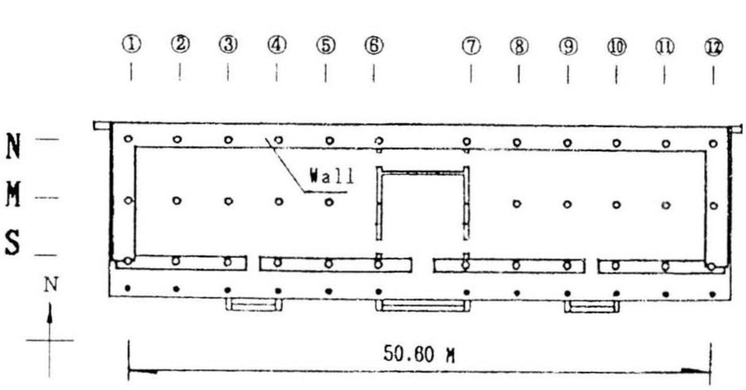


Fig.2 The Plane Viewe

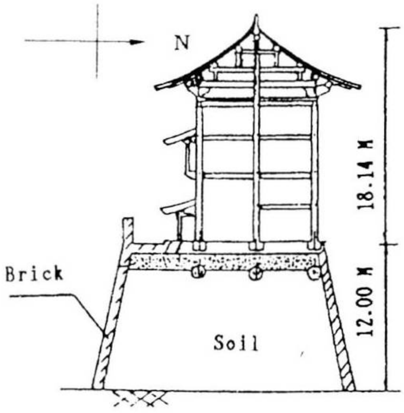


Fig.3 The Elevation

Xian City is located in a seismic zone and several strong earthquakes have occurred during its history. However, the Front Tower survived these calamities. It is therefore significance to study its dynamic characteristics in order to determine the inherent aseismic advantages and to prevent it from being destroyed during any subsequent earthquakes.

In the author's formal work[1], full scale vibration and model tests were carried out in order to determine the natural frequencies of the structure. These were determined to be 1.10Hz and 1.70Hz for basic natural frequencies of symmetrical and asymmetrical vibration mode shapes. In this paper, 3-D FEM dynamic analysis of the wooden structure was undertaken and the Dougong and joggle joint—the components' connecting method, as used in Chinese wooden structures for thousands of years—was modeled as flexible elements. Contrary to general structural analysis methods, Simplex Method was employed to determine the coefficients of the flexible elements in order to cause the calculated frequencies to agree with the tested ones.

After clarifying the natural frequencies and vibration mode shapes, the seismic responses of Middle Japanese Sea Earthquake were also calculated. The connectors were proven to be advantageous in the structure's aseismic aspect.

2. FEM MODEL OF THE FRONT TOWER

2.1 The Semi-Rigid Connectors and Flexible Element

In Chinese and Japanese wooden structures, the joggle joint (Fig.4) and Dougong (a system of brackets inserted between the top of a column and a crossbeam)(Fig.5) were used as component connectors. In this paper, this kind of semi-rigid connector was described by 3-D flexible elements (Fig.6) and the equation can be expressed as Eq.1.

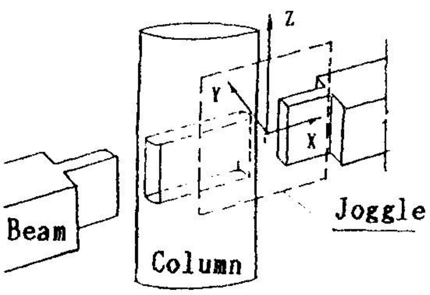


Fig.4 The Joggle Joint

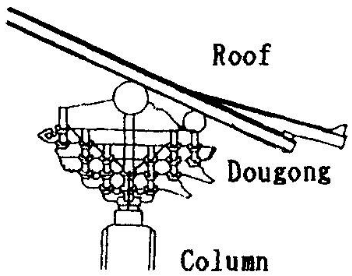


Fig.5 The Dougong

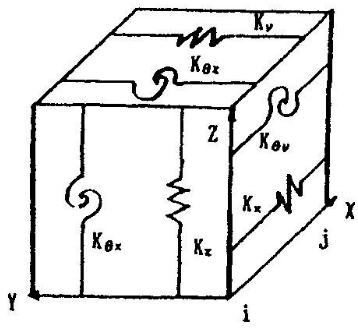
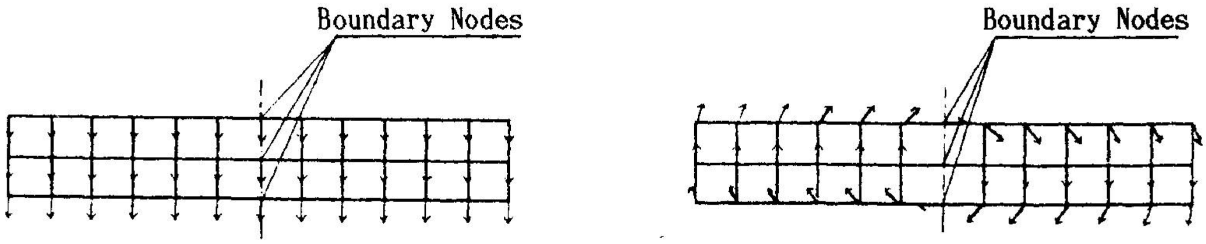


Fig.6 The Flexible Element

$$
\begin{Bmatrix} F_{xi} \\ F_{yi} \\ F_{zi} \\ M_{xi} \\ M_{yi} \\ M_{zi} \\ F_{xj} \\ F_{yj} \\ F_{zj} \\ M_{xj} \\ M_{yj} \\ M_{zj} \end{Bmatrix}
=
\begin{bmatrix}
K_x & & & & & & -K_x & & & & & \\
& K_y & & & & & & -K_y & & & & \\
& & K_z & & & & & & -K_z & & & \\
& & & K_{\theta x} & & & & & & -K_{\theta x} & & \\
& & & & K_{\theta y} & & & & & & -K_{\theta y} & \\
& & & & & K_{\theta z} & & & & & & -K_{\theta z} \\
-K_x & & & & & & K_x & & & & & \\
& -K_y & & & & & & K_y & & & & \\
& & -K_z & & & & & & K_z & & & \\
& & & -K_{\theta x} & & & & & & K_{\theta x} & & \\
& & & & -K_{\theta y} & & & & & & K_{\theta y} & \\
& & & & & -K_{\theta z} & & & & & & K_{\theta z}
\end{bmatrix}
\begin{Bmatrix} \delta_{xi} \\ \delta_{yi} \\ \delta_{zi} \\ \theta_{xi} \\ \theta_{yi} \\ \theta_{zi} \\ \delta_{xj} \\ \delta_{yj} \\ \delta_{zj} \\ \theta_{xj} \\ \theta_{yj} \\ \theta_{zj} \end{Bmatrix}
\qquad (1)
$$

2.2 FEM Model and Boundary Condition

Since the Front Tower was symmetrically constructed, the vibration mode shapes were divided into symmetrical and asymmetrical (Fig.7) and half of the structure is modeled



(a) symmetric vibration (b) asymmetric vibration

Fig.7 The symmetric and asymmetric vibration shapes of the Front Tower as Fig.8. The number of beam elements, flexible elements and nodes is 208, 162 and 280, respectively. The value of the flexible elements will be determined afterward. Because the columns of the structure have no other connections with the base stones but are only set upon it, hinges were used as the boundary condition. The nodes located in the symmetrical plane are constrained, as shown in Fig.9.

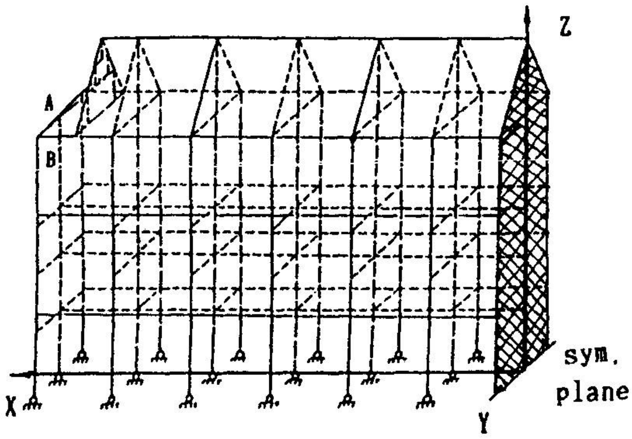
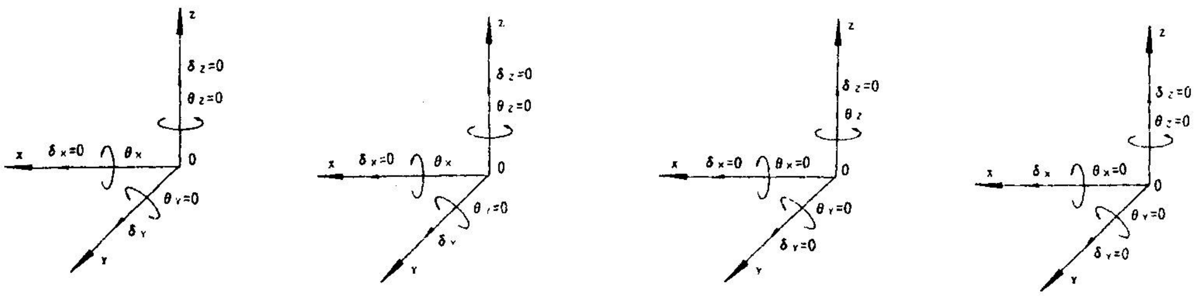


Fig.8 The FEM Model of the Front Tower



Middle Col. North and South Col. Middle Col. North and South Col.

(a) Symmetrical vibration (b) Asymmetrical vibration

Fig.9 The Boundary Condition of Symmetrical Plane

3. DETERMINATION OF THE PROPERTIES OF CONNECTORS

Since the Front Tower is over 600 years old, the elastic properties of each connector are different. It is also not necessary to consider each individual property, because what oncerns this study is the global characteristic of the Front Tower. In this paper, the average value of the elastic properties of Dougong (K_{TK}) and joggle joints located in x axial direction (K_{XA}) and y axial direction (K_{YA}) were determined.

In determining the stiffness of 3 kinds of flexible elements by means of the Simplex Method, the object function is formed as follows.

$$OBJ = \sqrt{(F_B - F_{BO})^2 + (F_T - F_{TO})^2} \qquad (2)$$

where

F_B : 1st symmetrical natural frequency by means of structural analysis

F_{BO} : 1st symmetrical natural frequency by means of test, $F_{BO}=1.100$ Hz

F_T : 1st asymmetrical natural frequency by means of structural analysis

F_{TO} : 1st asymmetrical natural frequency by means of test, $F_{TO}=1.700$ Hz

In each flexible element, let $K=K_x=K_y=K_z$ and $K'=K_{\theta x}=K_{\theta y}=K_{\theta z}$, and start the iteration when

K_{XA} $=3.0\times10^{9}$ kgf/cm
K_{YA} $=3.0\times10^{8}$ kgf/cm
K_{TK} $=3.0\times10^{7}$ kgf/cm
$K_{XA}'=3.0\times10^{11}$ kgf/cm^2
$K_{YA}'=3.0\times10^{10}$ kgf/cm^2
$K_{TK}'=3.0\times10^{9}$ kgf/cm^2

The object function OBJ and F_B, F_T change, as in Fig.10, when the iteration number N increases. The ranges of the connectors' properties which make the OBJ less than 0.02 were as follows:

K_{XA} $=0.10\times10^{9}\sim4.00\times10^{9}$ kgf/cm
K_{YA} $=1.59\times10^{9}\sim1.97\times10^{9}$ kgf/cm
K_{TK} $=2.12\times10^{8}\sim2.38\times10^{8}$ kgf/cm
$K_{XA}'=5.87\times10^{11}\sim9.77\times10^{11}$ kgf/cm^2
$K_{YA}'=6.44\times10^{10}\sim9.10\times10^{10}$ kgf/cm^2
$K_{TK}'=3.16\times10^{9}\sim7.17\times10^{9}$ kgf/cm^2

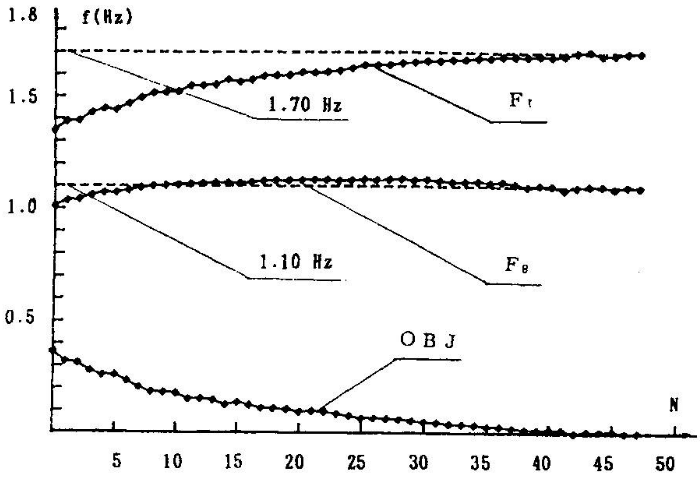


Fig.10 The Iteration of Simplex Method

4. THE DYNAMIC CHARACTERISTICS OF THE FRONT TOWER

By means of the determined properties of the flexible elements, the natural frequencies up to 10th mode (Table 1) and the vibration

Table 1 Natural Frequencies(Hz)

Mode	Symmtr.	Inv. Sym.
1	1.1089	1.7092
2	3.6093	5.6551
3	6.1694	6.7195
4	6.9043	8.1929
5	7.2118	9.2339
6	9.4001	13.0814
7	9.9288	14.7401
8	10.8743	15.1447
9	11.2877	15.9765
10	12.7344	16.9827

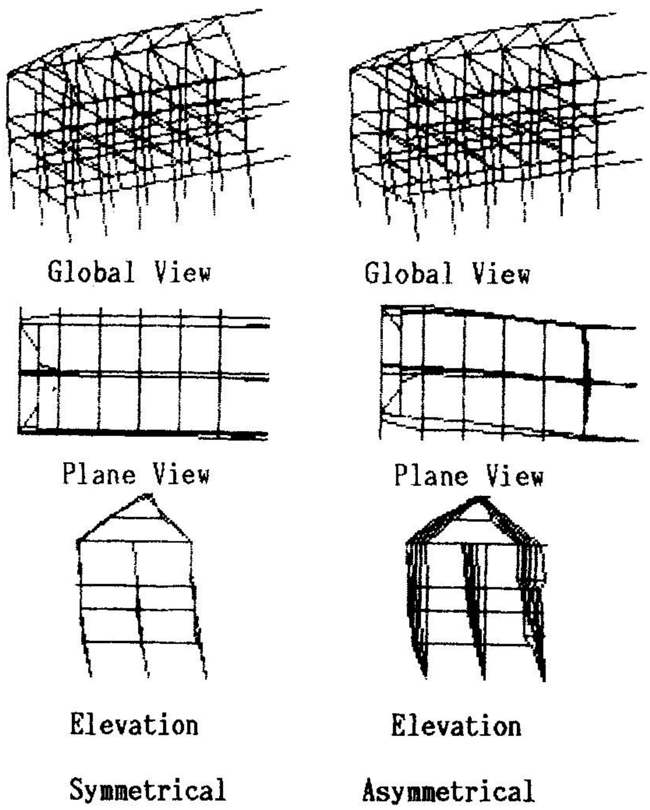
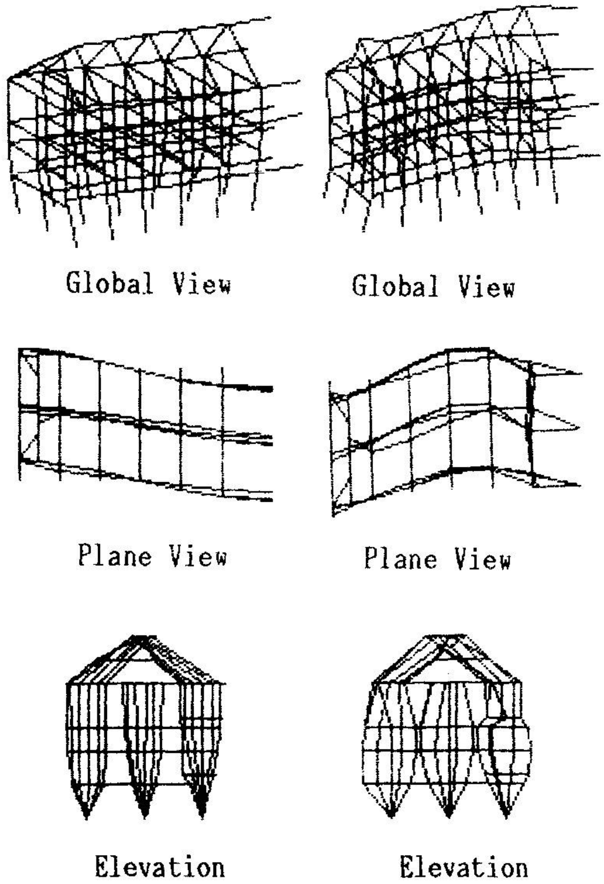
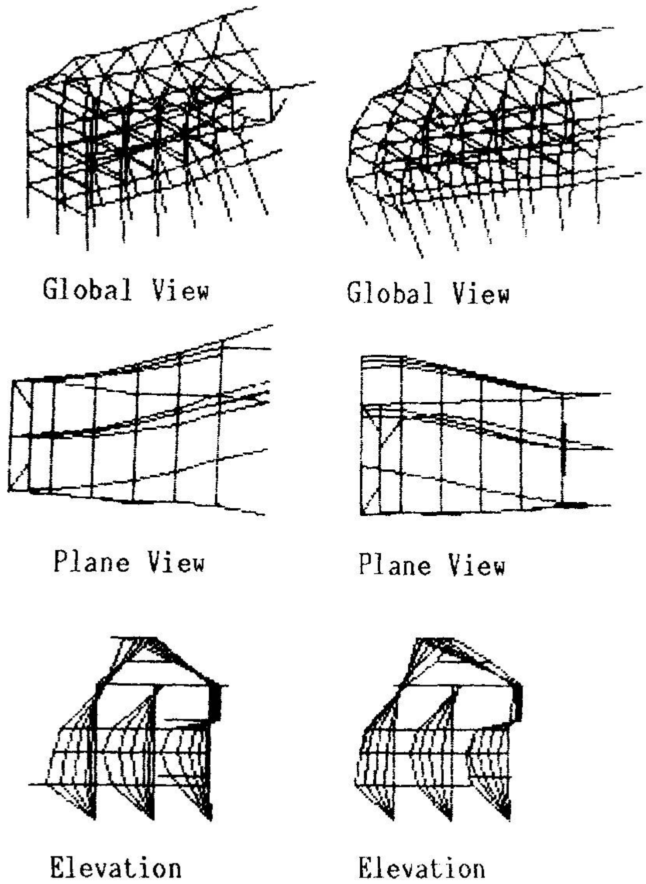


Fig.11 1st Vibration Mode Shape



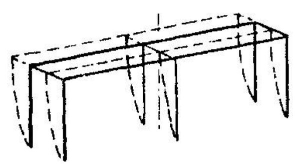
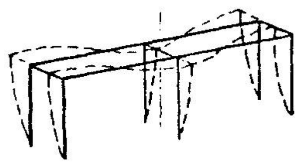
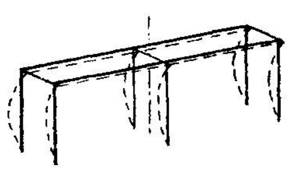
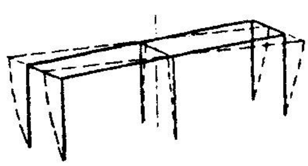
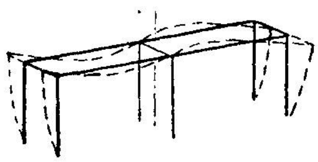
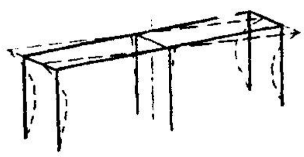
Symmetrical Asymmetrical
Fig.12 2nd Vibration Mode Shape



Symmetrical Asymmetrical
Fig.13 3rd Vibration Mode Shape

mode shape up to 3rd mode of both symetrical and asymetrical vibrations were calculated. Fig.11, Fig.12 and Fig.13 show the 1st, 2nd and 3rd vibration mode shapes. Referring to the tested results, the natural frequencies and the vibration mode shapes up to 3rd mode of both symetrical and asymetrical vibrations are shown in Table 2.

Table 2 The Natural Frequencies and Vibration Mode Shapes

Vibration Mode	1st Mode	2nd Mode	3rd Mode
Symmetric	 1.100 Hz	 2.725 Hz	 6.610 Hz
Inverse Symmetric	 1.700 Hz	 3.100 Hz	 7.200 Hz

5. THE SEISMIC EFFECT OF THE SEMI-RIGID CONNECTORS

By means of a 3-D dynamic analysis program (Flow Chart: Fig.14), the structure's earthquake response analysis was undertaken[2].

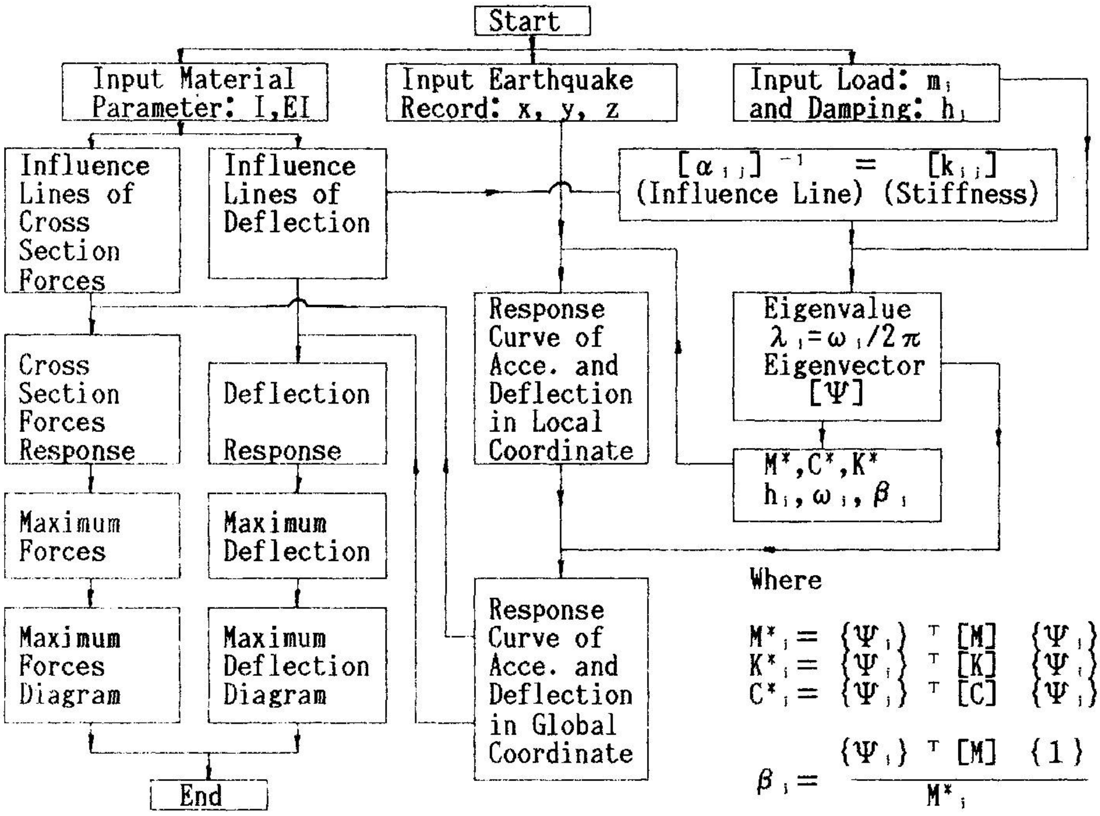


Fig.14 The Flow Chart of Earthquake Response Analysis

In order to understand the effect of the Dougong and joggle joint, the earthquake responses of the Front Tower with the connectors modeled as both rigid and semi-rigid joints were calculated (Fig.14). The deflection and acceleration responses were obtained from node A and the inner force responses were obtained from element B (referring to Fig.8), where the maximum response of the structure was recorded. It

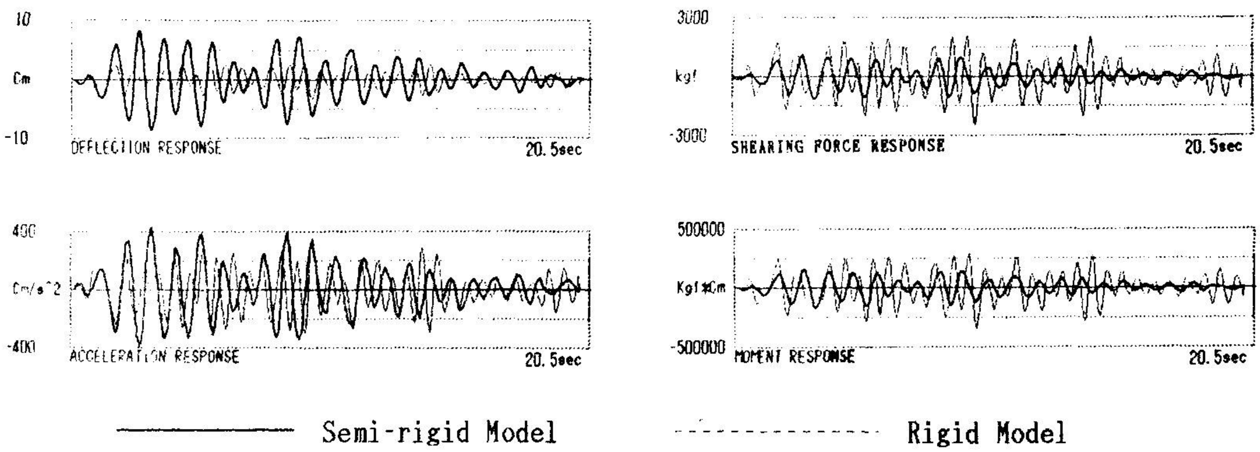


Fig.14 Earthquake Responses of the Front Tower

shows that the deflection response with the joints modeled as semi-rigid is twice that of the rigid modeled joint, and the inner force response is 2/3 that of the rigid modeled joint. Considering that the maximum deflection response is only about 8 cm, it could be said that the semi-rigid connectors offer an advantage for the structure's aseismic qualities.

6. CONCLUSION

There are many architectural heritages in the wooden structures of China, Japan and Korea. The understanding of their dynamic characteristics is of importance in structural preservation. The authors have taken years to try to clarify the behavior of the wooden structure's semi-rigid connectors, and, to understand their dynamic characteristics by means of full scale testing, model testing and 3-D FEM analysis[3][4][5].

In this paper, the elastic properties of the Dougong and Joggle joint, and the Front Tower's natural frequencies and vibration mode shapes, were clarified by means of the Simplex Method and 3-D FEM dynamic analysis. The earthquake response analysis shows that the Dougong and joggle joint are aseismic advantages. It should therefore be noted that the connectors should not be strengthened when this kind of valuable architectural structure is being repaired.

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Structural Assessment of Lisbon's Historical Buildings

Evaluation structurale des bâtiments anciens de Lisbonne

Tragfähigkeitsbeurteilung der historischen Bauten von Lissabon

S. POMPEU SANTOS
Civil Engineer
LNEC
Lisbon, Portugal

S. Pompeu Santos, PhD, is a senior research officer of LNEC and Head of the Structural Behaviour Division. Main fields of interest are structural assessment, as well as reinforced and prestressed concrete, precast, masonry, repair and strengthening of structures.

Mary MUN
Civil Eng.
LNEC
Lisbon, Portugal

Mary Mun, obtained her PhD degree from the Technical University of Lisbon. She is a research officer of the Structures Department of LNEC and has been active in consultancy work related to diagnosis and assessment of defective structures.

SUMMARY

Due to various reasons, far too many of Lisbon's historical buildings are being replaced instead of being repaired. Clearly a decision for preservation is needed, based on a correct evaluation of the situation. The paper presents a study aiming to develop a rational intervention strategy for the preservation of some of Lisbon's historical quarters. The envisaged actions, the methodology of evaluation and inspection are presented as well as the results of a preliminary survey.

RÉSUMÉ

Beaucoup trop de bâtiments dans les quartiers anciens de Lisbonne ont été remplacés récemment, au lieu d'être conservés. La décision d'une conservation doit être basée sur des données sûres quant aux dommages structuraux résultant des années. Ce rapport présente une étude visant au développement d'une stratégie d'intervention rationnelle pour la conservation de quelques-uns des quartiers historiques de Lisbonne. Les actions envisagées, les méthodes d'évaluation et d'inspection de même que les résultats d'une expertise préliminaire sont présentés.

ZUSAMMENFASSUNG

Aus mannigfaltigen Gründen werden zu viele Gebäude in den alten Quartieren von Lissabon ersetzt anstatt erhalten. Natürlich soll die Entscheidung über die Erhaltung eines Gebäudes auf zuverlässigen Daten über die im Laufe der Zeit entstandenen Schäden gründen. Dieser Bericht enthält eine Studie über die Entwicklung einer rationalen Interventionsstrategie mit dem Ziel der Erhaltung einiger der historischen Quartiere Lissabons. Die Vorgehensweise, die Bewertungs- und Untersuchungsmethoden sowie ein vorläufiger Expertenbericht werden vorgestellt.

1. INTRODUCTION

The majority of the buildings in the old quarters of the city of Lisbon are from after the catastrophic **earthquake of** November 1st 1755.

For the reconstruction of the areas affected by the quake, carried out under the supervision of then prime minister Marquês do Pombal, instructions were issued for what can be considered **the first aseismic buildings** distinguishable by the following features: incorporation of a three-dimensional well-connected wooden frame ("gaiola", meaning "cage") in the masonry walls; horizontal diaphragm action by means of timber trusses at floor and roof levels; limitation of building height to four storeys.

With time, memories of the 1755 earthquake dimmed. Buildings grew in height and volume whilst maintaining the same wall thicknesses or even reducing them; material and workmanship of inferior quality were used; the wooden "cage" degenerated into a system of poorly connected trusses and eventually was left out altogether, its purpose not well understood by constructors. In some of these latter buildings metal connectors were used to improve bonding of intersecting walls.

From the middle of last century until the full advent of reinforced concrete in the early fifties, a number of **identifiable structural typologies** evolved. The buildings of this period range usually from five to six floors above ground (attic included) and may stand isolated or integrated in a block of buildings. The load bearing walls are of unreinforced masonry, whether of stone, brick or rubble, single or double leafed; the floors initially of wood were later replaced by concrete slabs. With the diffusion of reinforced concrete and of steel structural work, mixed solutions were adopted. A typical example is the adoption of external masonry bearing walls together with internal beams and columns made of wrought iron. Many of the mixed solutions that can be seen nowadays are a result of enlargement/reconstruction operations where entire walls have been replaced by steel or reinforced concrete beams/frames or where additional storeys have been raised above top floors.

Buildings are, in themselves, a testimony to the history of urban development. For this reason some of the **older quarters** have been officially recognized as being of historical value and are now protected by municipal laws (eg. Baixa Pombalina, Alfama, Bairro Alto, Mouraria, Madragoa)[1].

Regarding the other **not so old quarters (circa 1880 - 1950)**, discussion is undergoing on whether to accept the costs of undertaking their preservation. It is clear however that if no active measures are taken towards this end and the situation is left to the dictates of the market based on economic considerations alone, ultimately, we will witness the demolition of most of the old buildings and their replacement by new highrises. The cost for achieving urban renewal this way would entail a loss of the architectural heritage and a change in the type of occupation with dwellings being replaced by offices. To support the process of decision making in this matter a study in now being carried out.

2. OBJECTIVES OF THE STUDY

A political decision for preservation is urgently needed. However, to be convincing, arguments have to be based on a correct evaluation of the situation. An answer must be given to the question: "What are the costs? For what results?". Most probably it is not possible to argue for full preservation, but based on objective data, it is possible to aim at more realistic goals.

It is the objective of the study [2] to develop a rational intervention strategy for the preservation of some of Lisbon's historical quarters. The approach envisaged for achieving this objective contemplates the following fundamental actions:

Phase 1:
- **Classification of the buildings according to a global rating based on "remaining structural capacity".**

Phase 2:
- **Definition of repair/strengthening measures needed to achieve an "acceptable level of safety" for gravity loads as well as for seismic actions; analysis of the costs involved.**

Phase 3:
- **Definition of a maintenance policy covering both preventive and remedial measures for the most common structural problems.**

As part of the strategy underlying the evaluation process it was decided to concentrate efforts on a restricted but characteristic area (**Avenidas Novas**) where the problems of structural deterioration are more acute, and, for this particular high priority area, develop a methodology which could later be extended to other areas.

3. SELECTED STUDY AREA. OBSERVED DAMAGE CONDITIONS

For the purpose of obtaining a general overview of the situation, a preliminary survey was conducted in 1986 [3] which enabled the definition of the main structural typologies and the identification of the higher priority buildings. Special buildings such as churches, monuments and mansions were not included in this survey. The assessment of the structural conditions of the over 1000 buildings was performed exclusively on the basis of visual inspections, carried out by a team of experienced surveyors under the supervision of two civil engineers, precautions being taken to ensure as far as possible the application of an uniform set of criteria. The general results are shown in Table 1.

The results of this survey show that of the 1028 buildings requiring intervention(*), 37% need corrective measures of some importance and 4% need major repairs.

Buildings	**Number**	
New or in very good condition	350	(24%)
In need of preventive measures(*)	611	(43%)
In need of corrective measures(*)	378	(26%)
In need of major repairs or for demolition(*)	39	(3%)
Under construction or undergoing overhauling	33	(2%)
Vacant lots from demolition of existing buildings	27	(2%)
	1438	

Table 1: Preliminary survey. Classification of buildings

Fig. 1: Two surviving buildings in the **Study Area**

In the **Study Area** constructions dating from the late 19th - early 20th century (1880-1930) predominate. These constructions have a characteristic structural typology (termed "gaioleiro"), featuring:

- external walls of irregular stone and weak mortar, single or double leafed, the exterior leaf made of brick masonry;

- internal walls of solid or perforated bricks, with or without a wooden frame (lower floors), and light wood panelling (upper floors);

- wooden floors and staircases; wooden roof structures;

- small inner courtyard (central or lateral) for admission of light and ventilation, enclosed by single or double leafed masonry walls;

- balconies at the back of the building, with tile flooring supported by a light (metal) structural frame.

The causes of damage are many, some having to do with normal lifetime deterioration and others with man's actions. The deterioration process caused by the exposure of the materials to environmental aggression is here drastically aggravated by the neglect of maintenance measures both at the preventive and the corrective levels. Of special significance are those needed to keep water away from masonry, resulting from leaky roofs or ruptured pipes. However very often interventions are performed only when an emergency is at hand.

Some of the observed damages suggest there may be inherent construction weaknesses which may be at the root cause eg. foundation weaknesses, overloading of certain critical areas due to the architectural layout (ie. the inner courtyard and the area affected to the balconies). A structural assessment may show that the level of safety is low due to these inherent weaknesses, even under the assumption that no strength degradation has taken place.

The most common damage inducing actions attributable to man were identified as:

- excavation works, both at the surface and underground, in the proximity of, or under existing buildings, resulting in localized differential settlements of the wall foundations;

- demolition of one of two buildings having a common separation wall, resulting in the destruction of the original connection between intersecting walls, both at the façade and at the back (Fig. 2);

- alterations to the existing structural system involving the total or partial demolition of load bearing walls (negative shoring-structure interaction);

- overloading through a change in building usage (eg. office archives in previous dwellings).

Fig. 2: Damaged masonry wall due to demolition of an adjoining building

4. METHODOLOGY OF EVALUATION

The evaluation of existing masonry structures in terms of remaining structural capacity involves an analysis for both gravity loads and seismic actions.

Where the latter are concerned, the more sophisticated approach to the problem would be the calculation of the time history inelastic response of the structure to ground excitation caused by a series of probable accelerograms using the Finite Element Method. The difficulties associated with this type of analysis are the great number of intervening parameters, the uncertainties related to them and the sensitivity of the results to these parameters. In view of these, where it concerns the systematic study of a great number of buildings, a linear elastic method of analysis will be used. Criteria will have to be established for the extrapolation of the results of elastic analysis to ultimate load behaviour.

The evaluation of the load bearing capacity of existing masonry structures must take into account both the type of construction and the extent and influence of deterioration on structural capacity. Information is needed regarding: construction details (eg. wall thicknesses, constituent materials, workmanship, degree of interconnection between walls and walls/floors); level of deterioration of the construction materials; extent and type of structural defects such as cracks and fissures, distortions, bulges and deviations, missing bricks and blocks, loss of interconnection between diaphragms, etc.

To obtain the relevant data, a programme has been established for in-situ and laboratory investigation of individual buildings, taken as representative of groups of buildings. The investigation will include a survey of the architectural and constructive features, a survey and assessment of the damages and a testing programme to determine the mechanical parameters of existing masonry structures.

For the identification of the dynamic behaviour of structures (natural frequencies, modal shapes and damping ratios), experimental in-situ tests will also be performed.

Because of the destructive or at best semi-destructive nature of the initial investigation (such as corings, removal of masonry blocks and flat-jack tests), operations will be restricted to buildings awaiting demolition or contemplating intervention.

5. IN-SITU MASONRY TESTS WITH FLAT-JACKS

The envisaged testing programme for the quantitative assessment of the residual mechanical characteristics of old masonry relies to a great extent on a technique based on the use of flat-jacks.

The flat-jack technique was originally developed for the determination of residual stresses at the surface of rock masses, particularly in tunnel and gallery walls. The method adopted at LNEC [4] consists in applying, by means of a thin copper sheet jack, a uniform pressure of up to 20 MPa to the walls of a slot cut in the rock mass by a diamond-edged disk. The state of stress is given by the pressure needed to cancel the deformation at the surface of the wall due to the cut. This technique can also be applied in masonry to determine the state of stress in walls for calibration of analytical models.

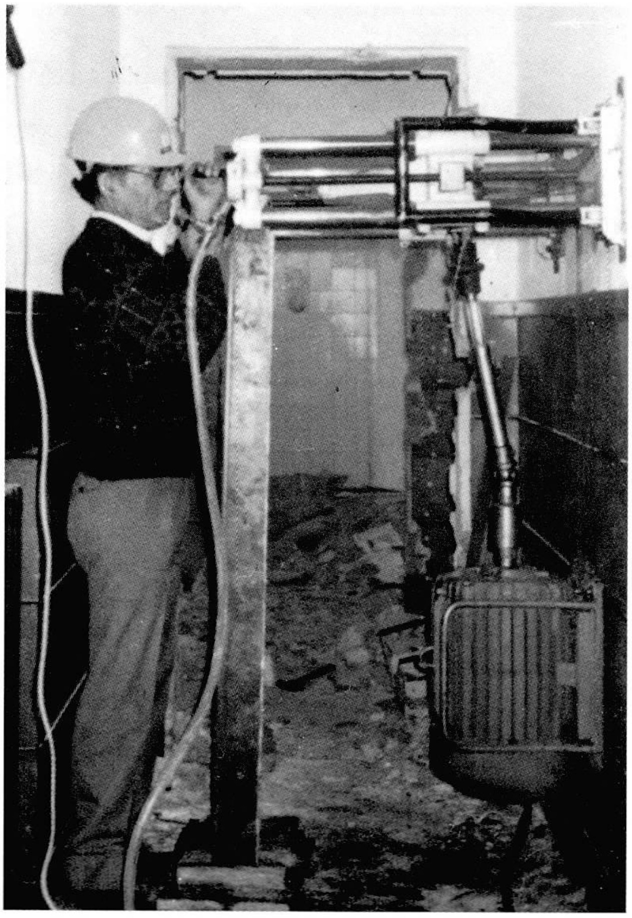
A relatively recent development of the flat-jack technique permits also the determination of the strength and deformability characteristics of in-situ masonry. The technique consists basically in the insertion of flat-jacks in two parallel cuts made in the surface of the masonry wall. By applying pressure through the jacks, the masonry block between the two cuts can be tested for deformability (axial and transverse) and compressive strength [5].

The LNEC testing equipment consists of a set of hydraulic jacks, a cutting machine (fitted with a 600 mm diameter diamond disk), a device for extracting the jacks, pressure gauges and deformeters. The available equipment permits cuts from depths of 100mm up to 240mm (Fig. 3).

6. INTERVENING PARTIES

The outlined study, still in its early stages of development, is being carried out by LNEC (Laboratório Nacional de Engenharia Civil) for CML (Câmara Municipal de Lisboa), Lisbon's Municipal Authority. This study is conducted in conjunction with normal consultancy activities on behalf of CML.

SNPC (Serviço Nacional de Protecção Civil), having established a programme for the seismic risk assessment of Lisbon's quarters, launched in the early eighties, has already shown interest in the results of the planned tests and field inspections. As work progresses, a liaison with SNPC will be established.



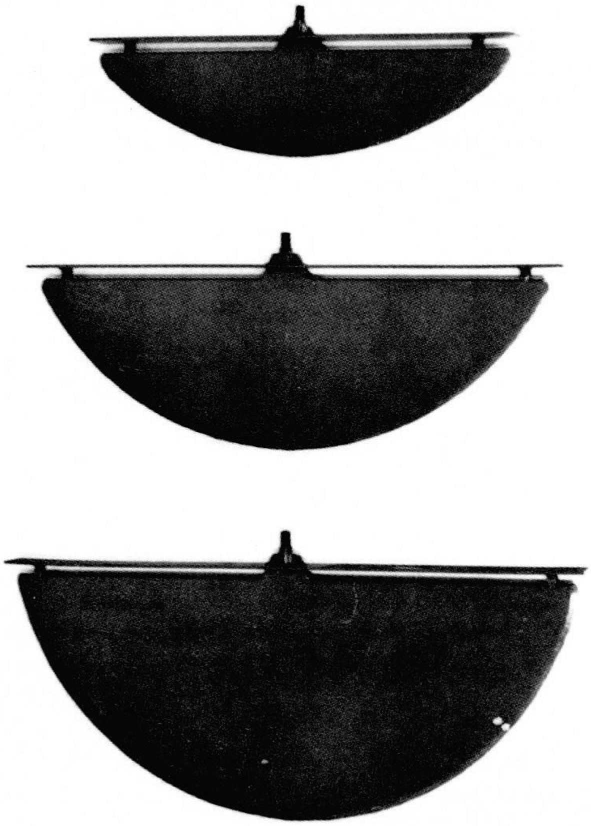


Fig. 3: Cutting operation in a flat jack test

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Modelling and Monitoring the Structure of Hagia Sophia in Istanbul

Modélisation et surveillance de la structure de Hagia Sophia à Istanbul

Modellierung und Überwachung des Tragwerks der Hagia Sophia in Istanbul

Robert MARK
Prof. of Archit. & Civil Eng.
Princeton Univ.
Princeton, NJ, USA

Ahmet S. ÇAKMAK
Prof. of Earthquake Eng.
Princeton Univ.
Princeton, NJ, USA

Mustafa ERDIK
Prof of Earthquake Eng.
Bogazici Univ.
Istanbul, Turkey

SUMMARY

Research is being conducted for assessing the seismic capacity of one of the most important historic buildings of Western civilization, the almost 15-century-old Hagia Sophia, by integrating data from on-site archaeology and measurements of both ambient and earthquake response into an interactive, numerical model. In addition to providing greater understanding of the behaviour of the building's complex structure, the study will also ascertain possible needs for seismic strengthening.

RÉSUMÉ

Une recherche est entreprise pour déterminer la résistance séismique d'un des monuments historiques des plus importants de la civilisation occidentale, Hagia Sophia, qui a près de 15 siècles. Celle-ci est réalisée par l'intégration de données résultant d'études archéologiques sur place et de mesures du comportement aux tremblements de terre dans un modèle numérique interactif. Cette étude permet d'avoir une meilleure compréhension du comportement de cet ensemble architectural complexe et permet de déterminer les besoins possibles d'un renforcement séismique.

ZUSAMMENFASSUNG

Gegenwärtig wird die seismische Tragfähigkeit eines der wichtigsten historischen Bauwerke der westlichen Zivilisation, der fast 15 Jahrhunderte alten Hagia Sophia bestimmt. Dazu werden die aus örtlichen archeologischen Studien mit Feldmessungen unter Umwelt- und Erdbebenerschütterungen gewonnenen Daten in ein interaktives numerisches Modell integriert. Die Forschungsarbeiten dienen dem besseren Verständnis des Verhaltens der komplexen Gebäudestruktur und klären den Bedarf nach gezielten Verstärkungsmassnahmen.

Historical Background

Because of the dual role that Justinian's great church of Hagia Sophia (Holy Wisdom) in Constantinople was to assume in both ecclesiastical and imperial liturgies, the architects, Anthemius of Tralles and Isidorus of Miletus, combined the traditional longitudinal basilican plan (a large rectangular hall having a high central space flanked by lower side aisles) with a great Roman central dome. Considering the close correspondence in scale between the original dome of Hagia Sophia and that of the Pantheon (ca. 118-128) it is likely that the earlier building provided the principal structural model for Justinian and his architects as they translated Roman concrete into Byzantine masonry [1]. Yet Hagia Sophia is a precedent-setting building: where the vast dome of the Pantheon rests on continuous, massive walls, four great arches and a like number of pendentives direct the weight of Hagia Sophia's superstructure to huge supporting piers (Fig. 1). The combination of large, glazed tympanum surfaces with a dome of monumental scale stands as one of the greatest architectural achievements of all times.

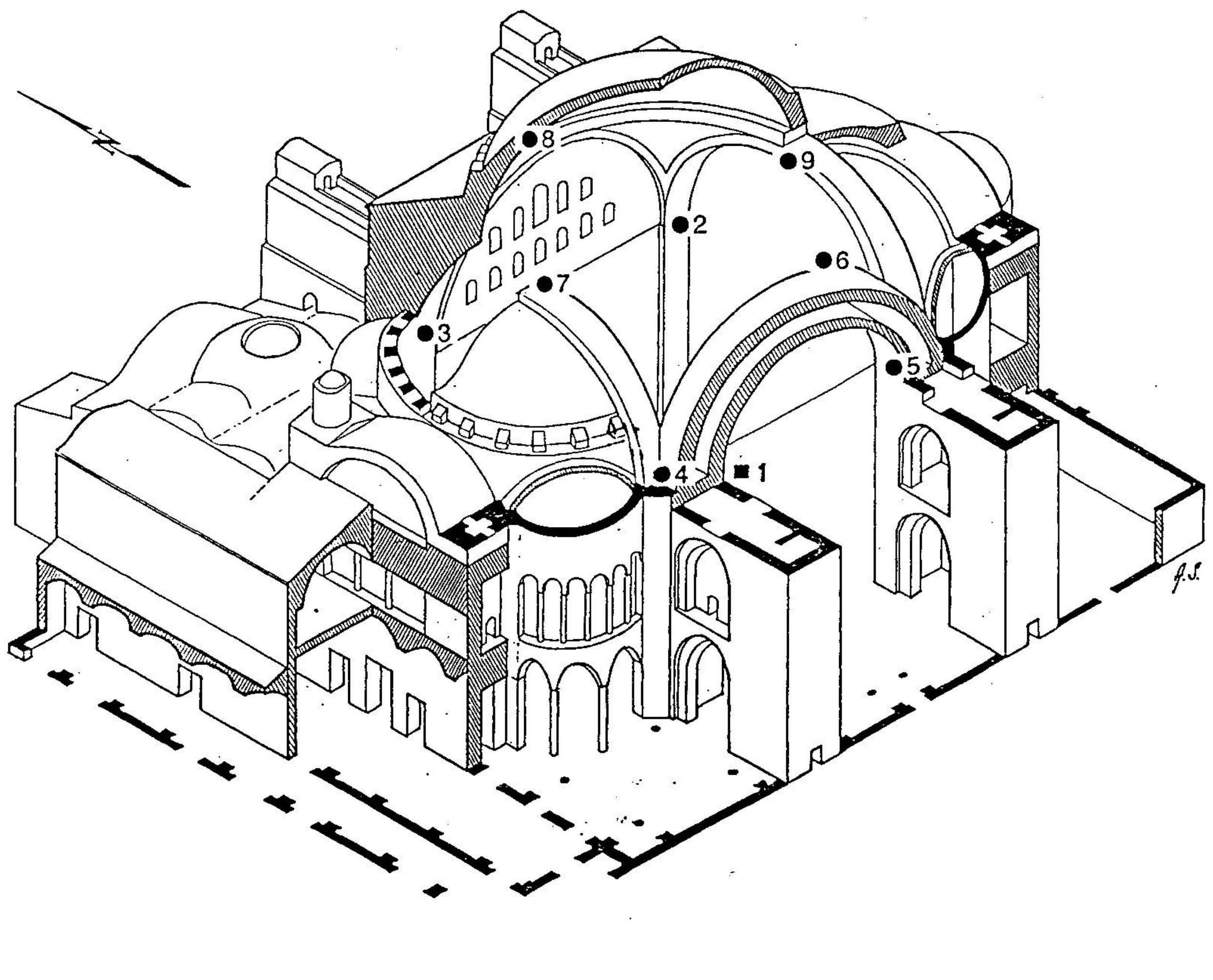

Figure 1. Analytical drawing of the Hagia Sophia structure (numbers indicate accelerometer locations).

In addition to the host of constructional challenges normally associated with such a major building project, political instability within the Empire required that Hagia Sophia, as the most visible symbol of Justinian's prestige in the capital, be completed as swiftly as possible. Begun in 532, construction proceeded in more or less horizontal layers until the erection in ca. mid 535, of the main arches, 31 meters in span and springing some 25 meters above the floor, to support the dome. Flying centering was probably used for assembly of these arches. And in all likelihood, this centering would not have been adequately tied to prevent enormous horizontal forces from impinging upon the upper portions of the main piers which then proceeded to tilt outward (the average, outward defection of the piers at the level of the springing is 45 cm). Before continuing with the construction, the exterior pier buttresses were reinforced and enlarged to their present height. The structure must have then seemed secure because the dome was raised in time to allow the vast building project to be completed in 537. Nonetheless, the great central dome fell in 558 after being subjected to two major earthquakes: the first one in August 553, and the second in December 557. A second dome having a higher profile than its predecessor was then erected in 558-562. Despite two partial collapses after earthquakes in the tenth century, and again in the fourteenth, the general form of the dome today remains unchanged from that of 562. But structural repairs associated with these incidents, as well as other adversities, have involved the placement of additional buttressing around the entire structure.

The present study is aimed at deriving a better understanding of the structural history of Hagia Sophia over its one-and-a-half-millennium life, including the strategies employed for its design and construction, and to determine the monument's current earthquake worthiness (and if necessary, recommend possible structural improvement). To accomplish these ends, several concurrent efforts are being undertaken which include: 1) creation of numerical models to account for both short- and long- term non-linear material behavior, including the consequences of cracking and effects of component deformation during the initial sequence of construction and subsequent structural modification; 2) determination, from physical and chemical tests, of the properties of the building materials, particularly the time-dependent behavior of early mortars; and 3) monitoring of measurements from accelerometers placed on the actual building structure under the action of vibrations produced by earthquakes.

Modeling

Two parallel types of numerical models are being formulated. The first, based on SAP 90 software, provides a purely linear elastic representation of the structure. The second, using the program FENDAC being developed by Colby Swan at Princeton, can account for the non-linear elasto-plastic behavior of masonry. Linear elastic models (including elastic models rendered non-linear by allowing cracking or weakening at specified tensile stress levels), although insensitive to values of elastic moduli, can provide essential information about overall stress distributions where the prototype is essentially composed of a single material (for example, to highlight regions of tension where cracking is likely to occur). The structure of Hagia Sophia, on the other hand, incorporates at least three major classes of materials: stone, brick, and mortar, the later containing brick dust and fragments that impart to it pozzolanic characteristics, but with a relatively long curing time (see below). In this case, criteria for modeling integrity are based on matching deformations; i.e., 1) predicted static deformations should agree in both form and magnitude with those observed in the prototype; and 2) natural frequencies and mode shapes computed by the models should match those determined from the on-site measurements. Because of the long curing time of the Byzantine mortar, however, both criteria cannot be simultaneously satisfied using the same material characteristics as is illustrated in the following.

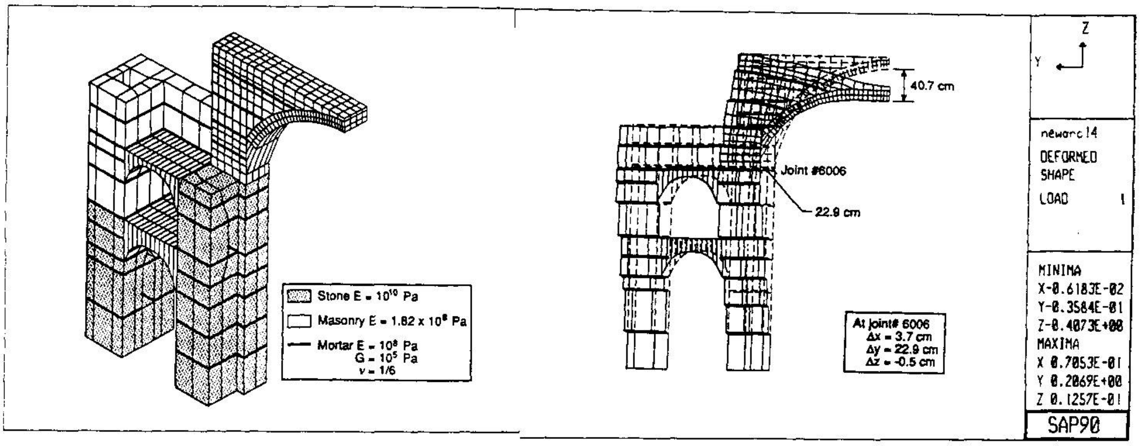

Figure 2. NEWARC 14: Partial finite element model of the Hagia Sophia structure under dead-weight loading.

Much of the effort to date has concentrated on the linear, SAP 90 models. A static example is afforded by the model designated as NEWARC 14, illustrated in Fig. 2. Mechanical properties of the constituent model materials are here determined from an inverse analysis that focused on the northeast main pier -- whose outward deflection at the point of springing was estimated to be 18 cm during initial construction, just prior to erection of the first great dome [2] -- the associated pier buttress, and adjacent great arches. The main piers are formed of stone, either limestone or a local granite, of up to about a meter in length and 45 cm thick. Mortar layers between stones are relatively thin, probably no more than several centimeters. For modeling, the stones are represented by elements whose thickness averages about 3 meters, interspersed with 25 cm of mortar. The pier buttress is assumed to incorporate similar stone and mortar layers up to the level of the first connecting arch, above which it is composed of brick masonry containing a large cavity for the existing stairwell. As shown in Figure 2, good results were achieved from the model by adopting the indicated values of Poisson's ratio and elastic and shear moduli. Using these properties for early material behavior, a full model was then constructed in three stages, allowing those portions of the structure in each stage to deform and weaken (at prescribed levels of tension) before the portions of subsequent stages were added. This model, while accounting for geometric non-linearity, is still elastic; nevertheless, it helps to reveal some of the characteristic behavior of the prototype that will influence future non-linear modeling.

Because Byzantine mortar today exhibits vastly altered physical properties from those displayed during the early stages of the building's construction, the indicated values need to be modified in order to allow the elastic model to meet the second, dynamic modeling criterion. This is exemplified by the eigen-value-analysis of the linear-elastic model designated as DYN26. With mechanical properties represented by: E (stone) = 10^{10} Pa; E (surcharge) = 2.5 $(10)^9$ Pa; E (tension areas) = 10^9 Pa; E (all else) = 5 $(10)^9$ Pa; and the density of the pendentives taken as three-fourths of the nominal density of masonry, the model indicated the first three vibrational modes described in Table 1 which also presents a comparison with measured ambient natural frequencies (from a test described below). In this context, it is interesting to note that the application of the material properties from the (static) NEWARK 14 model to the DYN26A model not only results in frequencies that are too low by a factor of seven, but mode shapes that are incorrect as well.

TABLE 1: Hagia Sophia Ambient Vibrational Modes

	Mode	Mean Measured Frequency	Calculated Frequency
1st:	east-west	1.8 Hz	1.97 Hz
2nd:	north-south	2.1 Hz	2.09 Hz
3rd;	rotational	2.4 Hz	2.38 Hz

Mortar Analysis

The critical role of mortar in the structure of the Hagia Sophia has led to a collaboration with researchers at the National Institute of Standards and Technology, Gaithersburg, MD. This investigation has the main objective of determining the composition of the material from which strength and deformation properties may be inferred [3]. Obtaining samples of mortar for such tests, however, presented some difficulty. Most interior surfaces of Hagia Sophia are covered by a thick layer of plaster on top of which are frescoes or mosaics. Brick and mortar are accessible in the interior passages of buttresses, but it is not always clear whether materials date from the original construction or from later restoration; nor is large-core drilling permitted. Fortunately, there is a significant collection of thoroughly documented samples from Hagia Sophia in the Dumbarton Oaks Museum, collected by Robert Van Nice during his decades long study of the building, and some of these have been kindly made available to us for testing.

The analysis, to derive both chemical composition and physical properties, involves a set of instrumental methods including quantitative X-ray diffraction, thermal analysis, and automated image analysis of polished sections under scanning electron microscopy. These conventional methods are supplemented by neutron diffraction and neutron and X-ray small angle scattering. Test results to date are consistent with pozzolanic mortar, the pozzolan being provided by the crushed brick. Pozzolan mortars offer far higher tensile strengths than mortars of pure lime; yet strengths for such mortars develop relatively slowly compared with modern Portland cement. Lea has reported tensile strength data for modern lime-pozzolan cements as 0.7-1.4 MPa after 7 days, 1.4-2.6 MPa after 28 days, 2.4-3.5 after 90 days, and 3.6-3.9 MPa after one year of curing [4].

The conventional civil engineering approaches for determining mechanical properties of materials are essentially ruled out here because of the lack of large specimens. Another generic problem with strength testing of brittle materials, ancient or modern, concerns finding a suitable specimen configuration for tensile strength measurement. A scaled-down modulus of rupture (bending) test seems the only feasible solution for the specimens at hand. The (static) modulus of elasticity can also be estimated from modulus of rupture measurements on thin disks using a method developed by Wittman and Prim [5]. And of course, the dynamic modulus of elasticity can be determined non-destructively by measuring the speed of sound waves through a specimen.

Monitoring

Two sets of measurements have so far been taken at the building site. The global dynamic behavior of the structure at low amplitudes was established from ambient vibration measurements taken at different locations within the building in 15 separate tests using a set of four seismometers. From preliminary model results, it was expected that the structure would exhibit greatest motion in the east-west direction. This first mode of vibration was confirmed by the ambient measurements, as were also the second and third modes, as shown in Table 1 above. Of import too for the overall building study, the measured frequency spectrum falls within the frequency content of typical earthquakes observed on the North Anatolian fault.

The structure of Hagia Sophia is expected to behave non-linearly at higher, earthquake-level forces. To delineate the higher-amplitude behavior, a network of digital strong-motion accelerometers (Kinemetrics SSA-2 accelerometers) having 30 channels of readout was permanently mounted at the locations shown in Figure 1. These were chosen to capture the motion of the main structural elements supporting the central dome including the four main piers and adjoining great arches. Ground motion at the base of the main piers is measured by the accelerometer labeled 1. (For purposes of analysis, it has been assumed that all pier bases lie on bedrock so that the effect of any base variation on the dynamic behavior of the response above is negligible.) Behavior of the main piers at the level of the vault springing is measured by the array labeled 2-5; and the behavior at the crown of the great arches is measured by the array labeled 6-9. The system response is then characterized by the motion of the two arrays relative to one another and to the base.

The strong-motion accelerometer system was triggered on 22 March 1992 by a 4.8 Richter-scale magnitude earthquake whose epicenter was at Karacabey, Turkey, 120 km south of Istanbul. This earthquake induced the type of horizontal acceleration component time history represented in Figure 3. While the traces indicate a non-stationary process, it is possible to consider three approximately stationary periods or time windows: the first subtending the beginning of the record up to about 10 seconds; the second, from 15 to 25 seconds; and the third, from 30 to 40 seconds. Spectral analysis [6] of the data led to something of a revelation: the observed frequencies indicate non-linear behavior for the masonry structure even at these rather low response levels. The peak frequency at the springing level of a main pier varies from a high of 2.03 Hz in the first window to 1.74 in the second and back to 1.84 Hz in the third, a pattern corresponding to that of a non-linear system in which the system characteristics vary according to the intensity of loading.

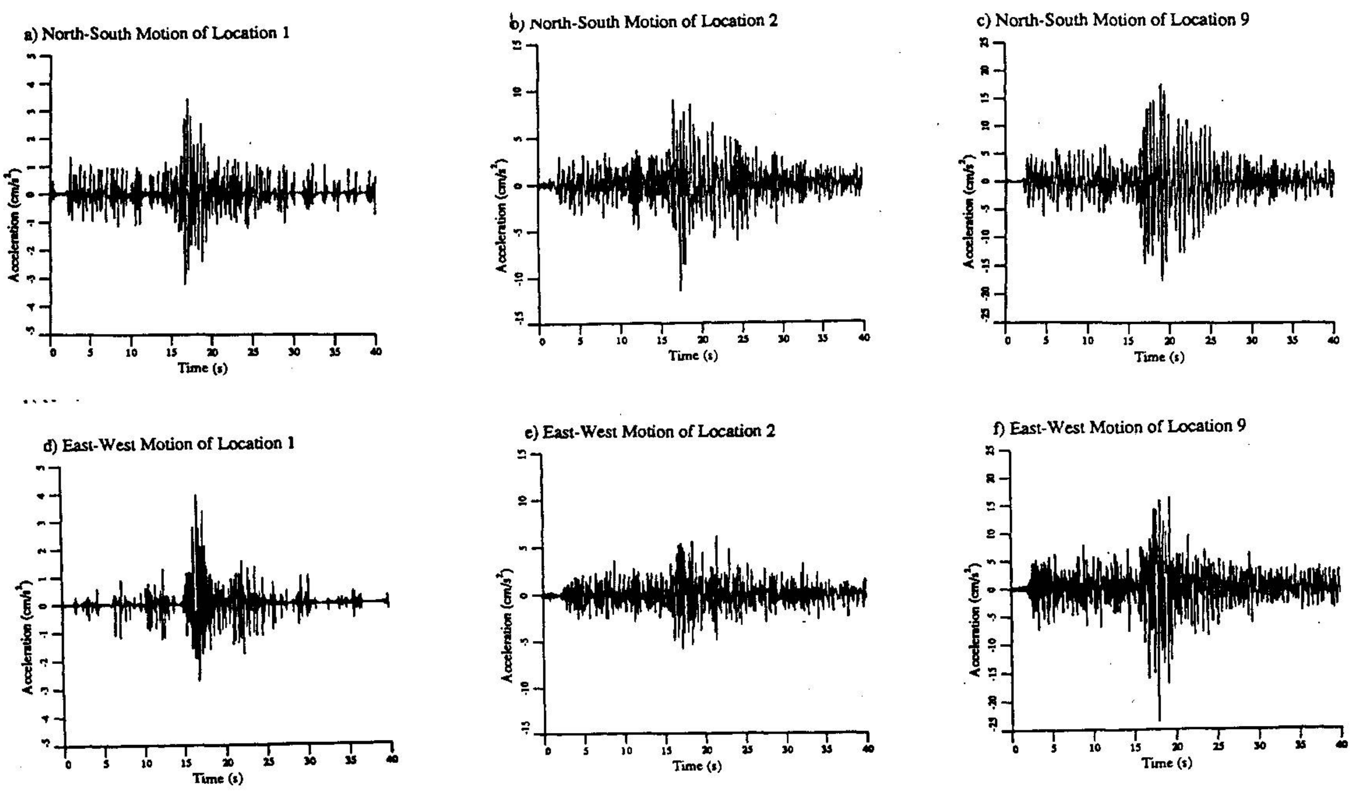

Figure 3. Horizontal acceleration component time histories in Hagia Sophia during earthquake of 22 March 1992.

If indicating somewhat lower frequencies, nominal vibration modes established from the earthquake data (Table 2) were similar to those established from the ambient tests (cf. Table 1):

TABLE 2: Earthquake-induced Vibrational Modes

Mode	Mean Measured Frequency
1st: east-west lateral	1.73 Hz
2nd: north-south lateral	1.85 Hz
3rd: rotational, about z-axis	2.3 Hz

Conclusion

Although much additional work on this project remains to be undertaken, some new perceptions about the structure of this magnificent monument are already coming to light. The first concerns the role of the slow-curing pozzolanic mortar during the initial construction process. The mortar allowed the development of early, large deformations, but its inherent plasticity also helped to reduce possible cracking. A second insight derives from the observed (in the numerical models) predisposition of the east and west great arches, that provide support to the central dome, to warp out-of-plane under gravity loading. With additional out-of-plane motion caused by an earthquake (in this regard note also that the lowest vibration mode is east-west), the basis for the collapse of adjacent portions of the central dome (in the east and west) at different times throughout the building's history becomes more clear.

Perhaps more important for historical interpretation of the Hagia Sophia structure is the finding that the changing of the first to the second dome configuration had only small effect on relieving the total outward thrusts on the main piers. This new understanding goes counter to almost every modern historical explication of the second dome form. Finally, data from the single, low intensity earthquake so far experience by the instrumented Hagia Sophia structure has revealed the non-linear response of its masonry.

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Diagnosis of Damage in a Wooden Vaulted Structure

Estimation des dommages dans une construction voûtée en bois

Schadensbeurteilung bei einer hölzernen Schalenkonstruktion

Paola RONCA
Assoc. Professor
Politecnico di Milano
Milan, Italy

P. Ronca got her Civil Eng. Master degree at the University of Waterloo, ON, Canada and her degree in architecture at the Politecnico di Milano. She teaches structural engineering at the Faculty of Eng. in Milano and strengthening of Structures at the Faculty of Eng. of Brescia.

SUMMARY

The methodology adopted in the diagnostic phase to understand the causes of the static anomalies in the wooden vaulted roof of the 'Palazzo della Loggia' in Brescia is illustrated. The paper underlines the usefulness for a correct diagnostic engineering interpretation of a mutual exchange of information between numerical and experimental findings. In particular, the realistic description of the constitutive law of the structural elements - needed in the numerical analysis - has been based upon particular experimental tests.

RÉSUMÉ

La méthode appliquée dans la phase d'estimation des causes des défauts statiques dans le plafond voûté en bois du 'Palazzo della Loggia' à Brescia est présentée. L'article montre l'utilité d'une estimation correcte et de l'interprétation du spécialiste pour un échange mutuel d'informations entre les résultats numériques et expérimentaux. En particulier, une description réaliste des lois constitutives des éléments structuraux qui sont nécessaires pour l'analyse numérique, a été réalisée sur la base d'essais expérimentaux particuliers.

ZUSAMMENFASSUNG

Die zum Ergründen der statischen Unregelmässigkeiten der hölzernen Gewölbeabdeckung im 'Palazzo della Loggia' in Brescia angewandte Methodologie wird vorgestellt. Der Artikel beschreibt die Notwendigkeit eines gegenseitigen Austausches der Werte aus numerischen und experimentellen Analysen, damit eine ingenieurtechnisch richtige Diagnose ermöglicht wird. Insbesondere die realistische Beschreibung der Werkstoffbeziehungen der tragenden Elemente - wie sie für die numerische Berechnung erforderlich sind - basiert auf eigens durchgeführten experimentellen Untersuchungen.

1. INTRODUCTION

Computer modelling provides a great tool in understanding the real behaviour of structures, either new or ancient. Neverthless, in particular for problems concerning old structures, the use of numerical methods are often unapplicable because of the lack of knowledge in the actual values of mechanical parameters. In situ tests or monitoring devices not allwais can be reliable in giving the required quantitative informations of the data usefull for an accurate analysis. In addition to these reasons, the structure under cosideration may present structural typology quite similar to many structures of the same historical period. In this respect the metodology for an adeguate understanding of the statical behaviour assumes a particular meaning, being the structure itself not only an historical monument, but also a prototype, the study of which may be usefull in many auther cases. With these premises the complete and correct metodology for the diagnostic interpretation needs a carefull and mutual exchange of informations between numerical and experimental findings.
The present work illustrates this kind of metodology used to understand the excessive deflection of the wooden vaulted roof of the Palazzo della Loggia in the city of Brescia. After a primal phase of inspection, monitoring and in situ non destructive tests able to give informations about the actual state of the materials and structure [1],[2], it has been necessary to make some ingegneristic interpretations, and to verity them or to search a different response by means of numerical models and experimental tests. In particular it was necessary to understand the efficency of the joints of the principal truss arches of the vault. Some prototypes of wooden joints have been constructed in the laboratory of the Departement of Civil Engineering to calibrate in a relistic way the parameters necessary to estimate the stiffnes of the connections for the numerical non-elastic analysis. As a matter of fact the - actual joints of the truss wooden arches are made by different series of - bolts. The efficency of which, as well as the behaviour of the connections arequite difficult to know beacause of the differential deformations among timbers, steel plates and bolts. The paper want to underline the importance of this metodology by describing the real structure and its damages, the diagnostic interpretation, the trials, by means of numericals modelling, to - adjust or confirm the ingegneristic hipotesys following the findings of the tests. Being the work still in progres, the present paper reports toughether with the numerical approach just a first part of the experimental research, which has provided nevertheless very significant results used in the numerical non-linear analysis of the structural behaviour.

2. THE WOODEN VAULTED ROOF OF THE LOGGIA PALACE

The figure 1 shows the principal facade (XVI th century) of the building. the actual roof is made by a vaulted wooden structure of important dimension - (fig.2), the hight of which reaches a maximum of 25 meters with a planar dimension shown in figure 3. The structural architecture of the vault consist of principal truss wooden arches and simple secondary arches (arch a and b); both are connected at the top by a truss made wooden beam (fig.4).
The semiarches of the short side of the roof end on the diagonal arches, to which are connected (fig.5). The principal arches, made as the entire structure, of hoak wood, are trussed structure, as shown in figure 4, made of two longitudinal curved beams a1 and a2 (the intradoss and the extradoss of the arch) connected by diagonal and straigth elements . The two curved beams a1 and a2 converge to forme a unique monolitical element at the base, which ends at two different levels.

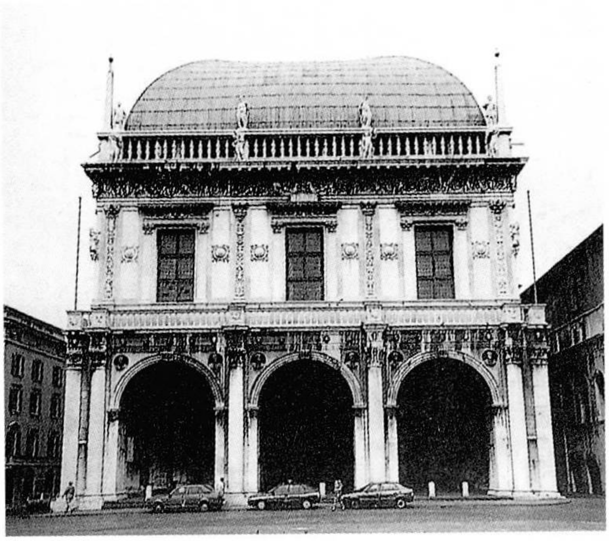


Fig.1 The main view of the Loggia

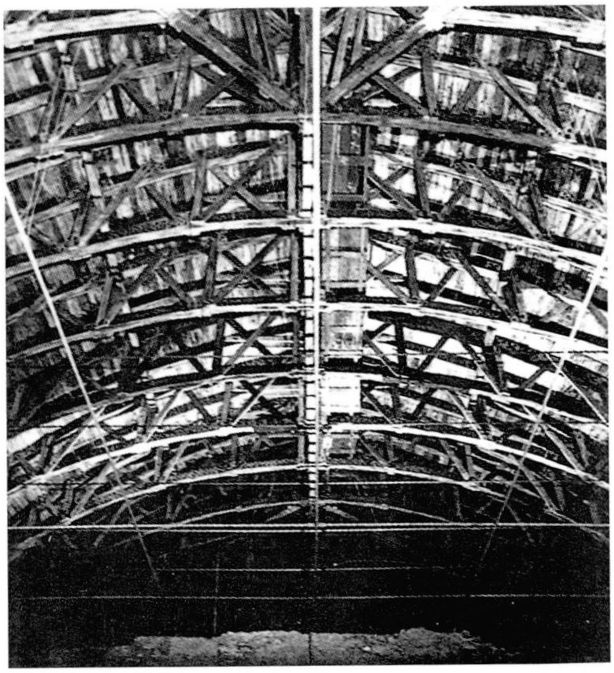


Fig.2 The wooden vaulted roof

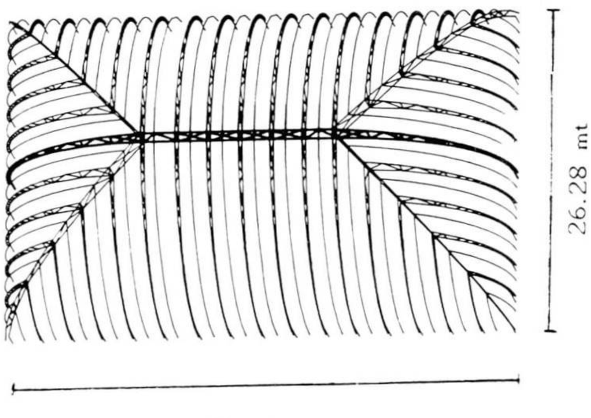


Fig.3 Planar drawing of the roof

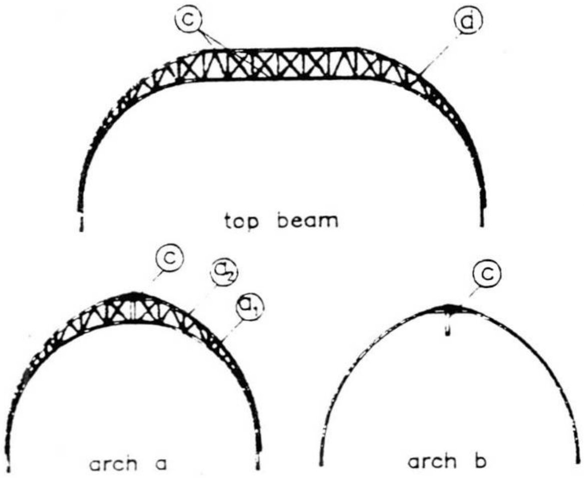


Fig.4 the principal structural element of the roof

The two curved beams a1 and a2 of the principal arches are made by the union of three lumbers of section 10x30 cm and length of about 2 meters, as shown in figure 6. The monolitical behaviour is provided by numerous steelplates and bolts, inserted probably in non calibrated holes. Originaly the transversal continuity of the element was assured by the friction of the lumbers, achieved by the tightening of the bolts. Essentially because of deformations of the holes and the time dipendent behaviour of the wood the bolts lost their tightening. As consequence, slips and mutual displacements either among the lumbers and among the elements of the truss joints has been permitted. As a - matter of fact signs of successive tightening of the bolts are visible.

Fig.5 The view of a diagonal arch

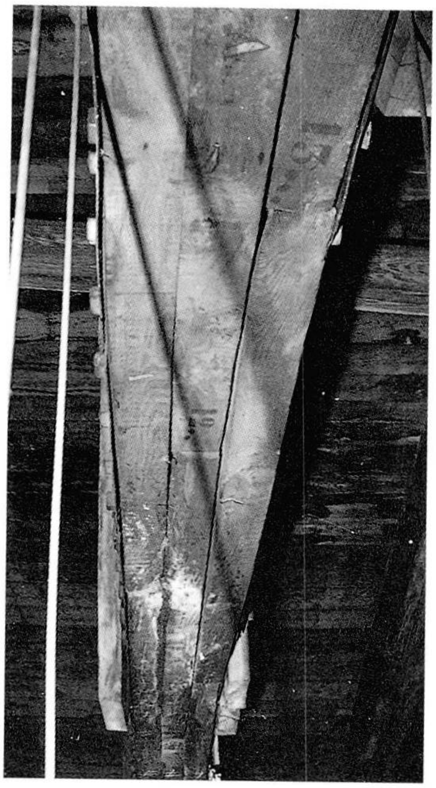


Fig.6 The lumbers and the steel-plates which form the extradoss and the the intradoss of the arches

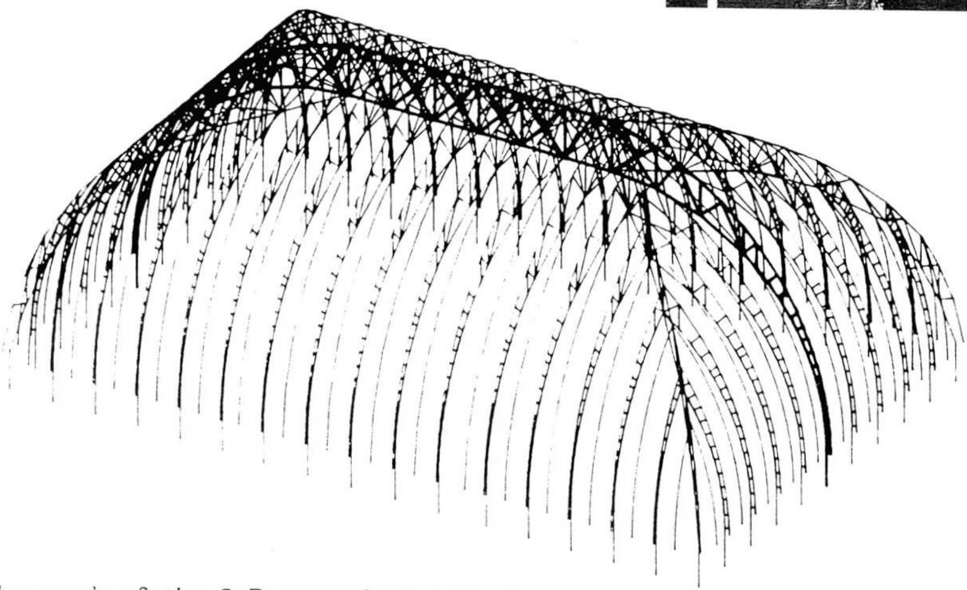


Fig.7 The mesh of the 3-D numerical analysis

3. SURVEYS AND STATIC INTERPRETATION OF THE DAMAGES

The structural damage of the vault is evidenced essentially by an important deflection of the longitudinal top beam and the connected arches. The diagnostic phase has required, as in most cases, the following different approaches:

i) measures of the effctive out-of-plane of the lateral walls

ii) the set-up of a monitoring system able to give informations about the possibility of progressive deformations and the grade of sensitivity at the umidity and change of temperature

iii)the check up of the grade of the deterioration of wood.

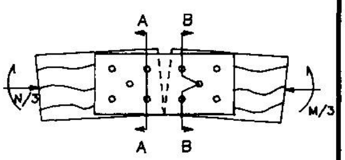
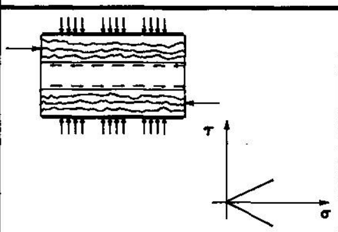
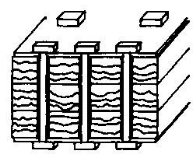
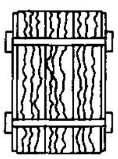
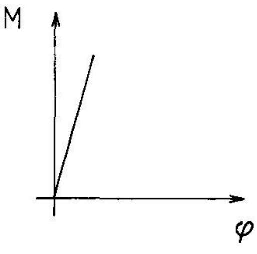
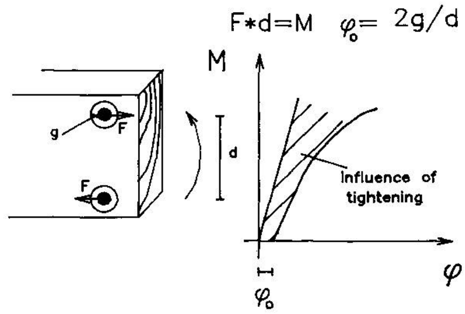
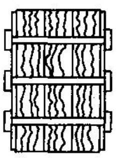
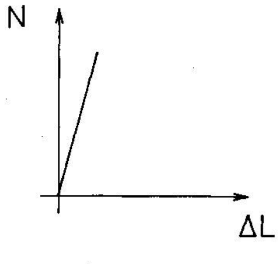
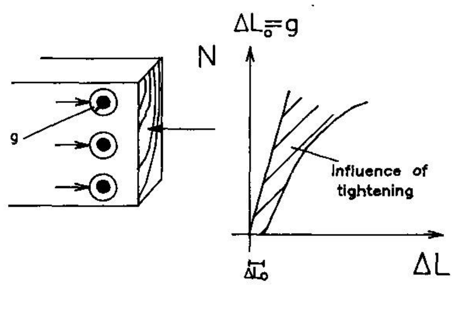
Desription of these procedures and their related tecniques and results are given in details in [1],[2],[3]. In parallel of these in situ methods of inspections, an analitical interpretation of the overal static of the structure has been performed assuming, by hipotesis, rigid joints, elastic behaviour and values of mechanical parameters known by the previous mentioned in situ test. As it was possible to guess, the elastic analysis in this way performed has been not able to catch the real deformations of the structure (fig.7). It has been confirmed that the actual deflection is due to non-elastic behaviour of the structure, and in particular of the joints.

To be able to perform a realistic numerical analysis by which obtain results in term of deformations quite similar to the actual, it has been necessary to know experimentally the real constitutive law of the joints. The non linear behaviour of the connections may be essentially due to the following points:

i)local damages of the wood by non statical causes (insects,fungi,decay)
ii)climate and time dependent deformations
iii)localized deformations of some bolts
iv)instant deformability of the wood in the head joint
v)non-efficient tightening of the bolts
vi)the procedures and successive phases of construction.

Probably all these causes occur, the combination of which determines the actual flexural and axial rigidity of the joints and is responsible of the weakness of the structure. In table I the influence on the constitutive law of the tightening action, toughether with the probably non calibrated size of the

TABLE I

Structural head joint	Force of tightening	Influence of deformed holes of the bolts
		
Section A–A 		
Section B–B 		

holes is summarized for a typical head joint. More discussion and analytical consideration about the influence of each cause on the local behaviour of the joint may be found in [4].

4.NUMERICAL ANALYSIS AND EXPERIMENTAL TEST

As already mentioned, a linear numerical 3-D analysis on the entire structure, has been performed,as a first stage. A mesh using beam elements for the principal truss-arches has been prepared for a non-elastic analysis (whit ABAQUS v.4.9). The prototype constructed and tested in the laboratory of the University has the same dimensions and characteristics of the real arch-element, as shown in the photo of fig 8.
The Figure shows also the loading device able to simulate the load conditions of the element in the real structure, known by the results of the 3-D elastic analysis previous mentioned. In figure 9 some particulars of the instrumentation are shown.

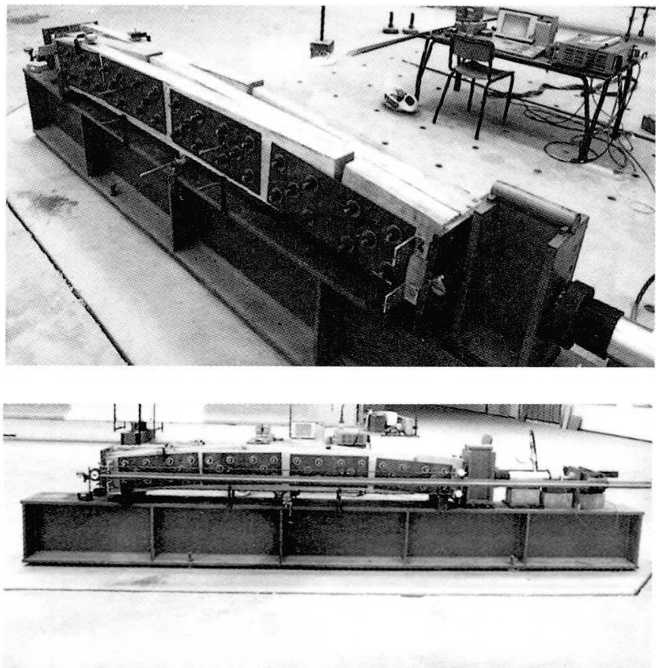


Fig.8 The model and the loading device of the test

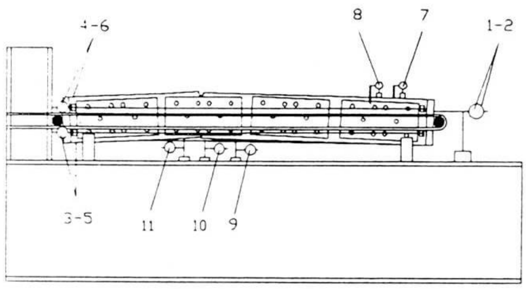


Fig.9 The model and the position of the instruments

The intensity of the actual tightening action of the bolts being unknown, it has been calibrated into four different steps, in each of them a different behaviour of the joints (and the entire element) at constant axial force has been observed and monitored during the test.
The maximum tightening force has been calculated such as the bolts on each head joint are able to transfer the axial force N by friction. According with Coulomb friction law and assuming the value of 0.35 for the friction angle, the maximum tightening force, in the case of an axial force N of about 7,000 Kg, must be equal to 700 Kg on each bolts (N/3 on the single lumber, see table I).
As mentioned the prototype as been tested giving four different intensity of tightening action, from the maximum to zero. The first test here reported concernes a prototype caracterized by holes of the same dimensions of the

bolts (g=0 in table I). The figure 10 shows the behaviour of the element loaded by an incresasing axial load centered at the two ands of the element (as shown in fig.8). Each curve represents the P-ΔL behaviour for a the definite tightening action. The diagram for the highest value of tighteting is linear elastic; in this case the maximum axial deformation of the element is of the same order of an equal monolytical wood element (ΔL_{exp} =0.3 mm, $\Delta L_{mon.}$ =0.24 mm). In the other cases of less tightening forces, it has possible to read from the shape of the diagramms an initial minor rigidity due, probably, to local deformation of the wood subject to the pressure of the bolts.

As mentioned we refer here just on a first series of results, as a part of a more comprehensive investigation. In any case on the basis of these experimental results it is possible to make a first quantitative hipotesis on the influence of the tightening action on the deformability of the basic element of the arch. As a constitutive non linear elastic law of the element, such results have been implemented in the numerical analysis of the principal arch. The figure 11 shows the deformed shape of the arch in the case of T_f=700 and T_f=0 (referred to the moment of the torquemeter T_f=25 Nm and T_f=0 respectively).

A second test is in progress with the aime of quantifying the influence of the tightening force on an element with non calibrated holes.

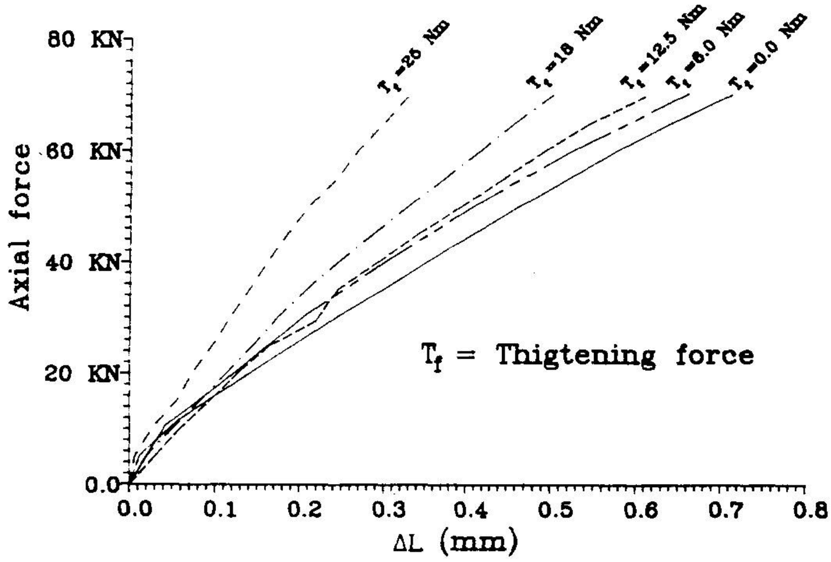


Fig.10 P-ΔL experimental behaviour for different tightening force

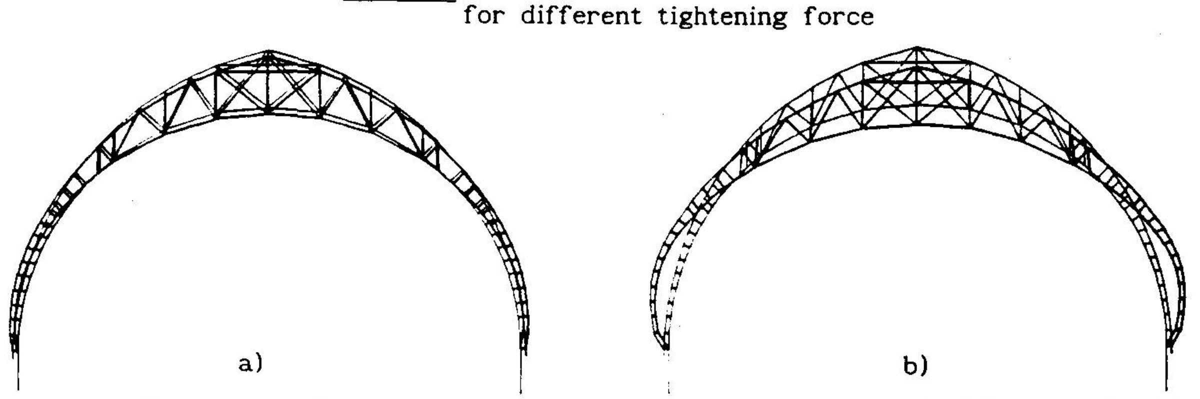


Fig.11 a)the deformed shape in the case of linear elastic analysis (T_f=700)
b)the arch deflection in the case of non elastic behaviour (T_f=0)

ACKNOWLEDGEMENTS

The author wants to espress her Acknowledgements to eng.M.Lucchini, chief of the Municipality of Brescia and to eng.M.Di Pasqua for his valuable contribution for the numerical simulation.

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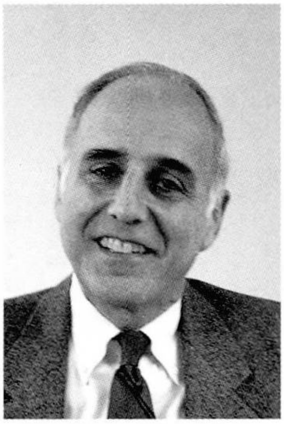
Non-Destructive Evaluation to Document Historic Structures

Essais non destructifs pour l'évaluation de bâtiments historiques

Zerstörungsfreie Untersuchungstechniken für historischer Bauten

Robert SILMAN
President
Robert Silman Assoc.
New York, NY, USA



R. Silman, born in 1935, has earned a BA degree at Cornell Univ. and BCE and MCE degrees at New York Univ. He is President of his consulting firm and an Adjunct Professor of Architecture at Columbia Univ.

Marie ENNIS
Associate
Robert Silman Assoc.
New York, NY, USA

M. Ennis, born in 1959, received her BCE from the Univ. of New Hampshire in 1981 and her MS Historic Preservation from Columbia Univ. in 1987. She is presently an Associate and Director of Preservation at her consulting firm.

SUMMARY

Five nondestructive evaluation techniques were investigated for use in documenting structural components of historic buildings. Traditionally, nondestructive evaluation has been used to search for structural defects. This research sought to extend the effectiveness of said techniques and to evaluate their usefulness in documenting framing systems. A brief description of each technique is presented coupled with an evaluation of the effectiveness.

RÉSUMÉ

Cinq méthodes d'essais non destructifs sont analysées en vue de leur application dans l'évaluation des éléments structuraux de bâtiments historiques. Ces méthodes ont été utilisées, dans le passé, dans la recherche de faiblesses structurales. La présente étude a pour objet d'étendre le champ d'application des méthodes d'essais non destructifs et de déterminer leur utilité dans l'évaluation des systèmes à cadres. Chacune des méthodes est brièvement décrite et leur efficacité est présentée.

ZUSAMMENFASSUNG

Fünf zerstörungsfreie Versuchsmethoden und ihre Anwendungsmöglichkeiten bei der Beurteilung von Tragwerkselementen in historischen Bauten werden analysiert. Diese Techniken wurden in der Vergangenheit für den Nachweis von Schwachstellen eingesetzt. Die gegenwärtige Studie zielt darauf hin, weitere Einsatzmöglichkeiten aufzuzeigen, vor allem hinsichtlich Rahmenwerke. Jede Technik wird kurz vorgestellt und ihre Zuverlässigkeit dokumentiert.

1. INTRODUCTION

The authors conducted a research program to evaluate nondestructive evaluation (NDE) techniques for purposes of determining size, location and type of structural components in historic buildings. Many investigators have employed NDE techniques with a good deal of success to locate deficiencies in structural materials in historic buildings. This paper describes extending the uses of NDE beyond diagnosing defects and into the realm of documenting existing framing systems.

Many, if not most, historic structures do not have original drawings of the framing and support systems. When engineers are asked to evaluate these buildings for purposes of a condition survey or for the purposes of knowing their structural capability to function as useful contemporary spaces, the absence of original drawings is a serious impediment. Traditionally the only method of determining the sizes and positions of framing member was to make destructive probes into the original fabric, to measure and record the results of these observations and to recreate framing plans and details. This method implies several serious shortcomings, the two most important being the destruction of original fabric and the need to repair it at probe locations and the limitation that the knowledge gained is only valid at the specific site of the probe and any attempt at extending the assumption to other parts of the structure runs the risk of being inaccurate.

This paper will present the results of research conducted by the authors' firm in partnership with the United States Army Corps of Engineers' Construction Productivity Advancement Research program. Five techniques used for examining concealed features of historic buildings were:

1. Radar
2. Impact Echo, Pulse Velocity and Spectral Analysis of Surface Waves.
3. Electromagnetic Detection
4. Infrared Thermography
5. Fiber optics.

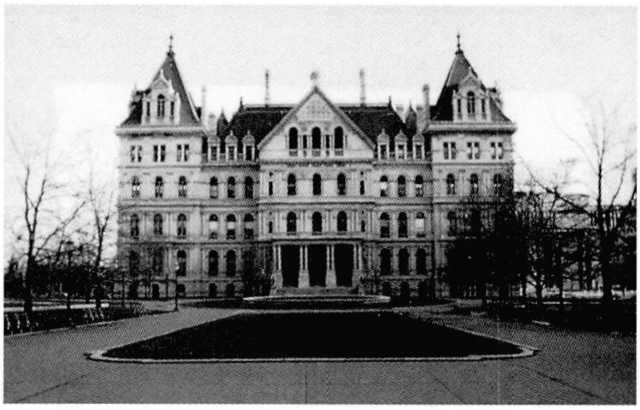


Fig. 1 New York State Capitol Building

The research was conducted in parallel with a structural survey being conducted by the authors' firm on the New York State Capitol building in Albany, New York (Figure 1). This large brick and stone masonry bearing wall structure was constructed during the period 1867 to 1899. It contains approximately 50,000 square meters of floor space on five floors plus attic and basement. Exterior building dimensions are approximately 95 m. by 115 m. The structural survey's purpose was to recreate the framing plans for the structure (the original 1911) and to evaluate the live load capacity of all levels of the structure including the foundation. For purposes of the original survey, destructive probes were made at many locations throughout this building to reveal thicknesses of masonry walls, brick arch and concrete slab floors; the location of wrought iron and steel beams and columns, pipes and ducts and flues; footing depth profiles. In conjunction with these destructive probes, NDE techniques were applied side-by-side so that the results could be verified exactly and determine the success or failure of the various methods.

2. RADAR

The surfaces of floors and walls were scanned with an impulse radar device known as subsurface interface radar. Initial calibration was obtained for the relative permittivity of stone (granite), concrete and brick masonry by using samples recovered from the building. The equipment consisted of a 900 MHz transducer which emitted short pulses of high frequency, low power electromagnetic energy into the subsurface and a receiver within the antenna. The emitted pulses were in the band from 50 MHz to 1.5 GHz and were 1.1 nanoseconds in duration. As the transducer was moved along the surface, reflected pulses were recorded as digital information on magnetic tape; this was later processed in a computer in the office.

Fig. 2 Radar Equipment in Use

The imaging technique used is known as backward propagation. The radar data is a function of the transducer position and the delay time experienced between the emission and reception of the signal. From this information a holographic record along the path of the transducer is produced. Ultimately images are reconstructed using an algorithm by superimposing these coherent backward propagations [1].

The waves reflect off of changes in materials, voids or discontinuities, the rear boundary of a material or a buried object. Thus radar is capable of determining many features beneath the surface of a historic wall or floor or ceiling. One of the most successful uses of the radar was in profiling floors where brick arches span approximately 900 mm between wrought iron rolled I beams. In other areas H shaped columns were located in walls and the orientation of the flanges was established. Where masonry walls were thoroughly bonded, their thicknesses were readily measured. The presence of steel reinforcement in concrete floors and metal ties in masonry was also recorded. Flues and pipes could also be detected in walls up to 800 mm thick. New techniques are being rapidly developed to enhance the processed images so that they bear a resemblance to the actual in situ features.

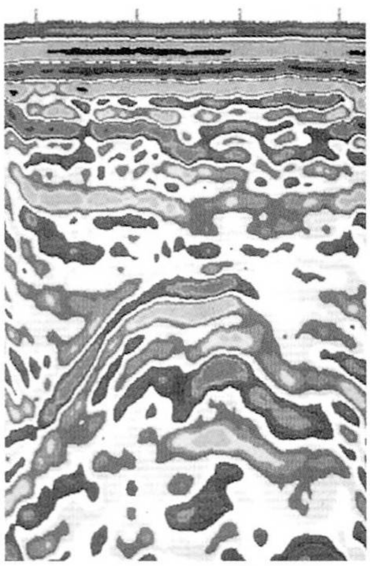


Fig. 3 Radar Output, Raw Data

Radar was tried as a means of profiling large 4.0 meter wide stepped, pyramidal stone footings. However due to the presence of a high water table the readings were inconclusive. In other projects, subsurface applications of radar have been useful in mapping footing profiles as well as locating buried pipes. Other features which causedthe radar method to fail were large quantities of metal near the surface, conductive surfaces such as slate and false floors or walls with air spaces.

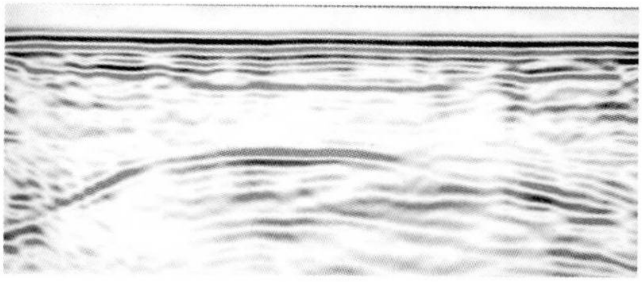


Fig. 4 Radar Output, Computer Enhanced Image of Data From Fig. 3

The radar equipment is expensive to purchase, requires a highly skilled operator and must be compiled with a sophisticated computer software program. Of course this is all quite expensive. In addition, there are several potential technical pitfalls such as accurate calibration of the pulse propagation velocity in a given material and errors due to the manual towing of the transducer across the surface.

3. IMPACT ECHO, ULTRASONIC PULSE VELOCITY & SPECTRAL ANALYSIS OF SURFACE WAVES

3.1 Impact Echo

This equipment consists of a hammer and receiver which are both wired to a computer which processes the data. The surface to be read is struck, the computer records the input energy, and the receiver picks up the reflected compression wave energy. In general, the more dense the material, the higher the wave velocity response [2]. This technique is the "high tech" version of sounding a material.

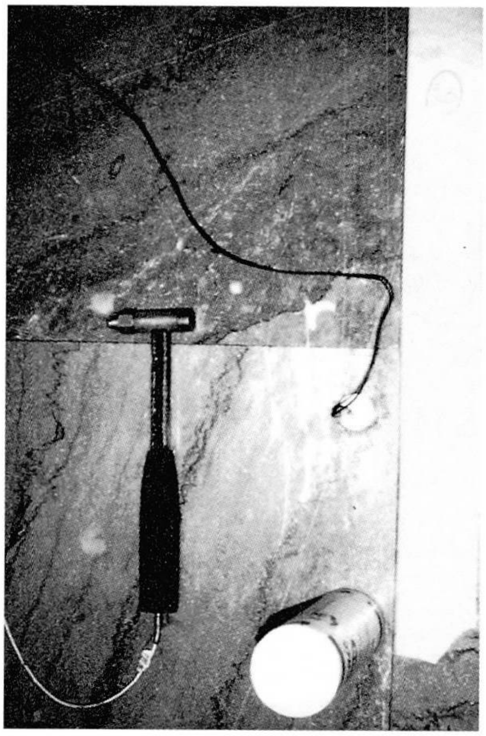


Fig. 5 Impact Echo Equipment

The equipment produced good data for reading thicknesses and integrity on granite and sandstone columns, veneer walls, and brick walls less than 600 mm thick. A steel column behind granite facing was located using this technique but the orientation of the flanges was not discernible. Brick walls over 600 mm thick could not be fully measured. Multiple layers (terrazzo floor over setting bed over brick) did not produce good results; brick arches and beams were not detected as they were using radar. The equipment is best used to detect cracking parallel to the surface (in this case, stone). Optimum results were obtained on homogeneous materials (solid stone) where thicknesses and hidden defects were easily read.

The data is, like radar, not easy to read. The output is plotted in linear frequency versus distance displaced per unit of force. A specially trained technician or engineer must interpret the results. The cost of the measuring devices is moderate; this does not include the computer equipment or the rather expensive software required.

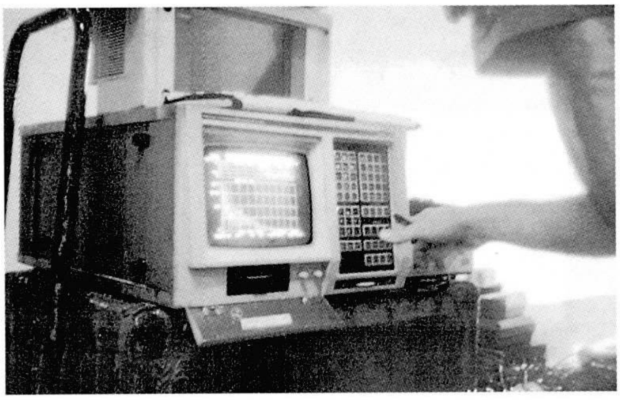


Fig. 6 Impact Echo Data on Field Computer

3.2 Ultrasonic Pulse Velocity

This technique is widely used to assess concrete quality (ASTM C597-83). The process uses a transmitter and receiver to pass ultrasonic energy through a test member; therefore, access is required from both sides of the object. The faster the measured velocity, the denser or stronger the material [3]. Characteristic velocities were obtained for various materials: brick, granite and sandstone. This was used as baseline data for sound materials.

Fig. 7 Pulse Velocity Equipment in Use

In composite walls, the velocity and signal strength dropped across mortar joints. Low frequency pulse velocity signals can provide good results for thick brick walls where multiple mortar joints would otherwise block the transmission of higher frequency signals. In solid materials, such as granite columns, the results were excellent for determining thicknesses and soundness of the material. In general the results correlated well with the impact echo results, giving better results for brick walls over 600 mm than impact echo. The two techniques should be used in conjunction; ultrasonic pulse velocity provides accurate baseline velocity measurement for the test material and impact echo assesses thicknesses (only need access from one side) and hidden damage. Since low frequency energy penetrates concrete and brick better than high frequency energy, signals were received using this method. The disadvantage is that the long wavelengths result in an

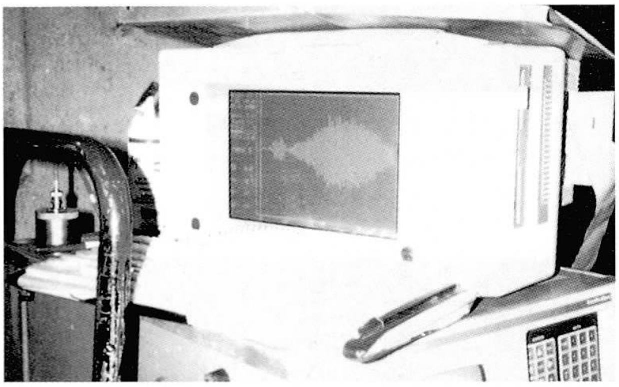


Fig. 8 Pulse Velocity Data on Field Computer

"averaging" effect which decreases the resolution of the method and does not allow the location of smaller scale damage such as individual brick unit de-bonding.

As with radar and impact echo, the data is not easily readable and requires a specially trained technician or engineer to interpret. The output is given in time (microseconds) versus signal amplitude (millivolts). The equipment costs are quite high.

3.3 Spectral Analysis of Surface Waves (SASW)

This is a seismic method developed at the University of Texas in Austin to read shear wave velocities and modulus profiles in layered systems (such as pavements and earth). In this technique the surface of the test material is struck on line with two surface mounted receivers to measure surface wave velocity as a function of wavelength. With increased surface shear wave velocity, the material modulus is higher, therefore, the quality of the material is better [4].

Access to only side of the material is required to perform measurements, as was the case for impact echo. This is important as a practical concern in the field where access is often limited. In its usual application for pavements, the SASW method can determine thickness of slabs on grade quite accurately. However where there is an air void behind the material or where a corner of structure occurs, a great deal of interference is experienced by the sensing equipment and accuracy is seriously impaired. For thick stone and brick walls the method was satisfactory in measuring thickness but for thinner walls and framed slabs, the SASW technique was not suitable. For both impact echo and SASW, the presence of energy absorbing materials (e.g. plaster) is an impediment.

As with the other techniques, the data is not easy to read. The output is given in frequency (KHz) versus phase angle (degrees). The equipment costs are moderately high.

4. ELECTROMAGNETIC DETECTION

Most available instruments are cover meters for locating rebar. Two instruments were tested: Profometer and the Fisher M-100 Meter.

The basic principle of operation is that an alternating current passing through a coil generates a magnetic field. When a magnetic material is encountered the field is disturbed. The magnitude of this disturbance is related to the size of the metal object and proximity to the probe. The probes are generally directional, that is there is a sharp maximum reading when the long axis of the probe and the long axis of the object are aligned.

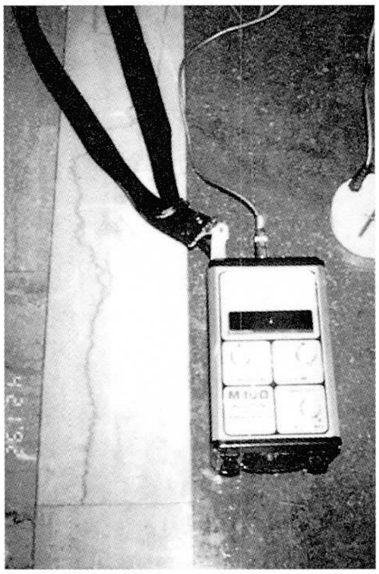


Fig. 9 Magnetic Detection Equip.

Both instruments worked very well at locating iron beams and girders supporting brick arch construction with about 9 inches of cover where no wire mesh was present. A steel column behind 8 inches of granite cover was easily located. Iron anchors in stone walls were also easily located. At the flat concrete slabs with draped wire mesh the results were variable. The instruments must be "fine tuned" to avoid reading the mesh only.

Fig. 10 Magnetic Detection & SASW Equipment

Costs are quite low for the equipment. The less expensive Fisher instrument worked just as well as the higher cost Profometer instrument for these purposes. Electromagnetic detection is recommended for use with all other methods to ascertain whether hidden objects are metallic. Specially trained operators are not required.

5. INFRARED THERMOGRAPHY

Infrared thermography is typically used to perform energy efficiency surveys of buildings and to assess roofing membranes for leaks. This technique could be referred to as heat imagery. The equipment consists of camera/video equipment with a special cooler containing semi-conductor crystals in liquid nitrogen. A photograph is taken of the subject and the resulting colors are indicative of surface temperature variations. The colors typically translate as black/violet being cool, and red/white being warm.

Fig 11. Infrared Equipment in Use

The basic principle is than an object having a temperature above absolute zero will radiate electromagnetic waves. Wavelengths fall within certain bands depending on temperature. At room temperature, typical wavelengths are 4 to 40 micrometers which is outside the visible spectrum. At very high temperatures the wavelengths are less than 1 micrometer and fall within the visible spectrum. Hence when metal is heated and begins to glow red, the electromagnetic waves emanating from the object fall within the visible range.

Water content reduces the transmission of infrared radiation, therefore thermoscans taken on a humid day may adversely affect the results. The ambient temperature and time of day are also important aspects of infrared thermography. The readings are only of surface temperature; the equipment is not capable of deeper penetration such as electromagnetic detection, radar, etc.

Hidden structure could not be detected through the roofs or walls. The technique did prove to be very useful for locating hidden pipes and flues which were at different temperatures within the thick walls. It is also useful for documenting surface deterioration of masonry which were at different temperatures. This is primarily related to the different moisture content in porous, deteriorated stone and brick.

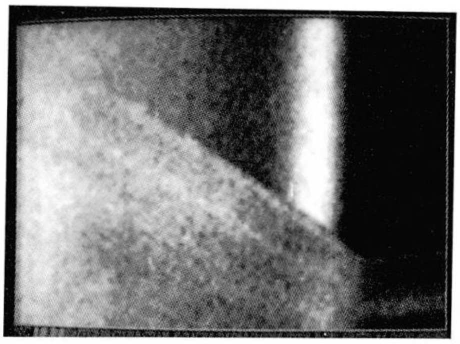


Fig. 12 Infrared Thermoscan Data

6. FIBER OPTICS

This technique was originally developed by physicians for internal examinations. An instrument called a fiberscope is constructed of a bundle of flexible optical fibers (or a borescope of rigid optical fibers), some of which carry high intensity light along their length to the end of the probe. Others of the fibers are used to view the object by means of focusing lenses. The viewing end or head can be rotated to give variable viewing angles.

Although these instruments have applications in evaluating historic structures, the type of structure is critical. It is mandatory that there be void spaces into which the fiber optic tube can be inserted for visual observation. Solid masonry structures are generally not suitable candidates for fiber optics because they have few voids. Good success has been experienced in timber structures where a small hole can be made in a plaster or wood finished surface, the flexible fiber optic tool inserted and a great deal of information gathered. One critical feature of the equipment is the focal length; for structural evaluation a long focal length is important.

Cost is fairly moderate, depending upon the number of accessories purchased.

7. CONCLUSION

Of the five NDE methods employed on this heavy masonry building , radar proved to be the most generally successful. Next in order of success were impact echo/pulse velocity and infrared thermography. Electromagnetic detection was very useful but its scope is limited to buildings which contain iron or steel and to locations where framing members can be isolated from pipes, conduits and other iron features. Fiber optics was found to be of minimal use in this type of building because void spaces were not present. A problem to be considered in the utilization of the three most successful methods is the need to have highly trained equipment operators present as well as sophisticated computer programs which can translate the raw data into meaningful results; these imply significant costs which must be borne by the NDE users. However there is great promise for NDE as a tool for exploring historic structures. The rapid changes and improvements in existing technology as well as the expectation for new techniques to be introduced are causes for future investigators to be extremely optimistic about the ability of NDE to delineate features of historic structures.

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Investigation of Historic Masonry by Means of Radar

Investigation de constructions anciennes en maçonnerie au moyen du radar

Erkundung historischen Mauerwerks mit Radar

Martin KAHLE
Civil Eng.
Univ. of Karlsruhe
Karlsruhe, Germany

M. Kahle, born 1960, received his civil engineering degree from the Univ. of Karlsruhe, where he became research assistant at the Inst. of Structural Engineering. He is working on the investigation of historic masonry with non-destructive methods.

SUMMARY

A radar method has been tested, improved and then employed on a number of buildings for the non-destructive investigation of the texture and condition of historic masonry. It has been used for the detection of vertical leaf boundaries, or leaf separations, voids and discontinuities as well as for the assessment of moisture contents.

RÉSUMÉ

Le procédé du radar a été testé afin d'examiner la structure et l'état de constructions anciennes en maçonnerie de manière non-destructive. Il a été développé et utilisé sur de nombreux bâtiments. Le radar permet de repérer les épaisseurs et les séparations des parements, les vides, et de constater l'hétérogénéité des matériaux, ainsi que le degré d'humidité des murs.

ZUSAMMENFASSUNG

Zur zerstörungsfreien Untersuchung von Struktur und Zustand historischen Mauerwerks, wurde das Radarverfahren getestet, weiterentwickelt und an zahlreichen Bauten eingesetzt. Mit ihm lassen sich Schalengrenzen, Schalenablösungen, Hohlräume und Materialeinlagerungen orten, sowie der Feuchtegehalt bestimmen.

1. INTRODUCTION

Unsatisfactory inspection and diagnosis of historical buildings often leads to inappropriate restoration measures, which cause excessive damage, a delay of time and additional costs.

By thorough investigations of the structure these mistakes can be avoided. However, in many cases it will not be sufficient to inspect surfaces, but to obtain data from the inside of a structural member. For this purpose, non-destructive methods are required, since data acquisition should be made on a very dense grid without causing damage.

2. THE RADAR METHOD

The main components of the equipment used are the control unit, a microcomputer containing specific software, a transmitter and a receiver. In the reflection arrangement both antennas are connected in one box which is moved along vertical or horizontal lines with scans every 2 cm approximately, fig. 1, 2, 3.

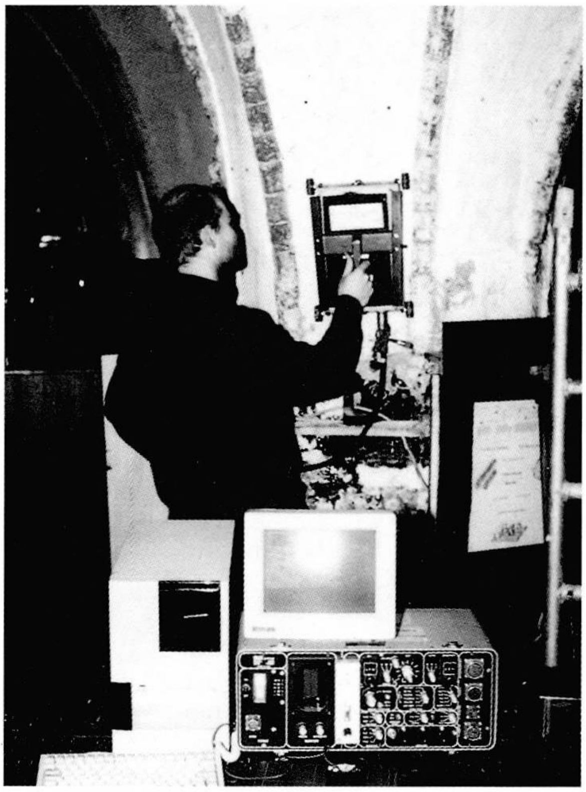

Fig. 1: The radar equipment

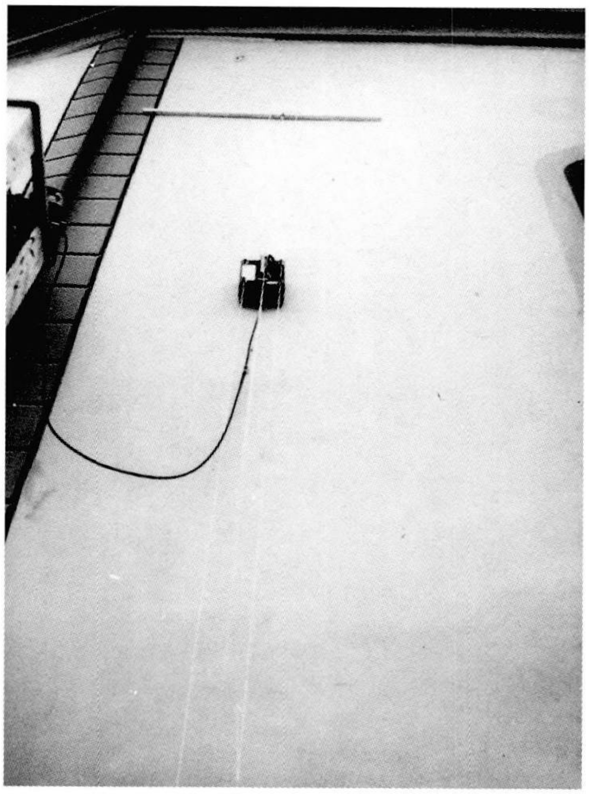

Fig. 2: The 500 MHz-antennas in the reflection arrangement

The transmitter directs pulses of low power electromagnetic waves of one and a half wavelength duration at frequencies of 100 to 1000 MHz into the structure. The waves propagate through the material with the specific velocity of

$$v = \frac{c}{\sqrt{\varepsilon_r}} \qquad \text{with: } c = 3\cdot 10^8 \text{ m/s} = \text{speed of light}, \quad \varepsilon_r = \text{relative permittivity} \qquad (1)$$

At an interface separating materials of different electrical properties, a portion of the energy is reflected back to the surface, where it is picked up by the receiver. For vertical incidence the reflected field strength is given by the reflection coefficient

$$r = \frac{\sqrt{\varepsilon_{r,1}} - \sqrt{\varepsilon_{r,2}}}{\sqrt{\varepsilon_{r,1}} + \sqrt{\varepsilon_{r,2}}} \quad \text{with: } \varepsilon_{r,1}, \varepsilon_{r,2} = \text{relative permittivity of material 1 and 2 respectively} \qquad (2)$$

The received signals are amplified depending on the delay time, transmitted to the microcomputer and finally plotted in radargrams, fig. 3.

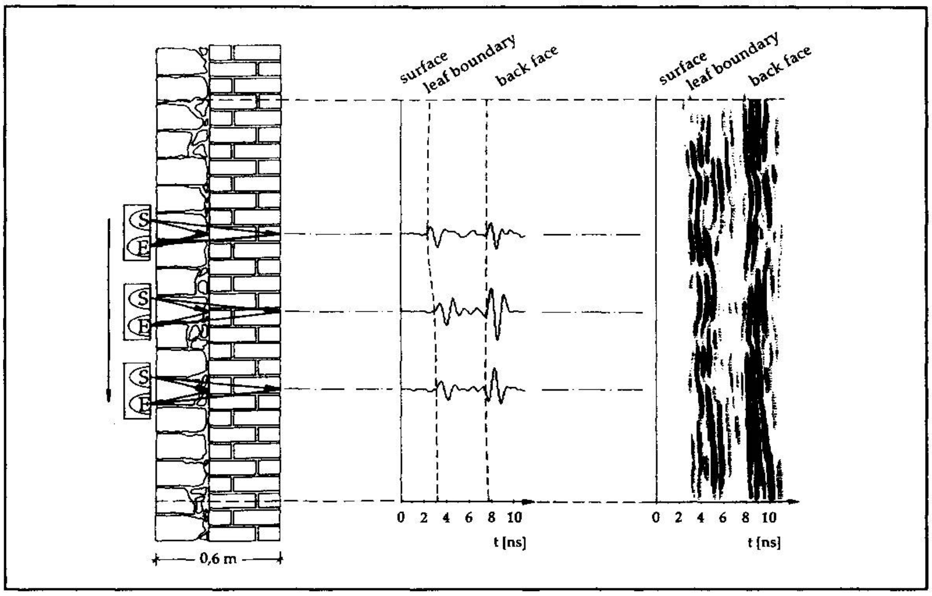

Fig. 3: The reflection arrangement on a multiple leaf wall; schematic cross section, received signals and radargram (S..transmitter, E..receiver, ↓.. direction of profiling)

The radargram provides a graphic profile presenting the amplitudes of the electromagnetic waves as a function of the delay time between the moment of entering and leaving the structural part. In order to condense the data display it is convenient to use dots, whereby the colour code or the colour density (in the case of grey as in the figures shown) is associated with the magnitude of the amplitude, fig. 3.

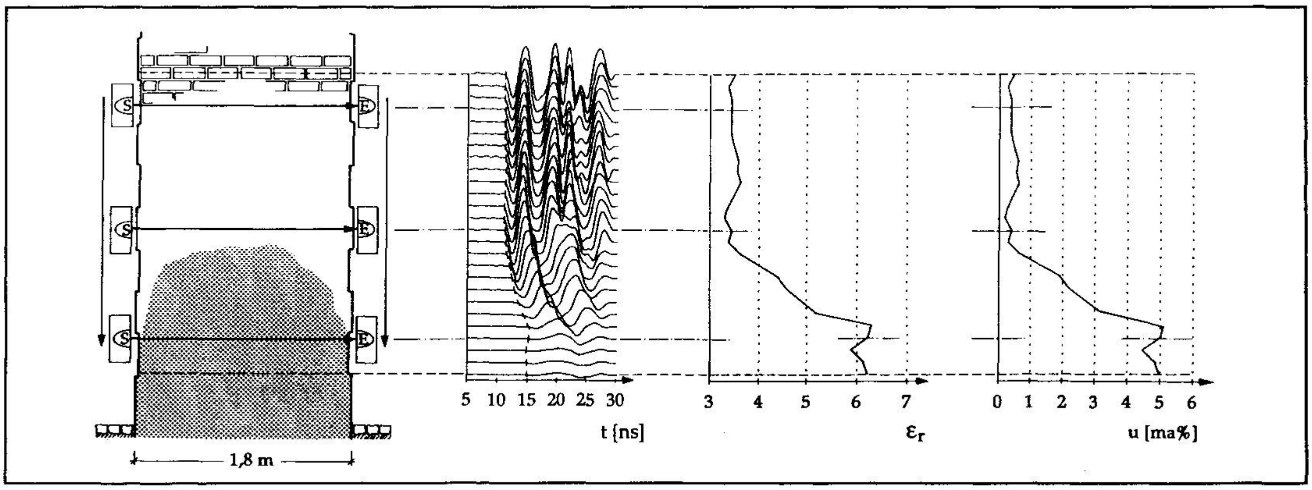

Fig. 4: The transmission arrangement on a column with rising moisture; schematic cross section, received signals, relative permittivity and moisture profile (S..transmitter, E..receiver, ↓..direction of profiling)

In the transmission arrangement, which is primarily employed for the assessment of moisture, transmitter and receiver are placed on opposing faces, fig. 4. Because of the difficult handling of the antennas, discrete scanning points at 5 or 10 cm spacing are chosen. The propagation time of the electromagnetic wave through the structural part is given by the location of the first deviation of the receiver signal, from which the velocity is calculated. Using equation 1 the relative permittivity is determined and from this value, the moisture content is derived (see §3 and §4.3).

3. ELECTRICAL MATERIAL PROPERTIES

The electric resistivity and the relative permittivity of the materials determine the velocity of the electromagnetic waves, the reflectivity at interfaces and the depth of penetration. Hence was examined the dependence of the electrical properties of materials commonly used in masonry from parameters such as: material matrix (chemical composition, texture and porosity), water content, concentration of dissolved salts, frequency of the waves, temperature and mechanical stress level. Since the literature provided unsufficient information, special laboratory tests had to be carried out.

The resistivity was determined on samples of different materials with the geoelectric direct current method. The resistivities for brick, given as a function of the moisture and salt content, are shown in fig. 5.

The relative permittivities were measured using the radar technique in the transmission mode. The dependence of the permittivity on the moisture content, as depicted in fig. 6, may be approximated by a linear relation. Dissolved salts cause a slight increase up to 10 % of the relative permittivity. The influence of temperature and pressure was found to be insignificant for the application of radar on historic masonry.

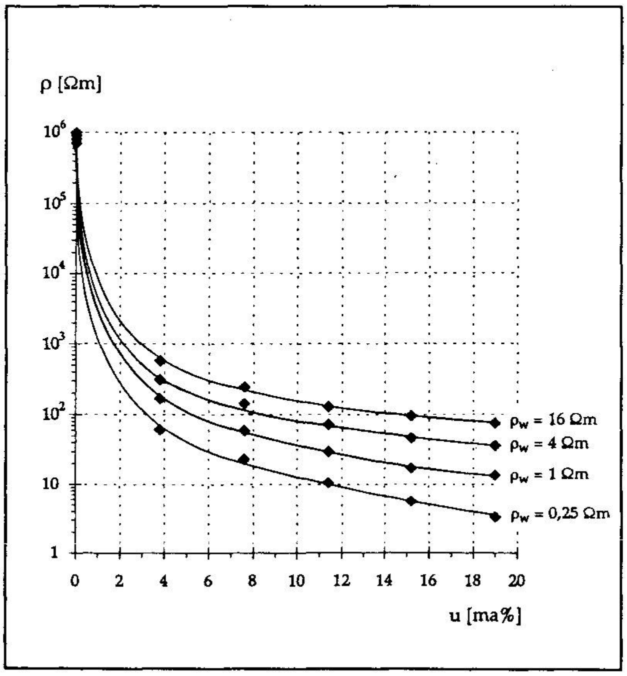

Fig. 5: Dependence of the electric resistivity ρ of brick on the gravimetric moisture content u; parameter ρ_w is the resistivity of the porous aqueous solution

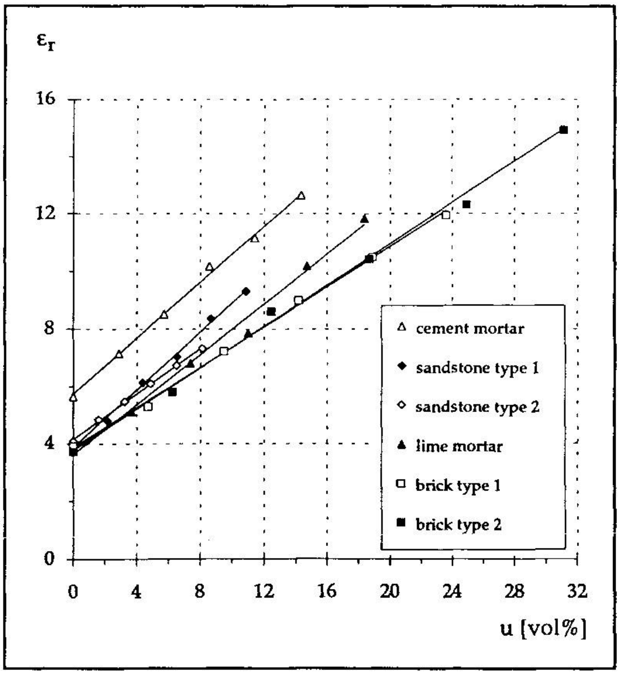

Fig. 6: Dependence of the relative permittivity ε_r of different materials on the volumetric moisture content u

4. BUILDING INVESTIGATIONS

4.1 Leaf Boundaries

The radar method is capable of detecting boundaries of multiple leaf masonry in the way depicted in fig. 3. The result of the investigation and the data quality is dependant on the nature of the boundary surface and the difference in the electrical material properties. Smooth boundaries yield clear and uniform reflection bands in the radargram. In this respect the example in fig. 3 is an ideal case.

Since the leafs are usually bonded and often consist of similar materials, the radargrams picked up from several multiple leaf stone walls looked like the one on fig. 7. The data is less distinct, but the leaf boundaries are imaged in short reflection bands and moreover, the leafs show different reflection patterns.

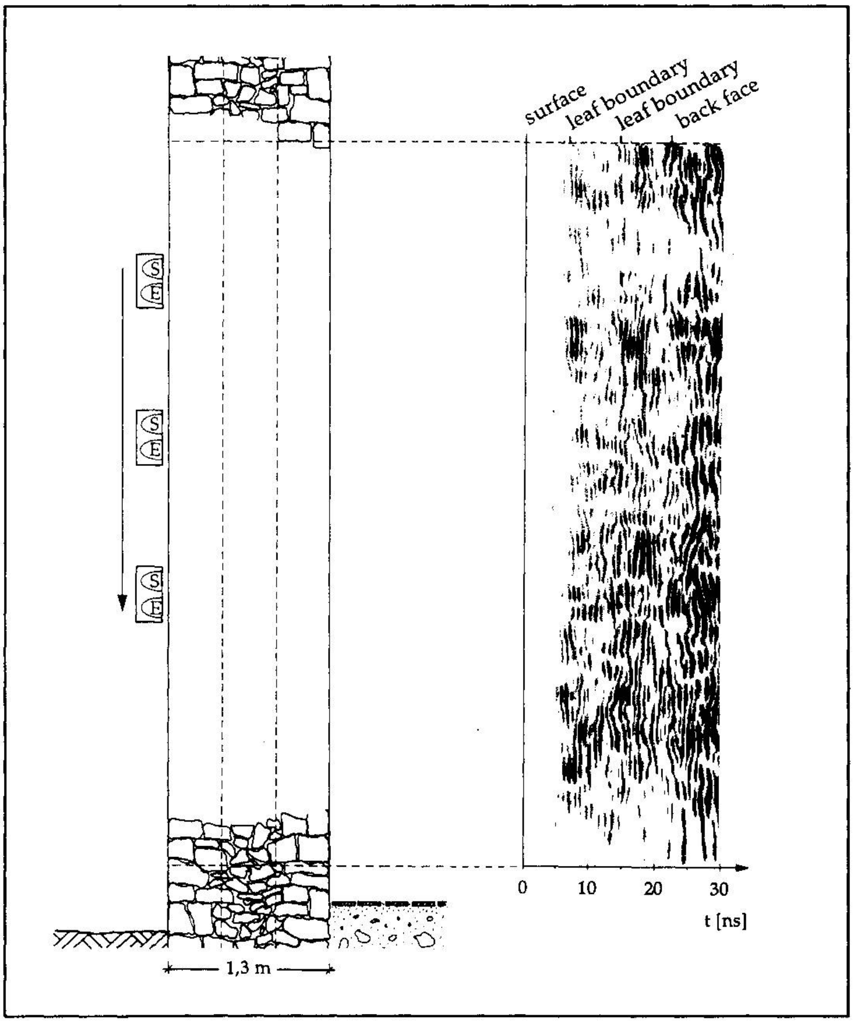

Fig. 7: Detection of leaf boundaries; wall cross section and radargram

Some investigations were carried out on walls with leafs which have separated, fig. 8. In these cases, because of the high reflection coefficient, the reflection from the interface stone/air was very strong with amplitudes in the order of 10 times as much as from comparable intact wall elements. If the gap was filled with debris the intensity was lower but still significant.

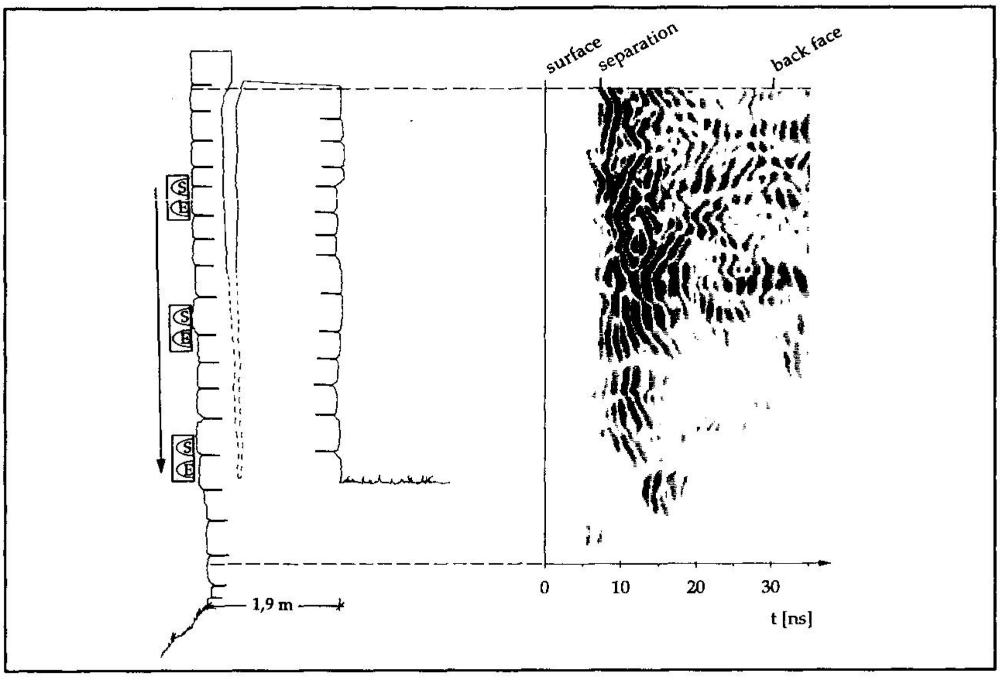

Fig. 8: Detection of leaf separations; wall cross section and radargram

4.2 Anomalies

Because of its high resolution the radar method is capable of detecting discontinuities due to voids or enclosed materials. The reflections from the boundary of the anomaly are imaged as diffractions in the radargrams, fig. 9. A charac-

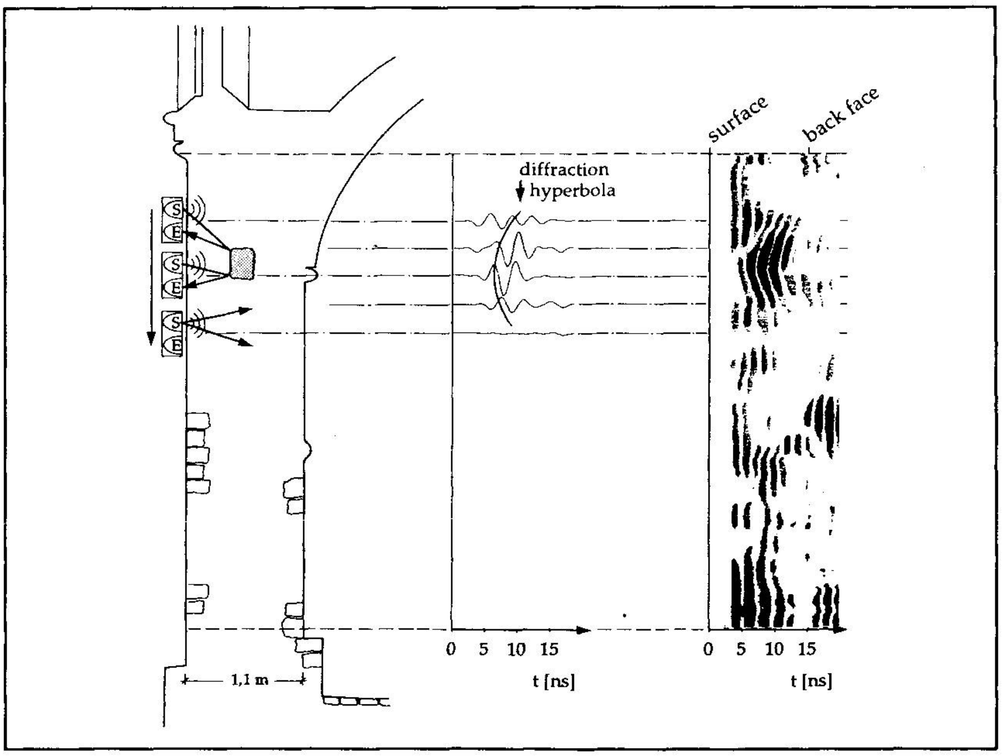

Fig. 9: The detection of anomalies with the reflection arrangement; wall cross section of the Matthiaskapelle in Kobern/Mosel, receiver signal and radargram

teristical hyperbolic shape is created because the anomaly is captured over a certain profile length while the distance between the anomaly and the antennas changes. At the Matthiaskapelle, shown in fig. 9, a long hole for a wooden tie beam was found by this means.

Several investigations on masonry containing voids or enclosed materials occupying 10 % of the wall width or less, served to show that the resolution is usually sufficient to detect all significant characteristics of historic masonry.

Radar measurements on the Jägertor in Potsdam were carried out in order to define its bearing structure which was not known and especially to reveal possible hidden metal ties or clamps. The radargrams, picked up from underneath the beam, showed distinct diffractions from singular metal bodies plus reflection bands from several joints. Thus the data was interpreted in the way presented in fig. 10.

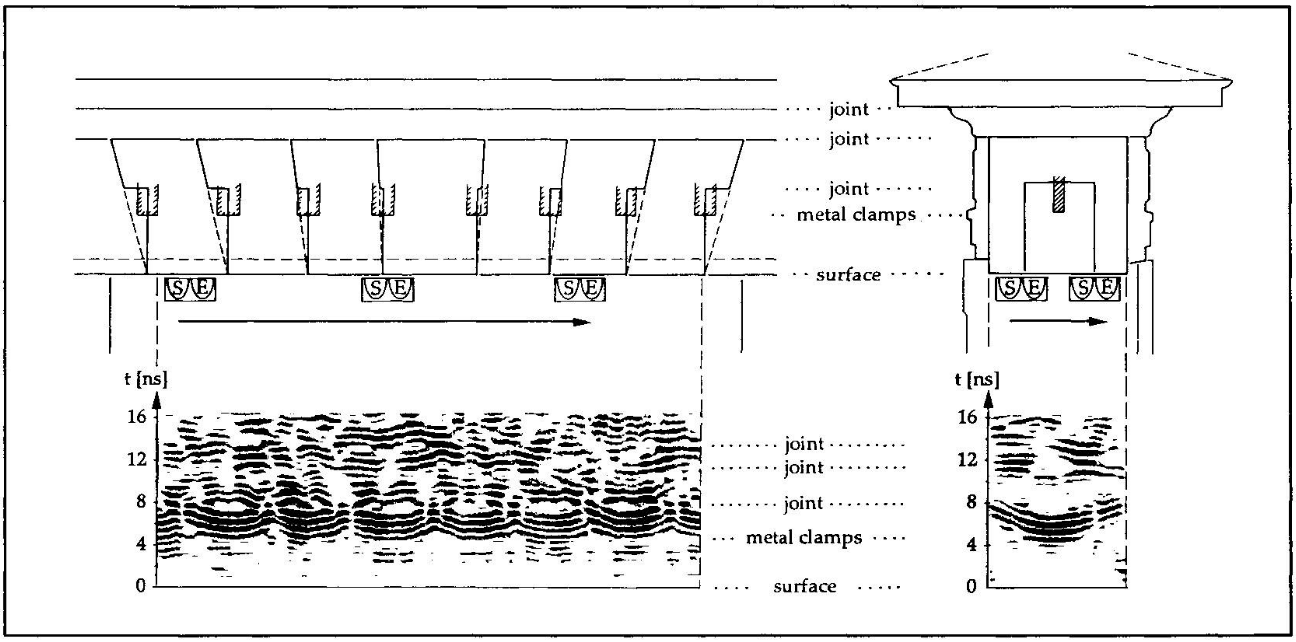

Fig. 10: Longitudinal and transversal cross section of the beam of the Jägertor in Potsdam and corresponding radargrams

4.3 Moisture

The possibility of measuring the moisture content of masonry with radar is derived from the dependence of the relative permittivity on the amount of water in the material matrix, fig. 6. The transmission arrangement usually yields the best data, but the reflection arrangement can be used on the condition that the reflection from the back surface is caught precisely and the width of the structural part is known.

The relative permittivities are calculated from the radar data using equation 1. From this values the moisture contents are determined by means of linear relations of calibration, which are either transferred from experience (S3) if applicable, or are derived from gravimetric moisture control on samples from the investigated member. In any case, a mean moisture content is measured from an averaged cross section and a lateral area according to the antenna size.

A comparison of the results from radar and the gravimetric moisture determination on drill debris from about 120 samples was made on a brick wall of a church. Fig. 11 shows 3 cross sections with the values from both methods, the profiles from radar being much smoother because of the larger measuring volume.

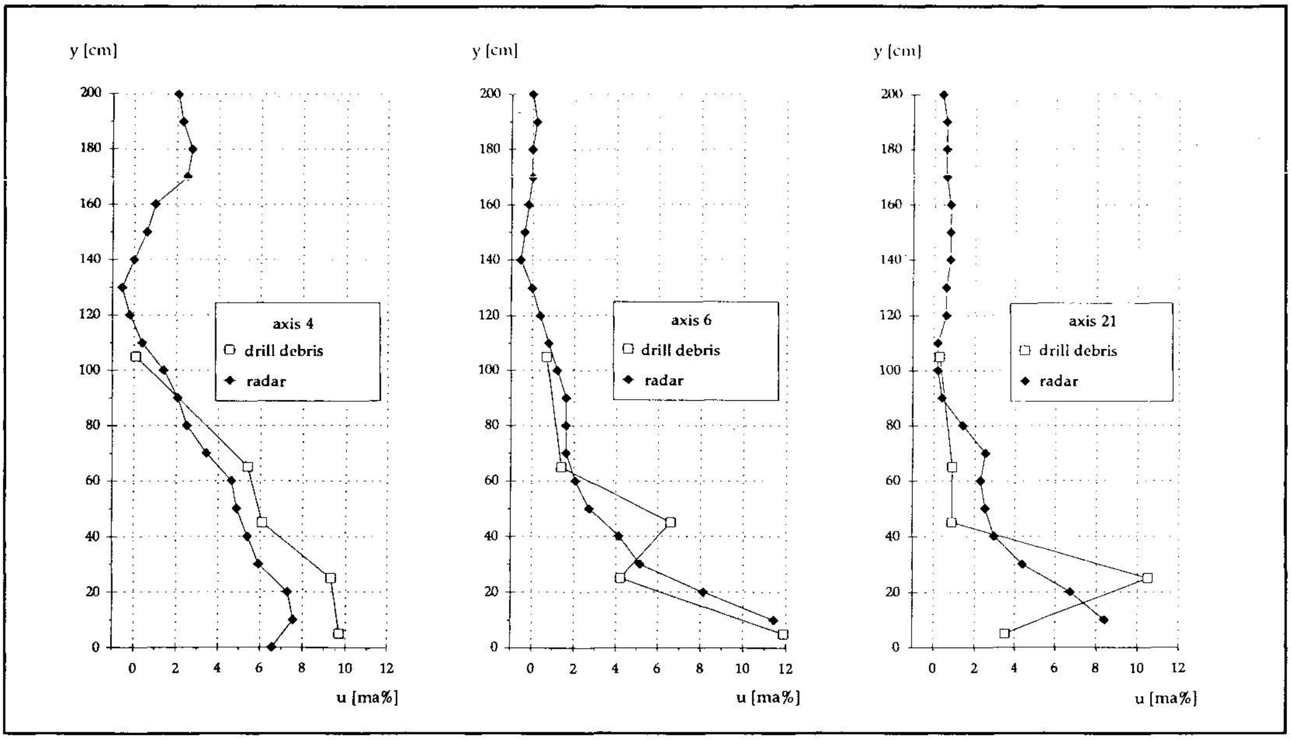

Fig. 11: Moisture content u of a brick wall at 3 cross sections, determined with radar or the gravimetric method on drill debris

5. CONCLUSIONS

Within the scope of the research program, concluded at the end of 1992, radar proved to be a versatile and efficient method for the investigation of texture and condition of historic masonry. It is expected and indeed hoped that the method will come to be used as a credible commercial tool, since this would be of benefit to the historic buildings and assist the engineers involved. The savings in the design and execution of restoration measures may easily surpass the costs of a radar investigation.

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Monitoring of the Brunelleschi Dome

Surveillance de la coupole de Brunelleschi

Überwachung des Brunelleschi-Doms

Gianni BARTOLI
Dr. Eng.
Univ. of Florence
Florence, Italy

Carlo BLASI
Professor
Univ of Florence
Florence, Italy

Vittorio GUSELLA
Professor
Univ. of Perugia
Perugia, Italy

G. Bartoli, born 1962, got his Civil Engineering degree at the Univ. of Florence. At the present time, Post-Doc in Structural Engineering at the Civil Engineering Department of the University of Florence.

C. Blasi, born 1948, got his Architecture degree at the Univ. of Florence. As researcher of Structural Theory and Technology, he has been teaching Rehabilitation and Reliability of Buildings since 1991, at the Engineering and Architecture Faculty of the Univ. of Florence.

V. Gusella, born 1957, got his Civil Engineering degree at the Univ. of Florence. Doctor in Structural Engineering. At the present time, at the Istituto die Energetica of the University of Perugia.

SUMMARY

Owing to cracks instruments have been placed for centuries on the dome of the Santa Maria del Fiore Cathedral in Florence. Recently, a digitally-controlled monitoring system has been installed to observe and control the evolution of cracks. The first four years of acquired data allow both an understanding of the mechanical and structural properties of the behaviour of the dome and a suggestion for some considerations about the use of monitoring systems in the field of the structural preservation of ancient monuments.

RÉSUMÉ

Suite à la présence de graves fissures dans la coupole, plusieurs méthodes de contrôle ont été utilisées pendant des siècles à la Cathédrale Santa Maria del Fiore, Florence. Un système de contrôle digital a été installé récemment, afin d'observer et contrôler l'évolution des fissures. Les mesures faites en quatre ans permettent de comprendre quelques-unes des propriétés mécaniques structurales du comportement de la coupole et de faire des réflexions sur l'emploi de systèmes de surveillance dans le domaine de conservation de monuments anciens.

ZUSAMMENFASSUNG

Aufgrund der Rissbildung an der Domkuppel von St. Maria del Fiore in Florenz, wurden während Jahrhunderten Kontrolleinrichtungen installiert. Jüngst wurde ein digitalgesteuertes Kontrollsystem aufgebaut, um die Rissentwicklung kontinuierlich zu überwachen. Zwei Aspekte ergeben sich aus den Daten der ersten vier Jahre: Erstens klare Hinweise auf das effektive mechanische und statische Tragverhalten der Kuppel; zweitens wichtige Informationen und Erfahrungen über den Einsatz von Kontrollsystemen im Rahmen der Tragwerkserhaltung bei alten Baudenkmälern.

1. THE CRACKS SYSTEM OF THE BRUNELLESCHI'S DOME

Brunelleschi's Dome is affected by a number of cracks that, appeared as soon as the monument was completed, have been evolving during time. Numbering the eight webs of the Dome starting from the one corresponding to the nave of the Cathedral and moving counter-clockwise, the main cracks (passing through both the two masonry domes constituting the monument) are located on even webs; they started from the skylight as far as the tambour. Moreover, other cracks, even though smaller, can be observed in the odd webs and in the corners between the webs.
These cracks have always been matter of apprehension during the years, so that from the beginning of this century, many control systems have been placed. At the present time, two of them are still working: the one installed by the Opera del Duomo (O.D.) in 1955 and the one placed by the Soprintendenza ai Beni Ambientali e Architettonici (S.B.A.A.) in 1987. These systems have revealed able to point out the long-term structural behaviour of the Dome, but not to reliably describe the short-term one and the link between environmental loads and the variations in width of the cracks.

2. DIGITAL MONITORING SYSTEM AND RECORDED SIGNALS

In order to better understand the behaviour of the monument, a new monitoring system have been installed in 1987 [1]. This system allows the recording of 165 different signals (by mean of instruments placed on the Dome) four times a day. The instruments are deformeters (placed across the main cracks), thermometers (measuring both the air and the masonry temperature), levelling instruments (placed at the tambour level), telecoordinometers (to control the verticality of the pillars) and a piezometer.
All the instruments are controlled by remote units connected to a central control unit, linked via modem to the Engineering Faculty of the University of Florence. This peculiarity allow to make a real-time control, in such a way to point out every possible ill-functioning of some components. The recorded data, after a preliminary control, are stored on magnetic supports, converted into mechanical units and checked, in order to perform successive analyses.
The system has been working since January 1988, and so as many as four years of recorded data are now available.

3. ANALYSIS OF THE EXPERIMENTAL DATA

The aims of the installed monitoring system are mainly three:
- the investigation of the structural behaviour of the monument, with a particular regard to the correlation between environmental actions and structural response;
- the study of long-term behaviour by mean of the trend analysis, based on the mean-rate estimation of the cracks amplitude variations;
- the assessment of a control procedure capable to give information about non-standard values.

The numerical analyses have then been carried out in order to give responses to the previous questions.
After a preliminary investigation about the recorded time-histories of all the different instruments, two types of correlation functions have been studied: correlation between signals recorded by instruments of the same kind (i.e. temperatures Vs temperatures, displacements Vs displacements) and between different ones (displacements Vs temperatures).
Figure 1 reports, as an example, some of the obtained functions referred to the web no. 4.
The following observation can be achieved:
- the cross-correlation curves between temperatures [Figure 1b] recorded by the internal instruments with respect to the external ones, show a time-shift compared to the auto-correlation function: this is related to the thermal diffusion between different layers of the dome and to the thermal inertia of the masonry. Nevertheless the curves look very similar, this implying that, in

view of a structural description, only one of the time-histories can be utilised as a first approximation;
- along the main cracks, the cracks themselves shows a different behaviour in the lower part with respect to the higher one; as a matter of fact, while the inferior part is opening, the superior is closing, and vice versa. This can be seen from the correlation graph [Figure 1a], which shows a phase delay of about 90° between curve no. 1 (auto-correlation of the instrument placed in the higher part of the Dome) and no. 4 (cross-correlation between the previous and the one placed in the lower part);
- in the radial direction, every crack shows a behaviour similar to that previously described, that is, its opening and closing are not in phase between the internal part of the Dome and the external one;
- the correlations between temperature values and cracks amplitude variations [Figure 1c] confirm what was previously asserted: for example, an increase in temperature (warm months) induces a closure of the cracks near the tambour (curves no. 1 and 2) and a growth in the upper part of the Dome (curves no. 4 and 5).

With regard to the trend analysis, this has been carried out mainly in order to understand the long-term behaviour of the cracks layout. A particular procedure has been set up, based on the following steps (see Figure 2):
- a first trend removal is performed, evaluating the best fit, in a least squares sense, of the experimental data through a linear plus harmonic function, the period of the latter being of one year; this was chosen because of the particular shape of the autocorrelation function, clearly showing this periodicity (Figure 2b);
- the autocorrelation of the residuals obtained in this way shows another strong periodicity of about six months (Figure 2e), so that a new fit with an harmonic function having the same period is performed;
- the new autocorrelation function of the residuals (Figure 2h) does no longer show any particular periodicity, and so the residuals can be considered as being delta-correlated (white noise) with a distribution very similar to a Gaussian one (Figure 2i); the obtained approximation seems to be sufficiently reliable, and the derivative of the linear component gives a trend estimation.

In order to evaluate the correlation between the obtained trends and the behaviour of the temperature data, the same analysis has been performed with regard to the data recorded by the thermometers.

The above described analysis will be utilised in order to define a control procedure that will be built in this way:
- by means of the previously recorded data, some approximation functions can be achieved, for the deformeter signals as well as for the thermometer signals. These functions can be utilised to set up a forecasting procedure; the comparison between the forecast values and the acquired one allow us to single out non-standard behaviour, that could depend either on an ill-functioning of the system or on a variation in the structural behaviour.
- moreover, it is necessary to set up a control procedure based on the correlations between different instruments, in order to single out if the recorded non standard value of mechanical quantities is due to particular thermal conditions or to different structural response of the monument.

4. COMPARISON WITH HISTORICAL DATA AND OTHER SYSTEMS DATA

The surveys carried out on the bases of 22 deformeters installed (on the main cracks at various heights) by O.D., provided a total of some 2,400 measurements manually taken four times a year since 1956. These data are particularly significant in that they cover a long period of time (more than 30 years) which includes such noteworthy events as the great flood of 1966, little earthquakes and some variations in climate. On these data, some analyses have been performed in order to evaluate the trend of the cracks mean width variation: a linear regression has been utilised (see Figure 3), thus

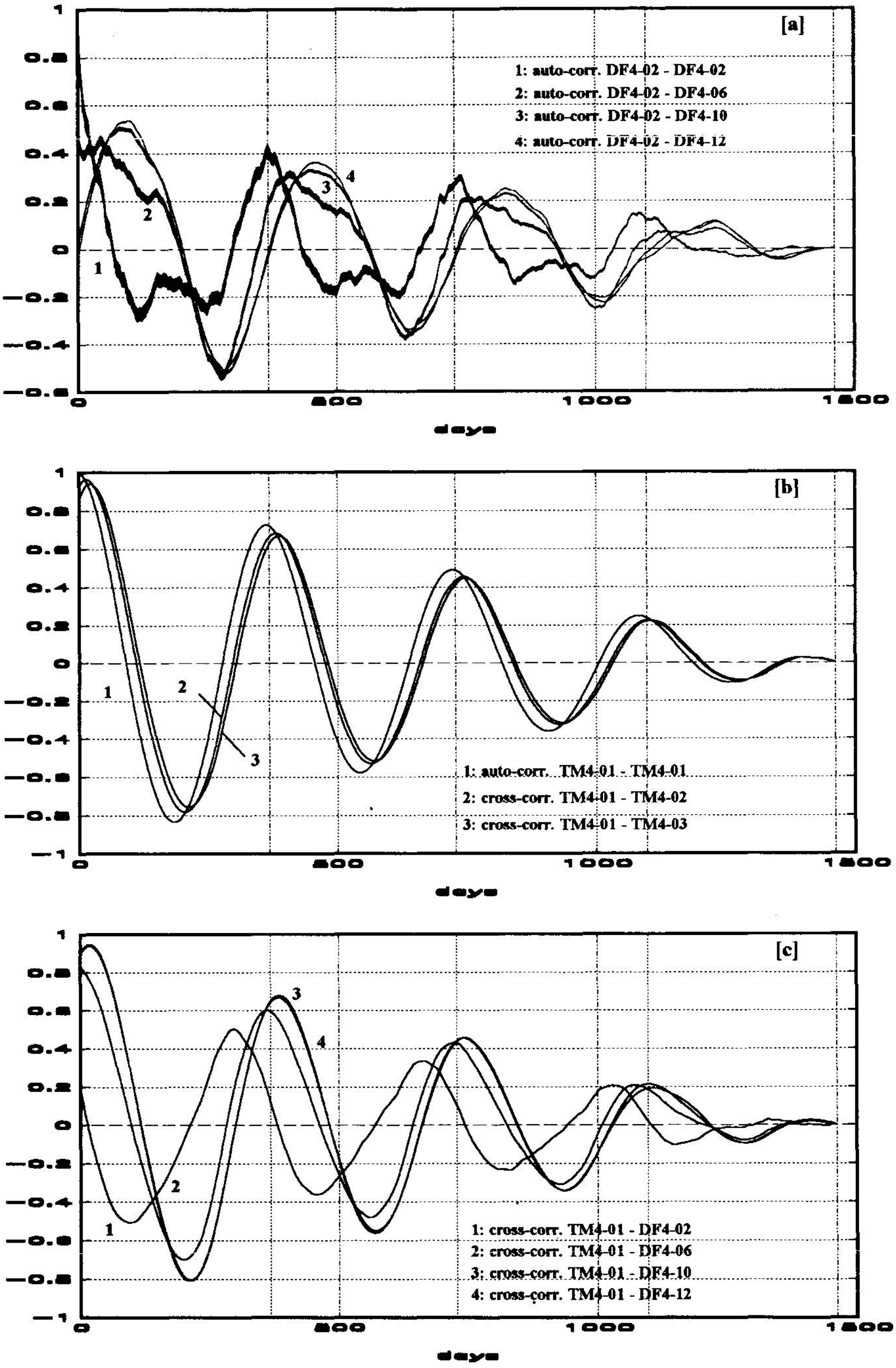
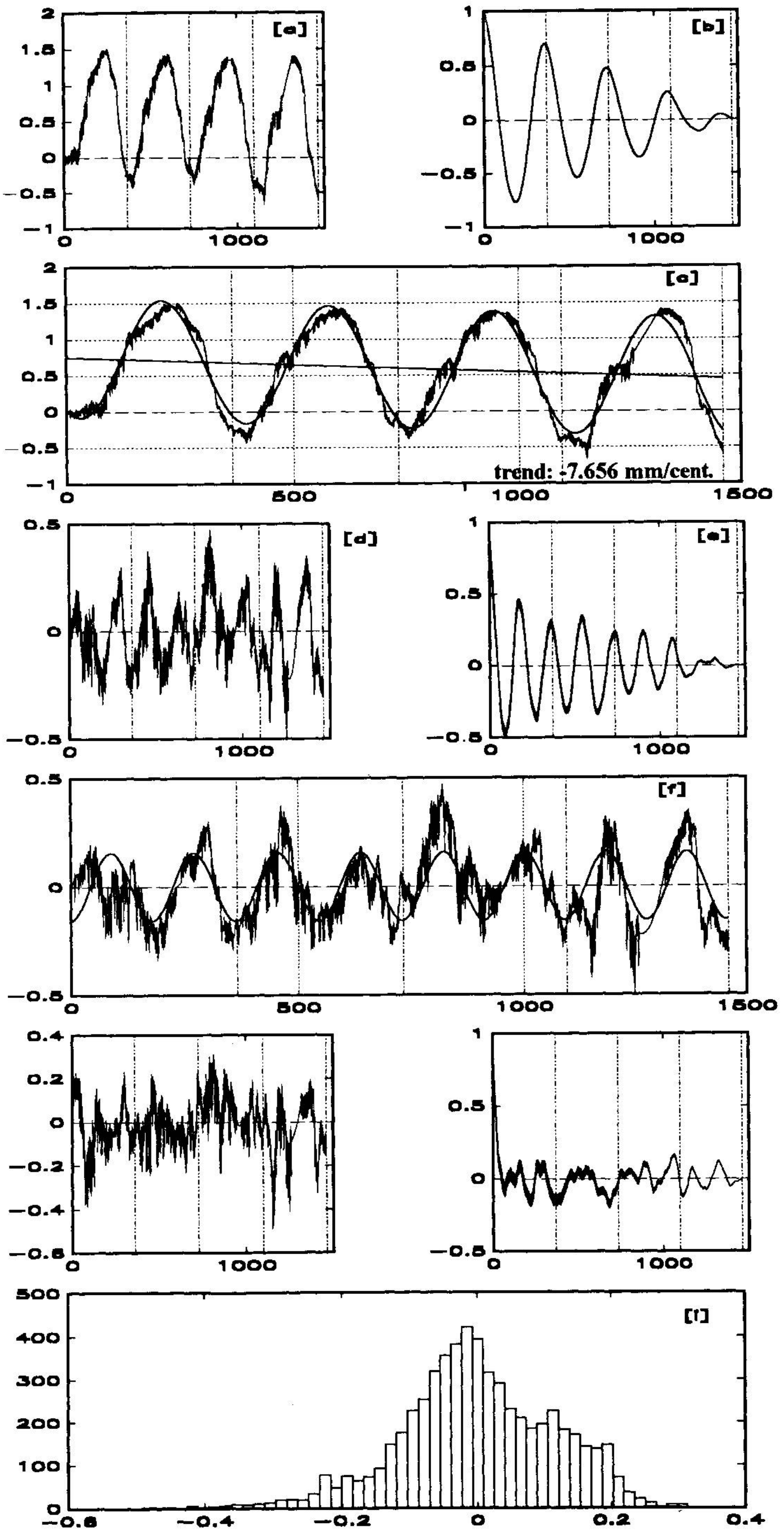


Figure 1: Correlation analyses on data recorded in web no. 4:
[a]: correlation between data recorded by deformeters
[b]: correlation between data recorded by termometers
[c]: correlation between data recorded by termometers and deformeters



[a]: recorded data (deformeter DF4-10)

[b]: autocorrelation function

[c]: approximation, step one: linear plus harmonic trend removal

[d]: first step residuals (recorded data minus linear plus harmonic approx.)

[e]: first step residuals: autocorrelation function

[f]: approximation, step two: linear plus harmonic trend removal on step one residuals

[g]: second step residuals (recorded data minus double linear plus harmonic approx.)

[h]: second step residuals: autocorrelation function

[i]: second step residuals: distribution histogram

Figure 2: Trend removal procedure; application on data recorded by deformeter no. DF4-10 (web no. 4, internal dome, lower part [negative values=cracks opening])

obtaining a trend value less than 6 mm per century (for the main cracks). A cubic regression has been performed too (Figure 4), so to single out the non-stationarity of the increases during the years; this is an important aspect that has to be considered when analyses utilising shorter period of time are taken into account.
The recorded data acquired from the 48 deformeter bases installed by the S.B.A.A. have been analysed too. These data are very interesting because they give information about the cracks near to the Dome scaffolding holes which have been utilised for the restoration of the frescoes. The data have been recorded twice a month since October 1987. An analysis similar to the one performed on the data recorded by the digital monitoring system has been carried out, thus estimating the increasing trend in the develop of the cracks: an example is shown in Figure 5.
Further analyses are ongoing in order to assess a comparison procedure between the data recorded from different monitoring systems (historical and digital).

5. SOME CONSIDERATIONS ABOUT THE STRUCTURAL BEHAVIOUR

The recorded data and the performed analyses allow to make some considerations about the overall structural behaviour of the Dome.
The variations in width of the cracks is mainly due to temperature effects, as regards both of the annual and the daily variations. As it can be seen, the signals recorded by the deformeters show daily variations as well as the external recorded air temperature does. On the contrary, the temperature values recorded inside the monument, both about in air and in the masonry, show a progressive smoothing moving from outside to inside. The recorded data analysis confirms results already known [2]: the external (thinner) dome plays the role of a thermal shield with respect to the internal (thicker) one, but the structural links between them allow the two domes to exhibit the same dependence upon thermal effects.
Nevertheless, as it was mentioned before, the cracks amplitudes seem to show some evolution in time, pointed out by the presence of a trend. This tendency is generally toward an increase in the cracks mean width, and so a particular attention must be devoted to the control of this phenomenon, even if the comparison with historical data shows that the behaviour is not different to the one recorded during last decades.
On the whole, the temperature seems to be the main cause of the movements of the edges of the cracks; last researches [3] showed (by numerical modelling and considering the historical evolution) that the main cracks have been caused by the self-weight of the monument and its distribution between the pillars and the arches sustaining it, and this fact can be considered as the main reason of the progressive increasing in time of the cracks width.
The overall structural behaviour is very difficult to understand and it is not possible to be summarised by simplified models. As an example, the main vertical cracks in the even webs, were thought to be thermal joints capable to permit the expansion of the webs in warm months. The analysis of experimental data have shown that its behaviour is not constant both along the height and along the radial direction and so the previous simplified hypothesis must be removed.
The differences between the behaviour along the height can be explained considering that actually the Dome is not behaving like a dome, that is with a radial symmetry, but as four almost independent sub-structures, separated in correspondence of the main cracks. As a consequence of this, flexural deformation can arise in the meridian direction, and then the behaviour along this direction is not constant.
Further researches and modelling are necessary in order to better understand the structural behaviour and to design possible intervention to preserve the monument.

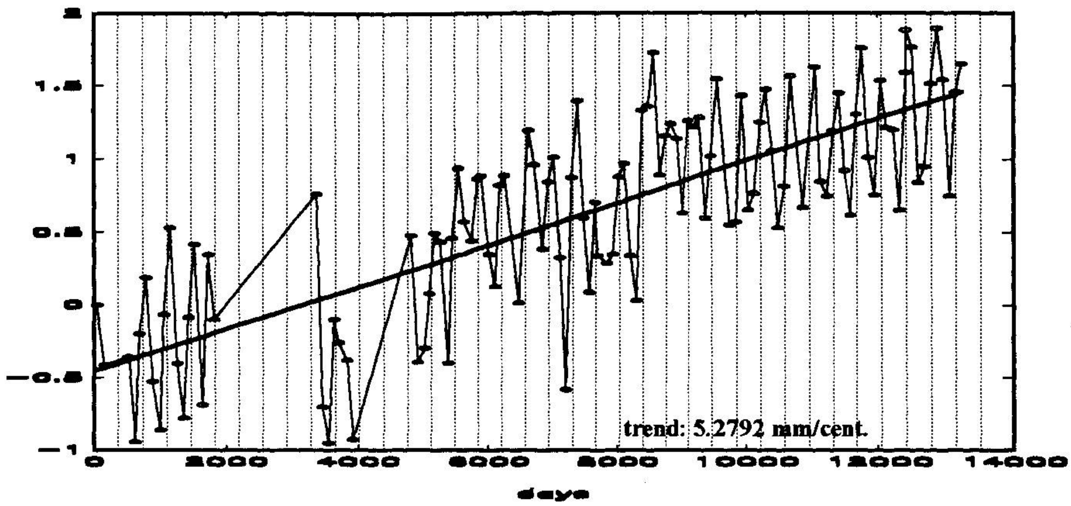


Figure 3: O.D. data (years 1956-1992), base no. 5: linear regression (values in mm [positive values=cracks opening])

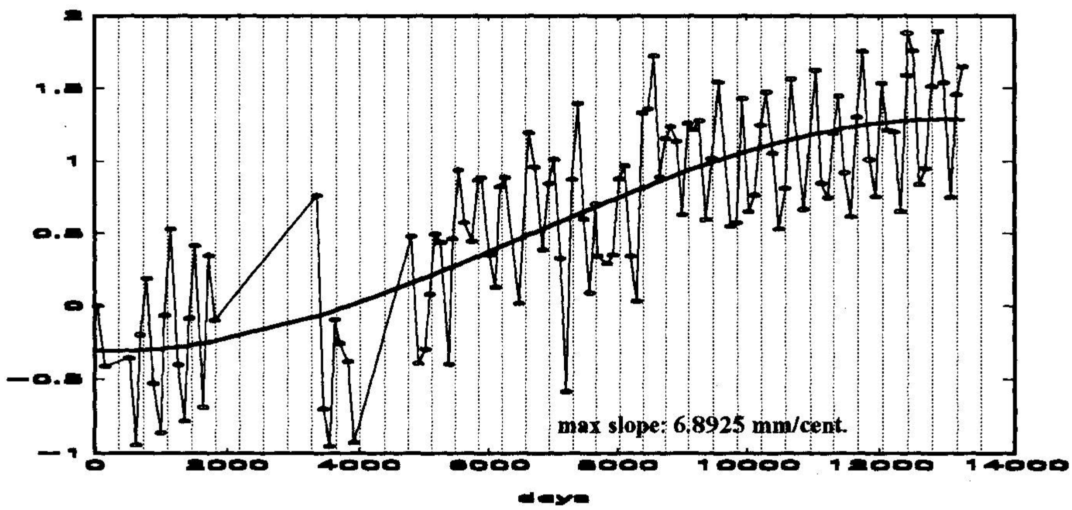


Figure 4: O.D. data (years 1956-1992), base no. 5: cubic regression (values in mm [positive values=cracks opening])

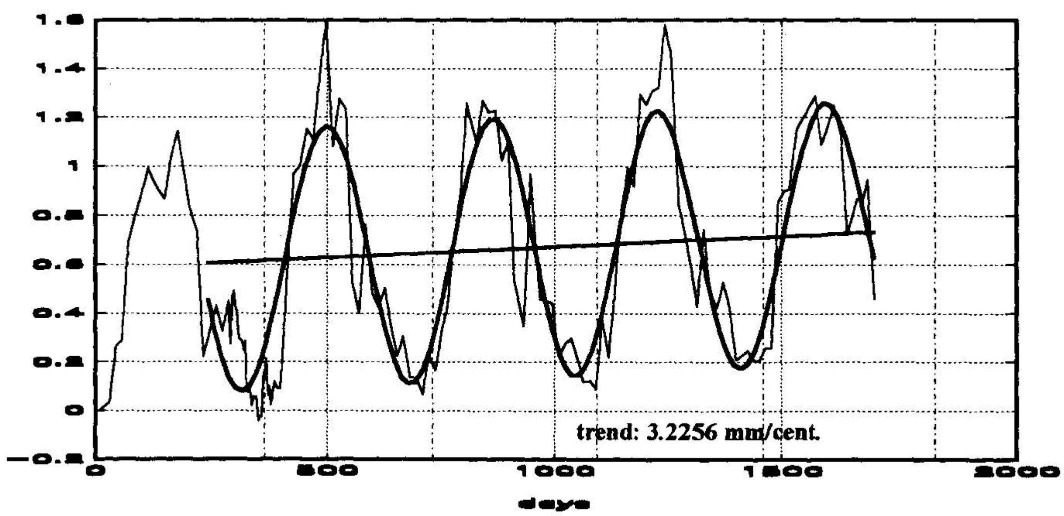


Figure 5: trend analysis of S.B.A.A. data (years 1956-1992), deformeter no. 6, web no. 4: linear plus harmonic approximation (values in mm [positive values=cracks opening])

6. CONCLUDING REMARKS

The digital monitoring system has been a fundamental tool in order to achieve a better knowledge of the Brunelleschi's Dome; correlation between environmental actions and periodical movements of the cracks as well as the presence and the values of the trends have been singled out.
Moreover, the recorded data and their analyses will allow to set up numerical models capable to forecast the structural behaviour under non standard events (such as strong earthquakes) and to evaluate the effects of some preservation designs.
Nevertheless, the results obtained from mechanical instruments as well as from the monitoring system at present installed, don't seem to give rise to any worrying about the evolution of the cracks layout.
Some considerations about the experience done can be carried out:

- the data acquired from the monitoring systems are not enough to make a reliable estimate of the trends of the increment of the cracks width. This can be seen from the fact that temperature data show some trends too and so the evaluated tendencies could be due to those recorded by the thermometers. In other words, a more correct assessment of the expected values could be done only when no trends (or a very small one) will be obtained from thermometers data; maybe other 1 or 2 years of records, could enable the procedure to give appreciable results. Nevertheless, the obtained values are in accordance with those estimated by the values recorded from mechanical instruments.
- as the historical data show, the increment in the cracks width hasn't been evolving in the same way during the years, but it showed a "stairs" course. In fact, there have been some periods in which the cracks did not show any variation while, in others, some increments occurred. The period investigated by the present monitoring system could, however, be too small to decide which of the previously described classes the present time-interval belongs.
- the data recorded by monitoring systems "poorer" than the one at the present time installed have shown their great utility in long-term evaluation of the phenomena. As a matter of fact, simple monitoring system can give appreciably results mainly for three reasons. The first one is that simpler systems are easier to be used, so that even a not skilled staff can utilise them, and this one is the circumstance related to the monitoring of historical monuments. The second reason is due to the fact that uncomplicated systems require less control with respect to most sophisticated ones. The latter, directly related to the first two, is the possibility of using them for long periods and so to acquire a considerably large amounts of data.

Large (and most expensive) systems can so be used for a limited period of time, in order to clearly understand the physical phenomena and so to properly design systems simpler but capable to give information about the quantities that have been revealed as the most interesting ones. The use of less expensive systems allow them to be used in as many situations as possible and so to control a great part of the historical monumental heritage that actually gives apprehension.

This study is a part of a research program involving the Civil Engineering Department of the University of Firenze and the S.B.A.A. of Florence, under the scientific supervising of Prof. Ing. A. Chiarugi.
The Authors wish to thank Arch. R. Dalla Negra (S.B.A.A.) for his active collaboration.

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Structural Assessment of St. Mark's Basilica, Venice

Evaluation structurale de la Basilique de St. Marc, Venise

Tragfähigkeitsbeurteilung der Basilika von San Marco, Venedig

Pier Paolo ROSSI
Engineer
ISMES
Bergamo, Italy

Franco VENTRUCCI
Engineer
Ministry of Public Works
Venice, Italy

Ettore VIO
Architect
Surveyor Basilica S.Marco
Venice, Italy

SUMMARY

The diagnostic analysis to check the static conditions of the St. Mark's Basilica is presented. The wide research program, now in progress, is based on a combined experimental-numerical procedure which includes geometric survey, historical analysis, geotechnical investigation, analysis of the structural and mechanical characteristics of the masonries, monitoring system and numerical modelling.

RÉSUMÉ

On présente l'analyse diagnostique pour le contrôle des conditions statiques de la Basilique de St. Marc à Venise. L'important programme de recherche, actuellement en cours, est basé sur un ensemble de procédures expérimentales et numériques comprenant relevé géométrique, recherche historique, recherche géotechnique, analyse des caractéristiques structurales et mécaniques des maçonneries, système de surveillance et développement d'un modèle mathématique.

ZUSAMMENFASSUNG

Es wird die für die baustatische Prüfung der Basilika von San Marco in Venedig unternommene Diagnoseanalyse beschrieben. Das gegenwärtig laufende, umfangreiche Forschungsprogramm ist auf ein integriertes experimentell-numerisches Verfahren gegründet und umfasst Aktivitäten wie geometrische Aufnahme, historische Nachforschung, geotechnische Proben und Untersuchungen, Analyse des Tragverhaltens und der mechanischen Eigenschaften des Mauerwerkes, Überwachungssystem und Rechenmodelle.

1. INTRODUCTION

The Basilica of St. Mark is a structurally complex monument in which three major construction phases can be identified. The first dates from 828 A.D. when the Ducal Chapel was founded to house the body of St. Mark Evangelist which the Venetians had recovered from Alexandria in Egypt. The second phase began in 976 A.D. when the Ducal Palace and the Church were burnt by the populace to kill Doge Candiano IV; the structure was rebuilt with new decorations by Doge Pietro Orsolo the Saint. The third phase began in 1063, when Venice gave the world a fine example of its power with the construction of the grandiose building designed by the most important Byzantine architects. The Church was consecrated and dedicated to St. Mark in 1094, just two years before the ninth centenary of the life of the Saint. Built over many centuries utilising pre-existing structures as far as possible, the Basilica is showing clear signs of deformation and damage to its walls, arches, vaults and domes, as well as the flooring. Numerous earthquakes exacerbated this situation, especially those in the XII century, which shook Venice and the Basilica to their foundations, as well as several devastating fires, requiring the rebuilding of entire sections of the monument; one can also mention in this context the stylistic modifications and the increased structural weight of the domes, joined, in turn, by all the other additions made over the centuries. The longitudinal section of the Basilica in fig. 1 shows the original masonry domes and the overhanging lead domes built in the middle of the XIII century. It was only in the early XIX century, following the fall of the Venetian Republic, that the Austrian government enacted systematic restoration works based on the project developed by engineer Fustinelli, which was examined by a specially-appointed government commission. Above all, attempts were made to restore the walls, vaults and arches, and to consolidate the centuries-old structure which, under the thrust of the weight of the lead domes, was literally splitting the Basilica in two lengthways.

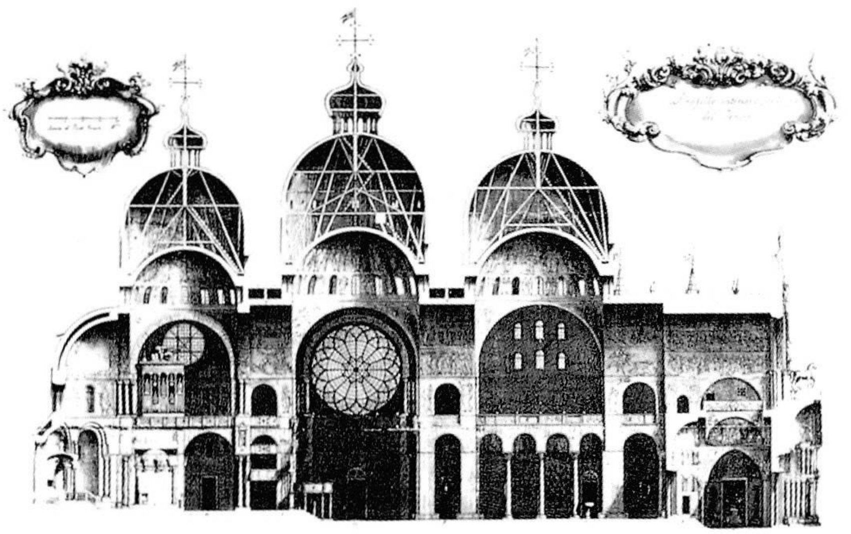

Fig. 1 - Longitudinal section (A. Visentini, XVIII century)

Thanks to the admirable attention of the "Procuratoria" (ancient name of the Surveyor Board of the Basilica) in the eighties a scientifically advanced restoration programme was planned by the Technical Department of the Basilica. A first programme of structural investigations, including the monitoring system, was financed by ISMES and ENEL in 1990. Then ISMES was commissioned by the Venice Water Board of the Ministry of Public Works to carry out a wide-ranging programme of surveys for the evaluation of the static conditions of the Basilica. This research programme, which is now in progress, is based on a combined experimental numerical procedure consisting of the following major steps:
– topographic and photogrammetric survey; – historical analysis; – detailed crack pattern survey; – geotechnical investigation and foundation survey; – analysis of the mechanical characteristics of the masonry structures by sonic tomography, radar, coring and video camera survey and flat-jack tests; – monitoring system; – numerical modelling

2. HISTORICAL SURVEY

The works performed on the Basilica in the past are documented up to the fall of the Venetian Republic in the Venice State Archives and subsequently through documents filed for the most part in the Historical Archives of the "Procuratoria" of St. Mark. These documents comprise texts, drawings and photographs. This material is extremely important and provides the essential basis for properly targeted restoration work; moreover, the Technical Department of the Basilica is well-informed of the works performed over the last 20-30 years. This material has to be organised in a manner which integrates knowledge of written documents with graphics and photographs and, especially, coordinated in a chronological and topographical fashion in order to provide rapid access, with the aid of computers, to the information concerning the various fields of intervention. Work therefore involves cataloguing these documents, in order to prepare report cards detailing the major restoration tasks implemented in the past.

3. TOPOGRAPHICAL AND PHOTOGRAMMETRICAL SURVEY

Unfortunately, no full-scale geometric survey of St. Mark's Basilica presently exists. Ferdinando Forlati, the famous Surveyor - Architect, indeed hoped that one would be able to conduct a scientific geometrical survey of the Basilica in his book on the Restoration of the Basilica (1974). Various attempts to survey the Basilica are extant, some of considerable prestige, such as those carried out by Antonio Visentini in the mid-1700s and later by the Surveyor - Architect Scattolin. The structure of the Basilica, however, is of such spatial complexity - dominated by curved surfaces embellished by figurative and decorative low reliefs and mosaics - that it cannot be reproduced faithfully without a careful survey of forms, dimensions, measurements and alignments which alone will make it possible to evaluate static conditions and verify historical hypotheses in order to target restoration work and conservation methods appropriately. In the case of St. Mark's Basilica, architectural photogrammetry is especially meaningful and pertinent, since it is the only means of ensuring absolute correspondence between graphic reproduction and actual architectural-spatial placement. The photogrammetric study, already begun in 1983, now requires the elaboration of all the photographs taken, in close relationship with the reference topographic map in order to ensure the dimensional accuracy of the drawings and provide structural analysis with definitive information concerning the deformation of the monument. The project envisages numerical data acquisition for the survey, so that this information can then be used for restoration work. By converting historical drawings into computer graphics, it will also be possible to compare manual surveys of past work with the current status of the building. By overlapping these visual elements, further information will be gained in order to complete the graphic reproduction of the Basilica.

4. SUBSOIL AND FOUNDATION SURVEY

A detailed survey of the subsoil of the Cathedral was carried out in order to define the stratigraphy of the soil as well as the mechanical characteristics of the different soil layers. Seven boreholes (30 m deep) were drilled around the perimeter of the Basilica and many undisturbed samples were taken for carrying out physical and mechanical laboratory tests. Static cone penetration tests were also carried out in three testing points, using a digital cone, in order to identify the mechanical characteristics of the different layers of soil. On the basis of in situ and laboratory tests, the stratigraphy of the soil was identified. As an example, fig. 2 shows the stratigraphic profile obtained by correlating the results of three boreholes drilled on the North side of the Cathedral. Under a surface layer of filling material, a silty-clay layer (about 5 m thick) has been observed. A

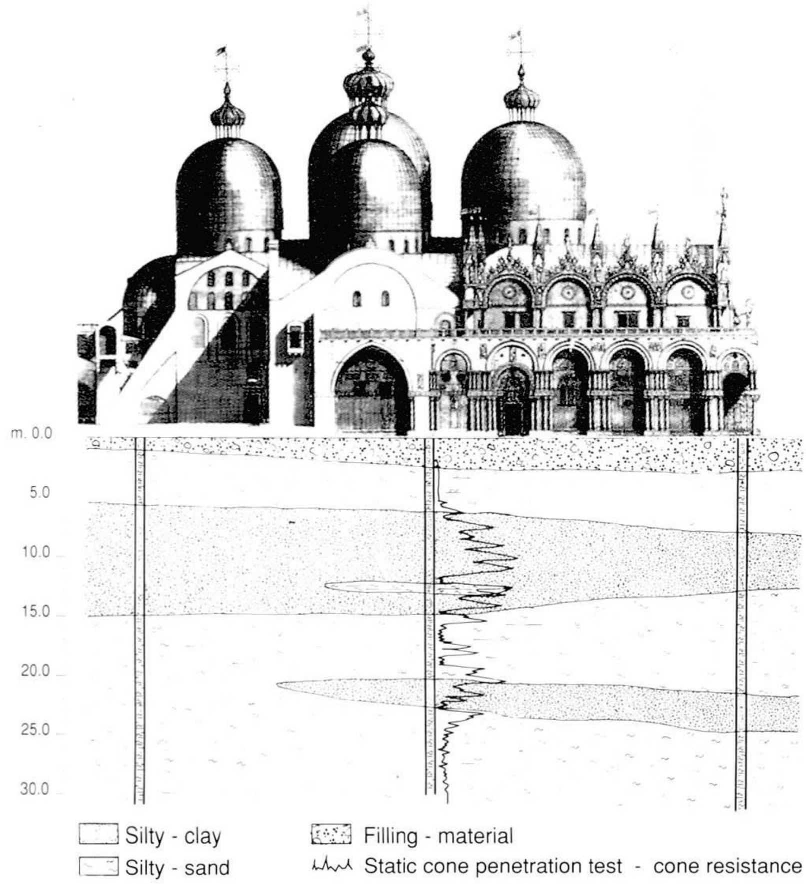

Fig. 2 - Stratigraphic profile along the North side of the Basilica. The results of static cone penetration tests are presented

silty-sand layer with higher mechanical characteristics is also present with a thickness varying between 5 and 10 m. Under the sand layer, silty-clay material was found. The presence of local lenses of sand inside the silty-clay material was observed as usual in lagoon-side environments. Three boreholes were equipped with electric piezometers and three with long base settlement-gauges connected to an automatic reading unit. This monitoring system will allow the settlement of the perimeter of the Cathedral to be controlled as a function of time and water-table variations.

An in-depth investigation was also carried out to analyze the geometric and structural characteristics of the foundation masonry of the perimeter walls and of the pillars. Several small boreholes (diameter 62 mm) were cored by using a light diamond saw and the lateral surface of the boreholes was surveyed by means of a colour video camera. Fig. 3 shows the drilling equipment installed at the base of a pillar for coring the foundation masonry. More than 50 subvertical boreholes were drilled to investigate the structural characteristics of the foundation masonries. In order to obtain qualitative information concerning the mechanical characteristics of the masonry structures, a sonic-log survey was carried out as well as cross-hole measurements by using two parallel boreholes. As a result of the coring and video camera survey, the typical structural scheme of the foundation masonry was determined (Fig. 4). At a depth of about 70-90 cm from floor level, a stone basement made with large blocks of sandstone is present with a thickness of about 2 m. Underneath this stone basement there is a wood plate (10 cm thick) which is about 3.0 m below the floor. This wood plate rests on short wooden piles which are very poorly preserved.

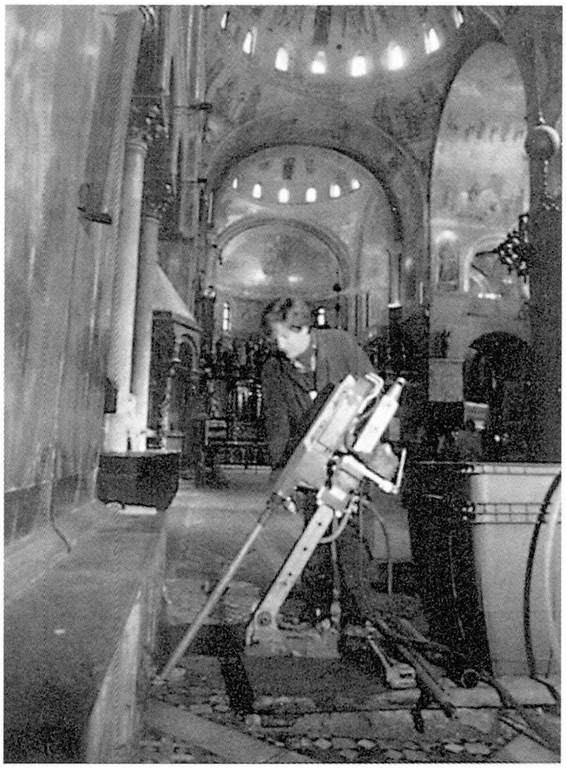

Fig. 3 - Drilling equipment for coring the foundation masonry

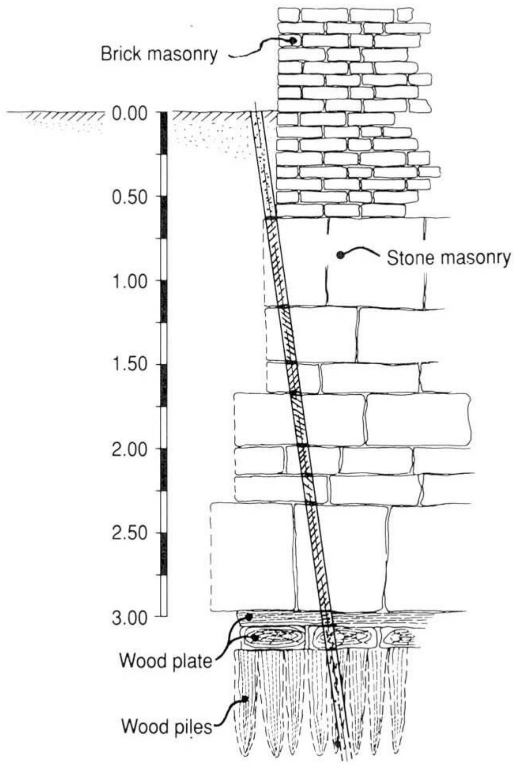

Fig. 4 - Structural scheme of the foundation masonry

5. ANALYSIS OF THE MECHANICAL CHARACTERISTICS OF THE MASONRY STRUCTURES

The pillars and the perimeter walls of the Basilica were intensively investigated by using both non-destructive and slightly-destructive tests. Special attention was devoted to the pillars which were investigated in the only unlined portion (about 1 m high) along the women's gallery. Under this layer, the pillars are lined with marble and over this layer the surfaces are covered by mosaics.

5.1 Sonic tomography and radar survey

At first a non-destructive investigation was carried by using sonic tomography and radar survey in order to check the presence of possible anomalies. The most significant results were obtained from the sonic tomography survey which provided a clear mapping of the different velocity zones. As an example, fig. 5

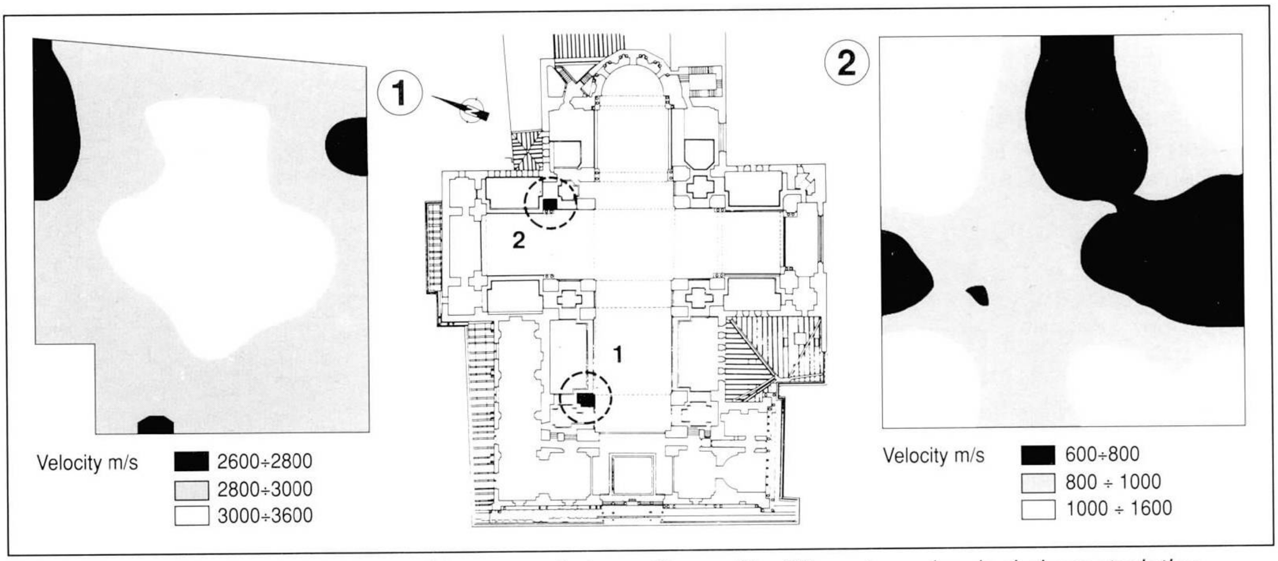

Fig. 5 - Results of tomographic survey in two pillars with different mechanical characteristics

shows the results for pillars 1 and 2. It can be clearly observed that pillar 1, which was consolidated by grouting about 30 years ago, clearly shows high velocity values, especially in the internal zone which was involved in the grouting operation. On the contrary, the velocity values obtained for pillar 2 show that the mechanical characteristics of this pillar are very poor (in fact no consolidation work was performed on this pillar in the past).

5.2 Coring and borehole video camera survey

In every pillar, horizontal and vertical boreholes (more than 60) were drilled by using light drilling equipment. Then a video camera survey was performed with a detailed description of the surface of the boreholes. Fig. 6 shows the equipment used for the video camera survey and the results of the inspection into the horizontal borehole of the pillar 2 indicated in fig. 5. A large number of voids can be observed inside the masonry, which confirms the results obtained by sonic tomography. By drilling subvertical boreholes (about 4 m long) starting from the level of the women's gallery, it was possible to investigate the lower part of the pillars lined with marble plates. Several boreholes were also carried out along the perimeter walls of the Basilica.

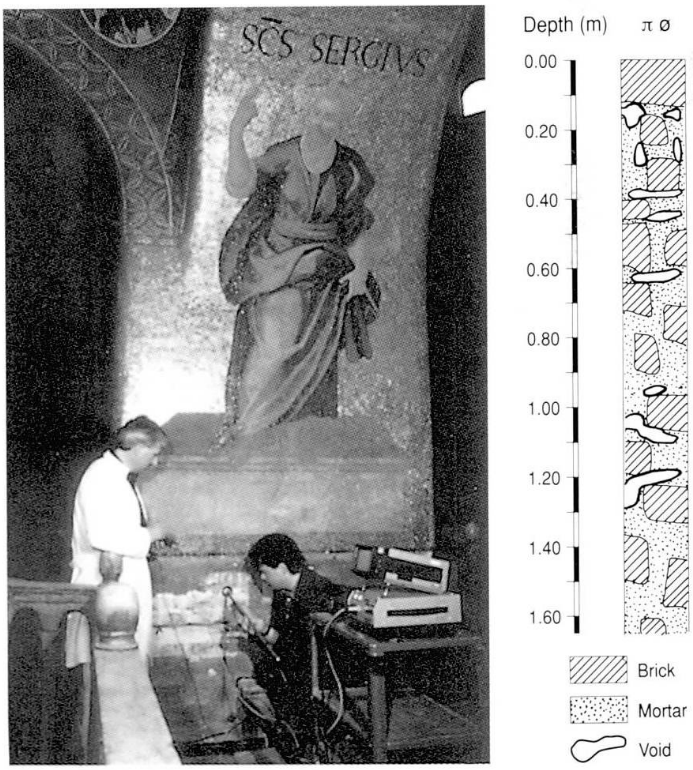

Fig. 6 - Video-camera survey equipment and result of the inspection into the horizontal borehole drilled in the pillar n. 2

5.3 Measurement of the state of stress and analysis of the deformability characteristics

For the evaluation of the static conditions of the Basilica it seemed advisable to measure the actual state of stress on the main supporting structures with special attention to the pillars. For this purpose the well-known flat-jack test was used. This non-destructive test, developed by ISMES about 15 years ago, is based on the release of the state of stress by making a small slot in a mortar layer and then reloading by means of a thin flat-jack inserted into the slot. This test is very simple and reliable, as proven by calibration tests carried out in the laboratory. More than 80 flat-jack tests were carried out (about 65 on the pillars and 20 on the perimeter walls). Fig. 7 shows the view of a flat-jack test on a pillar.

The stress values measured on the pillars are shown in fig. 8. A certain heterogeneity of the state of stress in the pillars can be observed, which is related not only to the position and size of the pillars but also to the type and extent of the consolidation works which some pillars have undergone in the past. High stress values (up to 1.1 MPa) were measured in some pillars. Furthermore, in the above-mentioned pillar 2 (the south-east pillar of the St. John's Dome), a state of stress of 0.95 MPa was measured. This value was considered very high in relation to the results obtained by sonic tomography, coring and video camera survey.

By using two parallel flat-jacks the deformability characteristics of the masonry were determined for the pillars and along the perimeter walls. Two flat-jacks were inserted in the masonry in order to delimit a specimen of appreciable size (40 x 50 cm) on which a uniaxial state of stress was applied. During the loading phase, the axial and transversal deformation of the specimen was plotted as a function of the applied load. This simple testing technique allowed the deformability moduli of the different types of masonries analyzed to be determined. These values have been used as input data for the finite element model.

Fig. 7 - Measurement of the state of stress in a pillar by means of the flat-jack technique

The deformability characteristics of the pillars are a function of the type of consolidation works performed on the masonry. The tests carried out on the original unconsolidated masonry show values for Young's modulus varying between 800 and 2000 MPa (for a stress range of 0.4 ÷ 0.8 MPa). The values determined on the masonry consolidated by grouting are higher (between 1600 and 4000 MPa) in the same stress range. In some pillars, where the masonry was completely rebuilt about 30 years ago, Young's modulus has an average value of about 5000 MPa.

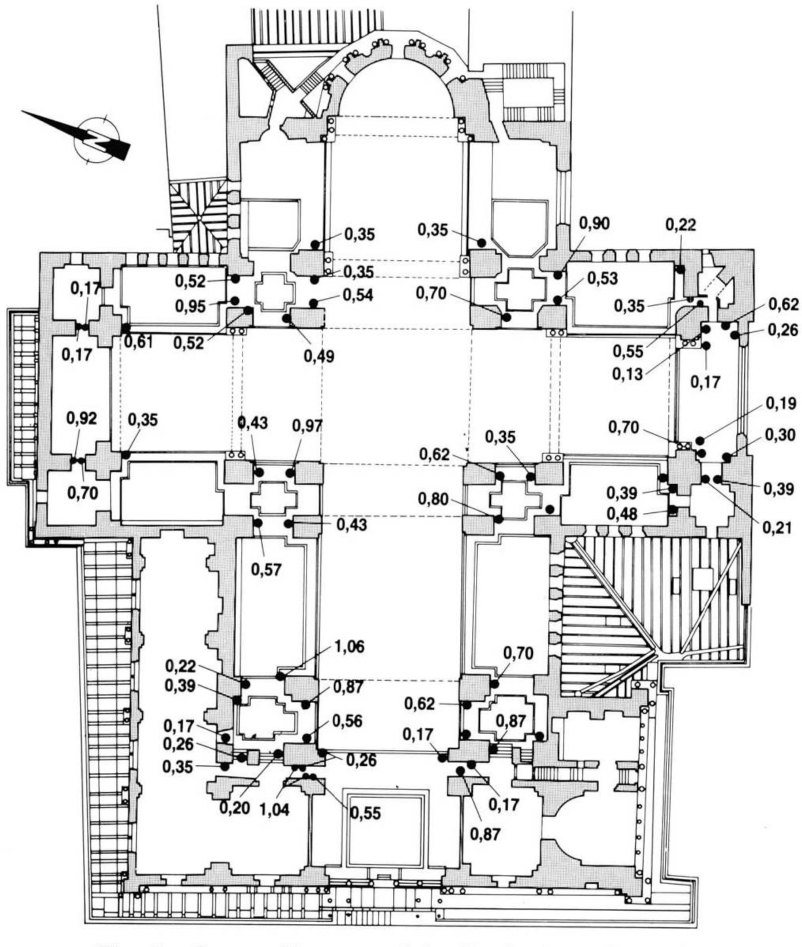

Fig. 8 - General lay-out of the flat-jack testing point with the indication of the measured states of stress

5.4 Supplementary investigation on the south-east pillar of St. John's Dome

The combined analysis of the results obtained by flat-jack test, sonic tomography, coring and video camera survey clearly shows that the pillar n. 2 (sout-east pillar of St. John's Dome) has to withstand very severe loading conditions in comparison with the poor mechanical characteristics of the masonry. For this reason, it was decided to carry out a more detailed investigation in the lower part of the pillar after removing the marble lining. At first a detailed crack pattern survey was carried out which showed the presence of several vertical small cracks, especially near the corners. Sonic tomography was performed at three sections of the pillar at different heights from the floor and the results obtained confirmed the low velocity values presented in fig. 5. In several points of the pillar, the state of stress was measured by flat-jack test and an average value of 0.85 MPa was found. Then, in two points, the deformability characteristics were determined by two parallel flat-jacks. The typical stress-strain curve of the masonry is shown in fig. 9, where axial and transversal strain values are presented as functions of axial stress. The average value of the measured state of stress is also indicated. It can be observed that the masonry presents a linear behaviour up to a stress level of about 0.8 MPa with a value of Young's modulus of about 2.000 MPa. For higher stress levels, up to 1.2 MPa, Young's modulus decreases to a value of about 900 MPa and for the stress range between 1.2 and 1.5 MPa the modulus is lower than 300 MPa. It can be observed that the average state of stress measured in the pillar exceeds the limit between elastic and plastic behaviour. This supplementary investigation clearly confirmed that the static conditions of the pillar are not satisfactory and for this reason it was decided to carry out urgent consolidation works which are now in the design phase.

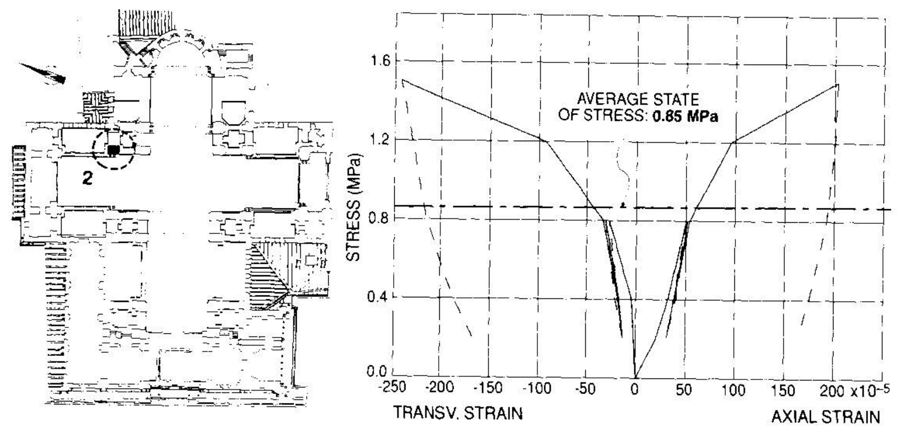

Fig. 9 - Stress-strain diagrams obtained by flat-jack tests on the pillar n. 2 with the indication of the measured state of stress

6. MONITORING SYSTEM

In order to obtain a continuous control of the stàtic behaviour of the Basilica, in 1991 a monitoring system was installed by ISMES. The principal features which are monitored are as follows:

- opening of the main cracks in the pillars (8 extensometers)
- relative horizontal movements of the pillars (11 long base extensometers)
- tilting of the vertical structure (4 inclinometers)
- internal and external temperature (5 temperature-gauges)
- vertical settlement of the soil foundation (3 long base settlement gauges)
- water-table variations (3 piezometers).

All the instruments are connected to an automatic data acquisition and recording unit which can quickly indicate possible anomalies in the structural behaviour. The analysis of the deformations as a function of time and temperature makes it possible to separate thermal effect from the deformations arising from other structural causes. As an example, fig. 10 shows the diagrams of the relative horizontal movements between some pillars measured by three long base extensometers during the first period of observation. It can be clearly

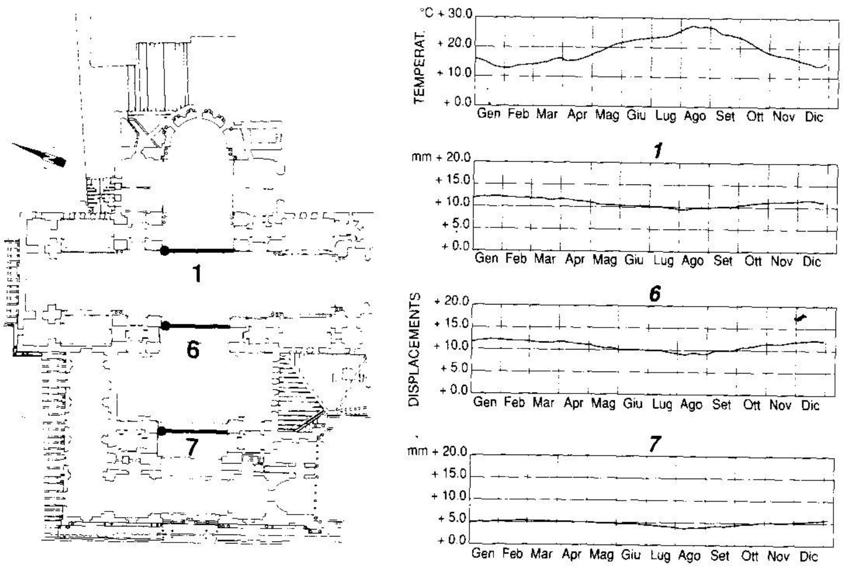

Fig. 10 - Diagrams of the relative horizontal displacements of some pillars during the first year of observation

seen that the deformation diagrams follow temperature changes and no permanent deformation is noticed at the end of the first yearly cycle.

7. NUMERICAL MODELLING

A significant contribution to the knowledge of the static conditions of the Basilica will be provided by the mathematical model which will utilise all the data and information obtained through in-situ investigations. This mathematical analysis, which is now in progress, has the following aims:

- to set up a mathematical model capable of providing a reasonable picture of the present stress distribution due to dead loads; the validity of the model in achieving this aim will be tested by comparing experimental stress measurements with analytical results in certain points of the structure;
- to estimate the effect that a reduction of stiffness in some structural elements could produce in terms of displacement and stress redistribution;
- to forecast the displacement trend under cyclic loads, such as thermal loads, in order to obtain a better understanding and a more meaningful interpretation of data provided by the monitoring system.

The high degree of complexity of the structure suggested the opportunity of considering the whole building as the sum of suitable substructures. Several analyses on simplified models have been performed in order to identify the most significant substructures. It now appears evident that a good representation of the global behaviour of the Basilica can be achieved by isolating five parts, each containing one dome; the boundary conditions of each single substructure will reproduce correctly the stiffness of the adjacent substructures.

8. CONCLUSIONS

The diagnostic analyses illustrated here represent an exhaustive example of the methodological approach which should be followed when studying the static behaviour of an important monument. This combined experimental and numerical procedure is effectively the only research tool for the study and design of restoration and consolidation work. The results of the experimental investigations (flat-jack tests, coring and video camera survey, sonic tomography, radar, etc.) have been used to evaluate the strength, the deformability and the composition of the masonry, as required for the construction of the numerical model, and to determine the present state of stress, which can than be used for safety checks and for calibrating the numerical models. This study is not yet completed, but a first important result has been obtained; the existence of a weak point in the structure has been established (the south-east pillar of St. John's Dome) where consolidation work is now in progress.

It may also be mentioned that once cataloguing is completed of the enormous quantity of written and graphic material detailing the Basilica, together with the data obtained from the present analysis and research work, an invaluable tool will be at our disposal to ensure historical and technical continuity in restoration procedures.

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Investigation of the Roman Aqueduct Structure of Acqui Terme

Recherche concernant la structure de l'aqueduc romain de Acqui Terme

Tragwerksuntersuchung am römischen Äquädukt in Acqui Terme

Vittorio NASCÈ
Prof.
Politecnico di Torino
Torino, Italy

Giuseppe PISTONE
Dr. Eng.
Politecnico di Torino
Torino, Italy

Roberto ROCCATI
Dr. Archit.
Politecnico di Torino
Torino, Italy

SUMMARY

The research concerns the remnants of a Roman aqueduct of stone block masonry at Acqui Terme in Piemont, Italy. Its aim was to acquire exhaustive data on the mechanical behaviour of the aqueduct, to serve as a basis for assessments concerning the safety and preservation of this monument.

RÉSUMÉ

La recherche concerne les vestiges d'un aqueduc romain constitué de blocs de pierre et situé à proximité de la ville piémontaise de Acqui Terme, en Italie. Cette recherche visait à acquérir des informations exhaustives sur les aspects les plus significatifs du comportement mécanique de l'aqueduc, afin de poser les fondements pour les évaluations en matière de sécurité et de conservation de ce monument.

ZUSAMMENFASSUNG

Die Arbeit bezieht sich auf die Reste eines antiken römischen Äquäduktes aus Bruchsteinmauerwerk, das sich in Acqui Terme, im italienischen Piemont, befindet. Ziel ist das Sammeln von Daten hinsichtlich der wichtigsten Aspekte des mechanischen Verhaltens des Äquäduktes als Grundlage für die Beurteilung der Sicherheit und Erhaltung des Monuments.

1. INTRODUCTION

The Roman aqueduct of Acqui Terme was built in the Augustean period to supply water from the Erro stream to the town. At one end, this structure, extending over 14 km, ran about 20 m above ground to bridge the depression created by the Bormida River. This portion of the aqueduct was supported by piers evenly spaced 9 m apart and stone masonry arches; only two portions remain of this part of the aqueduct: one, closer to the town, consists of seven piers and four arches; the other, next to a hill, includes eight, badly deteriorated shorter piers.

The investigation illustrated in this paper was conducted on behalf of the Archaeological Superintendence of the Piedmont region with the aim of acquiring data on the mechanical properties of the original material as a basis for an evaluation as to the monument's safety conditions and state of preservation.

2. COMPOSITION OF THE PIERS AND ARCHES

At the end of the 19th century (1896), when erosion had already damaged severely both the facing and the inner masonry (photo 1), restoration work was started and carried out in several stages, mostly on the arches.

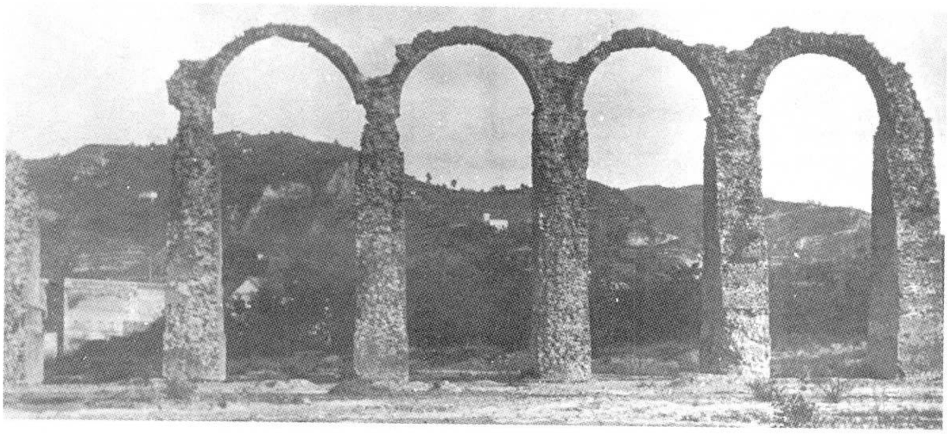


Photo 1 - General view of the aqueduct in 1896.

The composition of the original masonry may therefore be discerned best in isolated piers which underwent but limited repair work (photo 2). The facing layer consists of sandstone blocks, of variable width (25 cm on average) 8 cm thick, squared on the outer surface. Mortar joints, whether horizontal or vertical, have a thickness of between 2 and 3 cm, which is generally seen to increase towards the innermost part of the masonry. The average value of the ratio between the area of the mortar and the total area of the facing layer is about 0.3 (photos 3 and 4). The inner part of the masonry instead is made of sandstone blocks exhibiting irregular edges between the two parallel faces. Narrower and thinner than the facing blocks, all these elements were carefully laid horizontally one at a time. The mortar beds of the inner blocks reveal marked discontinuities and their thickness, on average, is greater than that of the facing blocks: the average value of the ratio between the mortar area and the total area of an inner vertical section, in fact, is about 0.5 (photos 5, 6). In the inner part of the masonry there are many cavities, mostly located in the median portion of the thinner vertical joints. These cavities were produced both by insufficient flow of the mortar from the bottom up as the blocks were laid and by insufficient filling of the upper mortar bed.

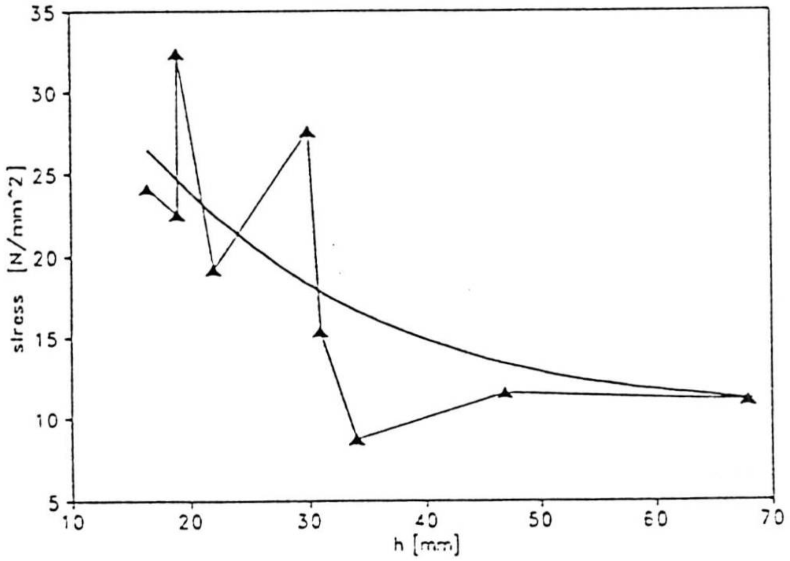


Fig. 1 - Strength as a function of the specimen height.

Photo 2 - Non restored isolated pier.

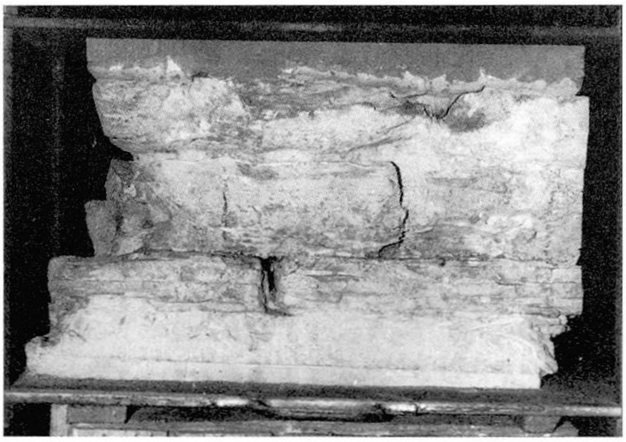


Photo 3 - Sample obtained from facing masonry.

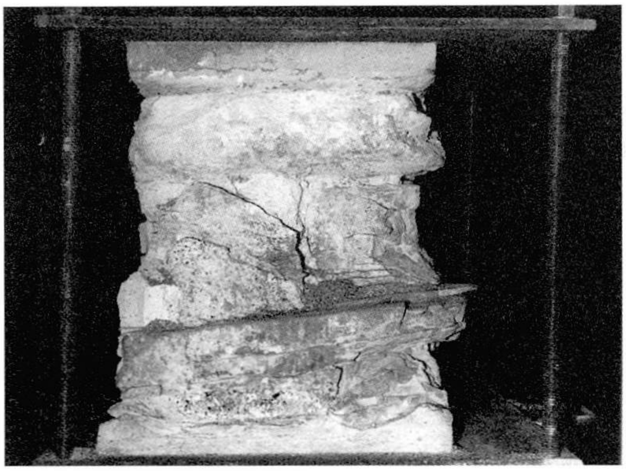


Photo 4 - Sample obtained from facing masonry.

Photo 5 - Sample obtained from inner masonry.

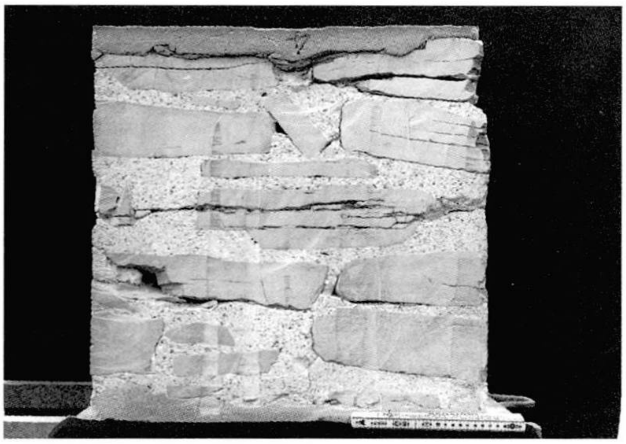


Photo 6 - Sample obtained from inner masonry.

This macro-porosity, already observed by Choisy in Roman masonry walls of this kind [1], was surely associated with the need to make limited use of water in the mortar mix and to improve its strength, which was very high in this particular case; another consideration was not to undermine the stability of the piers at the building stage. In all likelihood, the primary cause of erosion resulting in the monument's deterioration was this macro-porosity associated with freezing and thawing cycles; this was also one of the most significant factors, together with the thickness and discontinuity of the mortar joints, affecting the mechanical behaviour of the masonry as observed throughout the testing programme.

3. TAKING THE SAMPLES

With the agreement of the Piedmont archaeological authorities, three blocks were obtained from the ruins of the first pier in the set of eight without arches adjacent to the hill. Sample selection was performed so as to make sure the specimens would be representative of the overall conditions of the materials. The samples taken from the cladding included two specimens: Nos. 1 and 2, 38 and 50 cm high, respectively, including the concrete headers, with a height-to-width ratio of between 1.5 and 2 (see photos 4, 5). As for the inner part of the masonry, it was deemed necessary to obtain a large test piece which would clearly represent the irregularity of its fabric and be able to average out the singularities in mechanical behaviour entailed by the latter. Specimen No. 3 turned out to be almost cube shaped, measuring 55 x 57 x 56 cm (photos 6, 7). However, the typical behaviour of this masonry attenuates, at least partially, the influence of the pressure plates' transverse containment on the loading surfaces: the height/width ratio, of considerable significance when dealing with homogeneous materials or well laid brickwork, loses its importance when a limited offset of the vertical joints compared to the height of the stone courses promotes the early formation of cracks along such joints and causes the formation of resistant sections on which the influence of the plates is minimal.

In parallel with the taking of samples, cores were also drilled for chemical-physical analyses and mechanical tests on the stones and the mortar.

4. MECHANICAL PROPERTIES OF THE STONES AND THE MORTAR

The samples taken were analysed and tested by A. Frisa Morandini of the Geo-Resources and Territory Department of the Turin Politecnico.
Petrographic characteristics were deduced from microscopic visual examination of thin sections and carbonate measurements (through powder calcimetry).

The stone was classified as "calcareous cement sandstone" with an average carbonate content of 38%. Extensive outcrops of this sandstone, of the Tertiary Basin of Liguria and Piedmont, can be found in the hills surrounding Turin and in the Langhe and Monferrato areas where numerous small quarries were opened in the past for the production of cutting stones for the building industry. The mechanical properties of the stone (whose mean values are listed in Table 1) were obtained by testing to failure Ø 31 mm cylinders with an average height of 75 mm.

	Rock	Mortar
Weight by volume [N/m³]	26,750	18,100
Mono-axial compressive strength [N/mm²]	63	19
Tangent elastic modulus in compression [N/mm²]	11,000	9,900
Secant elastic modulus [N/mm²]	9,500	11,600

Table 1

The values of the tangent and secant elastic moduli were determined for a load corresponding to half the failure load under mono-axial compression.

A comparison between the physical and the mechanical characteristics of the different specimens revealed that the compressive strength of the less compact test pieces having higher porosity and higher soaking coefficient values was about 20% lower than that of the others. Mortar employs a binder consisting of air-hardening lime, with partially hydraulic properties, presumably produced by good baking. The aggregate was a rather course siliceous sand, with up to 10 mm diameter.

The average binder/aggregate ratio is about 1/3. The specimens were seen to contain no gypsum nor any other chemical alteration product and the presence of lichen colonies on the face joints reveals that to this day no chemical physical alteration has taken place in the mortar.

In the mortar, compressive strength and the elastic modulus were determined on cylinders of different heights: strength was seen to decrease with increasing specimen height (fig. 1), this being an aspect of special interest in the case being considered, on account of the sizeable discontinuities present in the mortar beds of the inner masonry, in terms of both thickness and area.

The high values of strength and of the elastic modulus in this Pozzolan free mortar may presumably be ascribed to a favourable aggregate grain size, combined with the binder's partially hydraulic properties, and the thorough working of the mortar in the presence of a low water/binder ratio.

The comparison between the mechanical properties of the mortar and those of the stone revealed a ratio of about 1/3 for compressive failure strength while the elastic modulus of the two materials, under a load corresponding to about 50% the failure load, was almost the same.

5. TESTS ON MASONRY ELEMENTS

Specimen behaviour was studied by applying the load in cycles of increasing intensity, starting from relatively low values. The tests were performed by means of a ball-joint press at the laboratory of the Structural Engineering Department of the Turin Politecnico. More precisely, the test pieces were subjected to three loading cycles of the same intensity and after that the load was increased in steps of between 1 and 2 N/mm². The readings were taken as a function of the load increments imposed. Unloading never fully relieved the pressure on the elements. The instrument used consisted of horizontal and vertical strain gauges: the vertical readings made it possible to detect the deformations taking place in the masonry specimen as a whole (thanks to gauge bases from 20 to over 30 cm long). Specimen faces in contact with the pressure plates had been levelled beforehand with gypsum. Load-displacement and stress-strain diagrams were plotted on the basis of the test results.

During the tests, local failure was seen to occur systematically in small portions of the masonry and various stones located at different places were also seen to fail, through additional shear and bending effects: all these phenomena gave rise to partial load distributions and the reduction of isolate stress peaks.

In the case of sample No. 1, though care had been taken to apply the load to its centre of gravity, a marked compression and bending effect occurred due to the presence of a stiffer and stronger core in one of the corner area. Failure took place at 1180 KN (8.6 N/mm²), during the second loading cycle of the last triplet, at a strain value markedly higher than that observed during the previous cycle where the specimen, however, had reached a load of 1300 KN (9.4 N/mm²).

In specimen No. 2 the compression and bending effects were much smaller, on account of a more regular composition (fig. 2), and failure occurred under a load of 1420 KN (13.1 N/mm²).

The behaviour of specimen No. 3 was relatively uniform, with no sizeable compression and bending effects: a series of diffused micro-cracks appeared rather early over all the faces. At 1000 KN (3.3 N/mm², 40% of the failure load), widespread crack formation was already evident and it kept increasing over the successive cycles, the load level remaining the same (photo 7). This phenomenon continued steadily until failure which occurred at 2400 KN (8.0 N/mm²).

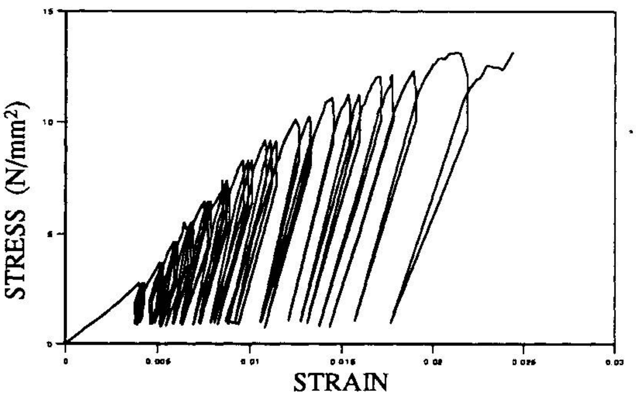


Fig. 2 - Stress-strain curve of strain gauge 7 - specimen 2.

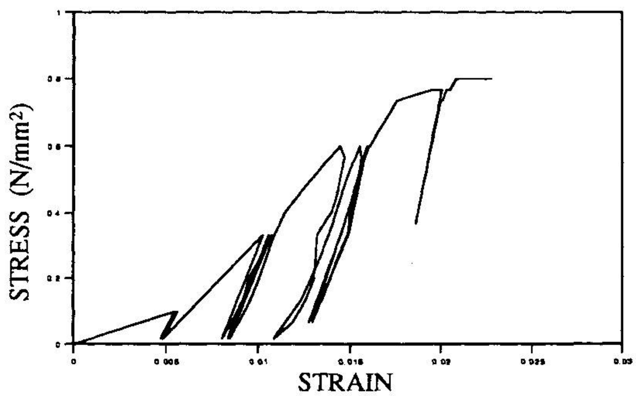
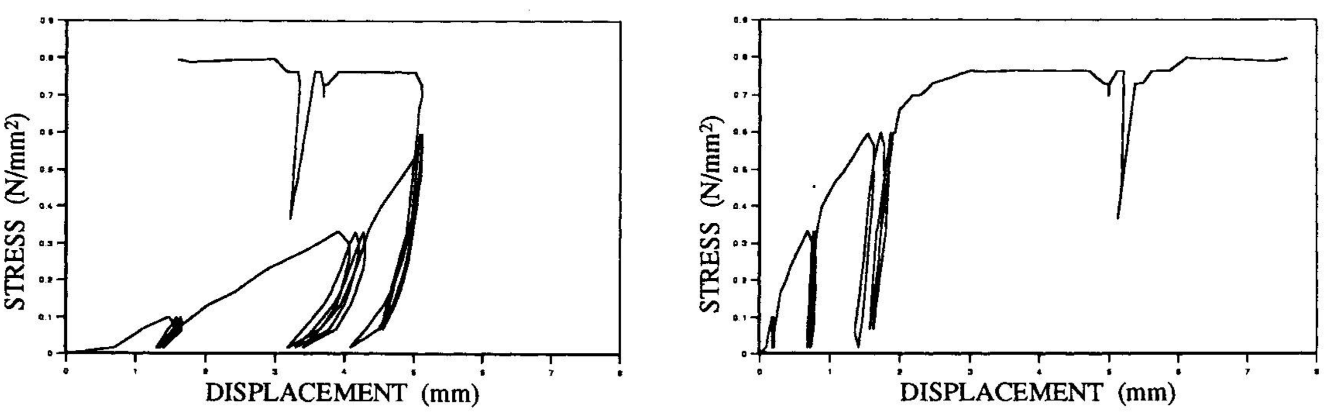


Fig. 3 - Average stress-strain curve - specimen 3.



Figs. 4 - 5 - Stress-strain curve of strain gauge 4 (vertical) and 9 (horizontal) - specimen 3.

The progressive opening of cracks did not prevent the material from exhibiting overall elastic behaviour in the individual loading cycles up to very high stress values, as confirmed by the diagram in fig. 3. This diagram illustrates a typical behaviour which was observed throughout the tests: the initial load increase, for each testing level, was accompanied by high strain values and marked residual strain, while in subsequent cycles (the load being the same) strain was lower and residual strain was virtually negligible.

A comparison between the diagrams plotted from the readings taken by two strain gauges of about the same length on the same face of specimen No. 3 (figs. 4, 5), - a vertical and a horizontal gauge, Nos. 4 and 9, respectively - shows that the strain values are of the same order of magnitude and the horizontal one entails a transverse strain modulus about three times as high as that of the masonry at the elastic stage. Moreover, the downward portion of the curve in fig. 5 reveals a major overall settlement of the specimen under the load, this behaviour closely resembling that of specimen No. 1 already described above (fig. 6). Thus we find that the transverse strains that have been observed are greater than would be expected on the basis of the Poisson modulus of the components, and are independent - even in terms of their direction - of vertical ones in the proximity of failure. Failure involved the masonry specimen as a whole, as shown in photo 13. Strain stabilisation was especially slow at a load level corresponding to 90% of the failure load and required a waiting period of about 20 minutes, this being a confirmation of the major influece exercised by the permanence of the load on the collapse of very irregular masonry.

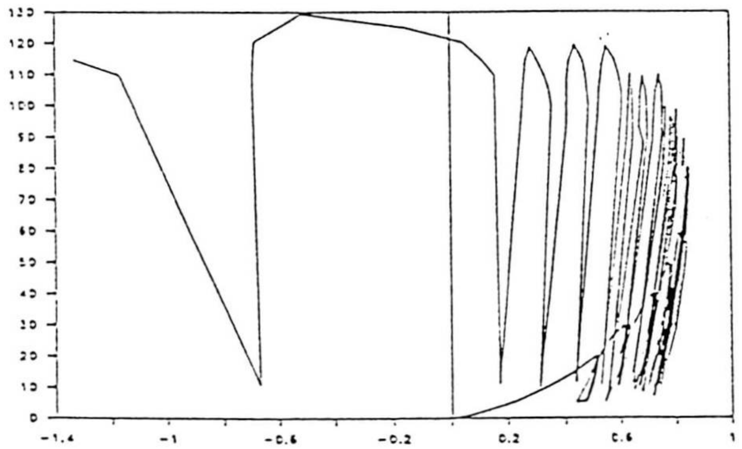


Fig. 6 - Load-displacement curve of strain gauge 5 - specimen 1.

Photo 7 - Collapse of specimen No. 3.

6. CONCLUSIONS

The stones and mortar employed in the original construction of the Acqui aqueduct have very similar deformability, as borne out by the values of the tangent and secant elastic moduli.

The deformability of the masonry is characterised by the fact that the material takes on a markedly elastic behaviour under repeated loading cycles with a pre-determined maximum stress limit, whilst it exhibits an anelastic behaviour each time the load is increased. This phenomenon, which is typical of masonry in general [2], was seen to be very pronounced in this case. We may therefore distinguish two different deformability moduli: one can be inferred from the envelope curve of the maximum values in the stress-strain diagrams, this curve being very close to the one that would be obtained under a steadily increasing load. The central portion of the curve is nearly linear and very long, with a value of between 600 and 800 N/mm^2, which is lower than 1/10 of the values for stone and masonry.

The other modulus characterises the elastic behaviour of the masonry during the loading cycles: it is appreciably constant almost up to failure, and its value is about 3000 N/mm^2 (1/3 that of the components). The ratio between the values of the modulus under a constantly increasing load and the values measured during constant width loading cycles is 1/4. The great difference observed in the deformability of the masonry and that of its components should be ascribed to the state of cracking which occurs in the masonry specimens at the early stage of loading; this isolates blocks and fragments within the specimen which undergo relative rotation and slip during the test; this conclusion is borne out by an examination of the cracks, almost always of variably width due to individual fragment rotation, and by the strain values read on the horizontal gauges, all of them much higher than would be observed in an elastic material.

In addition to being associated with size effects in the specimens, the noticeable decrease in the strength of the masonry compared to the strength of mortar and stone is caused primarily by the fact that the stone material is not homogeneous with limited overlapping, the influence of these factors being greater in the inner part of the masonry: the average stress value obtained by dividing the applied load by the specimen's area, in fact, does not account for the great differences in stress existing in the specimen at different points of the cross-section as a function of variations in local rigidity; it is precisely these high local stress values that result in the formation of circumscribed cracks and the material becoming plastic, with the consequent stress redistribution (a phenomenon that can be clearly observed in figs. 5 and 7), in addition to resulting in a reduction in the material's overall strength.

This also accounts for the lower strength of the inner masonry compared to the facing layers, though the latter were seen to be greatly influenced by their small size and by the composition of the facing elements.

7. REFERENCES

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2. PISTONE G., ROCCATI R., Mechanical Characteristics of Masonry Rebuilt with Ancient Bricks and Fresh Mortars, 9th International Brick/Block Masonry Conference, Berlin, Oct. 1991.

Strengthening of the Parish Church in Farigliano

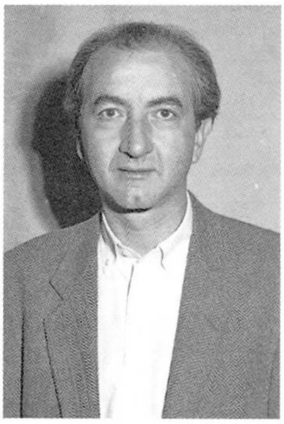
Consolidation de l'église paroissiale de Farigliano

Verstärkung der Pfarrkirche in Farigliano

Renato LANCELOTTA
Prof.
Politechnico di Torino
Torino, Italy

Giuseppe PISTONE
Dr. Eng.
Politechnico di Torino
Torino, Italy

Renzo ROVERE
Dr. Eng.
Public Works Ministry
Roma, Italy



SUMMARY

The foundations of the Parish Church of Farigliano are dug in soft, recently deposited materials with a high degree of compressibility. This explains the precarious behaviour exhibited in recent years by the static conditions of the structure. This report describes the urgent measures taken to protect the building, including the installation of a hydraulic levelling system.

RÉSUMÉ

Les fondations de l'église paroissiale de Farigliano reposent sur un sol souple constitué de dépots récents et ayant un degré élevé de compressibilité. Cela explique l'état précaire des conditions statiques de la structure au cours de ces dernières années. Ce rapport décrit les mesures d'urgence prises afin de protéger l'édifice, y compris l'installation d'un système hydraulique de mise à niveau.

ZUSAMMENFASSUNG

Das Fundament der Pfarrkirche in Farigliano wurde in einen weichen, aufgeschütteten Boden mit hoher Kompressibilität gegraben. Das erklärt das prekäre statische Verhalten des Gebäudes in den letzten Jahren. Die vorliegende Abhandlung beschreibt die notwendigen Massnahmen, um das Gebäude zu schützen, einschliesslich eines hydraulischen Hebungssystems.

1. INTRODUCTION

On February 29 1887, the ancient parish church of Farigliano was severely damaged by an earthquake and its stability was irremediably impaired. In his report of December 1887 to the Parish Priest, Ing. Chiechio, the designer responsible for static analysis, recommended the construction of a new church precisely on account of the exceedingly high costs to be incurred for the consolidation of the ancient building.

Having selected the site for the new church, it was immediately discovered that the soil was hardly suitable to carry the considerable loads to be transferred by the building (see the report of October 18, 1888, parish archives). As a result, under the guidance of In.g Chiechio, the entire area was excavated to a depth of 5 m, and 5 m long durmast wood piles were driven into the ground to correspond with the impost faces of the foundations.

From the archive documents, it can be seen that the site had been selected for various reasons, none them, unfortunately, of a technical nature.

The church was erected very rapidly and was completed in 1890 (fig. 1).

The building is fashioned in the shape of a Latin cross with a three-nave front body and a polygonal apse; at the confluence of the transept and the central nave rises the presbytery, surmounted by an octagonal drum covered by a dome (fig. 2).

Under the entire church floor there is a basement obtained from the digs carried out initially to improve the soil.

From the structural standpoint, the building has outer bearing walls of masonry made of pebbles and air-hardening lime, interrupted at regular intervals by horizontal brick courses, spaced about 80 cm apart, featuring excellent workmanship and strength.

Thick, continuous brick walls also define the entire presbytery area inside, while the naves are delimited by thin marble columns surmounted by arches and by a longitudinal masonry wall which defines the central nave. The columns continue down into the basement with square section brick piers.

The roofing of the basement consists of 12 cm thick triangular vaults and arches; "in folio" vaults cover the side aisles, while the central nave is topped by a wooden lacunar roofing.

These elements add up to a box section possessing great stiffness in the portion comprising the apse and transept, while the front portion of the building above ground is rather weak on account of the poor horizontal rigidity offered by the slender columns, the thin vaults and the lacunar covering.

2. STRUCTURAL DAMAGE

From the very outset, diffused fissures appeared in the thin vaults of the side naves, in addition to cracks opening in different points of the curtain walls.

In 1988, the settlements of the structure - which were already quite evident - were compounded by the flooding of the basement.

Fig. 1 - View of the church.

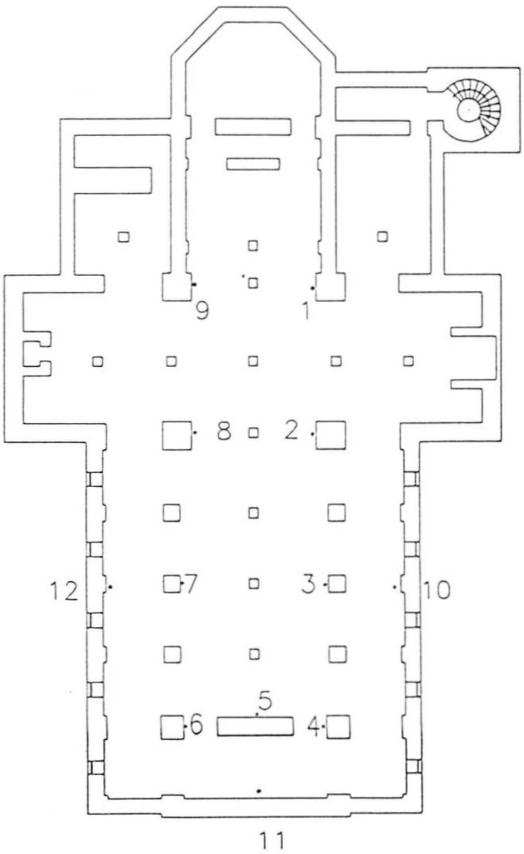


Fig. 2 - Plan of the basement showing the arrangement of the hydraulic levelling system measuring points.

Fig. 3 - Fissures in the thin vaults of the side aisles.

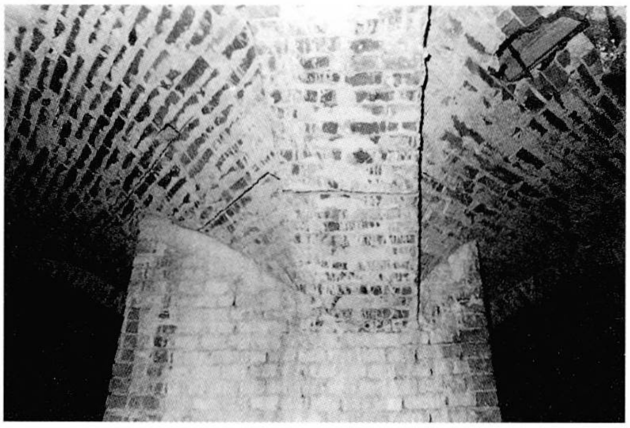


Fig. 4 - Fissures in the the 12 cm thick vaults on the underground level.

Fig. 5 - Fissures in the the wall defining the central nave.

At this point the overall picture was as follows:

- systematic fissures in the thin vaults covering the side aisles, of ancient origin, but probably made worse by recent damages; their width came to over 4 cm and their position in the building's plan revealed a markedly symmetrical arrangement; in particular, their 45° degree slant isolated, in each vault, a central part which was obviously in compression, as borne out by the relative slip of the crack edges and its enhanced curvature (fig. 3).
- a series of lesions, of recent appearance, involving the vaults and arches in the basement. The most important cracks were located all around the four main piers at the confluence of the transept and the central nave, and were as wide as 19 mm at the arches (fig. 4);
- a few major cracks affecting the church proper, in the wall at the top of the columns defining the central nave (fig. 5);
- full height lesions of more recent formation (August 1989) in the right-hand side lateral wall of the presbytery, where the transept meets the nave, and in the facade wall.

3. SOIL FOUNDATION CONDITIONS

From the outset, the pattern of lesions and relative settlements observed in the building structures was ascribed to the conditions of the subsoil. From the geological viewpoint, the foundation soil is on the edge of an alluvial terrace overlying the stiff clay of the nearby valley (of Miocene origin). Part of this terrace is of a colluvial nature, and is made of soft materials with great compressibility.

The data collected from 4 bore-holes have shown that the upper stratum of this recent silty clay of low to medium plasticity has a thickness of 17 m. Because of the chaotic structure of this foundation soil and on account of the scarce significance of laboratory tests performed on small samples, in situ compressibility has been evaluated through a back-analysis of the settlements observed in a lower level wall. This has yielded an operational value of soil stiffness as low as 1.5 MPa, which explains the large values of the cracks, fissures and settlements observed in the masonry structures.

Fig. 6 - Collegamento tra basamento del pilastro e testa dei micropali.

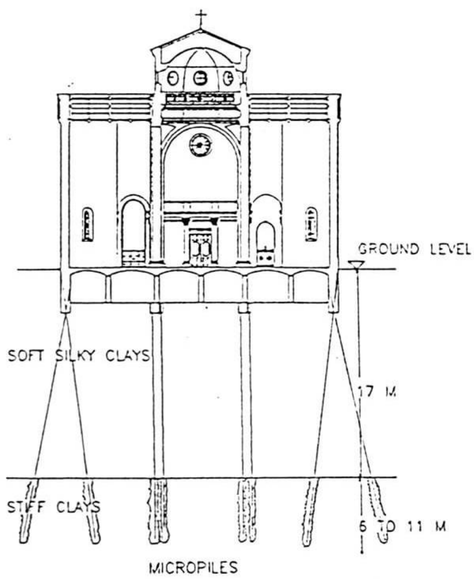


Fig. 7 - Cross-section at the transept showing the stratigraphic composition of the soil and micro-piles.

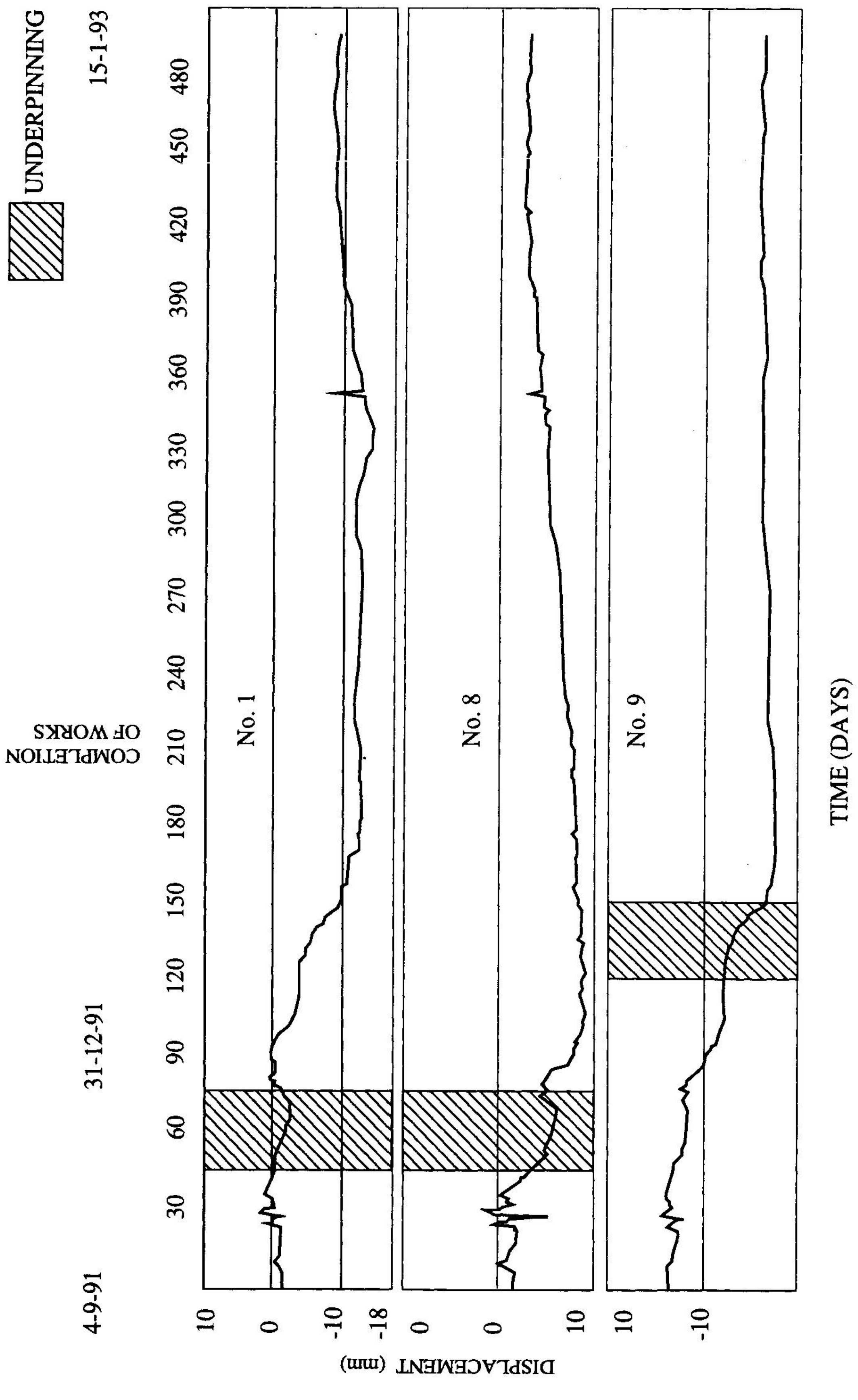
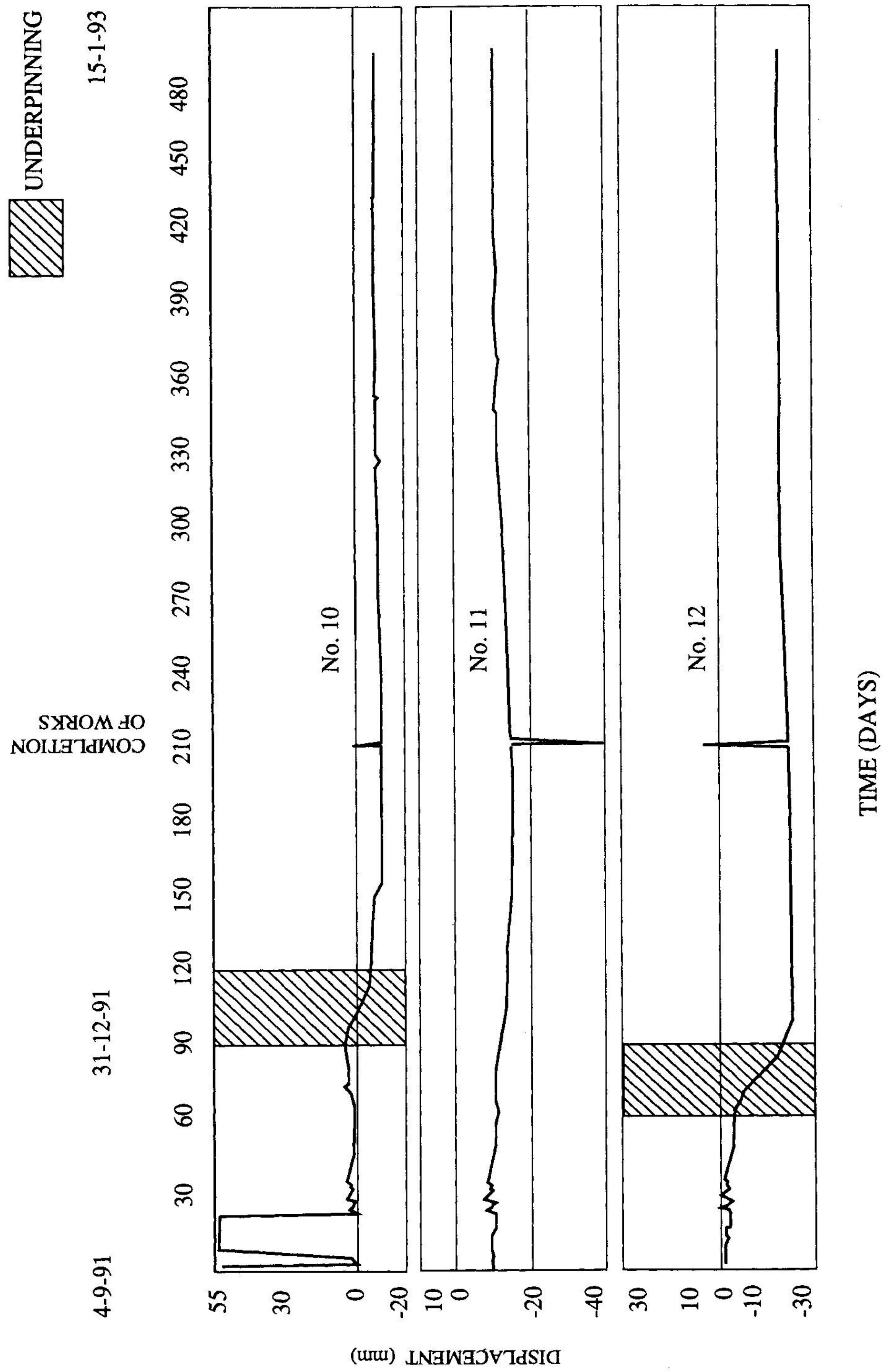

UNDERPINNING
COMPLETION OF WORKS
4-9-91
31-12-91
15-1-93
No. 1
No. 8
No. 9
TIME (DAYS)
DISPLACEMENT (mm)
30
60
90
120
150
180
210
240
270
300
330
360
390
420
450
480
10
0
-10
-18
10
-10

Fig. 8



UNDERPINNING
4-9-91
31-12-91
15-1-93
COMPLETION OF WORKS
No. 10
No. 11
No. 12
TIME (DAYS)
DISPLACEMENT (mm)
30
60
90
120
150
180
210
240
270
300
330
360
390
420
450
480

Fig. 9

In this situation, the first major protective measure taken was the underpinning of the foundations by means of micro-piles. To prevent the formation of undesirable lateral loads as would be produced by down-drag if the upper layer settled more than the micro-piles, raking piles were not used and the vertical piles were connected to the masonry foundations through reinforced concrete slabs (figs. 6 and 7).

The load applied to each of these micro-piles ranges from 130 to 400 KN and the negative skin friction varies between 120 and 200 KN.

According to these loads, the bearing length in the lower stiff-clay ranges from 6 to 11 m. In addition, in view of the large difference in settlements necessary to attain the desired resistance, only skin friction has been taken into account, neglecting base resistance, so as to prevent the occurrence of further movements of the structure.

4. MONITORING AND RELIABILITY EVALUATION OF THE PROTECTIVE MEASURES

Systematic measurements of the vertical settlements of the columns and lateral walls started in June 1991.

Figure 2 shows the arrangement of the hydraulic levelling system and the most significant diagrams are shown in figs. 8 and 9.

Due to the re-activation of the movements in 1988, the first part of these diagrams clearly shows that the settlements were still evolving in 1991.

The time periods during which underpinning work was underway are also indicated by shaded portions of the diagrams and it can be seen that after the work was completed the structure did not suffer from any further movement.

A few additional remarks should be made: first, in order to prevent down-drag, the bearing length of the micro-piles was limited to the lower part of the stiff clay stratum. This required a careful control of the injection pressure in the upper part, so as to reduce soil disturbance and prevent any increase in pore water pressure with subsequent consolidation.

Secondly, the settlement pattern shown here has to be understood in terms of relative settlements, because the values given are referred to point 11- a fixed point in the facade wall which certainly underwent minimal movements. This explains, in some cases (see points 1 and 8, for example), the unusual apparent reduction in the extent of the settlements with time.

ACKNOWLEDGEMENTS

The Authors wish to express their gratitude to Dario Zorgniotti who helped in the preparation of the drawings.

Damage Detection and Repair Control of Marble Elements

Détection des défauts et contrôle de la réparation sur des éléments en marbre

Schadenserfassung und Reparaturkontrolle bei Bauelementen aus Marmor

Angelo DI TOMMASO
Professor
Univ. of Bologna
Bologna, Italy

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

Francesco CIANFRONE
Techn. Dir.
Soc. Ital. Contr. e Collaudi
Padova, Italy

SUMMARY

This paper presents an experimental study carried out using the microseismic method on blocks of marble in which defects such as cracks and cavities were created artificially. The purpose is to evaluate the ability of the method to detect internal defects in stone blocks, and to judge the effectiveness of resin repairs.

RÉSUMÉ

Cet exposé présente une recherche expérimentale par la méthode microséismique effectuée sur des blocs de marbre contenant des vides et des fissures. L'objectif de la recherche était d'évaluer dans quelle mesure cette méthode permet d'identifier les défauts internes et de juger de l'efficacité d'une réparation effectuée avec de la résine.

ZUSAMMENFASSUNG

Der Bericht präsentiert eine experimentelle, nach der mikroseismischen Methode vorgenommene Forschungsarbeit an Marmorblöcken, die künstlich eingebrachte Fehler wie Risse oder Hohlräume aufwiesen. Ziel der Forschung war es festzustellen, inwiefern mit dieser Methode Fehler im Innern von Steinblöcken entdeckt und Reparaturen mit Kunstharz beurteilt werden können.

1. INTRODUCTION

The repair of stone elements in historic structures requires a complete knowledge of the type of damage. This is an essential premise in order to plan the intervention accurately. A simple visual analysis is normally not capable of providing reliable indications. In addition, once the repair has been done by means of resin injections, it is important to check the effectiveness of the repair. In both cases, it is necessary to have a method capable of identifying internal cracks, hollows and discontinuous areas.

Methods based on the propagation of elastic waves are undergoing a considerable evolution, also due to the availability of processing methods based on the use of appropriate software, such as for sonic tomography [1] and the impact echo method [2,3]. The use of radiography techniques requires extremely high power. The application of radar to this sector is still in the experimental stage.

This paper presents an application of microseismic analysis. These are a useful instrument for this type of analysis, but require the use of instrumentation, procedures and interpretation criteria suited to the purpose [4,5,6,7,8,9]. The paper presents some results of an experimental study carried out on marble blocks with the following aims:

- to evaluate the sensitivity of the method in identifying defects, by simulating defects of varying size and repeating the measurements in the same condition each time;
- to estimate the size of the defects based on velocity and attenuation measurements;
- to evaluate the adequacy of the method in checking the effectiveness of a resin injection, by comparing measurements taken on the same trajectories in the following situations: integer block, block with defect, repaired block.
- in the case of cracked stone elements, to use the appropriate experimental methods to evaluate the extension of the area in which the material has undergone damage.

2. EXPERIMENTAL PROGRAM

2.1. Characteristics of the samples

A quasi-homogeneous and not stratified Greek marble was employed, with grain between 2 and 3 mm. A block 34x34x20 cm^3 was utilized for the tests, four 4x4x16 cm^3 prisms were used to determine the mechanical properties of the material.

2.2. Material specifications

The tests run on the prisms provided the following average values:

Uniaxial cubic compressive strength	σ_{rc}	= 179	N/mm^2
Flexural strength	σ_{rf}	= 12.9	N/mm^2
Young's secant modulus (0-50 N/mm^2)	E_c	= 66250	N/mm^2
Volumic mass	m	= 2898	kg/m^3

The pulse velocity of the P waves in the tests was rather variable depending on the anisotropy of the material, and falls between 4000 and 5100 m/s.

2.3. Instrumentation

A microseismic analyzer was employed, the most important details of which are:

Piezoelectric transducers, either emitting or receiving, with a flat active surface (emitting or receiving) with a diameter of 30 mm .

– ***Two on-line commutators***, allowing the operator to activate any emitting or receiving probe, among fixed multiple transducers applied to the stone specimen.

- ***Operator-adjustable pulse repetition rate***, avoiding reverberation and resonance giving rise to disturbed measurements or to the impossibility of carrying them out.

- ***Pulse frequency***: this was set at 70 KHz, being the resonance frequency of the emitting and receiving probes, tuned and applied to the specimen.

- ***Pulse energy:*** the instrumentation was designed to power the emitting probes at various energy levels from 0.25 μJ to 50 μJ, according to the absorbing characteristics of the specimen and trajectory lengths. The emitting probes were powered respectively with 2.5 μJ for the smaller specimens (prisms) and 250 μJ for the larger ones (blocks).

- ***Amplification of received signal:*** amplification varying between -16 and +80 dB, in 1-dB steps, was used for the electric signal of the receiving probes. The amplification value was both digitally displayed to the operator and available as electric output for recording equipment during measurements.

- ***Transit time and intensity variations:*** the CRT (Cathode Ray Tube) of an oscilloscope was used for continual display of time function vibrations, as received by the probe and suitably amplified.

2.4. Experimental procedures.

The program of the experiment proceeded according to the following stages:
Initially the probes (three emitting and three receiving) were glued to a block in the positions indicated in figure 1.

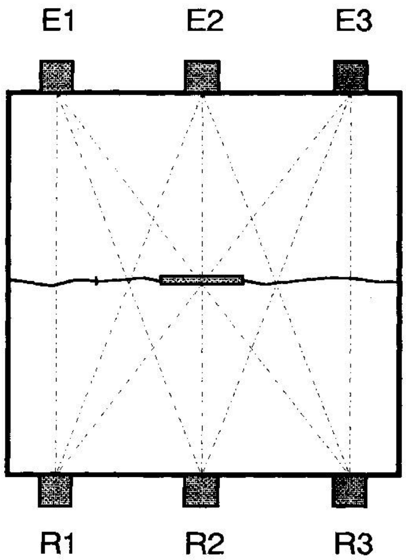


Fig. 1 Marble block and paths

The probes were glued on the block surface during the experiment in order not to introduce disturbing parameters due to repeated removal and repositioning of probes and thus to avoid variations in coupling points, acoustic coupling between probes and material, electric connections, etc. Using two specially-built commutators, each of the emitting probes was then connected to each of the receivers, and the microseismic parameters were determined (P-wave propagation velocity and attenuation) along the paths indicated in figure 1. An average tension of 12 N/mm^2 was applied to the sample.

The block was cracked horizontally, rotating it at 90°, and a splitting test was performed; the sample was then replaced in its initial position (see fig. 2). The same pressure was then applied once again, and the measurements taken along the same trajectories.

Without moving the lower part of the block, a circular cavity 20 cm in diameter was then created in the upper part by milling (see fig. 3). After replacing the block in its initial position, the pressure of 12 N/mm^2 was applied again, taking into account the reduction of the compressed area, and the measurements were repeated.

This operation was repeated several times, progressively increasing the diameter of the cavity, and always taking the microseismic measurements with a constant average pressure.

Finally, the crack and the cavity were sealed by applying a technique used for repairing damaged stone elements, consisting of the injection of epoxy resin after spackling the edges (see figs. 4 and 5). Once this procedure was completed, the measurements were repeated in the same conditions.

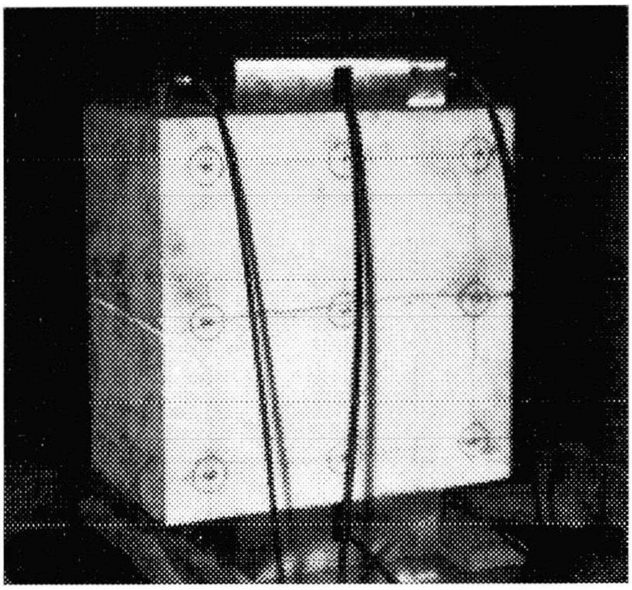


Fig. 2 Marble block with glued probes

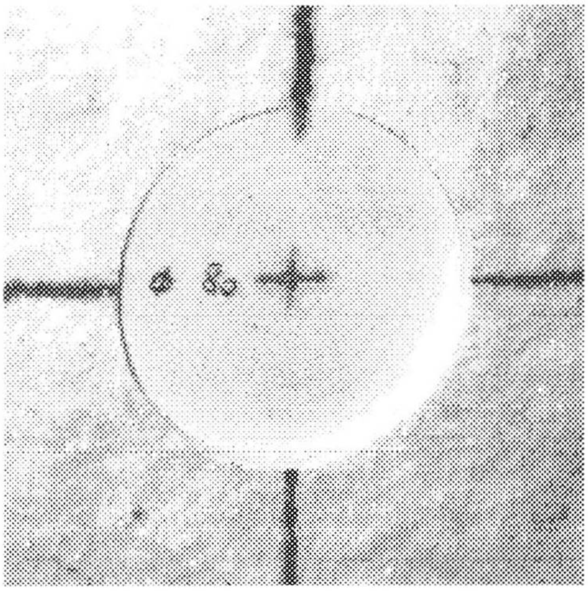


Fig. 3 Milling a circular cavity

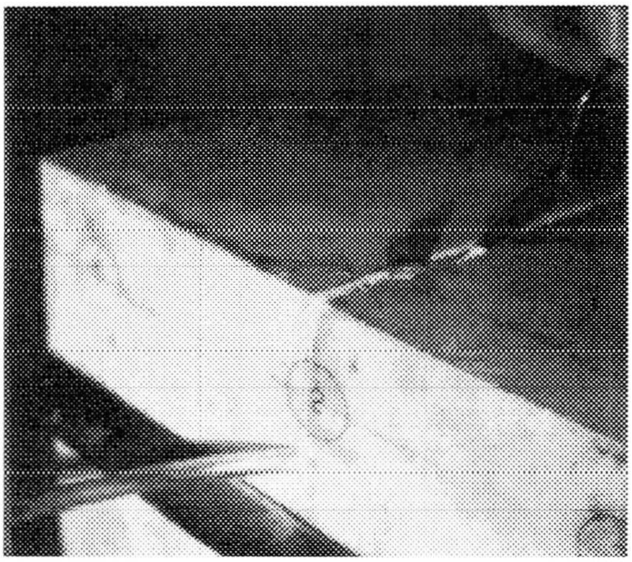


Fig. 4 Preparing the block for repair

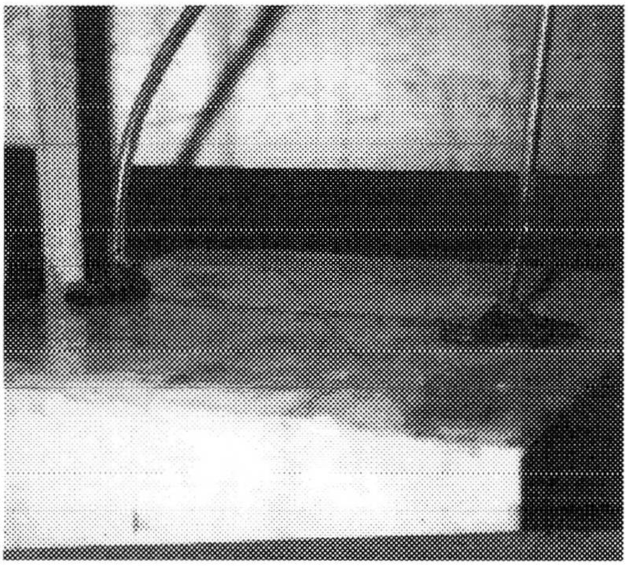


Fig. 5 Epoxy resin injection

2.5. Presentation of the results

The results obtained are shown in tables 1 and 2, and in graphic form in figures 6 and 7. In the latter, the data regarding the measurements taken along equivalent paths have been grouped together and averaged.

A: vertical path E2-R2, which intersects the cavity;

B: diagonal paths crossing the cavity (E1-R3, E3-R1);

C: diagonal paths that do not cross the cavity directly (E1-R2, E2-R1, E2-R3, E3-R2).

Cavity	Path								
Diam.(mm)	E1-R1	E1-R2	E1-R3	E2-R1	E2-R2	E2-R3	E3-R1	E3-R2	E3-R3
Integer	4625	4619	4516	4847	4802	4661	4822	4841	4695
0	4593	4578	4478	4835	4768	4613	4800	4777	4631
20	4600	4584	4478	4841	4768	4613	4795	4777	4631
40	4593	4584	4474	4836	4748	4619	4779	4777	4637
60	4600	4584	4450	4835	4715	4631	4758	4790	4637
80	4606	4590	4409	4828	4631	4625	4716	4803	4619
100	4587	4561	4386	4796	4569	4625	4649	4809	4631
120	4575	4538	4337	4758	4508	4596	4589	4777	4581
140	4508	4505	4268	4746	4397	4527	4564	4727	4544
155	4508	4477	4238	4715	4307	4466	4521	4691	4550
Repaired	4566	4523	4393	4709	4679	4523	4763	4764	4659

Table 1 Pulse Velocity (m/s)

Cavity	Path								
Diam. (mm)	E1-R1	E1-R2	E1-R3	E2-R1	E2-R2	E2-R3	E3-R1	E3-R2	E3-R3
Integer	32.5	37.5	45.0	40.5	32.0	40.0	47.5	36.0	33.5
0	35.5	38.0	46.0	42.0	33.5	41.0	49.0	36.5	37.5
20	35.5	38.0	46.0	42.0	34.0	41.5	49.0	37.0	38.0
40	35.5	38.0	46.0	42.0	34.0	41.5	49.5	37.0	38.0
60	36.0	39.0	46.0	42.0	35.0	42.0	49.0	38.0	38.0
80	36.0	40.0	46.0	42.5	36.0	44.0	49.5	41.0	38.5
100	36.0	42.0	47.0	43.5	37.0	44.5	49.5	41.5	38.0
120	36.0	43.0	48.0	44.0	40.0	44.0	61.0	42.0	38.0
140	35.5	46.5	67.0	46.0	41.5	48.0	66.5	43.5	38.0
155	35.5	48.5	70.9	47.0	42.5	52.0	70.9	45.0	38.5
Repaired	32.0	42.5	65.5	37.5	43.0	54.0	64.0	41.5	34.0

Table 2 Attenuation (dB)

Along the path A, the velocity in the cracked block is slightly lower than that in the integer block, and the attenuation slightly greater. This is due to the applied pressure, which creates good contact between the edges of the crack. In the presence of the cavity, the velocity decreases as the diameter increases, more so for diameters greater than 60 mm. The total increase is approximately 10%. The attenuation increases progressively, with a total increase of approximately 10 dB. After repair, the velocity returns to a value quite similar to that of the integer block, while this is not the case for attenuation, which remains very near to that found with the largest cavity surface area.

The paths B show a similar velocity pattern to that of the path A. For attenuation, the pattern shows a certain anomaly, due to the formation of a vertical crack.

The paths C do not intersect the cavity until the diameter exceeds 100 mm. Beyond this value, the velocity graph shows a limited decrease. Attenuation instead shows the defect from the very beginning, increasing progressively.

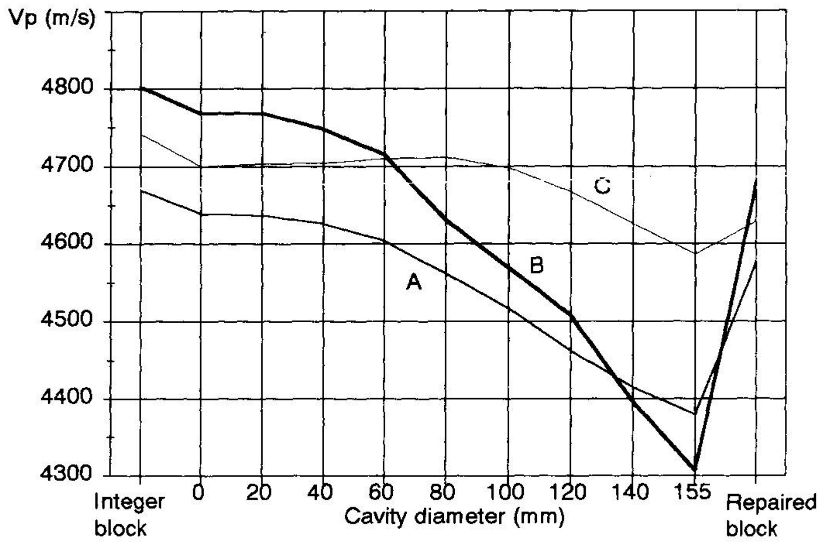


Fig. 6 Pulse velocity along different paths

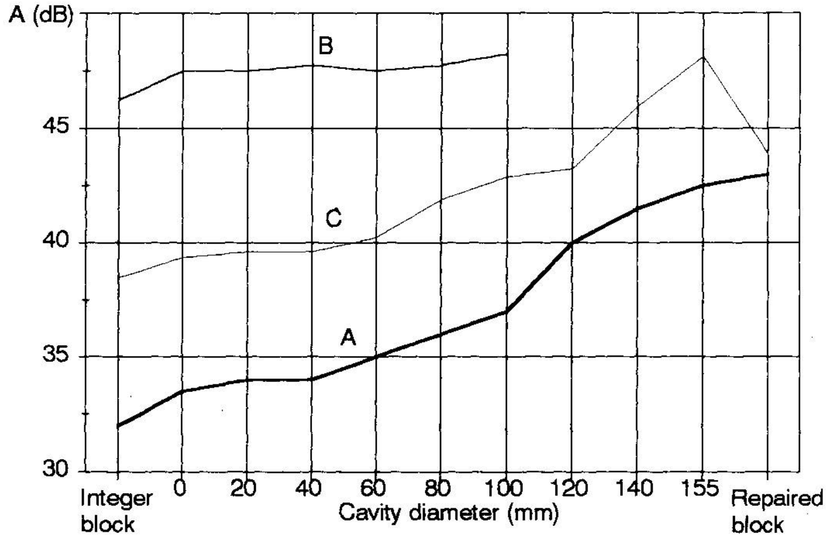


Fig. 7 Attenuation along different paths

3. INTERPRETATION OF THE RESULTS

The graphs in figures 6 and 7 show a different response by the two parameters *velocity* and *attenuation* according to the presence and size of the internal cavity. The cavity reflects a part of the energy, and thus causes an increase in attenuation even if the defect is not directly on the line joining the two probes. The transit time, and thus the apparent velocity, show instead a change only if the joining path crosses the defect. Variations in attenuation may therefore indicate the presence of defects, while in order to determine the position and dimensions it is more appropriate to perform a series of velocity tests.
The following formula is often used to evaluate the size of a defect:

$$d = L \cdot \sqrt{\left(\frac{\Delta t_1}{\Delta t_2}\right)^2 - 1}$$ where:

d = diameter of the defect;
Δt_1 = transit time along the trajectory containing the defect;
Δt_2 = transit time along a trajectory of the same length, in the integer material;
L = distance between the probes.

Based on changes in the transit time along the trajectory E2-R2, the known diameters are compared to the calculated diameters using the above formula. The graph in figure 8 indicates that the estimate becomes reliable beyond 80 mm in diameter.

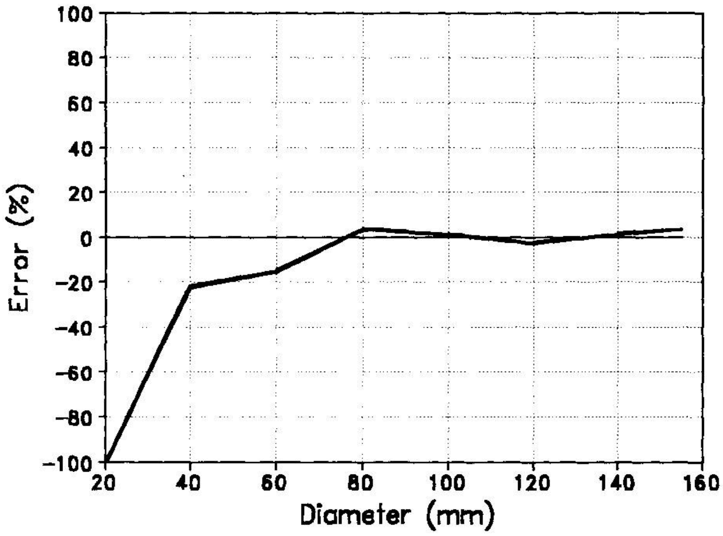


Fig. 8 Error in the evaluation of the diameter

Another purpose of the study was to examine the ability of the method to determine whether a resin filling has been successful. When this technique is used to repair internal cracks or cavities in blocks of stone or marble, it may occur that the filling is incomplete and air bubbles remain, or that the resin separates from the edges of the hole while hardening. In the case in question, the filling was performed correctly, as was subsequently verified by dividing the two parts of the block along the adhesive surface. By examining the graphs in figure 6, we note that in the repaired block the velocity returns very close to the values of the integer block. This would not have been the case if internal cavities were still present, due to a faulty filling . The *pulse velocity* parameter is therefore capable of indicating that the material is once again continuous. The graphs in Figure 7 indicate instead that the filling, even when performed correctly, does not return the attenuation to the value found in the integer block, probably because the resin, which has a different acoustic impedance from that of marble, still causes a considerable part of the energy emitted to be reflected. The *attenuation* parameter therefore is not useful to this purpose.

4. CONCLUSIONS

The study carried out leads to the following conclusions:

- the presence of internal defects is revealed by variations in velocity and, especially, in attenuation;
- both types of measurements are useful in determining the dimensions and position of the defects;
- for evaluating the success of a resin filling, velocity measurements seem more effective.

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Posters - Session 2

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Consequences of Small Earthquakes on the Artistic Heritage

Conséquences de faibles séismes sur l'héritage artistique

Auswirkungen schwacher Erdbeben auf das Kunsterbe

Salvatore D'AGOSTINO
Prof.
Univ. Federico II
Naples, Italy

Giorgio FRUNZIO
Prof.
Univ. Federico II
Naples, Italy

The methodical study of defence from seismic risk of artistic heritage has considerably spreaded in the last years, increasing researchers' interest about problems concerning monumental building.

In spite of this diffused interest there are not scientific and methodical contributes examining the problem of work of art safety. The only research to consider as a specific reference point in this field can be that one performed at Getty Museum of Malibu [1,2]: in Italy, a country full of works of art and with a large part of its territory exposed to seismic risk, there are few researchers interested in the subject [3,6,7].

At last, there are not national or international Codes imposing behaviour to obtain sure results.

As a first step it is obviously very important to classify the wide range of objects according to their form, dimensions and way of exhibition.

The general purpose of this research program .is to give indications suitable for a great part of the situations that can occur in a Museum. This paper, as a first step, examines the single object as a rigid body simply supported on the main structure (leaving out of account the filter' s effect due to the action of structure on the show-case).

Based on the results of precedent studies concerning a generic body and the ones mentioned about the specifity of the objects of art [4,6], this paper reports the results of a set of experiences performed with the aim of establishing the variableness of the effects with the variation of friction conditions between the objects and the fixed floor, the object' s frequency and the increase of dissipated energy when there are oscillations and collisions between the objects and the show-case.

These experimental investigations are completely originals. They have been made using reproductions of vases and small statues supported on a plate oscillating with sinusoidal law.

Experiences have attested the *theorical* results obtained and they march side by side with that ones made in Japan on parallelepipeds.

Different material interposed allowed to obtain the expected results about *sliding and tipping,* as to measure the effective displacements with frequencies and period of simulation near to the real one.

Effective displacements are very important if the object's sliding is established as a desirable situation. In this case it is necessary to evaluate possible relative displacements in order to avoid incidental collisions.

Among different hypotesis made, it is very interesting the combination obtained increasing dimensions of the base by applying a plexiglass pedestal and controlling the friction between object and show case interposing a film of plastic material.

This study has been carried out contacting some Directors of Italian Museums, periodically subjected to sismic events of different intensity. Every one agreed about the necessity of carefully examining these themes and extending them to all the classes of objects, not only the simply supported ones, because there are no indications and the few intervents made depended on the sensibility of experts.

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Automatic Monitoring of Buildings Deterioration

Surveillance automatique de la détérioration de bâtiments

Automatische Überwachung der Schadensbildung an Bauten

Dario ALMESBERGER
Eng.

Antonio RIZZO
Eng.

Federico FABBRO
Arch.

Milenko TONICICH
Eng.

SER.CO.TEC
Trieste, Italy

SUMMARY

The time control of structural failures should be accomplished by monitoring systems which can provide data in real time. Using such a system we are able to follow the evolution of structural cracking , thermohygrometric conditions, structural yielding and other sources of material and structural deterioration occuring within a time domain.
The data acquisition unit connected to the measuring transducers is based on an intelligent system endowed with a programmable microprocessor which received analogic signals transformes into digital values. This unit, using a modem, submits the data by phone to a central control point where the received data are read and processed by a personal computer. The central control point can be located in a designer office or any other place where monitoring data are required. This acquisition data system is fully automatic permitting remote control of data readings. The system, once installed, permits monitoring continuously in time without staff assistance at the control point both for receiving and processing of data.
Autors examine the real possibility of monitoring today and present the last significant applications they made in Italy.

1. INTRODUCTION

Step 1. Find the Deterioration

In practise, this apparently obvious and simple statement may present a very subtle problem when defects are not visible. For example, timbers and timber pilling can be damaged by insects or other organisms, virtually to the point of collapse, without exhibiting any external evidence; corrosion of steel can be difficult to detect because it occurs, principally, in the most inaccessible parts of the structure.

Step 2. Determining the Cause

This is, by far, the most difficult and important step. There are no rules or procedures for determining the cause or causes of deterioration, each case is an individual problem. However the analysis of cracks, the appearance of the surface of concrete, the inspection of timber and steel structures, furnishes an idea for further investigations. It is important to study the structure in bad weather as well as in good. Also it is important to investigate the problem sufficiently deeply to discover any hidden or latent defect.

Step 3. Evaluating the Strength

Usually, the building being investigated is in active use, and it is necessary to determine, as quickly as possible, if it is safe to continue to use it, or if the facility should be restricted to some less severe usage.

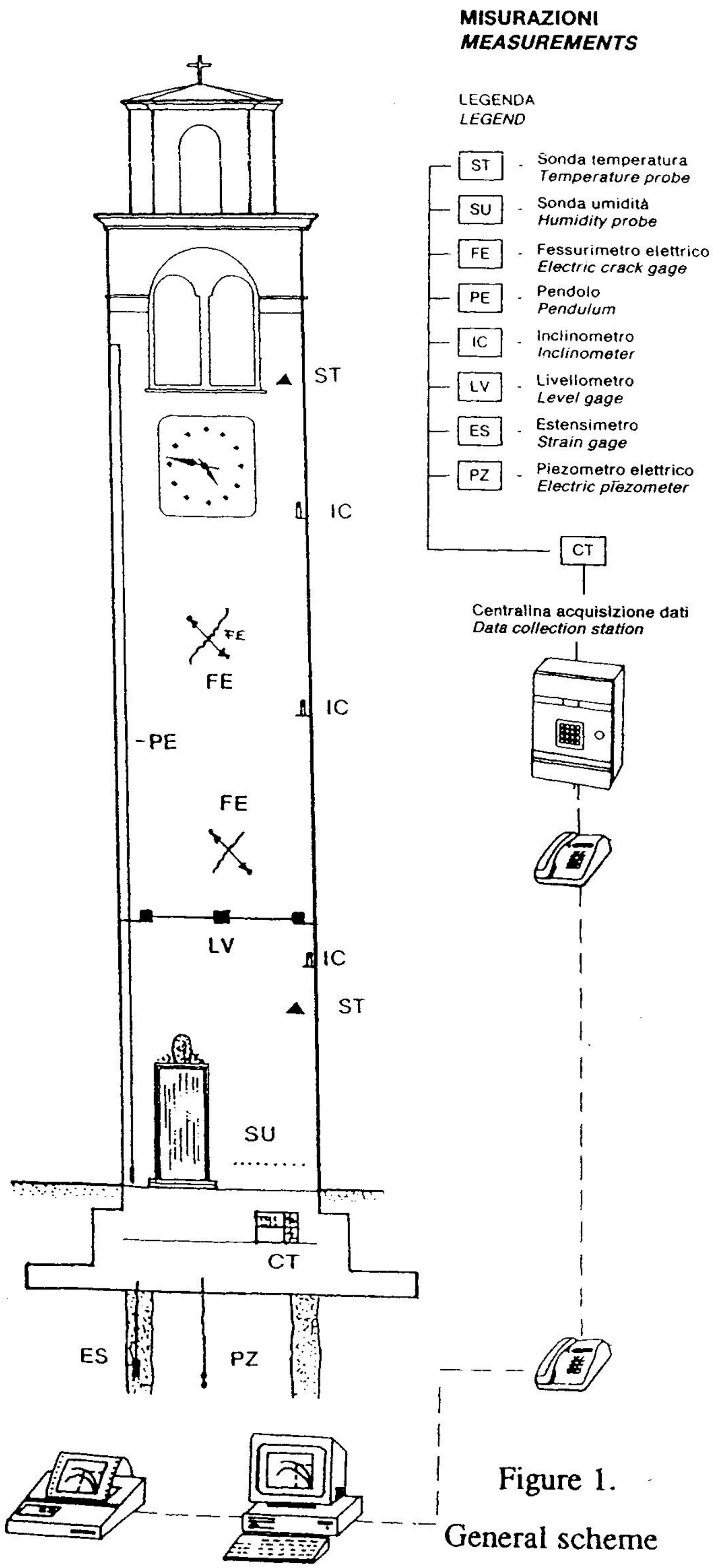


Figure 1.
General scheme

All the difficulties encountered in the steps for detection and diagnostic can obtain a powerful aid by monitoring the main parameters of the structure with non destructive techniques.

The modern technique and technology of measurements made the controls and monitoring easy. Electrical transducers transform every important parameter in numeric digital signal, data-loggers can retain for further manipulation, computers and suitable software elaborate data in real time, modems diffuse every information or data requested all over the world.

This acquisition data system is fully automatic permitting remote control of data readings.

The system, once installed, permits monitoring continuously in time without staff assistance at the control point both for receiving and processing of data.

Although this system and the corresponding software are very sophisticated they are permanently upgraded and further developed.

In the example (Figure 1) we examine the actual possibilities to control a church tower. Transducers continuously measure the variations of: width of cracks, temperature, moisture content, global and partial verticality, stresses, strains, relative displacements. All the transducers are connected to a central controller that by modem release actual values of measured parameters.

This arrangement of instrumentation allows to evaluate the evolving of deterioration in the time and also to study the importance of the cyclic variation of temperature and moisture content of the walls and foundations.

The last application we made in Italy concern the evolution of cracks in the upper part of Castel S. Angelo in Rome, Chiesa S. Paragorio in Noli and Chiesa S. Lorenzo Maggiore in Naples.

Strength Assessment of a Brick Masonry Building of 1895

Détermination de la résistance d'un bâtiment en maçonnerie de 1895

Widerstandsbestimmung eines Backsteingebäudes von 1895

Kazuhiro KANADA
Taisei Corporation
Japan

Kimio UDAGAWA
Taisei Corporation
Japan

Toyakazu SHIMIZU
Ministry of Construction
Japan

Akiyoshi SATO
Ministry of Construction
Japan

1. Introduction

Recent severe damage to old brick masonry buildings in Japan as a result of even moderate earthquakes has emphasized the need to retrofit such buildings to enhance their seismic performance. This paper describes a collaborative project to investigate the seismic performance and appropriate retrofit measures for a famous 3–story brick masonry building, 130m long, 45m wide and 15m high and containing many interior walls. The building was completed in 1895 and is the only remaining masonry structure in a Tokyo business district where it is part of a government office complex.

This paper consists of two parts. Part 1 provides detailed information on the strength of the brick walls. Part 2 examines the seismic performance of the building by using earthquake response spectrum methods and the strength results.

2. Test program

To evaluate the seismic resistance of the masonry, flexural, shear and compressive monotonic loading tests were conducted on the walls. The walls were constructed of bricks 240*115*65 mm with 10 mm thick bed joints. And tests, as follows, were conducted

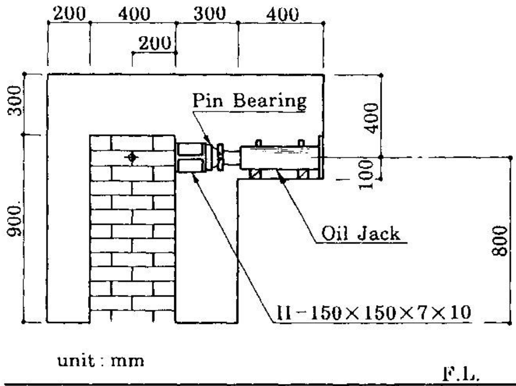


Fig.1 In–plane flexural test

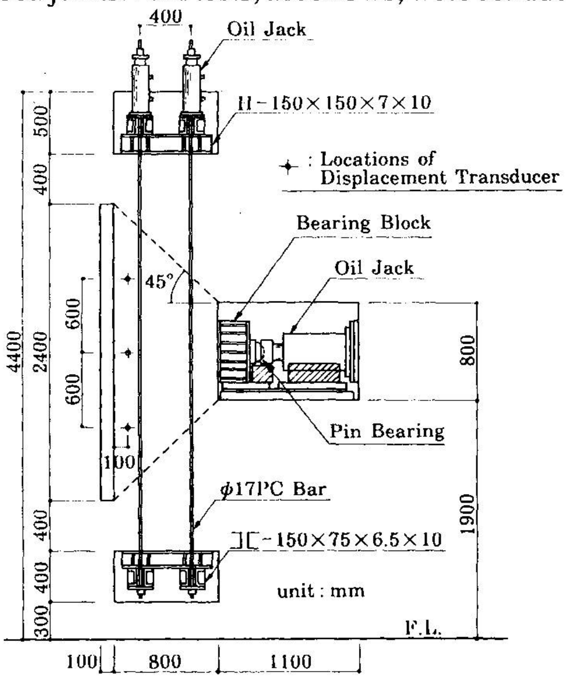


Fig.2 In–plane shear test

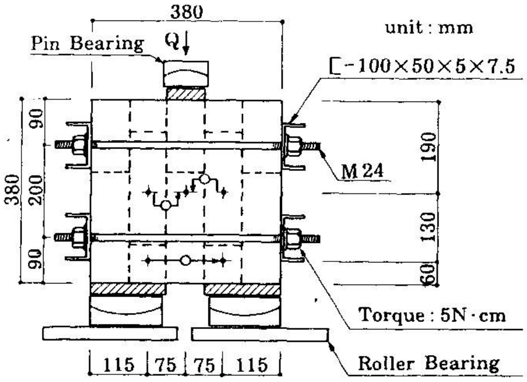


Fig.3 Double shear test

on the actual brick walls and on 380 mm cubic specimens cut from the walls:

a. In–plane flexureal and shear tests of the walls (Fig.1 and 2) ;
b. Double shear tests on cubes loaded parallel to the horizontal joints (Fig. 3) ;
c. Compressive tests on cubes loaded perpendicular to the horizontal joints ; and
d. Compressive, splitting and modulus of rupture tests on individual bricks

3. Test results

The setup for the two in–plane shear tests is shown in Fig. 2. The loading jack was inserted in a 800*1100 mm hole cut in the wall and the area between that jack and a 2400*100 mm slot also cut in the wall was preloaded to a mean vertical stress of 0.4 Mpa. Horizontal load, Q , versus horizontal center displacement, δ , relationships obtained from this test are shown in Fig. 4. The slope of the curves, which represents the stiffness of the walls, gradually decreased as the loading increased. The load carrying capacities degraded asymptotically once severe diagonal shear cracks initiated along the joints. For specimen B–1 the maximum load was 230 KN at a horizontal displacement of 9.9 mm.

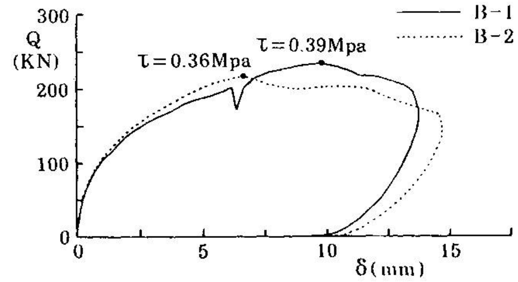


Fig.4 Load – displacement relationships for In–plane shear test

In the double shear tests, displacements were measured parallel(slip) and perpendicular (separation) to the horizontal joints. Fig. 5 are the resultant load – displacement relationships for specimen C–1. Slips and separations were not observed until immediately after the maximum load of 98 KN was reached. Then large plastic displacements occurred without significant reduction in load carrying capacity. The other three specimens exhibited similar behavior but their capacities and stiffnesses were slightly smaller than for C–1. The mean shear strength at the joints derived from this test was 0.28 Mpa.

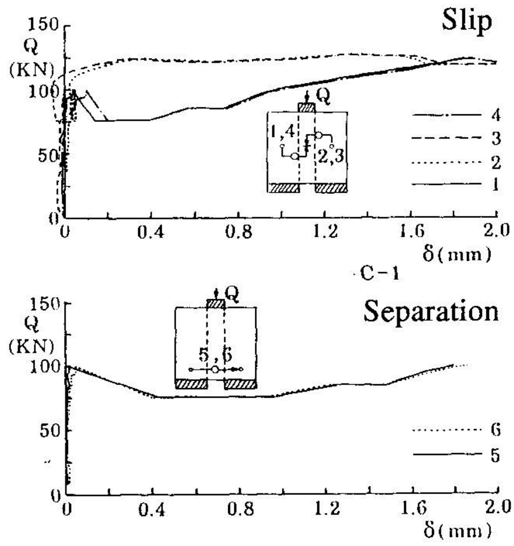


Fig.5 Load – displacement relationships at the joints

Shown in Fig. 6 is a comparison of the stress–strain relationships from the compressive tests. The vertical strains at the peak load for each specimen differ due to differences in the maximum strengths of the fragile joint mortar. The initial stiffness for the walls, defined as the secant modules at one third of the maximum strength, averaged 2800 Mpa .

4. Summary and conclusions

The test results showed that the strength of the joint mortar had a significant effect on the behavior of the brick walls. That strength was reduced by long term deterioration effects. The following conclusions were drawn as to mechanical properties of the brick walls:

1. Flexural strength $\sigma t=0.15$ Mpa ;
2. Shear strength $\tau u=0.38$ Mpa ;
3. Compressive strength $\sigma u=6.4$ Mpa ; and
4. Modulus of elasticity $Ew=2800$ Mpa

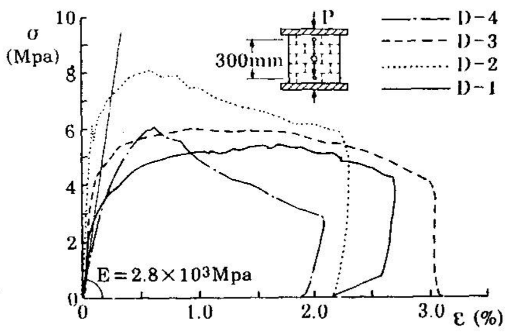


Fig.6 Stress – strain relationships from compressive tests

Survey of Statical Conditions of Churches in Rome

Etat des conditions statiques d'églises de Rome

Statischer Zustand römischer Kirchen

Marco MENEGOTTO
Prof. Struct. Eng.
Univ. La Sapienza
Rome, Italy

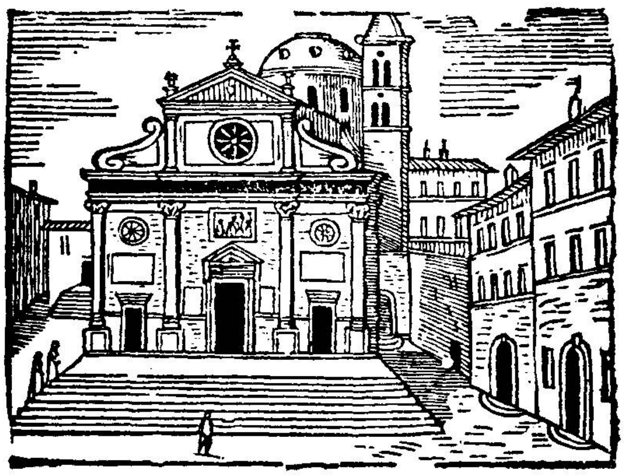
Sergio TREMI PROIETTI
Consulting Eng.
SE.I.CO. srl
Rome, Italy

Historic public buildings have often undergone limited spot repairs, following the emergencies rising time by time. Such maintenance is in the end poor and expensive.

Ancient architectures were often built over the remains of former buildings, or taking advantage of existing structures; seldom at once or according to one design, but rather in multiple phases, due to changes in available cash, sponsors and taste. Their overall statical conditions are not well known, in lack of information about construction materials, soil and foundations, modifications and repairs made in the past, state of conservation and even drawings showing the exact geometry of the structure. Thus, investigations are needed in order to understand and to assess these essential aspects of th life of the buildings.

The objective of the authorities concerned with monumental buildings is now to set organic plans of preservation, foreseing the needs of the building in a global view and assigning the priorities, in order to pursue a logical and economic sequence of maintenance and upgrading, despite fragmented funding.

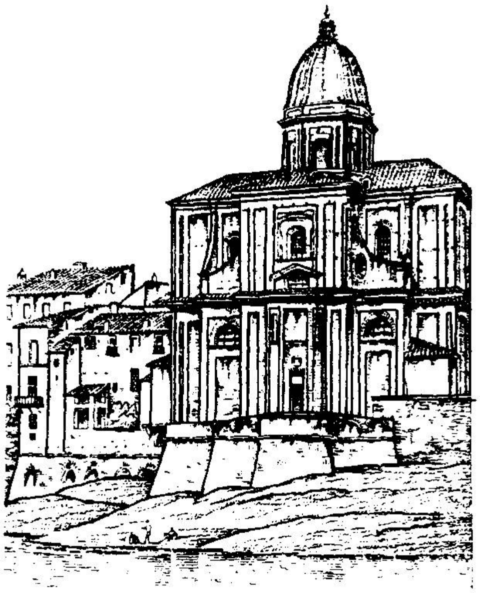
In this context, the writers have been commisioned by the Ministry of Public Works to undertake surveys and investigations on the statical conditions of three main basilicae in the heart of Rome: St. Augustine's, St. John of the Florentines' and the Gesù.



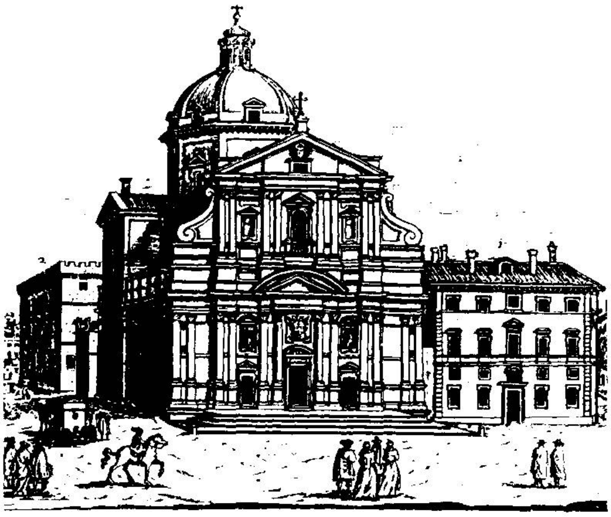
St Augustine's has been called the "living room" of Rome. This church lies between the Tiber and the stadium of Domiziano (piazza Navona) and was built by the Augustinian hermit order who, in the 13th century, acquired some land with a small church, S. Trifone, and started the construction of a greater one, for completing it only in 1483. The new church incorporated the older walls in its transept, but it is not yet clear where and to what extent.

In 18th century, the structure was in bad shape, with severe cracks in the central part, including the drum sustaining the dome. Then, L.Vanvitelli completely renovated it, consolidating all foundations, dismantling the whole dome and replacing the old bell-tower (both still appearing in the engraving).

In the middle of 19th century, the internal ornament work was renewed and brought to the present look.



St John of the Florentines' was built by the state of Florence in the beginnings of the 16th Century and completed in the following, involving several of the most important architects of the time. It lies on the Tiber left bank, opposite the Vatican. A large foundation plate was built into the river bed, then without embankments (see the engraving), giving the church a lenght appropriate to its importance.
Subsequent architects chose different layouts among centric and longitudinal plans, while construction was going on. Finally, the longitudinal scheme prevailed with A. da Sangallo. Some older structures were incorporated such as those of a chapel; but they are not on the same plate as the church and may suffer differential settlements, also due to the high changes in water level of the nearby river.
Later, the church underwent strenghtenings several times.
The Gesù (the Holy Name of Jesus'), Mother-Church of the Society of Jesus, is perhaps the most important Renaissance church in Rome. A small chapel, S. Maria della Strada, assigned to theSociety on the site, was soon demolished, yielding place to the new majestic architecture. Several designs were examined, including Michelangelo's, implying various positions, before the actual one by G. Della Porta was agreed upon, its façade rising in front of the *Papal Street* and chancel in the place of the former chapel.



Documents describe difficulties and cost of founding *in the water.*
The dome is low, not to impose upon the view of the façade; it had masonry ribs also on the inner face, that were demolished in the 17th century for frescoing the ceiling (a report says that the masonry of the vault was *good and strong*, needing no ribs!).
"Instructions" in 27 items, written by the Jesuites, codified the construction works and are testimony to the state of the art at the time.
The survey has begun with historic research on structural data, geometric measurements and visual inspections. No evidence appears on overall statical weaknesses, but typical degradations are present in all buildings, namely, roof timber aging and leakings, detachings of lead plates from domes, humidity from soil, settlement cracks in masonry. Particularly difficult is checking the foundations. In fact, inside the churches the underground space is filled with historic graves and, outside, streets and buildings hinder inspections. The survey will proceed by sampling the structural materials and monitoring the deformations, where needed. Finally, a maintenance plan will be established.

Testing the Steeple of the Peter and Paul Cathedral in Saint Petersburg

Contrôle du clocher de la cathédrale de St. Pierre et Paul à Saint Petersbourg

Kontrolle des Glockenturms der St-Peter und Paul-Kathedrale in St. Petersburg

Jossif RACHA
Civil Eng.
Institute of Steel Structures
Saint Petersburg, Russia

Lubarov BORIS
Civil Eng.
Institute of Steel Structures
Saint Petersburg, Russia

Jouravlev ANATOLY
Mechanical Eng.
Institute of Steel Structures
Saint Petersburg, Russia

The steel steeple with a cross of 60 m in height (from el. + 60.0 to el. 120.0) was erected in 1857 in place of a wooden one destroyed by fire. When examining the cathedral structures in 1991, cracks were found in the masonry at an el of 40.0 m in a place where the steeple anchoring beams had been built in.

The dynamic loading tests of the steeple structures were performed for experimental determination of the following:

— natural frequency of the first mode of oscillation and its comparison with the design mode;
— decrement of oscillation for calculation of dynamic forces;
— values of the revealed crack;
— an oscillograph and a cinecamera started working simultane-ously with the radio command.

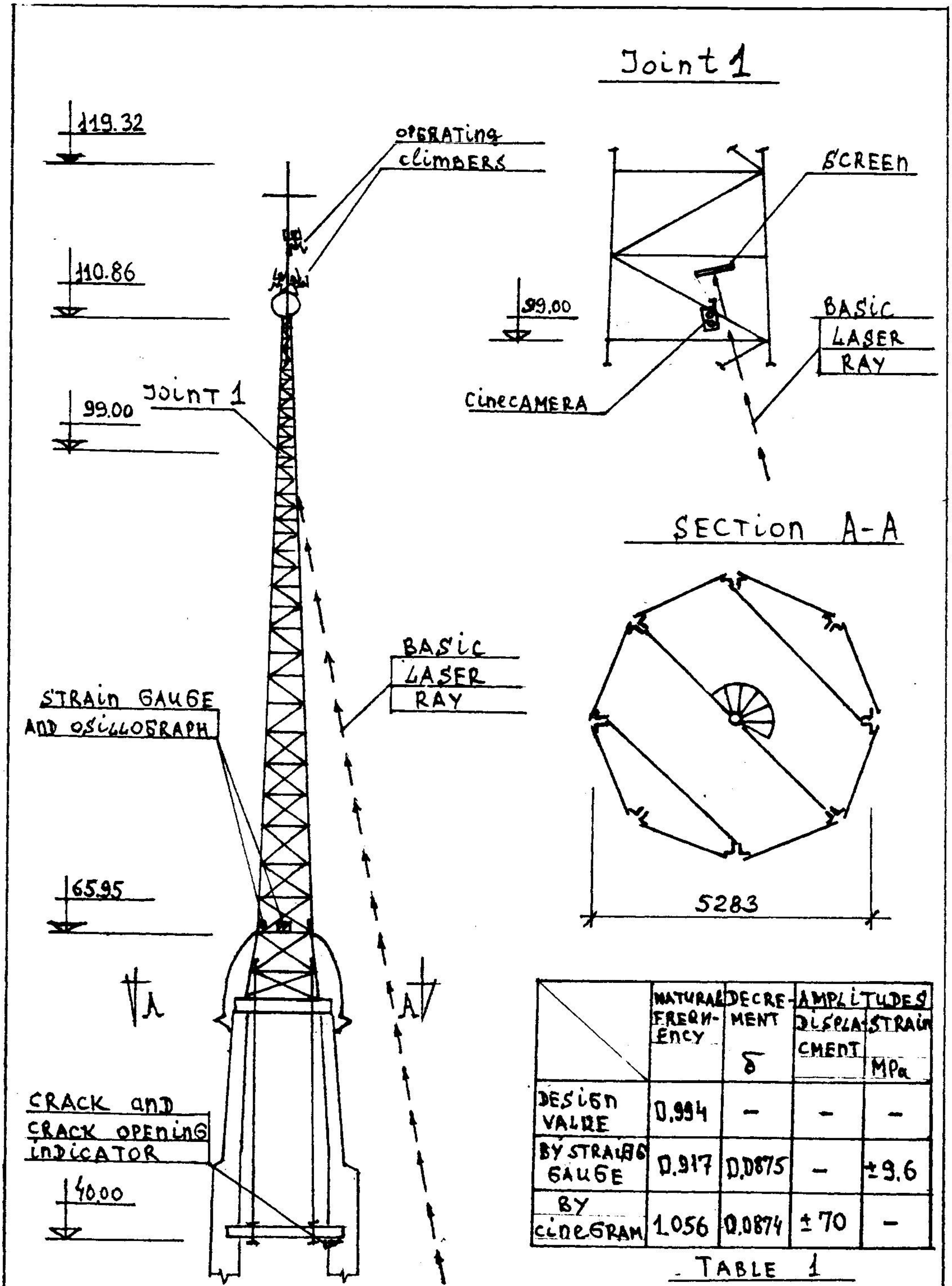
The tests were performed eight times.

The valute $\delta = 0.0875$ with an amplutide of strain $\pm$ 9,6 MPa is explained by the fact that the copper sheets of the steeple displace in oscillation which be clearly seen and dissipation of the oscillation energy accelerated due to dry friction.

In the cracked area at an elevation of 40.0 m, vertical displacement was recorded of the steel supporting beam relative to the brick masonry with an amplitude of $\pm$ 0.01 mm at a maximum amplitude of the steeple 70 mm at an el. of + 115.0.

The diagram of the tests is shown in Fig. 1. The results are presented in Table 1.

The tests showed coincidence of the design and the field strain characteristics of the steeple structures. The verification calculations, the results of examination and the field tests made it possible to give a comples assessment of the condition of one of the main historical structures of Saint Petersburg.



	NATURAL FREQUENCY	DECREMENT δ	AMPLITUDES DISPLACEMENT	STRAIN MPa
DESIGN VALUE	0.994	–	–	–
BY STRAIN GAUGE	0.917	0.0875	–	±9.6
BY CINEGRAM	1.056	0.0874	±70	–

TABLE 1

Fig.1 STEEPLE TEST DIAGRAM