INTERNATIONAL CENTRE FOR THE STUDY OF THE PRESERVATION AND THE RESTORATION OF CULTURAL PROPERTY
CENTRE INTERNATIONAL D'ETUDES POUR LA CONSERVATION ET LA RESTAURATION DES BIENS CULTURELS

ICCROM

STRUCTURAL CONSERVATION OF STONE MASONRY

CONSERVATION STRUCTURELLE DE LA MAÇONNERIE EN PIERRE

International Technical Conference

Athens, 31.X. – 3.XI.1989, Athènes

Conférence internationale technique

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PREFACE

Nous sommes tous conscients du fait que la conservation est un domaine pluridisciplinaire, auquel les différentes sciences doivent collaborer. Nous pensons normalement à la collaboration entre sciences humaines et sciences techniques, exactes et naturelles. Mais si on regarde de plus près le problème de la collaboration nécessaire, on voit combien on a encore à faire pour mettre au point une coopération entre les représentants de branches très voisines de la même science. Le génie civil et l'architecture, qui depuis l'époque de l'antiquité jusqu'au siècle dernier ont été unis, se sont développés et sont devenus des disciplines souveraines. Mais la conservation architecturale ne peut se réaliser qu'avec la participation conjointe des architectes et des ingénieurs. Le point central de cette collaboration est la conservation structurelle des bâtiments historiques, sujet de notre colloque international à Athènes.

C'était une rencontre interdisciplinaire importante, tant par son contenu que du fait qu'elle a permis de dialoguer autour d'une table ronde. Je crois que nous avons fait un pas important vers une meilleure collaboration des différents spécialistes co-responsables de la conservation des structures des bâtiments historiques. Il nous faut protéger, développer et coordonner cette collaboration à un niveau international. L'ICCROM considère avoir à jouer un rôle important dans ce domaine et la présente publication doit concourir à cette tâche. Sa forme est simple et n'a pas fait l'objet de travaux d'édition afin de produire la publication au plus vite pour qu'elle serve immédiatement. Elle n'est pas un point final de la discussion, mais un instrument de travail et de réflexion.

Le colloque a pu être organisé grâce à la collaboration harmonieuse de quelques organismes que j'aimerais remercier cordialement au nom de l'ICCROM: le Ministère hellénique des affaires culturelles, l'Université nationale technique d'Athènes et l'Unesco. La grande compétence, le dévouement et l'hospitalité cordiale de nos collègues grecs, partenaires dans l'organisation de la conférence sous la présidence de notre ami le professeur Tassios, mérite un grand merci de la part de tous les participants à ce colloque.

A. Tomaszewski
Directeur, ICCROM

We are all very conscious that conservation is a pluridisciplinary area where various sciences must work together. Normally we think about cooperation between the human sciences and the technical, exact and natural sciences. But if we look more closely at the problem, we see how much there is still to be done to promote fruitful interchange among representatives of very close branches within the same scientific area. Construction engineering and architecture, which have been united since antiquity until the last century, have developed and become separate disciplines. But architectural conservation cannot be implemented without the joint participation of architects and engineers. The central issue in this cooperation is the structural conservation of historic buildings – the subject of our international conference in Athens.

This meeting was a very important interdisciplinary event, both for its contents and for the occasion it provided for round-table dialogue. I believe that we have taken an important step towards better collaboration among the different specialists that are jointly responsible for the conservation of the structures of historic buildings. We must protect, develop and coordinate this cooperation at an international level. ICCROM considers its own rôle in this area as very important, and the present publication must support this task. Its shape is simple and modest, with little or no editing. It was produced as quickly as possible in order to put it into service. It does not represent a final point of the discussion, but an instrument for work and reflection.

The conference took place thanks to the harmonious efforts of a few organizations which I would like to thank cordially on ICCROM's behalf: the Greek Ministry of Culture, the National Technical University of Athens, and Unesco. The great amount of competence, commitment and cordial hospitality of our Greek colleagues, partners in organizing the meeting under the leadership of our friend Professor T. Tassios, deserve much gratitude from all the participants in the conference.

A. Tomaszewski
Director, ICCROM

PREFACE

This International Conference was intended to bring together engineers and architects involved in research and application of the structural conservation of historic stone masonry buildings. The aim of the conference was a scientific analysis of the various aspects of this subject: diagnosis and assessment, structural modelling of damaged, repaired and strengthened stone masonry buildings, problems related to execution and durability, and the relevant codes and recommendations.

It was felt that these aspects had not always been given due attention at the numerous and otherwise fruitful cultural-scientific events related to the conservation of monuments and historic buildings. Therefore, this conference focused on the *structural* capacity of masonry rather than any other property, unless the latter had a clear impact on it.

It is hoped that the scientific-technical topics of this conference will contribute to a sound basis for the assessment and rehabilitation of historic and traditional masonry buildings. Thanks to a more rational analytical modelling and a more systematic presentation of the problems encountered, the structural intervention may now be seen more as a technical scientific endeavour rather than as an empirical exercise. Thus, a larger number of engineers will be better qualified to design, as well as to carry out the implementation of such interventions in a safer and more economical way. It is believed that this conference was another opportunity towards such a goal.

The call for papers for this conference met with a very good response: more than 100 summaries from as many as 12 countries were submitted for possible presentation. The international Advisory Committee tried to select the best of them; a second selection was then made on the basis of the full papers. In addition to these papers, appropriate general reports and keynote addresses were also presented by distinguished experts.

The effort to make the full pre-prints of all accepted papers available to participants prior to the meeting provided the basis for truly constructive discussion during the sessions. Thanks to the present publication of the proceedings, a broader readership will now consider the output of the conference.

On behalf of the Organizing Committee and the International Advisory Committee, we wish to renew our thanks to ICCROM, to Unesco and the Greek Ministry of Culture, as well as to the National Technical University of Athens, for their sponsorship and collaboration. It is hoped that further similar events will be organized in the future, focusing on the fundamental *structural* aspects of conservation.

T.P.Tassios
National Technical University of Athens

Cette Conférence internationale visait à réunir des ingénieurs et des architectes s'occupant de recherche et d'applications en matière de conservation structurelle des bâtiments historiques en maçonnerie de pierre. La conférence avait pour but d'analyser scientifiquement les divers aspects relatifs à ce sujet: diagnostic et évaluation, modelage structurel des bâtiments en maçonnerie de pierre endomagés, réparés et consolidés, problèmes liés à l'exécution et à la durabilité des interventions, et normes et recommendations en la matière.

On estimait qu'il n'avait pas toujours été accordé à ces questions l'attention qu'elles méritent lors des nombreuses manifestations culturelles et scientifiques, par ailleurs fructueuses, qui portaient sur la conservation des monuments et bâtiments historiques. C'est pourquoi cette conférence est centrée sur l'aspect *structurel* de la maçonnerie plûtot que sur toute autre propriété, à moins que cette dernière n'ait sur lui un impact évident.

On espère que les sujets scientifiques et techniques de ce colloque contribueront à établir une base solide pour l'évaluation et la réhabilitation des bâtiments historiques en maçonnerie traditionnelle. Grâce à un modelage analytique plus rationnel et à une présentation plus systématique des problèmes rencontrés, l'intervention structurelle peut être maintenant considérée plus comme une entreprise scientifique qu'un exercice empirique. Ainsi, un plus grand nombre d'ingénieurs seront mieux qualifiés pour concevoir ainsi que pour exécuter de telles interventions de façon plus sûre et plus économique. On estime que cette conférence offre une nouvelle occasion de progresser dans cette direction.

Il a été répondu très favorablement à la demande d'exposés pour cette conférence: plus de 100 résumés provenant de pas moins de 12 pays ont été soumis en vue de leur éventuelle présentation. Le Comité consultatif international a essayé de sélectionner les meilleurs d'entre eux; une seconde sélection, sur la base des exposés complets, a été effectuée ultérieurement. Outre ces exposés, des rapports d'ordre général et des discours pertinents ont été aussi présentés par d'éminents experts.

Les efforts déployés pour mettre à la disposition des participants, avant la réunion, les publications préliminaires de tous les exposés retenus ont permis des débats réellement constructifs pendant les sessions. Grâce à la présente publication des actes, un évantail plus large de lecteurs sera maintenant en mesure d'examiner le résultat de la conférence.

Au nom du Comité organisateur et du Comité consultatif international, nous renouvelons nos remerciements à l'ICCROM, à l'Unesco et au Ministère hellénique des affaires culturelles, ainsi qu'à l'Université nationale technique d'Athènes, pour leur appui financier et leur collaboration. On espère que des manifestations similaires seront à nouveau organisées à l'avenir, portant sur les aspects *structurels* fondamentaux de la conservation.

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General Report to Topic

DIAGNOSIS AND ASSESSMENT OF EXISTING STRUCTURES

by Professor Dr.-Ing. W. Mann, Technical University of Darmstadt, Germany

1. Work on Old Building Structures

Work on old building structures is normally far more painstaking than the construction of new buildings. The absence of exact design documents and drawings, data regarding the existing building materials, the uncertainty of the present condition of the construction and the structural loadbearing systems, which can often only be calculated with difficulty, and then only approximately, make work to such constructions complicated and tedious.

Fig. 1:
Porta Nigra in Trier,
roman construction

On the other hand such structures often represent irreplaceable riches. They tell the story of long-forgotten times, of those who built them, their customs, their power and their knowledge. It is always masonry which survived until today, masonry constructed in stone or in bricks, thousands of years old, built for bridges, towers, castles, and cathedrals, where man could seek protection or feel at ease or could pay homage to the gods. My country, Germany, has also old buildings constructed in stone masonry. Not too many, as the country was destroyed too often, and our climate does not help; for instance the Porta Nigra in Trier (Fig. 1), one of the oldest stone masonry constructions, left by the Romans; or the King's Hall in Lorsch, an extremely precious building, inaugurated by the emperor Charlesmagne (Fig. 2). All buildings of this kind are very worth being preserved and protected. In this way work on old buildings, despite of the difficulties which may be encountered, can give deep insight into history and also a great deal of satisfaction.

Fig. 2:

King's Hall in Lorsch, inaugurated by Charlesmagne

There are always the same reasons for engineers and architects to apply their knowledge to old building structures:

Damage which becomes visible and gives rise to anxiety so that one must carry out repair work in order to prevent even worse happening;
Alteration and renovation of buildings in order to adapt them for present-day usage, which usually means higher loading values, larger rooms, more technical installation; or
Preventive measures in order to anticipate future damage from, for example, vibration caused by traffic, earthquakes, etc.

All these measures can only be efficiently planned and properly carried out if the existing construction is known as well as possible. Especially, damage can only be properly countered and repaired if one knows the cause. Thus the diagnosis and assessment of existing structures is of basic importance for all such work.

2. General Fundamentals of Diagnosis and Assessment

A site measurement of the building should be carried out at the beginning of the work, with a graphic representation in the form of plans, etc. The dimensions of rooms, walls, beams and columns should be documented as well as the materials and their condition together with details, such as intersection points, bearing positions and connections. This sounds simpler than it normally is in reality. The points of interest of the construction are often difficult to reach or lie hidden under plasterwork or facings. Alteration work in the course of time often results in the various construction elements no longer being homogeneous, so that findings made at one position cannot necessarily be accepted as being valid for any or all other parts of the building.

The condition of the materials and their strength can vary greatly and can only be assessed by completely laying bare of the whole construction. I have experienced this fact often; just at the present time I am involved with an old Carmelite cloister in the valley of the river Neckar, where one was of the opinion that one could carry out some superficial repair work with relatevely small means, as no great damage was visible externally. On placing new installation lines one came across, almost by accident, timber beams that were nearly completely decayed. The whole concept of renovation now has to be totally changed and enlarged. A large danger factor was recognized just in time and can be eliminated, even it the work involved is greatly increased.

The history of the building construction should also be documented, as one can gain important knowledge from this. Changes in the construction caused by additions and alterations can be seen and explained more easily, the historical value of the building substance can be better classified, the chance of approval for changes can be commented on and justified more clearly.

Of great importance ist the knowledge one can obtain regarding the nature of soil and the foundation conditions. Especially in the case of damage such as cracking, which can possibly be due to settlement, or in the case of additional loading from alteration work, one should not attempt to save money by doing without a soils report; it can give important information as to the cause of damage or on the measures necessary to prevent damage. The foundations are the base of a building structure, and omissions made during the construction phase can only be made good with great difficulty and extra expense at a later date, compared to carrying out such remedial work during the original building time.

Some weeks ago I stood in front of an old and valuable building that had been completely altered and renovated. Soon after completion of this work, severe damage occurred caused by settlement resulting from new loads on unexpectedly poor foundation conditions. The building had to be completely emptied again and the foundation conditions subsequently improved. One could have saved a lot of money and, probably more important, a lot of trouble, if one had carried out this soil investigation at the beginning.

The modern environment conditions can also have considerable effect on buildings and be the cause of damage. It is well known what a dangerous effect pollution of the atmosphere can have on building materials; or the effect of vibration from vehicular traffic on buildings, or even only adjacent construction sites and the thus necessary changes in the groundwater table. Such influences can be the cause of damage; one should therefore investigate and document them.

Fig. 3:
Gothic church in Worms, 15th century

Let me give an example of the effect of vibration from vehicular traffic. An old church in Worms on the river Rhine (Fig. 3) had already weathered many storms in the course of centuries. A few years ago a main road was to be so diverted that all the heavy traffic was to pass in the direct vicinity of the church. Tests were carried out by driving heavy lorries past the church, and at the same time vibration measurements were taken, which was reported on in [1]. These measurements (Fig. 4) showed that this traffic route would increase the dynamic loading of the building by a factor of 10, if you compare the acceleration at the end of diagramme with that while passing. This relation combined with observed small cracking and slight plaster damage was so convincing that the town council not only dispensed with the planned diversion but also banned lorries from using the road altogether. A sign that many are prepared to help protect historic buildings from modern dangers, even if this means extra effort.

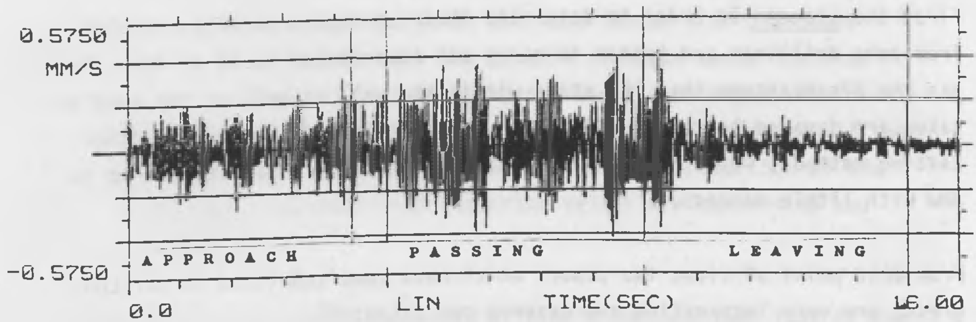

Fig.4: Typical diagramme of speed in the construction as lorry is passing

There is a very interesting paper in the preprints, by Ugur Ersoy, who summarizes all the basic steps, which should be done before any permanent intervention.

He asks for general rules and description of measures, because special codes are too difficult for all these unique problems.

In Germany we also believe that we don't need special codes for old structures, because our normal codes are sufficient and can be used for old structures too.

But let me mention that a working group in CIB, the group number W 23, just now tries to develop such a special code for old structures.

3. Stone Masonry Walling and its Strength

Masonry walling consists of two components: stones and mortar. For masonry walling built using natural stone, a third component is decisive for the strength, namely the position and construction of the joints depending upon the dressing of the stones. Exactly this last point causes a great deal of difficulty as one can only see the external face of the masonry, and the internal conditions cannot be seen. Is it a solid wall? Or does it consist of two outer stone leaves, with some mortar and broken stones in the middle? How to determine the strength of such a wall? Let us have a look to the different parts of a wall.

First the stones: In order to determine their strength, normally samples from core drillings are takten to carry out compression tests on them. This has the disadvantage that the stone within the wall as well as the samples taken are damaged by the drilling. One has therefore tried to find other testing methods, which are non-destructive as much as possible; but up to now with little success.

From this point of view, two papers which have been submitted to our Congress, are very interesting and deserve our attention.

Facaoaru reports in his paper on two methods of non-destructive testing. For this he uses the ultrasonic pulse technique, with the help of which the characteristics and the filling degree of joints with mortar can be examined, and also the rebound method based on the use of the Schmidt-hammer.

Van Gemert, Van Mechelen and Dereymaker use the geo-electric method, where the electrical resistance of the material is measured. They show how this resistance changes when, for example, masonry work is injected. Such measurements can thus be used to investigate the homogenity of the masonry walling and the success of injections.

Both papers emphasize that these methods must still be improved upon.

Especially the main interesting relationship, namely that between the measured values and the strength of the masonry will still require a great deal of further research. At the present time one readily uses combinations of the various methods of testing.

In order to complete the possibilities of diagnosis I would like to mention the infra-red foto. We use it in Germany for instance to have a look behind plaster. By that mean it is possible to see - without destroying the plaster - if a wall is more or less homogeneous or interrupted by a frame-work or by other constructions.

A third paper is of great interest here: Blasi and Sorace report on their experience with the long-term behaviour of marble. They come to the conclusion that the tensile strength of marble subjected to long-term loading is considerably reduced. A reduction of the tensile strength to 85 % and 75 % could be proven in tests even after only a few days. Further tests are being carried out. An extrapolation and experiences with old stone material lead to the statement that the tensile strength would diminish after many years to about 25 % of the original tensile strength. It is well known that long-term loading, for example with concrete or masonry walling constructed in artificial stone, brings about a certain reduction of the compressive strength, but such high reduction values are surprising and deserve our attention.

The strength of the mortar is even more difficult to test than that of the stones. Several problems have to be faced: The deviation of the mortar characteristics, the geometry and degree of fill of the mortar joints, and, most of all, the fact that one can hardly obtain suitable testing pieces from the existing mortar. So one very often is left with a superficial testing of the hardness of the mortar.

Facaoaru also reports here on non-destructive methods, namely the application of the ultrasonic pulse technique and the rebound method using the Schmidt hammer.

Tassios, Vacliotis and Spanos describe two methods to measure the strength of mortar:

At first the scratch-width method, which can be applied in situ. A steel bar is dragged along a mortar surface by a certain pressure. The width of the scratch, thus produced in the mortar, is to be measured. It can be translated into compressive strength, if it is calibrated by scratch-tests and comparing conventional tests in laboratory. A rather reasonable method, although it gives only the strength of that surface, which was scratched. I admit that I often use a similar but much more primitive way, when I rely on my test by a pocket knife scratching the mortar.

A second method described in the paper is the fragments-test method, where small pieces of mortar are subjected to direct tension.

The biggest problem, namely the strength of the bonded building material masonry walling, is not dealt with at all by a paper at this Congress. We know that, apart from the strength of the two components stone and mortar, primarily the construction of the joint, i. e. the bond and dressing of the stone, play an important role.

In order to be more specific in this problem, we have carried out theoretical research as well as tests on the loadbearing properties of the joints and reported on this in [2], in connection with the revision of the German masonry standards. The parameters used were not only the strength of the mortar β_M, but also the geometrical parameters of the bond, namely relationship of joint thickness to joint length h/d, inclination of the joint α and the ration ü of stone area to cross-sectional area in a horizontal section.

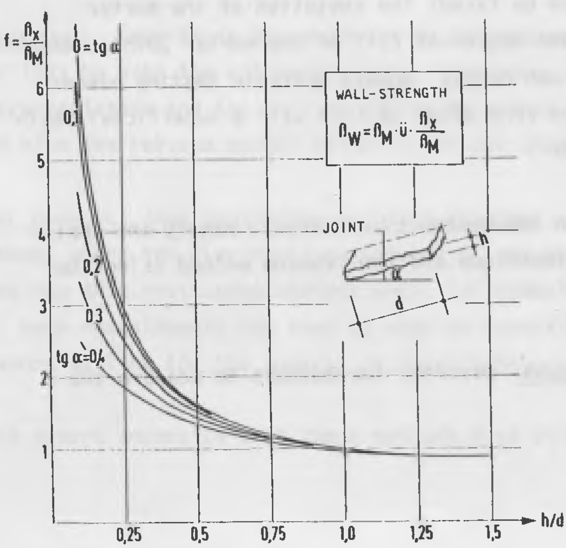

Fig. 5:

Form-factor f of the mortar joint and strength of quarry stone walling β_W

h/d and α = parameter of joint; $\ddot{u} = \overline{F}/F$ = ratio of stone area to cross-sectional area [2]

Even if one can thus derive generally valid relationships for the strength of the joints (Fig. 5), which are primarily conclusive for the strength of quarry-stone walling, one can nevertheless approximately determine the geometric parameters for existing masonry work. For only the surface of the wall is visible, and from it one must judge the quality of what is within. Sample drillings may possibly give further information.

4. Chemical Investigation

Chemical investigation of the mortar can occasionally be helpful, sometimes absolutely necessary. The proportion of binders and the type of binder permit the drawing of conclusions regarding the strength of the mortar. With injections one must firstly check if the existing material is compatible with the injection material, which is proposed to be used.

Capannesi, Seccaroni, Sedda, Majerini and Musco report in their paper on the analysis of mortar samples, which were taken from various positions in an old building near Rome. The various building construction stages from which the samples were taken are clearly reflected in the different results of the analysis - in other words, an important aid in the classification and restoring of the masonry.

Chemical analyses to determine the cause of damage are always then necessary when chemical reactions within the material can lead to dangerous changes. Well known in this regard is the formation of expanding minerals (Fig. 6), e.g. gypsum $CaSO_4 \cdot 2H_2O$

ettringite $Ca_3H_2\ (CO_3/SO_4)SiO_4 \cdot 13H_2O$

or thaumasite $Ca_6Al_2\ \left[(OH)_4/SO_4\right] \cdot 26H_2O$.

They all contain sulphur in the form of sulphates as a dangerous element. These materials have the tendency to store water in their crystalline form. They thus increase their volume so much that the surrounding material bursts and is thus destroyed. Damage of this type has also been observed when injecting masonry work: If the injected materials are not compatible with the existing materials, then chemical reaction and the formation of expanding materials with subsequent damage result.

At the moment we are observing another interesting example of damage which ist probably caused by the formation of expanding minerals. I have already reported of that in [3]. In an eighty-year old railway tunnel, the bricks with which the tunnel cross-section was lined, are breaking off in layers parallel to the surface of approximately 1 to 3 cm thickness. Chemical investigation has shown that the mortar in the joints has changed into gypsum mortar. Probably the steam-driven locomotives used in the past produced SO_2, which was deposited on the surface of the masonry. The condensing steam and the moisture from the hill simultaneously soaked the mortar, so that the SO_2 could penetrate the joints. It reacted with the existing lime and produced gypsum. The increase in volume probably led to the breaking off. We are attempting to confirm this hypothesis for the damage by systematic tests.

Fig. 6:
Expanding mineral,
microscope photograph

5. Deformation Measurement

The deformation of building structures can lead to serious damage, such as cracking or dangerous deviation from plumb. Typical causes of such deformations are: changes of temperature, varied settlement of the foundations, shrinkage or creep deformation with new building materials, or overloading, whereby deformation can be an indication of imminent failure. The knowledge of such causes is important in order to carry out suitable countermeasures. The long-term taking of measurements can be of great assistance here.

Rutenbeck reports in his paper on such measurements, taken at a national monument in Florida, and he discusses the conclusions which can be drawn from such measurements. They mainly give information on whether the movements are lessening or, dependent on temperature change, are changing, or if they are even constantly increasing, which could be extremely dangerous and be a sign of impending failure. If one knows the cause, then a remedy is easier to find. Often one can accept the cracks without having to take remedial measures, if one knows that they are not linked to any immediate danger.

6. Statical Investigations

Statical investigations are mostly the simplest and cheapest way of obtaining a picture of force flow and of the stress conditions within a structure. The results can be of assistance in numerous ways: One can, for instance, determine those parts of a building, which are especially highly loaded, so that one can carry out further investigation at these points or use more precisely defined examination. The existing factor of safety can be given numerically, and the measures necessary for strengthening the structure can be reliably estimated and justified.

Naturally, such calculations are often difficult, just in the case of old constructions. Three-dimensional loadbearing systems, e.g. vaults, or multiple statically indeterminate systems, or the estimation of not exactly known material constants often do not permit the expectation of too much precision. Nevertheless these calculations are for us an indispensable aid for the diagnosis.

Bernardini, Gori and Modena offer in their paper an interesting example for the application of statical tests in order to estimate the vulnerability of building groups to earthquakes. The seismic vulnerability of historical centres is estimated by determining the resistance of the individual buildings against seismic forces, using simplified statical models, and is weighed against additional influencing factors. The magnitude of the vulnerability and the necessity of supporting measures can be clearly demonstrated.

7. Diagnosis and Assessment

Based on the diagnosis, which could not be fully listed, the reasons for the damage must be recognized and the necessary measures for their remedy be determined. Naturally this is not easy, as often the findings are not clear-cut, sometimes several causes overlap one another so that the analysis is difficult.

We can show success and failure in our efforts concerning old building structures. Technical progress has offered us many new possibilities, for example in the science of building materials, or in construction, for instance in the technique of drilling and dowel fixings, or in the possibilities of the statical calculation.

Many buildings could be saved thanks to this technical progress, but also occasionally a setback occurs, when the step forward was too big and not adequately guarded against.

At all times man is in the centre of what happens: His experience with materials and constructions, his willingness to commitment and his attitude regarding old building structures are all decisive. Building works have at all times mirrored human culture. And our attitude to old buildings is also a mirror of our present culture.

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DIAGNOSIS, ASSESSMENT AND EMERGENCY INTERVENTIONS FOR HISTORIC MASONRY STRUCTURES

UĞUR ERSOY(*)

SUMMARY

This paper summarizes the basic steps to be taken in the assessment of historical masonry structures. It also includes a brief discussion on temporary emergency interventions to be made. The paper emphasizes the importance of a sound assessment and diagnosis prior to any permanent intervention, like repair or strengthening.

Although assessment and diagnosis should be made by experts, especially for old masonry structures, and although each case presents a unique problem, the author believes that some flexible rules and a check list can be drafted.

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1 - INTRODUCTION

Historical buildings should be well preserved. Most of these buildings have both architectural and historical importance. Anatolia has been the cradel of many different civilizations all the way from the early history. As a result of this the number and variety of old historical structures are very high. Many of these structures (especially the very early ones) have suffered damage or have been completely destroyed. Repairing, strengthening or rebuilding all of these structures is practically impossible. In Turkey, priority in intervention is determined considering the following aspects :

- symbolic value of the structure
- aesthetic value (art value) of the structure
- value of the contents (like frescoes, mozaics etc.)
- present social function of the structure
- danger of being lost in a near future. For example the area will be covered with water due to a dam construction.

Prior to any kind of intervention, including temporary interventions, an assessment should be made.

2 - PRELIMINARY ASSESSMENT

When priority of intervention is given to a certain historical building, a preliminary survey is essential. The main objective of the preliminary survey is to get acquinted with the structure and to see if temporary emergency interventions are needed. The preliminary survey team should consist of architects, archaeologists (or art historians) and structural engineers. If the structure has foundation problems, a soil engineer should also be included in the team. The preliminary assessment team should try to get all the documents related to the structure in question before going to the site. These documents include historical records and reports on previous interventions or assessments if any.

In general no measurements are made at this stage, visual inspection is usually adequate. In addition to getting acquinted with the structure, the structural engineer tries to make an estimation about the safety of the structure. If there is an immediate danger to total or partial collapse, either under the present actions or expected actions like earthquake, some emergency measurements should be taken.

The most common and most practical emergency intervention is providing temporary strutting and sporing. Generally timber is used for this purpose. If there is a settlement problem and this problem is related to water leaking from a pipe line or storage tank, this leakage should be stopped. Also if frescoes or mozaics are endangered due to water leakage, precautions should be taken to stop this. Sometimes a certain portion of the monument, which has a very high artistic or historical value, may be in danger of falling down. These should be immediately shored. If shoring is not possible, that portion should be removed and stored at a safe place.

Emergency interventions should be designed and recommended by the structural engineer. However the final decision should be taken in consultation with the architect and the archaeologist (or art historian), because the engineer can not very well judge the artistic, architectural and historic values.

Before proposing any emergency measurements, the engineer should understand the basic structural behaviour well. Any intervention made without understanding the behaviour may not be helpful, but worse than that, may accelerate the damage. An example of such a wrong emergency intervention was seen by the author in Alaeddin Mosque, Konya (Tankut-Ersoy, 1982). This mosque is one of the earliest Moslem monuments in Anatolia, built in 13. century by Seljuks. The main hall of the mosque has a timber roof supported on one pierce stone columns. The foundation of each column consists of a inverted column capital sitting on a timber grilage. As a result of this, columns were almost pin ended and could undergo a rocking motion. About 15 years ago, a huge watertank and connecting pipe-line was constructed in the vicinity of the mosque. Water leaking from the tank and the pipe-line penetrated underneath the mosque, causing settlement problems. The differential settlement resulted in severe cracking on masonry walls and tilting of the stone columns. Engineers who were called in for preliminary investigation decided to take some emergency measurements. They recommended strutting and shoring of the whole building by timber members. This was a sound decision. However in addition, they recommended to cast reinforced concrete slabs at foundation and roof levels where the roof was being supported by stone columns. These recommendations were immediately put into action. Placing a reinforced concrete slab was of course objectionable from architectural and artistic points of view. In addition it was also objectionable from the structural point of view. Joining the tops and bottoms of columns forced a monolithic behaviour and resulted in severe shear cracking in most of the columns. Experts called later pointed out that this has been a severe mistake, but what was done.

3 - ASSESSMENT

Assessment of a historical structure is made with the following objectives :

a - To keep a record of the structure for the future use.
b - To find out if urgent intervention is necessary to save the structure or the contents.
c - Decision has been made to repair or strengthen the structure. Assessment of the present condition is needed for a sound intervention.

In this paper most of the discussions made will be related to case (c), in other words, assessment for strengthening or repairing will be the main objective. Also the emphasis will be on structural problems.

Similar to preliminary assessment, the detailed investigation and assessment should be made by a team composed of structural engineers, architects and archaeologists (or art historians). If foundation problems do exist, a soil engineer should also be included in the team. The engineer should discuss all his findings and assessment with his team members. The work to be done by the structural engineer is summarized below.

Structural Survey

Before starting the structural survey, all documents and drawings related to the structure under consideration should be collected. If accurate drawings are available, the engineer can make use of them. If such drawings are not available or if they are not complete, a survey should be made measuring all dimensions. Later, scaled drawings should be prepared.

Before taking any measurements, the engineer should make a visual inspection. While making the visual inspection the engineer should try to understand the behaviour of different structural components and the behaviour of the structure as a whole.

Next the engineer should try to record the damages observed in a systematic way. To do this, he should either have a drawing or a sketch of the building. The damages observed should be marked on the drawing. Each type of damage should be designated by a letter on the drawing and these should be numbered. On a separate sheet the damages observed should be fully described, including the measurements made (for example, crack width and crack length). Common damages observed on masonry structures are listed below (Penelis, 1984).

- Cracking. It is important to distinguish between old and new cracks. Weathering in old cracks serves as a good criterium. Crack pattern should be carefully noted, giving the length, inclination and crack width. Type of cracking should also be noted. If this is properly done, then it will be possible to see the direction of primary compression in the masonry, which is very important.
- Crushing. Crushing of the masonry, splitting and spalling are caused by high local compressive stresses or local weaknesses.
- Deformations. Relative movements in masonry wall will usually result in opening of joints or local slips at joints. The amount and the direction of each deformation should be noted.
- Tilting. In masonry structures tilting of walls is very common. Magnitude and direction of tilting should be noted.
- Deterioration due to weathering or chemical action. The degree and extent of deterioration should be recorded. If necessary a map of deterioration should be given.
- Local Collapses. Partial or local collapses should be fully described.
- Differential Settlement. The direction and magnitude of differential settlement should be measured and recorded.
- Ties. The condition of ties in he masonry structure should be noted, including slips if any.
- Safety of Contents. If the contents, like frescoes, mozaics etc. have great artistic or historical value, the safety of these should also be inspected.

Material Survey

To analyze the masonry structure in question the engineer needs data on the material (density, strength, modulus of elasticity, chemical composition etc.). If possible, material samples should be taken and tested in the laboratory. Since less homogeneity and uniformity exist in historical buildings as compared to new ones, the engineer should try to take his samples from different locations (Yorulmaz-Çılı-Ahunbay, 1989). However in doing so, he should definitely get the approval of the archaeologist (or art historian) in his team. In many cases taking samples or adequate number of samples will not be possible. In such cases non-destructive tests can be performed (rebound hammer or ultrasonic pulse velocity).

Soil Survey

Soil properties can play an important role in the structural analysis, especially for seismic analysis. Depending on the importance of the structure, a visual inspection by opening ditches and simple tests on disturbed samples may be adequate. However for important and large structures boreholes should be drilled and tests should be made on the samples taken. The soil engineer should give a report on the soil condition. The detailed soil survey is unavoidable if the masonry structure has suffered settlement problems.

Long Term Observations

If some damages on the masonry structure have been observed due to time dependent actions, like settlement, then long term observations should be made. After stopping the cause of deformations (for example stopping the leaking water penetration to the foundation), the structure should be put under observation for an extended period of time to be sure if the cause has really been eliminated, before making any kind of permanent interventions. Long term inspection should not take less than 1 year, to include the effect of seasonal changes. Such a long term inspection is being made for Alaeddin Mosque, where the main cause of damage was settlement due to leaking water. After stopping the water, the building was instrumented to record deformations and displacements for a period of one year. It was decided to repair the building at the end of this period. The instrumentation consisted of electronic transducers and optical equipment. Transducers were used to measure displacements of walls, tilting of walls and opening of cracks. Continuous recording is being made. Settlement is being checked by optical measurements. As an emergency intervention, the root has been shored and voids in the soil underneath the building were filled by injecting concrete.

Structural Tests

In seismic regions, structural analysis should also be made for the seismic action. For such an analysis natural period of the structure should be known. This can either be estimated from knowledge on similar structures or can be determined by tests. Natural period of the structure at low amplitudes can be determined by making ambient vibration tests. It is also possible to determine the mode shapes if records are taken at different heights.

Some of the masonry structures have a complex structural system and it is difficult to predict how the loads are transferred. In such cases it may be feasible to make a model and test it. Such tests can reveal the load shared by different structural components and the load path followed.

4 - EVALUATION OF RESULTS AND DIAGNOSIS

The engineer should gather all the information obtained from inspections and performed tests. He should carefully evaluate every bit of information and every piece of evidence. Considering each structural component and structure as a whole, he should try to understand the behaviour and the damage mechanism. He should formulate hypotheses and then check these hypotheses considering the evidences again and again. He should not proceed until he is able to answer the following questions clearly.

1. Is the overall structural behaviour and contribution of each component to the load carrying capacity well understood?
2. Are the causes of damages well understood? In other words does the engineer have a sound diagnosis?

If the answer to these two questions is not positive, further investigation, testing and evaluation is necessary. No attempt for repairing or strengthening should be made until these matters are settled.

5 - NEED FOR STANDARDIZATION

Assessment of historical structures is a professional job and should be made by experts. It is very difficult to formulate rules and standards for assessment and interventions, because each structure has unique problems. However some general rules and measures can be specified if the historical masonry structures can be classified into categories. These rules and measures should not go into details. Measures can be brought with regards to the degree of damage. For example, the degree of damage can be related to the crack width and dimensionless deformation or tilt. Also it is possible to make check lists for the operations to be carried out for assessment of masonry buildings. Simple and general rules can be formulated for the structural analysis. These may include; (a) modelling, (b) load and material factors to be used, (c) seismic loading and (d) acceptance criteria.

A committee should be formed to study the possibility of classification of masonry buildings and to state some basic rules and procedures for assessment.

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ACKNOWLEDGEMENT

The author would like to thank Prof. M. Yorulmaz for the documents he provided and for his valuable suggestions.

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MONITORING STRUCTURAL MOVEMENTS IN HISTORIC STONE MASONRY BUILDINGS

TODD RUTENBECK*

SUMMARY

Structural movement monitoring can distinguish between seasonal movements and the more threatening progressive movements. Measuring changes in crack width, settlement, tilting, and deflection for at least 1 year can aid in conservation decisions. The author references projects from his 12 years of field experience in structural monitoring of historic structures and prehistoric ruins throughout the United States on investigations funded by the Department of the Interior, National Park Service. The effectiveness of various monitoring methods is discussed. Structural monitoring can save money and prevent unnecessary structural modifications by determining that some apparent structural defects do not require repair.

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INTRODUCTION

Structural movement monitoring is a valuable tool in the conservation of historic stone masonry buildings. It is used as a management method to make the best use of repair funding and to prevent the use of unneeded modern intrusions. Most historic stone buildings show some signs of structural distress such as cracks, settlement, tilting, and deflections. These signs of structural distress are the result of past structural movements. Such movements can be caused by temperature changes, insufficient foundations, overloading, or masonry deterioration. If these movements took place in the past, but are not currently active or are due to temperature changes only, the building may be structurally sound in spite of cracking and other signs of distress. If, however, the structural movements are currently continuing or accelerating, the building may collapse unless appropriate repairs are made. Structural movement monitoring data can distinguish between structural defects that are inactive and those that are currently active. It can also establish the current rates of movements in an active building. Comparing these rates with structural geometry can establish an estimate of time to failure. Thus, structural monitoring can warn of gradual movements and resulting failure. It cannot warn of instant collapse from overloading. In conserving historic buildings, negative monitoring results are useful and desirable. Data showing that major structural intervention is unnecessary or can be postponed help to maintain the integrity of the site in spite of the existence of apparent structural faults. Data showing movement toward failure, however, may require immediate action.

PROJECT EXPERIENCE

A current project at Castillo de San Marcos National Monument, St. Augustine, Florida, illustrates the use of structural movement monitoring in conserving a stone masonry structure. Castillo de San Marcos is a Spanish fortification constructed of coquina stone in the late 1600's on the shore of Matanzas Bay to protect the city of St. Augustine. The fort was built in the shape of a square with an arrowhead shaped bastion at each of the four exterior corners. It is exposed to heavy rains, salt spray, and the effects of rising and falling tides which raise and lower water levels in the wet moat on the western portion of the Castillo. Numerous structural cracks exist on the western half of the Castillo (see Figure 1). The largest cracks are vertical cracks on the exterior walls of the northwest and southwest bastions. Cracks also exist between firing steps on the terreplein and the exterior walls. Possible causes of the cracking include foundation settlement, pressure from the soil fill in the bastions, internal wall failure, and settlement of the terreplein. In 1986, waterproof mechanical dial gages were installed at 13 locations to monitor changes in crack width. The gages have an accuracy of 0.01 mm and a range of 5 mm. The five terreplein gages are easily accessible for the monthly readings. The exterior wall (scarp) gages are mounted high above the wet moat and are read with a tripod mounted telescope from the edge of the moat. The structural movement monitoring data will be used along with other site investigations to make sound management decisions concerning conservation and stabilization work planned for the near future.

Figure 1 indicates which cracks have shown progressive increases in width during the 2 years of monitoring. Figure 2 presents data for one of the currently active cracks. Changes in crack width since installation of the gage are shown in the upper part of Figure 2. Actual crack width is not determined with the gage. Corresponding ambient temperatures are shown on the lower part of Figure 2. The data show two definite trends. First, there is a seasonal change in crack width. In the cooler winter months, crack width increases. Second, there is a permanent increase in crack width. The plot does not return to the zero line at the end of each year, but ends each year at a yet higher point on the plot.

Figure 3 is not based on monitoring data, but on site measurement of total crack size (including repair material) since construction. Crack widths and offsets are given for the top, gage location, and bottom of the vertical wall cracks. The top offset measurement only is given for location 4A, a crack important to defining bastion movement, but not containing a gage. Figure 3 is not to scale, and movements are exaggerated to show the mechanism of failure. The solid lines show segments of the bastion wall that have moved (from their original dotted line positions) as indicated by total crack dimensions. Figure 3 indicates that the upper portions of the wall are tilting outward. Site observations indicate also that the southwest tip of the southwest bastion is at a lower elevation than other parts of the Castillo. It appears that a combination of tilting and settling have caused the structural distress in the southwest bastion. Similar results were produced for the northwest bastion. Likely causes include foundation movements and pressure from soil fill in the bastions. Thus, site measurements have established the directions and possible mechanisms of structural movement. Structural movement monitoring has established that such movements are currently occurring progressively. Future structural monitoring will establish if stabilization work is successful in reducing or eliminating progressive structural movements.

The Castillo de San Marcos project illustrates the usefulness of monitoring structural cracks. Similar successful monitoring projects have been accomplished to measure other types of structural movements. The objectives and data analysis are similar to those for crack monitoring, but the instrumentation is of a different type. In Glen Canyon National Recreation Area, the effects of a rising reservoir and of heavy tourist traffic on prehistoric American Indian stone dwellings were measured. Settlement rate at several sites was determined using precise leveling surveying methods. Reference points were installed and precise leveling circuits were run over a period of several years. The precise leveling method allows direct readings in elevation to 1/10 mm. At Tonto National Monument, Arizona, changes in a tilting wall of an American Indian stone cliff dwelling were determined using a plumb bob and permanently installed reference brackets. At Schuylerville, New York, changes in tilt are being measured in the stone, 154-foot-tall Saratoga Monument with a digital electronic level at several permanent reference surfaces. At Keet Seel, a prehistoric stone cliff dwelling in Navajo National Monument in northern Arizona, electronic gages (linear variable differential transformers) were mounted on stands set in bedrock to determine if the entire multi-room structure was sliding down the sloping base of the cliff alcove. Dial gages would have worked for this application, but it was necessary to have a remote readout to prevent the necessity of entering the fragile prehistoric rooms for each reading as required with dial gages. At Pueblo Bonito, a prehistoric stone village in Chao Canyon, New Mexico, a dial gage was used to monitor wall tilt by placing it between a leaning multi-story wall and an adjacent perpendicular wall. Results indicated impending collapse and braces were installed. Plots of wall profiles were made to establish how much movement could be tolerated before collapse. No matter what the monitoring technique, the objectives are similar. It is necessary to detect progressive movements (as opposed to seasonal movements), determine their rates, estimate how far the structure can move before failure, and recommend corrective action.

EVALUATING THE USEFULNESS OF STRUCTURAL MONITORING METHODS

Structural monitoring is useful only if the results are sufficiently reliable to define structural movements. Choice of instruments, location of instrumentation, and overall project design affect reliability. For example, Figure 2 reports 2 years of data of crack width change and temperature readings taken approximately every month. There are fluctuations in readings throughout any given day and many fluctuations between the monthly readings. Assurance is needed that plots such as Figure 2 represent true trends and are not merely random samplings of daily fluctuations. Some investigators may argue that only continuous readings can give an accurate view of movement trends. It is true that continuous readings are the ideal method. It is not true that continuous readings are always necessary or always desirable. In other technical areas, many important parameters are measured intermittently rather than continuously with sufficiently accurate results. In medicine, pulse, blood pressure, temperature, and other parameters are frequently measured intermittently. In driving, speed and engine gages are read intermittently. Only under critical and unusual circumstances are continuous readings needed. The same is true in structural monitoring. Continuous readings are justified on a structure about to collapse, or a structure where it is necessary to record events of unknown frequency such as sonic booms, earthquakes, or traffic. While continuous readings at Castillo de San Marcos would have been desirable to help interpret data, they simply were neither practical nor necessary. They were not practical because automatic data acquisition systems and electronic gages are unreliable and difficult to maintain in high humidity and salt water spray conditions. Additionally, cables leading to the gages would have been subject to vandalism. Continuous readings were not necessary because it was not necessary to record intermittent events at unknown times and because the validity of data such as Figure 2 could be established in other ways. The approximate magnitude of daily fluctuations was established by taking readings throughout a single day. Progressive movements indicated in data plots far exceeded daily changes. In addition, readings were usually taken about the same time of day throughout the year to minimize the effects of daily variations.

While continuous readings are desirable, they frequently are not practical and are seldom essential. While it is theoretically possible to obtain a plot like Figure 2 that is in error because of a random sampling of daily fluctuations, it is statistically unlikely.

Another important factor in obtaining useful structural monitoring data is the accuracy of the instruments. The data in Figure 2 were collected with a gage having a precision and accuracy of 0.01 mm. It was therefore possible to detect many minor changes in crack width within the total movement of approximately 3 mm. Had the measurements been taken with a ruler with an accuracy of 1 mm, the total movement would be only three times the instrument accuracy and a much longer monitoring period would be necessary to establish reliable trends. This may be acceptable in long-term monitoring programs where large structural movements can be tolerated before the danger of failure occurs. In general, however, it is best to use an accurate, but economical instrument. Mechanical gages provide excellent accuracy. The more expensive electronic gages are not necessarily more accurate, but are sometimes necessary either for remote readouts or for continuous recording.

Some structural monitoring projects fail because they have no clear purpose. There must be well-defined conservation decisions that will be based on the data produced. For example, the leaning Pueblo Bonito wall was close to failure. Monitoring showed it to be currently still moving toward failure. The decision was to immediately brace the wall. Conditions at Castillo de San Marcos are not so critical. While walls are currently moving progressively, movements far in excess of the total movements over the last 300 years would be necessary for large-scale structural failure. Even so, conservation decisions will be based on the data. Any stabilization action that can slow the movements will help to save the structure through future centuries. Structural monitoring will determine the effectiveness of these actions. It will also help in planning effective actions. For example, if cracks are filled with a flexible material to exclude water, use of monitoring data will indicate that the work should be done in winter when cracks are widest. This will help to keep the repair material in compression throughout the year. The basic mechanisms of bastion failure revealed by the study (walls tilting outward from the center of mass of the bastions) will also assist in corrective structural stabilization decisions.

CONCLUSIONS

Structural movement monitoring is a valuable method in the conservation of historic stone masonry buildings. It can prevent unneeded modern intrusions and can determine the best use of funding for stabilization work. Proper monitoring project design, appropriate instrument types, appropriate instrument accuracies, and well-defined conservation decisions to be based on the data are all essential to a successful structural monitoring project. Monitoring projects often more than pay for themselves because previously planned stabilization work is determined to be unnecessary in spite of apparent structural defects.

CASTILLO DE SAN MARCOS
WEST SECTION

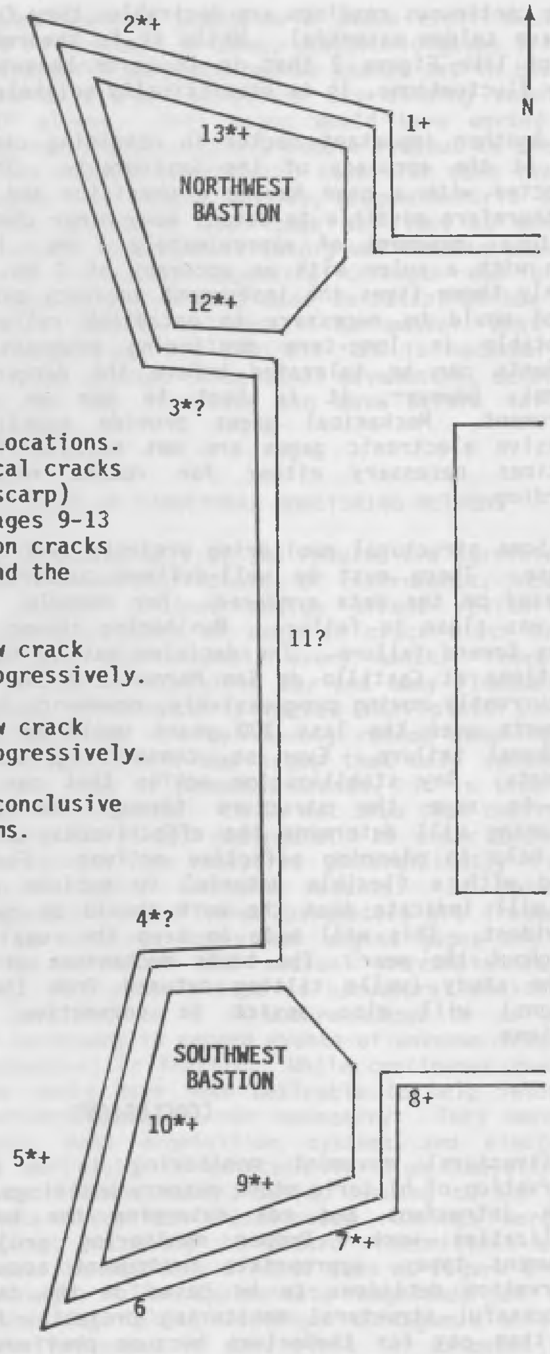


Figure 1. - Dial Gage Locations. Gages 1-8 are on vertical cracks on the exterior wall (scarp) above the wet moat. Gages 9-13 are on the terreplein on cracks between firing steps and the exterior wall.

*First year's data show crack width is increasing progressively.

+Two years of data show crack width is increasing progressively.

?Second year's data inconclusive because of gage problems.

FIGURE 2.

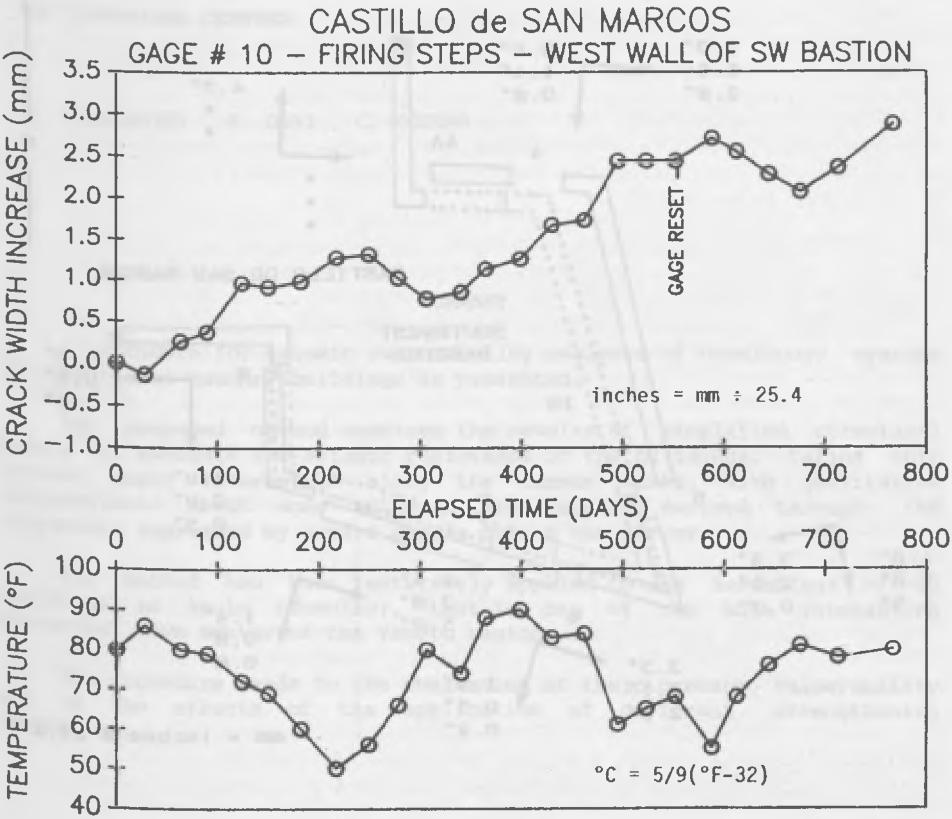
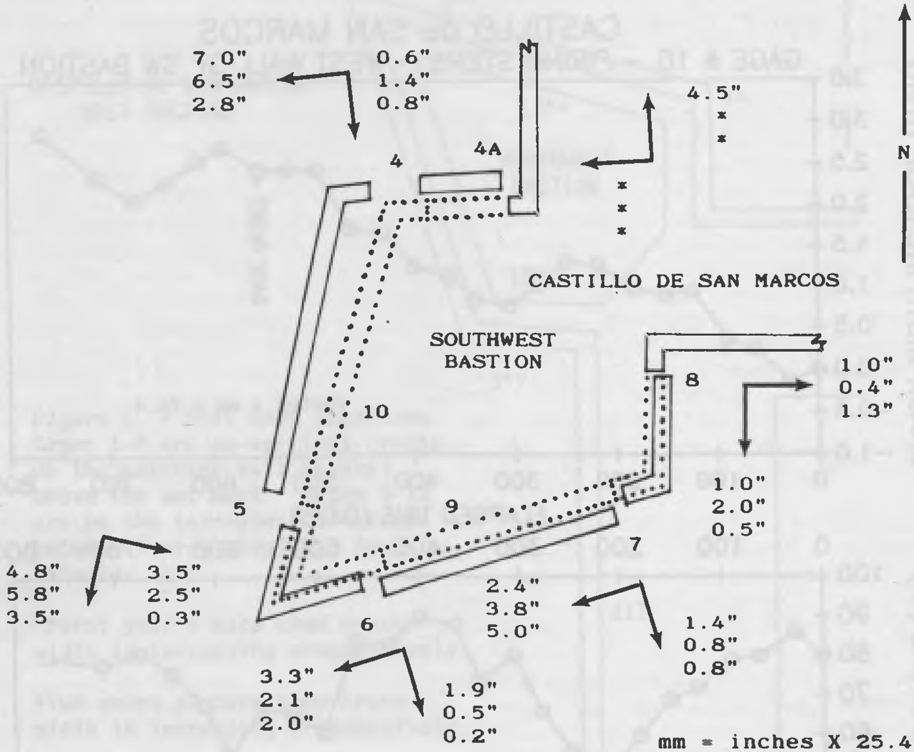

CASTILLO de SAN MARCOS
GAGE # 10 – FIRING STEPS – WEST WALL OF SW BASTION
CRACK WIDTH INCREASE (mm)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
−0.5
−1.0
GAGE RESET
inches = mm ÷ 25.4
0
100
200
300
400
500
600
700
800
ELAPSED TIME (DAYS)
TEMPERATURE (°F)
100
90
80
70
60
50
40
°C = 5/9(°F-32)

Figure 3. - Southwest Bastion Movements Since Original Construction



A STRUCTURAL MODEL FOR CONTINUOUS MASONRY BUILDING SYSTEMS IN HISTORICAL CENTRES

A. BERNARDINI , R. GORI , C. MODENA

SUMMARY

A procedure for seismic vulnerability analyses of continuous systems of traditional masonry buildings is presented.

The proposed method combines the results of simplified structural models to evaluate the seismic resistance of the buildings, taking into account their interaction along the common walls, with qualitative informations about some vulnerability factors derived through the judgements expressed by expert people during the survey.

The method has been tentatively applied to an assemblage of 20 buildings in Asolo (Treviso), that is one of the more interesting historical town center of the Veneto region.

The procedure leads to the evaluation of their present vulnerability and of the effects of the application of different strengthening techniques.

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INTRODUCTION

The structural behaviour of existing masonry buildings in the historical centres is greatly influenced by their contiguity causing the formation of continuous systems.
Buildings constructed in different periods, with different materials and for different uses are in fact grouped along the streets and around the squares; they form complex assemblages with often complicated and irregular distributions of the structural elements (walls and floor slabs) in plant and in elevation.

Such kinds of problems are being faced at the University of Padova in the ambit of a research program on the seismic vulnerability of historical centres in the Veneto region.
A simplified approach, derived from a model previously used for the analysis of single masonry buildings (Bernardini and Modena, 1986), is tentatively applied. The assemblage is subdivided into a number of separated structural units, on the basis of such aspects as the height, the period of construction, the type of employed materials, and so on. Rules are then given to take into account the interaction due to their connections along common walls.
The method is aimed to treat and combine in a precise and consistent formalistic way quantitative and qualitative informations on the structural consistency of every surveyed building.

In the following the methodology is briefly presented and the problems arising interpreting the results are particularly discussed considering a typical and very significant example, the town centre of Asolo , on the hills surrounding the Veneto plain and in a zone of medium seismic hazard.
The seismic vulnerability of the historical buildings grouped in a continuous systems along a street is evaluated, and the effects of the possible application of alternative strengthening techniques are discussed.

STRUCTURAL MODELS AND QUALITATIVE JUDGEMENTS

The observed damages of buildings stroken by earthquakes and the known results of experimental researches demonstrate that two principal modes of failure of the masonry walls occur under seismic excitations: the in plane shear mode of failure and the out of plane collapse; the second one is more frequent for the considered typology, particularly due to inefficient connections of each wall with transverse walls and floors.

The complex resistance mechanisms are supposed to be evaluated through two corresponding indexes, I_1 and I_2, representing the resistance of the building structure against an equivalent static, one directional and linearly variable with the height, horizontal seismic action (Bernardini et al., 1988) and measured by the ratios between the base shear at failure and the total weight of the building.
The shear mode of failure usually occurs at the first storey of perforated walls, due to diagonal cracks on masonry piers. This well known failure mechanism has been extensively studied by means of experimental investigations, and a simple formula has been derived to evaluate the

corresponding critical value of the base shear, and then the value of I_1. More difficult and complex is the evaluation of I_2 : many different resistance mechanisms and limit states must be considered for every wall, each one depending on the local critical resistance of the connections to the transverse walls and to the floors.

In the proposed model such mechanisms are separetely considered for the vertical and horizontal masonry strips and their resistances are simply added to evaluate I_2.

When the buildings are grouped in a nucleous, the values of I_1 and I_2 of each building are determined considering in an approximate manner the interaction effects between the adjacent ones.
In the evaluation of I_1 the commom walls are subdivided proportionally to the weight deriving from the adjacent buildings, i.e. approximately proportionally to the number of stories. In the evaluation of I_2 the restraining effects of all the walls of the adjacent buildings are considered.

The procedure to take into account qualitative informations on the buildings has been suggested by the possibility of using large data bases which are beeing created by means of a survey methodology largely employed in Italy to derive a vulnerability index of each building (Benedetti - Petrini,1984).
It assumes that the vulnerability can be measured through the wheighted sum of the values S measuring the "size" of eleven factors. The "size" S can assume the four values 0, 15,30,45, where the higher values correspond to the worst condition for the building . Trained operators performing the survey choose the value on the basis of simple rules or judgements.

Some of the factors are clearly related to a quantitative measure of the resistance of the building; they are here already included in the described structural models and are then disregarded in the following. The remaining seven parameters represent qualitative judgements on structural properties not considered in the models. They are listed in Table 1 with their proposed weight, and are used to calculate a new index

$$I_3 = \Sigma_i w_i S_i / (45 \cdot 3.15) \qquad (1)$$

which gives a synthetic measure, varying from 0 to 1, of the structural deficiencies of the considered building: the best conditions (I_3=0) correspond to the fact that code and good practice provisions are respected.

Table 1 Vulnerability factors and corresponding weights

1 - Wall system quality	0.15
2 - Soil and foundations interaction	0.75
3 - Floors interaction	0.5
4 - Elevation regularity	0.5
5 - Roof interaction	0.5
6 - Interaction of not structural elements	0.25
7 - General maintenance conditions	0.5
Total	3.15

The survey methodology considers furthermore that in some cases it is very difficult to derive the informations from direct inspections, which could be too long and expensive (e.g., when plasters and ceiling hide the connections between the walls and the floor structures).

In these cases it is required to express a judgement on the reliability of the informations regarding the seven parameters of Table 1, influencing I_3, and on the two parameters f_t (masonry tensile strength) and p (walls to floors restraining force per unit length) of the mechanical models, on which essentially I_1 and I_2 respectively depend.

VULNERABILITY EVALUATIONS

A safety criterion must be chosen to evaluate the reliability of each building, whose expected structural performace is assumed to be described by the three indexes I_1, I_2 and I_3, when subjected to earthquakes of given intensity . According (Bernardini and al.,1988), the reliability f_S (or the vulnerability Vu) is analytically expressed by the formulas

$$f_S = 1 - Vu = 0 \qquad \text{if } I_1/A \le c1 \text{ or } I_2/A \le c1$$

otherwise:

$$f_S = 1 - Vu = 1 - (1 - u)^{1/(a + 1)} \qquad \text{if } 0 < u < 1$$

0

$$f_S = 1 - Vu = 0 \qquad \text{if } u \le 0 \qquad (2)$$

$$f_S = 1 - Vu = 1 \qquad \text{if } u \ge 1$$

$$u = (c3 + c1 - c2 + \sqrt{(I_1/A - c1)(I_2/A - c1)}\ ) \ / \ (2\ c3 + a\ c4)$$

where A is the maximum base shear response devided by the weight of the building.

In the plane of the two parameters I_1, I_2 , a zone is defined between the safe (f_S = 1) and unsafe (f_S= 0) regions where the values of the reliability gradually varies from 1 to 0, with constant values on a family of hiperbolic curves (u = const , variable from 0 , boundary of the unsafe region, to 1, boundary of the safe region).

Such hiperbolic shape and the values of the parameters c1 and c2 allow to take into account the interaction between I_1/A and I_2/A due to both the two-directional characteristics of the ground motion and the mutual influence of the resisting mechanisms.
The parameters c3 and c4 define the maximum width of the zone between the safe and unsafe regions; c3 represents the positive or negative influence of the uncertainties of the assumed models to evaluate I_1, I_2 and A; c4 represents the maximum influence of the vulnerability factors described in Table 1.

When the values of c1, c2, c3, c4 are chosen (for the more common type of buildings in Italy it appeared to be reasonable to assume c1=0.5, c2=1, c3=0.1 and c4=1) the width of the zone corresponding to values of u in the range 0<u<1 and the values which f_S assumes depend on the qualitative judgements expressed on each building , through a relation between the index I_3 and the parameter a.

The evaluation of the vulnerability of each building requires the definition of rules to merge quantitative and qualitative informations. Two aspects must be solved: the relation between I_3, synthetizing qualitative judgements on the characteristics of the building and the parameter a (on which f_S depends); the influence of the quality of informations on the final measure of f_S.

The fuzzy set theory has been adopted fur such purposes: it gives in fact general formal rules to treat vague informations which in recent years have been already applied in the field of structural reliability. Details of the procedure are given in (Bernardini et al., 1989), and the results can be given, for each value of the seimic intensity A, and for each building, through the membership function of the fuzzy subset of interval of variation of f_S.

The building can be assigned to fixed classes of vulnerabilty using numerical techniques for ordered classification of the fuzzy subset fS. In the application five classes of vulnerability, i.e. VERY SMALL, SMALL, MEDIUM and GREAT, VERY GREAT to which the fuzzy subset are associated, are considered.

The results can be used not only to compare the vulnerability of every single building of a group but also the vulnerability of different groups of buildings; in the second case the statistical distribution of the buildings of a sample in the classes can be used.
Both the types of comparison are useful when retrofitting interventions are to be planned in a region, given for every group of buildings the expected intensity of the seismic action.

VULNERABILITY ANALYSIS IN ASOLO (TREVISO, ITALY)

The described methodology has been applied to the group of 20 buildings of the assemblage shown in Fig. 1.
The buildings have been built up in a period stretching from the 16th century to the 19th century, with a substantially homogeneous masonry

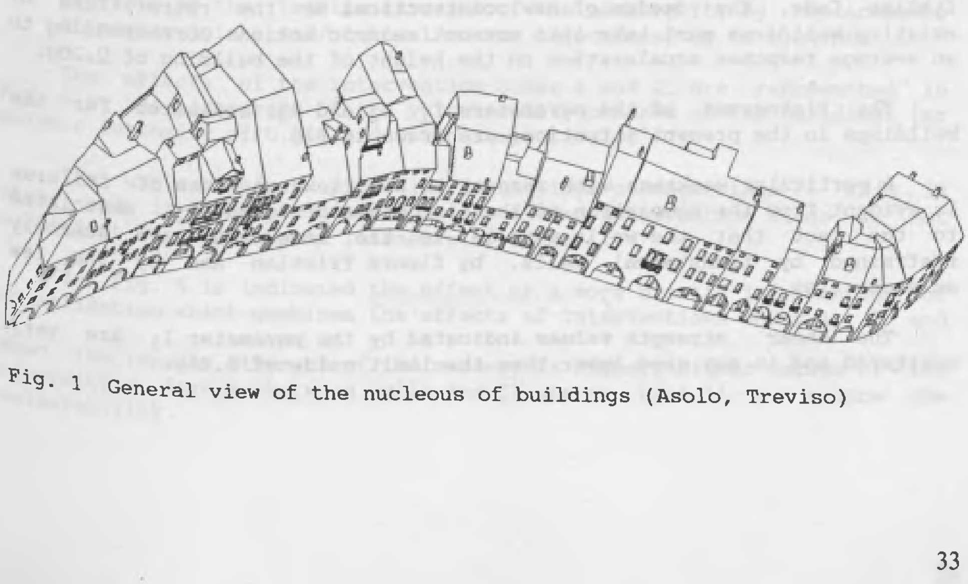

Fig. 1 General view of the nucleous of buildings (Asolo, Treviso)

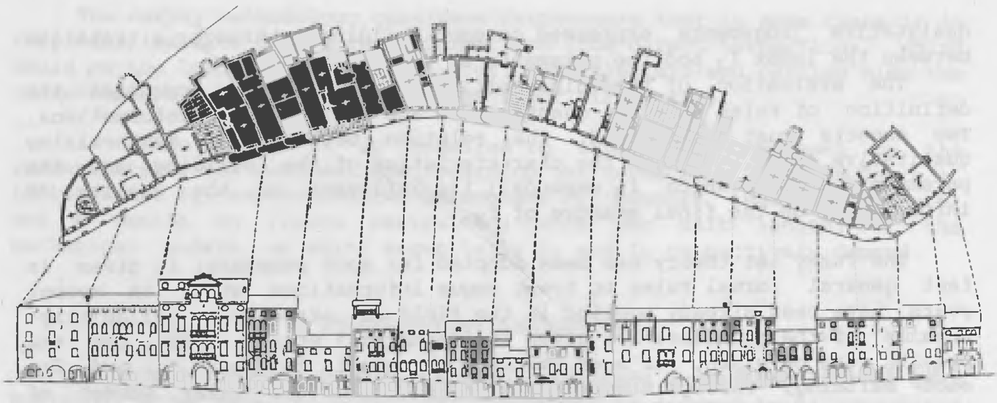

Fig. 2 Plant at the first storey and subdivision in single buildings

tipology, that is square dressed natural stone with mortar of lime and light wooden floors.

Many of the buildings have been retrofitted, probably after the occurence of past earthquake, by anchoring the masonry walls at the floor level by means of iron ties.

Some of the buildings have been recently restored: the light floors were partially or totally substituted by reinforced concrete floors, but no particular masonry strengthening was undertaken. In the present situation the average value of the masonry tensile strength f_t is estimated to be 0.08 MPa.

Typical traits of the considered group are the presence of the "porch" at the ground level of the main facade of any single building and a reduced ratio between the minor to major plant dimensions (Fig. 2). Both such characteristics reduce the shear strength of the walls system in the direction parallel to the facade.

Asolo is sited in a medium intensity seismic area. According to the Italian Code, the design of new constructions or the retrofitting of existing buildings must take into account seismic actions corresponding to an average response acceleration on the height of the building of 0.28g.

The histograms of the parameters I_1, I_2 and I_3, evaluated for the buildings in the present situation, are drawn in Fig. 3.

A particular weakness with respect to the flexural modes of failures is evident from the comparison of the histograms of I_1 and I_2, associated to the fact that the walls parallel to the long side are scarcely restrained by transversal walls, by floors friction and by the few existent ties.

The shear strength values indicated by the parameter I_1 are very scattered and in any case lower than the limit value of 0.28g.

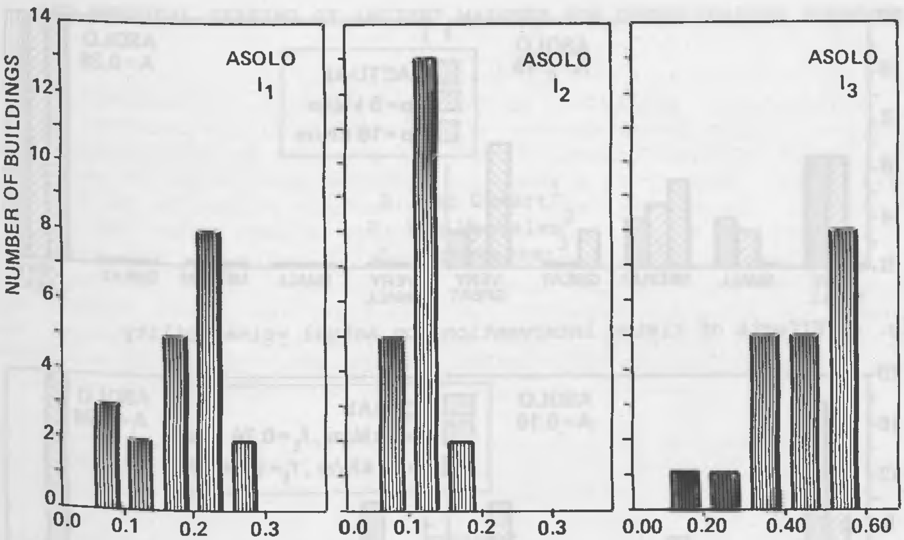

Fig. 3 Histograms of the I_1, I_2 and I_3 indexes for the buildings

Three types of building reinforcement interventions are considered:

1 - ordinary maintainance for the improvement of the existing situation, i.e. elimination of local weakenings for every building (e.g. holes for pipes or chemneys); such intervention reduce the "size" of the vulnerability factors listed in Table 1 and improve the reliability of the informations, and then modifies the "fuzzy measure" of the parameter I_3;

2 - systematic improvement of the wall to floor connections, and eventually also stiffening of the floors, so that the restraining force p can be increased up to values of 5 or 20 kN/m;

3 - increase of the tensile strenght of the masonry ft, by replacements of mortar joints and/or local mortar injections, up to 0.24 MPa.

The effects of the intervention under 1 and 2, are represented in Fig. 4 by the histograms of the vulnerability classes of the buildings for seismic intensity of 0.16g and 0.28g respectively.

The uselessness of providing connections stronger than 5 kN/m is clear and it is also evident that the tieing intervention alone is not sufficient for the seismic intensity of 0.28g.

In Fig. 5 is indicated the effect of a more drastic intervention of consolidation which combines the effects of interventions under 1, 2 and 3.
When the tensile strength of the masonry is higher, higher values of the restraining forces between walls and floors are usefull to reduce the vulnerability.

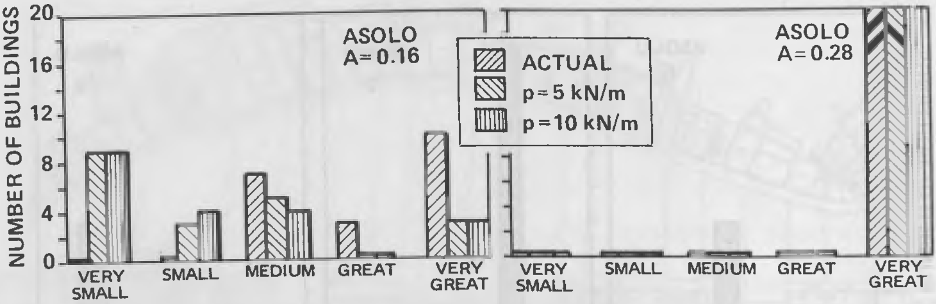

Fig. 4 Effects of tieing interventions on actual vulnerability

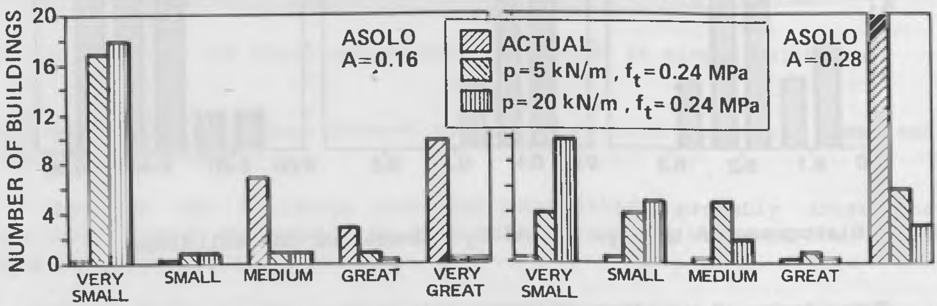

Fig. 5 Effects of tieing and masonry strengthening on actual vulnerability

The strenght increase obtained in such case with p = 20 kN/m seems to be adequate for most of the buildings, except those whose layout characteristics in plane require that new resisting structural elements must be designed and included.

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GEO-PHYSICAL TESTING OF ANCIENT MASONRY FOR CONSOLIDATION PURPOSES

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D. Van Mechelen[2]
J. Dereymaeker[3]

SUMMARY

Geo-electric measurements in combination with calibration corings are used for the survey and quality control of consolidation injections in historic masonry. The geo-electric measurements, executed with an asymmetric Dchlumberger disposition, give the distribution of the resistivity over the masonry mass. By means of calibration corings the resistivity data are related to void content, porosity and composition. After injection the same procedure is used to check the grout consumption and the uniform quality of the treatment. Two prototype consolidation and control works are presented : the castle of the counts of Flanders (12th Century) and the tower of the St. Mary's Basilica at Tongeren (15th Century).

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INTRODUCTION

Injection of hydraulic or polymeric grout is frequently used nowadays for the consolidation of degraded historic masonry. The principle of a consolidation by means of grouting is quite simple (Van Gemert 1986). Masonry is a composite material made of bricks or stones and mortar. The bearing capacity of the masonry is due to the strength of the stones and mortar, and to the adhesion between mortar and stones. This adhesion forces enable the masonry to resist to the splitting forces which arise in the masonry at loading. By injection of a polymeric or hydraulic grout in the pore structure of the masonry one can reduce the splitting forces and at the same time increase the adhesion between stones and mortar. This will result in a strong increase of the compression strength and the durability of the masonry.

To be effective the grout must be uniformly distributed over the masonry mass. Moreover, the grouting works will only be economically executable if the grout consumption can be estimated on before as a function of the masonry characteristics and of the required consolidation rate. The geo-electric sounding technique, combined with calibration corings before and after injection, can provide the necessary informations on the masonry to make such a design possible. The scheme was tried out on two prototype restoration sites : the castle of the counts of Flanders in Ghent (12th Century) and the tower of the St. Mary's Basilica at Tongeren (15th Century).

PRINCIPLE OF GEO-ELECTRIC SOUNDING OF MASONRY

The geo-electric method is based on the measurement of the electrical resistance of the stone mass (Van Gemert - Van Mechelen, 1988). The method is derived from the sounding techniques, used in geology for the examination of soil stratification. The basic equation is the law of Pouillet about the electrical resistance

$$j = E/\rho^{e},$$

where j = current density (Ampère/m²)
ρ^{e} = specific resistance (Ohm.m)
E = field intensity (Volt/m)

The equations are adapted for the three dimensional situation. In this three dimensional technique the specific resistance is determined as a function of the volume of masonry involved and of the distance L between the current electrode S1 and the potential measuring electrodes P1 and P2 (see fig. 1).

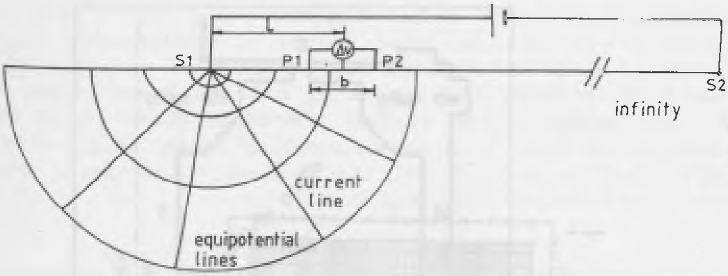

Figure 1. Geo-electric measuring scheme (asymmetric Schlumberger disposition).

The perfect contact between the masonry and the electrodes is obtained by dipping the electrodes in a $CuSO_4$-solution. A spherical tension field is created around the current electrode S1 like around a point source. By changing the distance L between the measuring electrodes P1 and P2 and the current electrode S1 the evolution of the resistance of a masonry volume with approximate dimensions L along the depth in the wall and b along the surface is measured. At the cross-points of a mesh, spread out over the masonry surface, the evolution of the resistance with the distance or depth L is measured. Two typical curves are shown in figure 2. In each measuring point the left end of the curve corresponds to the resistance of the parament. The curve for measuring station 1 in figure 2 shows that the resistance of the parament is high. The resistance of the following layers of material is lower because the measured mean value for the total masonry volume involved is decreasing. The rising curve in measuring point 4 indicates the presence of large voids just behind the parament. At greater depths the resistance is decreasing again, which results in a descending curve of the resistivity.

From the resistivity curves, which in fact are the mean values of the resistance of a growing slice of masonry, the resistivities are calculated as a function of the depth in the masonry.

After grouting the resistivities are measured again. In figure 2 the curves b show the change of resistivities for the same measuring stations 1 and 4 after a cement grout injection (Van Gemert - Van Mechelen, T.O.W.B., 1988).

The influence of the cement injection can be seen immediately. The curves b after injection are about horizontal, and in both stations the magnitudes are about the same. The grout has filled all the voids in the wall, and by its lower resistance has lowered the resistivity of the masonry.

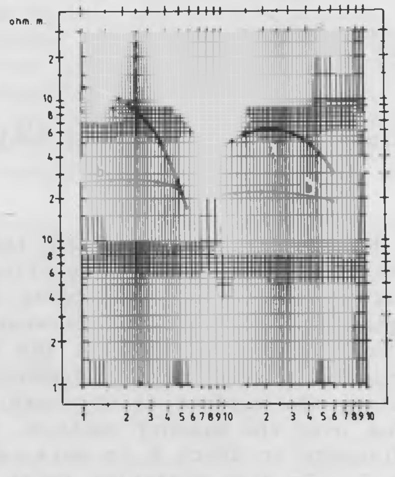

Figure 2. Evolution of resistivities in stations 1 and 4 before and after cement injection (curves b).

MASONRY EVALUATION BY MEANS OF GEO-ELECTRIC SOUNDING

The specific resistance (Ohm.m) or resistivity distributions in the masonry walls are calculated from the resistance curves (Fig. 2), and plotted in contour maps. An example is shown in figure 4, for the resistivities in the pillar NE of the tower of St. Mary's Basilica in Tongeren, Fig. 3.

In drawing the contour lines the resistivity of the interior parament with a thickness of 0.25 m has been subtracted. For example in this NE-pillar the calibration corings have been taken at the heights of 1 m, 3.5 m and 8.5 m above floor level, as indicated in figure 4. The corings are taken in those locations where the resistivity curves indicate changes in the properties. In this way the number of corings can be significantly reduced, and the corings are made in the right locations.

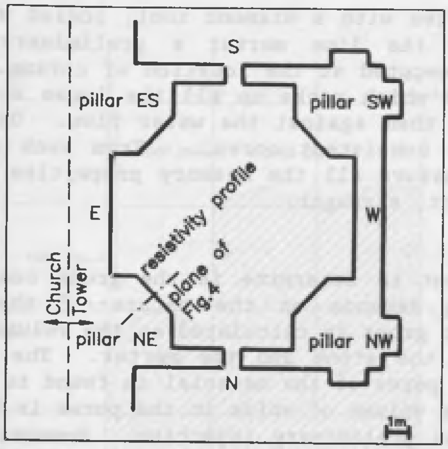

Figure 3. Cross-section of tower at Tongeren.

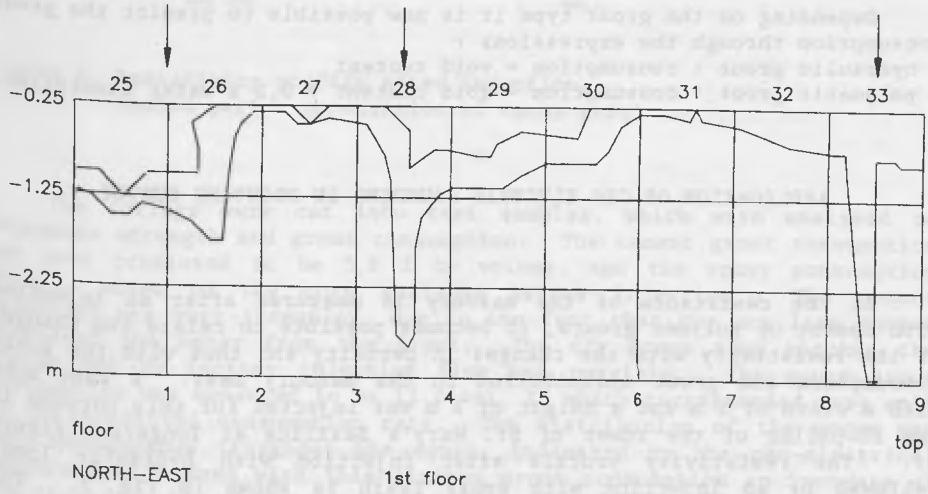

Figure 4. Resistivity contour lines in pillar NE of St. Mary's tower.

The corings are taken with a diamond tool, cooled with water. To avoid the wash-out of the lime mortar a preliminary injection of polyurethane resin is executed at the location of coring. At hardening this resin forms a foam which packs up all the loose stone and mortar particles, and protects them against the water flow. Only in this way it is possible to take consistent cores. From such cores the test samples are taken to measure all the masonry properties : composition, porosity and void content, strength.

An important element to determine is the grout comsumption. The grout consumption also depends on the nature of the grout. The comsumption of hydraulic grout is calculated as the volume of voids, not including the pores in the stone and the mortar. The penetration of hydraulic grouts in the pores of the material is found to be negligible. The determination of the volume of voids in the pores is hindered by the polyurethane foam of the preliminary injection. However, the foam can easily be removed by calcination at 400°C. At this temperature the polyurethane foam is burned, whereas the transformation of limestone ($CaCO_3$) into lime (CaO) and CO_2 is negligible. If an epoxy resin is used as consolidation grout, we must account for a penetration of resin in the material pores. For epoxy resin this penetration rate is found to be about 20 % of the pore volume.

Depending on the grout type it is now possible to predict the grout consumption through the expressions :
- hydraulic grout : consumption = void content
- polymeric grout : consumption = void content + 0,2 x water absorption.

APPLICATION OF GEO-ELECTRIC SOUNDING IN GROUTING SURVEY

If the resistance of the masonry is measured after an injection with cement or polymer grouts, it becomes possible to relate the changes of the resistivity with the changes in porosity and thus with the grout consumption and grout distribution in the masonry mass. A test zone with a width of 1 m and a height of 2 m was injected for this purpose in the ES-pillar of the tower of St. Mary's Basilica at Tongeren (figure 3). The resistivity profile after injection with hydraulic lime, followed by an injection with epoxy resin is shown in Fig. 5. The shaded part shows the zones where the injection grouts have penetrated.

The coring, taken in the zone with only cement injection, revealed to be very incoherent whereas the corings with combined cement and epoxy injection and with epoxy injection were very coherent.

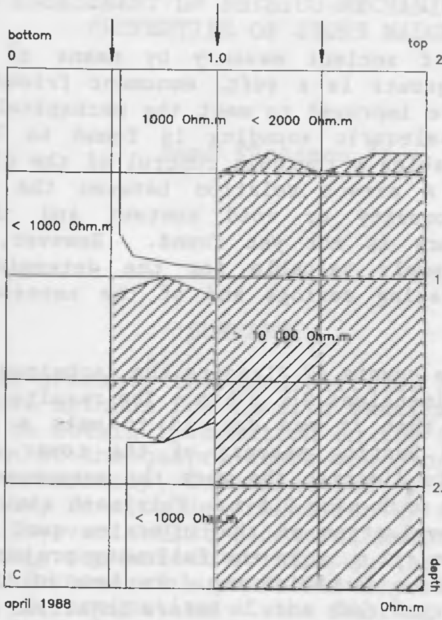

Figure 5. Resistivity profile after injection.
Shaded part : penetration of epoxy grout.

The corings were cut into test samples, which were analysed to determine strength and grout consumption. The cement grout consumption had been predicted to be 5,8 % by volume, and the epoxy consumption 10,2 %, based on the core analyses before injection. The cement injection was very irregular, due to the fact that the dry lime mortar extracted the water from the grout. The dry grout than blocked the cracks and no further injection flow was possible. The epoxy grout consumption was measured to be 12,6 vol. %, which corresponded very well to the predicted consumption rate. The distribution of the grout was very homogeneous troughout the zones, indicated by the geo-electrical measurements. Even with this limited grout consumption an increase of the strengths from 1 N/mm² to 3,5 N/mm² was achieved.

CONCLUSIONS

The consolidation of ancient masonry by means of injection of hydraulic or polymeric grouts is a soft, monument friendly technique. The altered materials are improved to meet the mechanical and physical requirements. The geo-electric sounding is found to be a reliable control method, which enables a thorough control of the distribution of the injection grout. A direct relation between the value of the resistivity and the porosity or void content and the mechanical properties of the masonry is not yet found. However, the maps of contour lines are extremely valuable for the determination of the locations of the calibrating corings and of the necessary number of corings.

In the castle of the counts of Flanders the technique was tried out on a wall segment with dimensions 2 m x 3 m. The results of these tests were very reliable, so that it was decided to make a geo-electrical survey of all the load bearing masonry of the tower of St. Mary's Basilica in Tongeren. To diminish the work the measurements were done with a coarse mesh, with mesh size 0,5 m. This mesh size was to big to allow for an accurate evaluation of the injection quality on a small test zone (dimensions 1 m x 2 m). In the following projects the overall measurement will be used to determine the locations of the calibration corings and of the injection test zone. Before injection this test zone will again be sounded with a fine measuring mesh, so as to detect all smaller details of the internal composition of the masonry in the test zone. In this way we will try to quantify the relations between changes in resistivities and the amount of grout consumption and strength increase.

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ON THE USE OF NON-DESTRUCTIVE METHODS IN THE DIAGNOSIS AND ASSESSMENT OF PHYSICO-MECHANICAL RESIDUAL PROPERTIES OF STONE MASONRY

Ioan Făcăoaru *)

SUMMARY

The paper presents the possibilities offered by different non-destructive methods for in-situ examination of stone masonry, in order to obtain information on the following matters: identification of the quarry where the stone comes from, degree of stone anisotropy, determination of elasto-dynamic constants of stone, in-situ determination of compressive strength of stone and its variability, determination of structural damages, following mechanical actions on the masonry, determination of the thickness of superficial layers, damaged by aggressive actions, determination of the filling degree of joints, with mortar, determination of mortar compressive or tensile strength, information about the development of cracks at the interface between mortar and stone.

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3o March 1989

1. Introduction

The stone masonry is a macro and micro heterogenous material made from two components: stone and mortar. Its behaviour, under loads and in time, depends on the behaviour of its components and on the properties of the interface layer. Non-destructive methods can inform about the properties of each component but can give only few informations about the properties of the interface layer, this being their limitation.

2. Estimation of stone properties

The main advantage when using non-destructive methods for testing stone is the direct in-situ testing, without any damage. The problems which can be solved are:

a). Identification of the origin quarry.
b). Establishment of the anisotropy degree of the stone.
c). Computation of the elasto-dynamic constants of stone such as: Young modulus, shear modulus, Poisson's ratio.
d). Estimation of compressive strength of stone.
e). Identification of structural damages following high loads.
f). Thickness estimation of the damaged superficial layer.

Identification of the origin quarry can be done using ultrasonic pulse technique and measuring the pulse velocity along the three main direction of anisotropy: V_{L1} ; V_{L2} and V_{L3}. If these values are compared to those characterising different quarries, the quarry of origin can be identified.

The degree of anisotropy of the rock is given by the ratio:

$$\alpha = \frac{V_{max}}{V_{min}} \tag{1}$$

where: V_{max} - is the maximum pulse velocity on the block
V_{min} - the minimum pulse velocity on the same block

The degree of anisotropy gives an ideea about the possible variation of mechanical properties along different directions.

The computation of elasto-dynamic constants of stone:

- Dynamic Young modulus (E_d) by using relation (2):

$$E_d = V_L^2 \frac{\gamma}{g} \frac{(1 + \nu_d)(1 - 2\nu_d)}{1 - \nu_d} = V_L^2 \frac{\gamma}{g} f(\nu_d) \tag{2}$$

where: V_L - is the longitudinal pulse velocity
γ - the apearant specific weight
ν_d - the Poisson's ratio
g - the acceleration of gravity

- Dynamic Poisson's ratio (ν_d) by using relations (3):

$$\nu_d = \frac{1 - 2a^2}{2(1-a^2)} \quad \text{where} \quad a = \frac{V_T}{V_L} \tag{3}$$

V_T being the transversal pulse velocity
- Dynamic shear modulus G_d by using relation (4);

$$G_d = V_T^2 \frac{\gamma}{g} \tag{4}$$

or from the two elastic constants E_d and ν_d.

The compressive strength of stone can be estimated by one of the following non-destructive methods:
- ultrasonic pulse method
- mechanical surface methods
- combined non-destructive methods.

The ultrasonic pulse method is based on the measurement of longitudinal pulse velocity, by using a surface technique and on the correlation between this velocity and the compressive strength,which depends on the nature of the rock. An example of such correlation,for limestone of Podeni-Constanţa, is represented in fig.1. The accuracy which can be expected is about ± 15%.

The mechanical surface methods,which can be used,are various. We will refer further on to the methods used in Romania:

a). The rebound method based on the use of Schmidt hammer.The method suppose the existence of a plane, roughless surface loo x loo mm,which, if doesn't exist, must be prepared. In such zone at least 6 shots must be performed. After applying a selective criterium,the mean value of the rebound index is computed. In this case also a graph of correlation rebound index - compressive strength must be used. An example of such a graph, for the same rock,is presented in fig. 2. The accuracy which can be expected is about ± 2o%.

b). The pull-off method, based on the measurement of the force necessary to pull-off a metalic discus, sticked by an epoxy resin, to the stone surface. A graph of correlation pull-off force - compressive strength allows the computation of the last.

The combined method used in Romania is based on pulse velocity and rebound index measurements. A graph having in its two axes these two properties and being crossed by equal strength curves allows the determination of the compressive strength. An example of such a graph is given in fig. 3. for the limestone of Podeni. The accuracy to be expected is ± 12%.

If we are looking for structural degradations such as microcracks, due to extraordinary mechanical loads, for example those produced by seismic actions, the pulse velocity measurements must be performed by direct transmission, both in supposed damaged and undamaged zones. The damaged zones are identified by using the following criterium:

$$V_{dam} < o,8\ V_{max} \tag{5}$$

where: V_{max} - is the maximum measured pulse velocity
V_{dam} - the velocity which identifies the damaged area

All tests must be performed in the same direction of rock anisotropy. A chart is drawn, showing the supposed damaged area

This must be compared to the one resulting from load considerations, to avoid the influence of stone heterogenity.

The estimation of layer thickness damaged by physical or chemical agressive actions can be done if the block is long enough to allow pulse propagation both in damaged and undamaged layers, according fig. 4. The layer thickness is:

$$x = \frac{L_o}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}} \quad (6)$$

where: L_o - is the distance at which mixed propagation, in damaged and undamaged layers, is quicker that straight line propagation in the damaged layer
$V_2 = tg\,\beta$ - the pulse velocity in the undamaged layer
$V_1 = tg\,\alpha$ - the pulse velocity in the damaged layer

3. Estimation of mortar properties

The problems to which non-destructive method can give an answer, in this field are:

a). Estimation of filling degree of joints with mortar.
b). Estimation of mortar compressive or tensile strength.

The filling degree of a masonry joint,included its uniformity, can be estimated by using X-ray or gamma-ray radiography. When using usual hard rays,the thickness of stone masonry which can be investigated is limited to 4o-5o cm. The method is expensive, due mainly to the film cost and of low productivity, due both to the exposure and processing time needed. The rules of hazard radiation must be observed.

In special circumstances the filling degree can be estimated also by ultrasonic pulse technique. This method by using a surface technique, according fig. 5, can give the depth of an empty joint, from the surface (h_v), applying the relation (7):

$$h_v = \frac{L}{2}\sqrt{\left(\frac{T_v}{T_p}\right)^2 - 1} \quad (7)$$

where: L - is the distance between the transducers
T_v - the propagation time across the empty joint
T_p - the propagation time across a filled joint

Mortar strength estimation might be performed by using one of the following non-destructive methods:

a). Superficial hardness methods.
b). Ultrasonic pulse method.

The superficial hardness appear as most adequate, due to the small thickness of the mortar in the joint. Methods belonging to this cathegory are: hellenic method, Windsor method,rebound methods, identation methods. The rebound methods, largely used in Romania, will be discussed.

When testing the mortar in the joints an adaptation of the existing equipment for concrete is needed. The impact bar surface must be reduced from 2o mm diameter to maximum 6 mm and the shock energy from 1 N.m. to maximum o.o5 N,m.

The ultrasonic pulse method can be used for mortar strength estimation in a joint if the following conditions are observed:

a). The joint is completely filled with mortar.
b). The mortar is adherant to the stone.
c). The stones, adjacent to the joint, have flat surfaces.

The testing schema, based on a surface technique, is illustrated in fig. 6. It implies three measurements:

a). A time measurement (T) on a path length L, across the joint, composed of three parts: two L_p on each neighbouring stone and one L_m on mortar (fig. 7a) according the relation:

$$L = L_{p1} + L_m + L_{p2} \qquad (8)$$

b). Two time measurements (T_1) and (T_2) on stones 1 and 2. The pulse velocity is given by the relation (9):

$$V_M = \frac{L - (L_{p1} + L_{p2})}{T - (T_1 + T_2)} \qquad (9)$$

Further on either the rebound index N_M or the pulse velocity V_M,both measured on mortar,are transformed in mortar compressive strength, by using graphs of correlation. Such graphs are obtained by testing specimens destructively and non-destructively.

4. Properties of the interface layer

The information are restrained to the signalisation and eventual the estimation of the cracks, on the interface stone - mortar, in joints perpendicular to the masonry surface.

If the joint is not completely filled with mortar,from the total depth (h_v), given by relation (7), the depth unfilled with mortar (h_n) must be substracted:

$$h_f = h_v - h_n \qquad (1o)$$

5. Conclusions

Non-destructive methods can bring useful informations about the bearing capacity and the actual state of a stone masonry. This paper has aimed to give a review of the existing possibilities,in order to stimulate the use of such methods. Our own experience is not large due to the small extent in which stone masonry is used in our country. It is our hope that countries where such a building system is largely used,will take into consideration the possibilities offered by non-destructive methods.

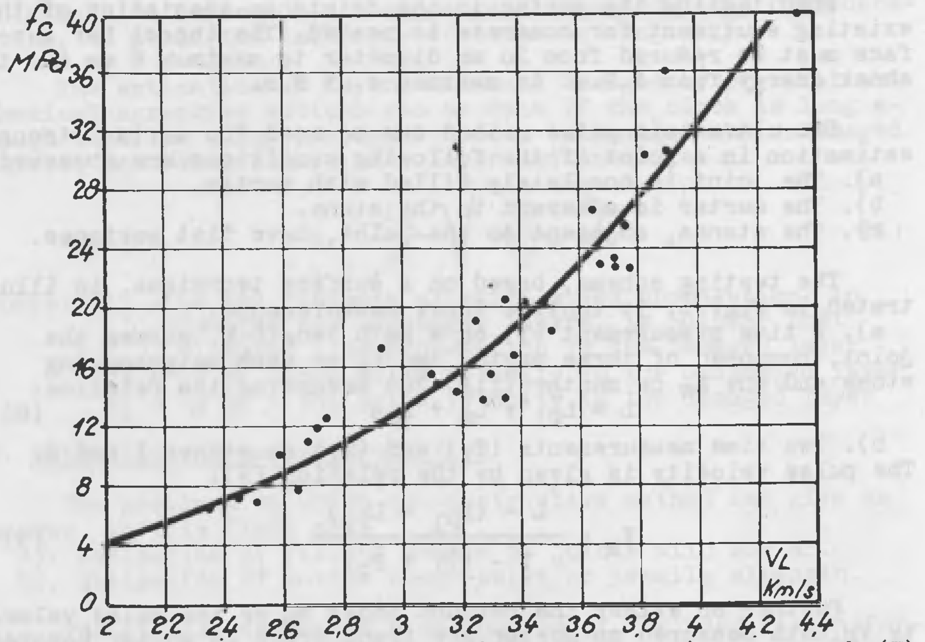

Fig. 1 - Relation ship pulse velocity - compressive strength (P)

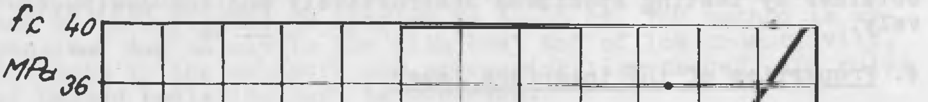

Fig. 2 - Relation ship rebound index - compressive strength (P)

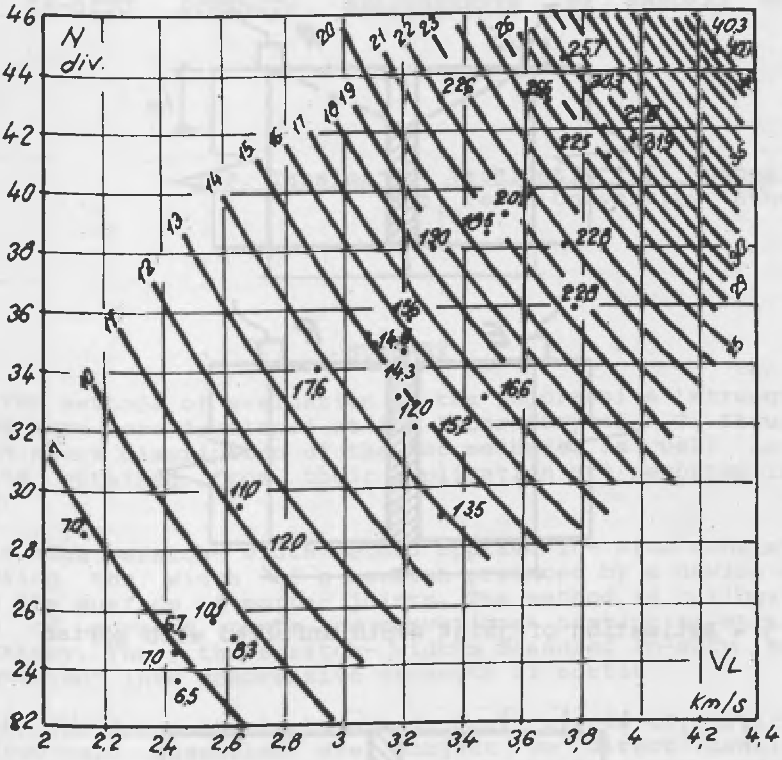

Fig. 3 - Equal strength curves in plane V_L - N (limestone P)

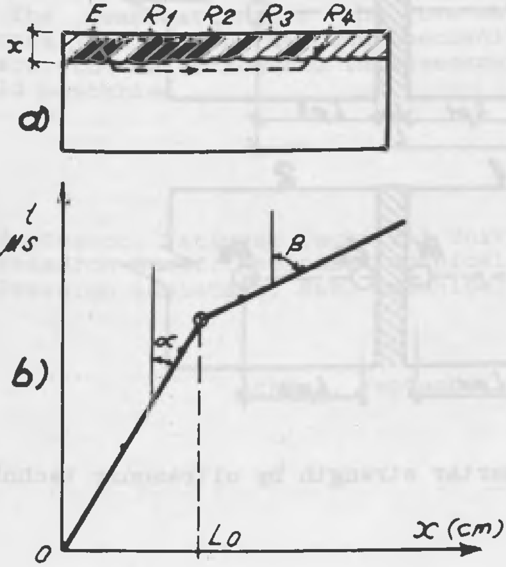

Fig. 4. Estimation of thickness of damaged layer.

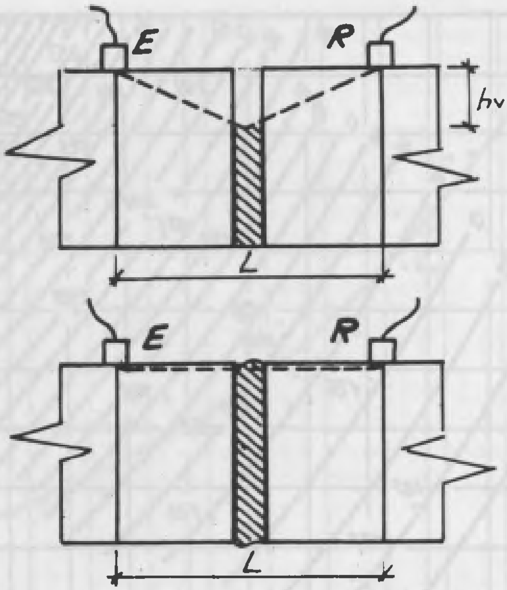

Fig. 5 - Estimation of joint depth unfilled with mortar

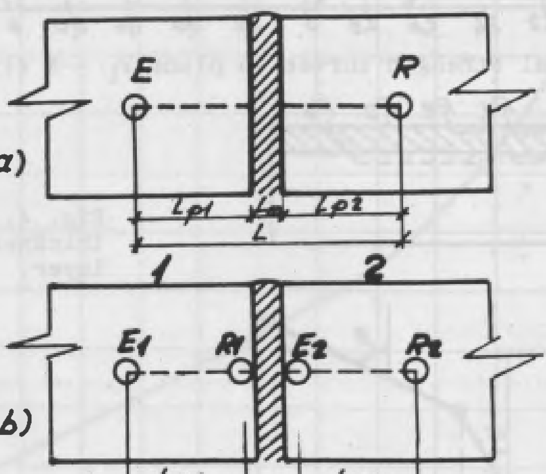

Fig. 6 - Estimation of mortar strength by ultrasonic technique

IN-SITU STRENGTH MEASUREMENTS OF MASONRY MORTARS

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SUMMARY

Two methods of evaluation of the compressive strength of old mortars were developed at the Laboratory of R.C. Structures NTUA. A short discription of the two methods as well as some results obtained from their application are reported in this paper.

a) The scratch- width method applied in- situ consists of measuring the width of a scratch produced by a devise moving along the surface of mortar joints. The method is calibrated by means of scratch tests and convetional testing of mortars in Laboratory. Thus, the scratch- widths measured in-situ may be "translated" into compressive strength of mortar.

b) The fragments-test method: Small pieces of mortar taken out from old masonries are subject to direct tension by applying a testing method developed at the Laboratory of R.C.

The combination of the two methods allows for a rather accurate estimation of the mechanical characteristics of mortars to be introduced in assessment, analysis and redesign of old masonries.

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Athens, September 1989.

1._INTRODUCTION

a) The need to estimate the strength of mortars used in old masonries cannot be overemphasised: If the compressive strength f_{mc} of the mortar is estimated and the average width of joints is measured, then the compressive strength f_{wc} of the masonry may be easily estimated by means of relevant empirical formulae.

However, since it is almost impossible to take and test adequately large samples of mortars (especially from ancient monuments), the necessity of developing <u>in-situ</u> measurements of f_{mc} becomes apparent.

b) To this purpose, two complementary methods were developed in the Laboratory of R.C. Structures NTU,Athens:

- The "scratch-width" (SW) method, as desribed in § 2,leads to a (roughly approximated) compressive strength of the mortar.
- The "fragments-test" (FT) method, as described in § 3, allows for the determination of the tensile strength of the mortar, thus to an indirect estimation of its compressive strength.

2._ THE SCRATCH-WIDTH METHOD

a) The method is successfully used for the compressive strength estimations of stones (Tassios, Mamillian, 1986.)

A standard pointed steel-bar (retained against the wall by means of a standard load, Fig. 1) is dragged along a mortar surface; the width of the scratch produced in the mortar is subsequently measured by means of appropriate lenses at 10 to 20 equidistant points. The most frequent value w_s of scratch widths is subsequently empirically related to f_{mc}. A set of different standard loads are calibrated to the purpose, in order to suit different levels of mortar strengths (Fig.2).

The same principle is now dublicated for in-situ applications: The pointed bars are the same, but the load (applied, this time, horizontally) is secured by means of a compressed air chamber contained in the instrument. The dragging of the instrument along a joint is made by hand; a sliced aluminium plate is fixed on top of the masonry face to facilitate this rectilinear movement (Fig. 3).

b) An application example, taken from measurements on the fifteenth- century church of Saint George (Megara, Greece), is shown in Table 1 and Fig. 4. An estimated compressive strength of mortar, equal to 3.00 up to 3.50 N/mm² has been found.

Table 1 : Scratch width measurements in Saint George Church, Megara.

No	Load [N] 5.00	Load [N] 10.00
1	1.9	2.0
2	1.1	1.9
3	1.1	1.2
4	1.2	1.5
5	1.0	1.7
6	1.0	1.7
7	1.3	1.8
8	1.2	2.0
9	1.3	
10	1.2	
11	1.1	
12	1.1	
13	1.2	
14	1.1	
most frequent value W_s [mm]	1.2	1.9
f_{mc} [MPa]	3.5 (from 2.2 to 5.1)	2.95 (from 2.5 to 4.5)

3._ THE FRAGMENTS-TEST METHOD

a) Since full cores of mortar cannot be taken from masonry monuments (and since the width of mortar joints is frequently inadequate to allow for shaping a reliable compressive specimen), the following hybrid method has been developed in the Laboratory of R.C. Structures, NTU Athens.

Small mortar-fragments (gravel size) are taken from the joints and arranged in a special mould within a strong matrix (an epoxy resin or a much stronger mortar) as shown in Fig. 5a. An easily applied direct-tension test (Fig. 5b and Fig. 6a) is carried out, as developed in the Laboratory of R.C. Structures, NTU Athens (Katsaragakis, 1987).

b) Thus a tensile strength of mortar's fraqments is determined, denoted $f_{mt,fr.}$, which differs of the:

$f_{mt,int}$ direct tensile strength of intact mortar specimens, and

$f_{m,flex.}$ flexural tensile strength of intact mortar specimens.

It has to be recognised that fragments, theoretically at least, should exhibit higher strengths than intact specimens. However, inevitable eccentricities in the proposed "fragments - test", may produce a reduction of test results; yet for the actual state of the art (and for low- srtength mortars) a preliminary investigation has shown that:

$$f_{mt,fr} \nless 0.70\ f_{mt,int} \qquad \text{(Equ. 1)}$$

Besides, if for the same category of mortars

$$f_{mt} = \frac{1}{3}\sqrt{f_{mc}} \quad \text{[MPa]} \qquad \text{(Equ. 2)}$$

then, a rough estimation of mortar's compressive strength may be carried out by means of the following expression.

$$f_{mt,fr} = \frac{1}{4}\sqrt{f_{mc}} \quad \text{[MPa]} \qquad \text{(Equ. 3)}$$

4._CONCLUSIONS

Research is followed-up in this direction; besides, carbonation effects have to be studied.

However, it is believed that the combination of the scratch-width method and the fragments-test method may considerably improve our knowledge on the in-situ strength of old masonry mortars.

R E F E R E N C E S

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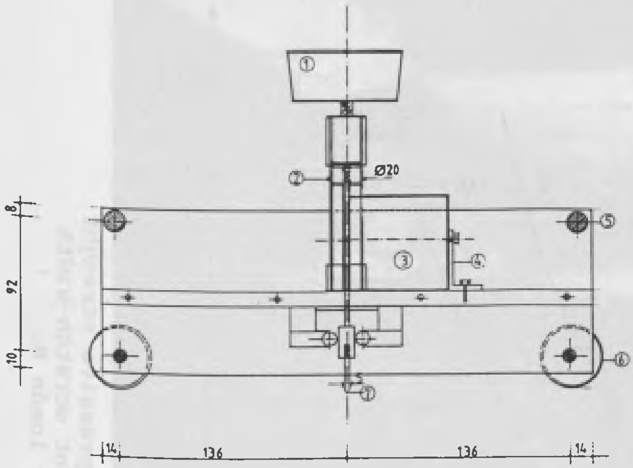
Tassios T.P., Mamillan M. : " Valutazione strutturale dei monumenti antichi ", Edizioni Kappa, Roma, 1986, (Full english text included).



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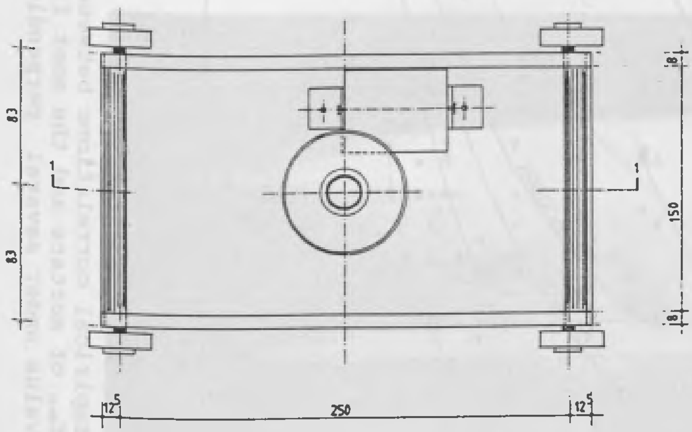

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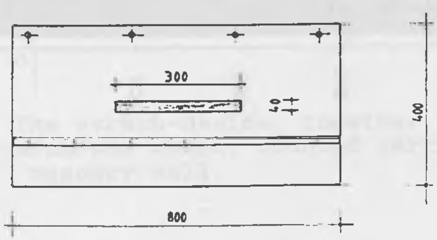

Guiding sheet AL 1:100

1. manometer
2. piston Ø20
3. pressure chamber
4. L 20-30-3
5. handles Ø14
6. ball-bearing Ø40 SKF
7. steel point (angle 60°)

Fig. 1: The dragging device for scratching on mortar surfaces. A constant load (secured by means of compressed air) acts on the scratching steel point.

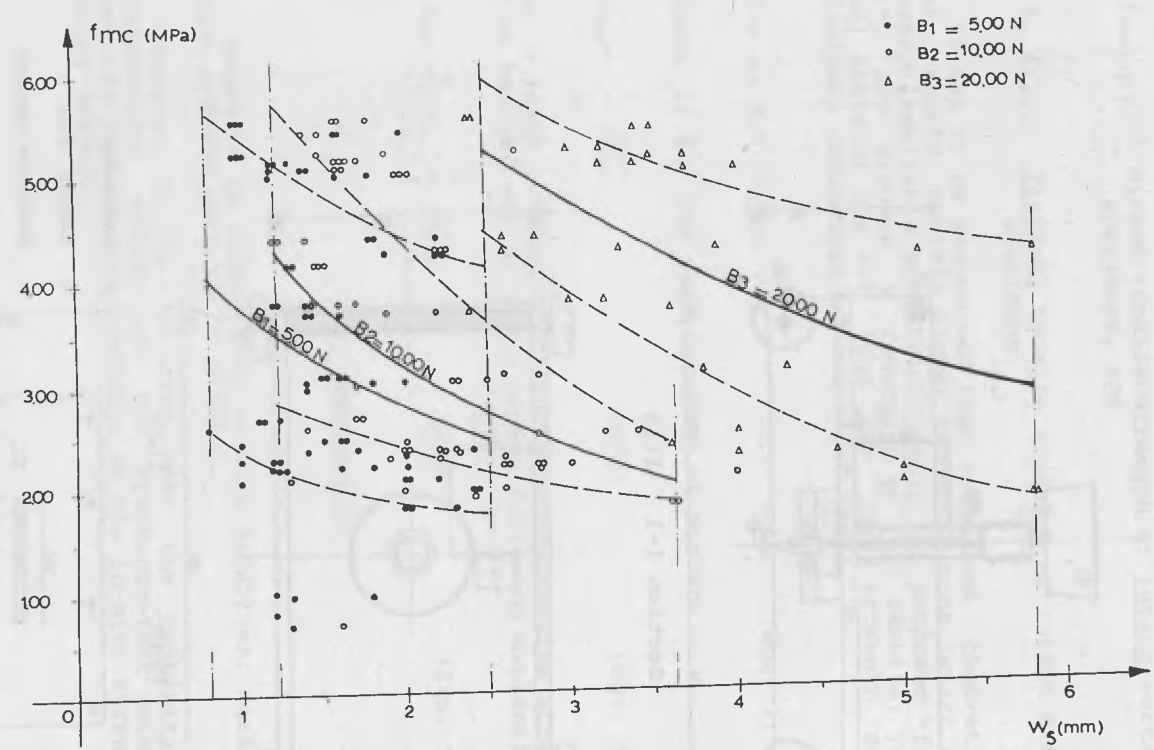

Fig. 2 : Empirical correlations between compressive strenght f_{mc} of mortars and the most frequent scratch-width value under several perpendicular loads B.

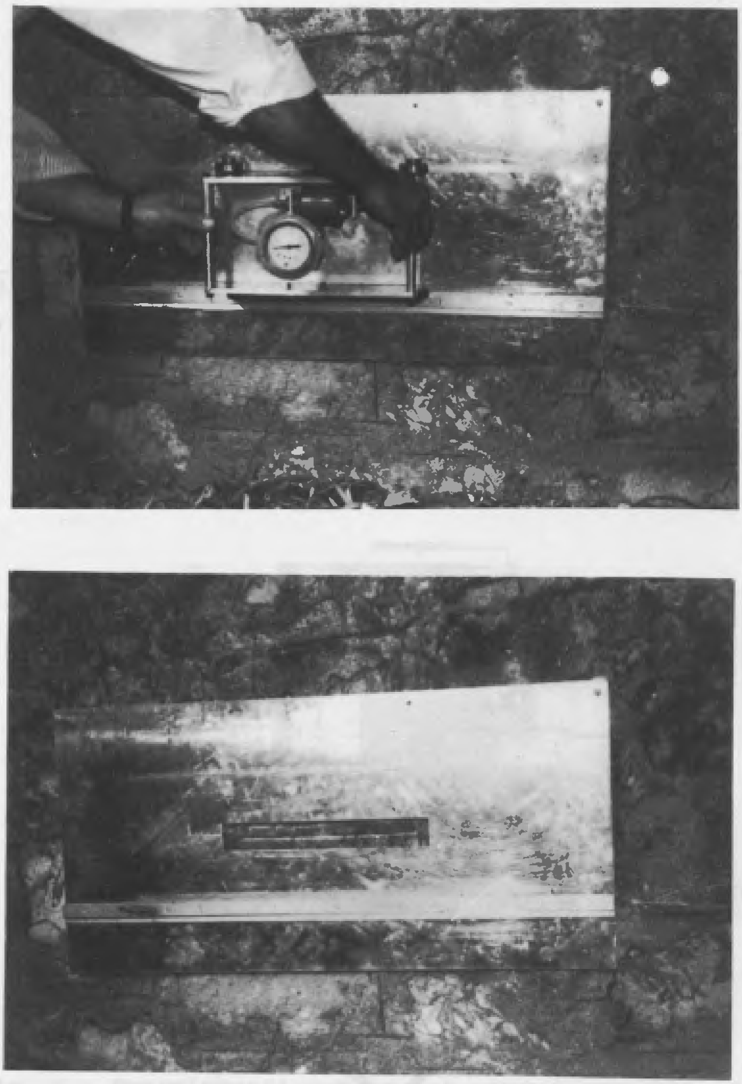

Fig. 3 : The scrach-davice, together with its guiding aluminum sheet, mounted vertically on a brik-masonry wall.

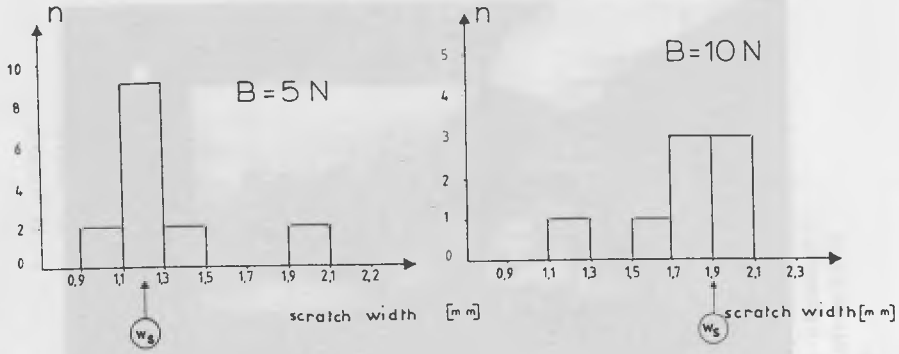

Fig. 4 : Examples of histogrammes of scratch width measurements (Saint George Church).

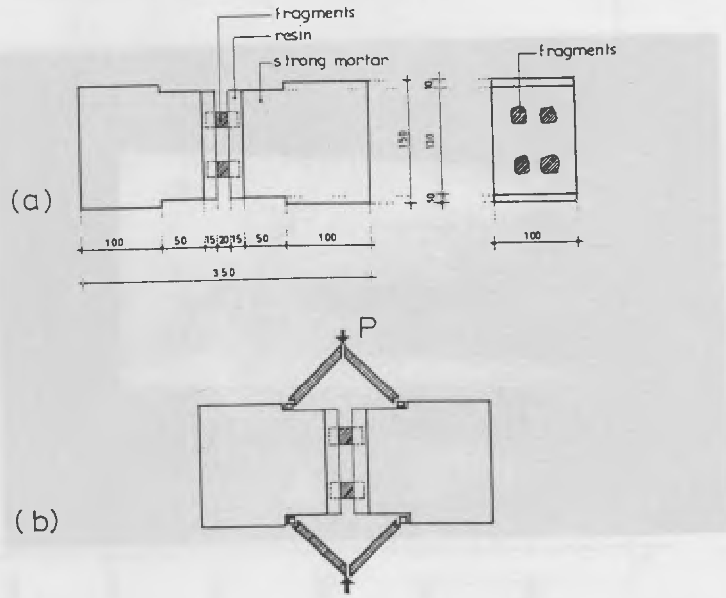

Fig. 5 : Test specimen's geometry (a), and direct tension arrangement (b) of the "fragments- test".

Fig. 6 : Experimental set-up (a), and specimen's cros-section after rupture (b).

CHARACTERISTICS AND ASSESSMENT OF OLD MASONRY STRUCTURES

Rowland J Mainstone *

SUMMARY

Different types of stone masonry found in historic buildings are reviewed and their characteristic structural behaviour discussed, noting both the differences and similarities in this behaviour.

The alternative equilibrium and elastic approaches to structural modelling and assessment are then presented and the choice between them considered.

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10 October 1989 (revised 14 November)

TYPES OF MASONRY

Masonry in columns, walls, piers and architraves

The Classical Greek masonry of the Parthenon consists of large blocks of marble accurately dressed to provide near-uniform horizontal bearing either over the whole bedding faces (in blocks destined to be used in stylobates or walls or as architraves or lintels) or (in column drums) over continuous broad circumferential bands. Wall blocks and architraves were similarly dressed around the margins of their vertical faces to give closely fitting vertical joints, except that the hidden rear faces of the blocks were left undressed where there was to be more than one skin. Accurate location and close jointing of the blocks were ensured by means of metal cramps and wooden dowels.

This masonry was not unique. Although some details suggest direct timber prototypes, it owed as much or more to an ancient masonry tradition that went back for almost two millennia to early dynastic Egypt. In the cautious first major use of stone there in the pyramid complex at Saqqara, the blocks were mostly little more than facings to retain masses of rubble behind. But masonry composed of much larger blocks dressed on all faces became structurally autonomous a century later in the Valley Temple of Chephren at Giza, prototype of all masonry column-and-beam structures with monolithic columns. Then, in temples of the New Kingdom, masonry of broadly similar character to that of Classical Greece was used, differing from this in little more than the use of different types of stone and the manner of cramping the blocks. Such masonry is found again in the *opus quadratum* of late Republican and early Imperial Rome and in the walls and piers of many major buildings of the Eastern Empire and its successors.

But, alongside this tradition of excellent ashlar, we find masonry which differs considerably, either in the dressing of the individual blocks of stone and the way in which they were fitted together or in the uniformity of construction through the thickness.

Other types of dressing ranged from none at all, or very little, to full dressing of the face combined with dressing of the meeting surfaces only around the outer margins to give close jointing only at the face. Where there was little or no dressing, the blocks were selected to form stable assemblages, usually with smaller stones wedged into the larger gaps to assist. In most early partially dressed masonry, the blocks were polygonal on the face rather than squared. More recently they have usually been squared, the close jointing on the face sometimes deceptively giving an appearance of fine ashlar (Fig.1).

Only occasionally in these other types of masonry was the whole thickness constructed of similar blocks similarly fitted together - as, for instance, in the free-standing massive defence walls at Tiryns where internal passageways and other openings were bridged by corbelling and rudimentary arch-like forms.

More commonly there was a separate skin on each face and some fill between. This fill might be composed of anything from earth and small stones to something closely akin to modern concrete, a weaker mortared rubble being probably the commonest in much post-Roman building (Fig.2). The inter-bonding between skins and fill also varied, usually being dependent on either the natural irregularities of the internal faces of the two skins or the deliberate incorporation of stones projecting well into the fill at suitable intervals. An even greater departure from a desirable homogeneity could result from changes after construction such as encasing an existing pier in new masonry to strengthen it (Fig.3). Akin to the composite masonry that resulted from this practice is that found in many Gothic piers, in which the main bulk of the pier, already far from homogeneous, was surrounded by slim monolithic shafts of a harder stone set against the cleavage.

Arch and vault masonry

For more than two millennia, the only true arches seem to have been of brick, those of significant span being found chiefly over gateways in defence walls constructed of the same material. The ample abutment for the arch thrusts which these provided on each side was, no doubt, an important factor in permitting modest increases in span over what had been achieved originally.

The initial transposition into stone - with voussoirs dressed to an accurate wedge shape over their whole bearing faces - seems to have taken place in about the early 3rd century BC in Greek defence walls. This form then spread widely throughout the Roman empire and continued to be used in most later stone architecture, either as a simple arch or as an arched rib in vaults of various kinds. Profiles varied. But the only variations in the masonry of the arch or rib itself that are worth noting here were in the dressing of the upper surface of the voussoirs and in provisions for keying block to block. Sometimes the upper surfaces were dressed to bond with superimposed masonry, while sometimes they were cut to a continuous smooth curve parallel to the intrados. Usually there were no provisions for keying block to block. Less frequently, dowels were used to impede relative slipping, or the meeting faces of the blocks were joggled instead of being cut on a single radial line, presumably for the same reason. In one more variant, long narrow blocks following the curve of the arch were set side-by-side like links in a chain separated by other narrow blocks that served as hinges.

Similar masonry to that of the voussoir arch is found in those barrel and groined vaults which are merely arches extended laterally or a pair of arches so extended and intersecting. It is likewise found in many domes, semidomes and domical transition elements like the squinch and pendentive.

In ribbed vaults however, including ribbed barrel vaults, the infills between the ribs were often constructed of less accurately cut masonry, of mortared rubble, or even of brick.

The use of mortar

In most early masonry using large blocks of undressed or only partly dressed stone, the blocks rested directly on one another. In masonry of well cut ashlar the same was true, or they rested only on the thinnest of mortar beds which probably served chiefly as a lubricant to assist in final positioning of the larger blocks. Slightly thicker mortar beds served to give more uniform bearing where the dressing was less precise.

Conversely, in rubble masonry and in mortared-rubble fills, larger quantities of mortar were necessarily used, with the result that the structural characteristics of the masonry became much more dependent on those of the mortar.

Roman concrete and some comparable later work may be considered as a special case of mortared rubble in which the hydraulic mortar used had excellent strength and durability and gave to the structural mass an almost monolithic character. But, except in the earlier period of its development, the coarse aggregate was usually broken brick and the facing was usually of brick also, even if this was given an outer skin of thin sheets of marble.

The use of cramps, dowels, embedded timbers, and continuous tie rods

The use of cramps and dowels, to which some reference has already been made, goes back at least to the Ancient Egyptian practice of holding blocks in place during construction by means of wooden dovetail cramps. Iron cramps continued to be used extensively in cut stone masonry in Roman and later times, but chiefly in string courses and cornices rather than throughout the masonry.

Embedded longer lengths of timber also have a long history. They were used chiefly to reinforce masonry of poorer quality, and, where they have since rotted away, may now betray their previous presence only by leaving voids where they once were.

Iron tie rods spanning freely between the springings of arches were used at least from the early 6th century in Constantinople and perhaps earlier by the Romans. Continuous chains of iron around the bases of domes do not seem to have been introduced until the 15th century, though there is evidence before this of embedded circumferential timbers and, as noted above, of stone cornices in which adjacent stones were joined by iron cramps.

Mixed construction

Finally, it must be noted that, in any but the simplest structures, different types of construction were often used in association with one another. The use of timber for floors and roofs is not of direct concern here. But the combination of different types of masonry was almost as common. The use of rubble infills in ribbed vaults has already been mentioned, as has the association of monolithic column shafts with piers constructed of coursed masonry. Other instances are the common practice of alternating courses of brick with courses of stone in walls and piers and the use of carefully constructed ashlar masonry in those parts of a structure subjected to the greatest anticipated loads but elsewhere using more economical alternatives - usually with a higher proportion of mortar and with small rubble or brick in place of cut stone. In the Flavian Amphitheatre in Rome, for instance, fine ashlar masonry of travertine and tufa supports vaults of concrete, while in the 6th century church of Hagia Sophia equally fine ashlar of the local limestone and greenstone in the main piers both supported brick vaults and gave way, at the higher levels, to construction almost wholly in brick.

Subsequent changes or repairs have often resulted in even more varied combinations.

SIMILARITIES AND DIVERSITIES OF STRUCTURAL BEHAVIOUR

The response of each of these types of masonry to load - whether self weight of the structure or environmental or other externally applied load - depends, of course, on the manner of loading and hence (among other things) on the type of structure and their place in it. To a varying extent, it also depends on past loadings and exposures which will largely determine the present state of the masonry, in particular the extent of cracking and the condition and properties of the mortar. Where there is much mortar, sequences of construction may also be relevant since loads will have changed while it gained strength and stiffness.

The most significant differences are probably between masonry which is fairly uniform through the cross section and masonry in which the core is very different from the outer skins, between masonry with little or no mortar and masonry with much more of it, and between masonry with high quality mortar and masonry with mortar of poorer quality. Mortar in thin beds between courses of good ashlar is subject to triaxial compression, so that its strength as normally measured is relatively unimportant. But when it was used more liberally in beds of uneven thickness, or equally liberally in uncoursed work, a significantly greater compressibility than that of the stone would easily lead to load-shedding and to highly non-uniform stress distributions over the cross sections.

Masonry of all kinds will almost invariably be cracked wherever it has been subjected to appreciable tension, whether as a direct result of past loading or as a result of differential settlements, other imposed movements or thermal strains. The cracking is caused not only by primary load-induced tensions such as the hoop tensions around the lower part of a dome but also by the orthogonal tensions associated with principal compression under compressive or shear loading and by separations of supports or of boundary elements such as the boundary arches of a vault.

In structures built of large blocks simply resting on one another, it may be observed as the splitting of individual blocks or as separations between blocks. In other structures it will either run through both stone and mortar - as a few clearly defined and widely separated cracks in good quality masonry or as a larger number more widely distributed in poorer masonry - or occur largely or wholly in the mortar. In arches and other arched forms it is often seen as the partial V-shaped opening of joints between the voussoirs, often accompanied by some relative slip.

Even structures that were not articulated *ab initio* by virtue of their construction become articulated by this cracking (Fig.4).

The structure as a whole is usually fairly obviously deformed by this cracking and by a combination of early plastic flow and later compressive deformations of the mortar. Such deformation is particularly characteristic of arched and vaulted structures with relatively slender vertical supports (see Fig.4 again).

Collapse, if it occurs, will usually result from a progression of the cracking and of the associated deformations and relative displacements (including any at foundation level) until the stable articulation becomes an unstable mechanism (Fig.5). High compressive stresses at points of hinging articulation may accelerate the change from stability to instability by causing local crushing, splitting or slips. Even the apparent exception - sudden collapse of a pier or wall under essentially vertical load and without rotation of the base - may be regarded as a special case of the same process involving a final rapid increase in vertical cracking (or splitting) under the action of the orthogonal tensions that are associated with the primary compression and of any tensions due to the outward pressure exerted by a fill that has lost its cohesion.

Dynamic response at low amplitudes (such as it is feasible to observe in tests) will be essentially elastic if the structure has suffered only limited cracking and otherwise still has a near-monolithic character. But the increased amplitudes associated with a damaging earthquake will lead to further cracking. Elastic response will then progressively give way to one in which the blocks of masonry separated by the cracks and any other dicontinuities will rock relative to one another, and this rocking will entail major changes in natural frequencies and in the forces that result from particular ground motions. Most seriously, there will be a tendency towards out-of-phase responses, with blocks alternately moving apart and coming together again. This will allow debris that falls into the cracks to jam them further and further open and will similarly allow arches and other spanning elements to jam their supports further and further apart. Or it may lead to repeated impacts where no such jamming can occur. Decreasing natural rocking frequencies as amplitudes increase will tend, however, to limit the movements, and any partial collapses of spanning elements will stabilise the supports by relieving their loading.

STRUCTURAL ASSESSMENT

A standing structure will, by its survival, have demonstrated that it has had the capacity to support all the loads it has hitherto experienced. Nevertheless, manifest present stability may not necessarily guarantee continuing future stability.

To test future stability it is necessary to make some assessment of safety margins under possible future loads. Analytically, the present choice is essentially between elastic models and equilibrium models, whether static or dynamic.

Analytical models for static behaviour

The typical equilibrium model is an articulated assemblage of blocks whose equilibrium is ensured by a static balance between the interactive and other forces acting on each. These blocks simply rest on one another, relative rotation being prevented only by the loading, by the overall configuration, and by any restraining ties. The assumed configuration should take note of any existing deformations and it could similarly take note of assumed possible future increases in these. But it ignores relative deformations along alternative load paths.

This is the model that was implicitly or explicitly assumed in all early theoretic studies of static stability, especially that of the arch (Figs 6 and 7).

Its ignoring of relative deformations, together with uncertainties about boundary conditions, makes the precise internal forces indeterminate and calls for repeated analysis to establish upper and lower limits for critical forces and reactions and their lines of action. In practice, however, these indeterminacies are reduced by the cracking and numerous effective articulations of most masonry structures (see Fig.7 again). If these are taken into account in the model, potentially critical forces can be estimated within much closer limits (Fig.8).

The typical elastic model is a continuum which responds to load by elastic deformation. Thanks to its recognition of the influence of relative stiffnesses, it leads, even for a hyperstatic structure, to unique internal forces and stresses for a given geometry and loading and given properties of the materials. Recent developments of the finite element approach and in the power of computers have greatly increased the range of possible applications. Even the development of cracking can be followed by an iterative procedure, and it should in principle be feasible to include the effects of sequential construction and time-dependent properties of the materials. If the necesary assumptions and idealisations are not to throw grave doubt on the significance of the results, the amount of computation called for tends, however, to limit its use to the study of simple structural elements like domes or sub-assemblies like ribbed vaults subjected to load only on final completion of construction.

For any but the simplest structures, there is also the difficulty of ascertaining, sufficiently accurately and comprehensively, the relevant support conditions and elastic moduli and strengths. This difficulty is especially marked where the support conditions have changed over the life of the structure and cross sections are heterogeneous. As a result of this, and of the ignoring of the effects of construction sequence and loading history, calculated internal loads and stresses are likely to be of very limited relevance to any actual state of the structure. Indeed, the true distributions are likely always to be indeterminate and subject to continual change.

Safety under static load

Neither model provides, by itself, an assurance of safety, and the elastic model can do no more than indicate where cracks will form and where high stresses may otherwise pose a threat to stability.

Safety of the equilibrium model must be assessed on the basis of the ability of the cross sections to carry the internal forces without local failures that could precipitate collapse. Either that ability must be judged ad hoc in terms of safe eccentricities for the forces in relation to the stress levels and the whole character and condition of the masonry. Or, after first assessing the strengths of the relevant sections, these strengths (expressed in terms of combined direct stress and either bending or shear) can be assigned to the members and joints of an equivalent linear frame. Analysis must then proceed with step-by-step increases in the loads until a mechanism is formed. The problem thus becomes one of defining the relevant cross-sectional strengths, though there may also be problems with three dimensional modelling.

Analytical models for dynamic behaviour

The elastic model again represents the structure as an elastic continuum which reacts to any input by elastic deformations.

The equilibrium model, subjected to inputs such as seismic ground movements, directly follows the dynamic rocking response of an assemblage of rigid blocks. It has been used in recent analyses and tests of the seismic response of multi-drum columns and similar forms. Although the computational requirements have, as far as is known, so far precluded its use to explore the post-cracking and post-elastic behaviour of more complex structural forms, it seems, here also, to offer the way forward to a fuller understanding - with the cracked state of the structure after the initial shocks as input.

Safety under dynamic load

Only the equilibrium model directly indicates collapse. Since it ignores the possibilities of local failures at points about which rocking occurs, it may be expected to overestimate safety and to do so to a variable extent depending on local compressive strengths, though the available evidence does not suggest much overestimation for structures of high quality masonry. In any event, the response is highly dependent on the precise nature of the input motions, which means that safety can be considered only in terms of probabilities of survival.

The elastic model cannot adequately represent the structural response after significant cracking, and the elastic response that it assumes is, by definition, a safe one. But it may be indirectly useful in helping to establish structural parameters that cannot be established more directly.

Choice of approach

Any assessment of present safety should start with an examination (including any desirable monitoring) of all that the structure itself can show us about its condition, its history of construction, the ways in which it is still able to carry the loads upon it, its strengths, and its potential weaknesses. Patterns of cracking and deformation can be particularly revealing. Analytical models should be used to clarify and give precision to what can be learnt in this way rather than serving as a substitute for it.

What has been said above suggests that equilibrium models are usually to be preferred to elastic ones. This is not to deny all value to the latter. They will be most directly applicable when the construction is most homogeneous through the cross section and there is least cracking - a situation more likely to be encountered in structures of Roman concrete or the best brickwork than in most stone masonry structures. But their wider value seems to lie chiefly in the deeper qualitative insights that they give into typical modes of response well short of possible collapse. Whichever type of model is chosen, more will depend on how the necessary simplifications and assumptions are made than on anything else.

If the initial appraisal shows a need for strengthening to conserve the structure for the future, it becomes highly desirable to clarify the precise aim. Is it to be merely to conserve the outward form? Or to conserve the whole structural organism as far as possible, with all the evidence this provides of the past?

If it is merely to conserve the outward form, it may be feasible - though not necessarily most economical or lasting - to reproduce it, in part at least, in a material such as reinforced concrete. Well proven standard design procedures then become available. But it should be recognised that this approach may bring about changes in the modes of response to future loads that are sufficiently significant to invalidate earlier analyses and that partial interventions of this kind may introduce new hazards that are not envisaged by the standard design procedures and are difficult to appraise fully.

If this alternative is not acceptable, it becomes necessary to seek out not only possible weaknesses but also the strengths that have ensured survival hitherto, and to concentrate on safeguarding, and if need be restoring or supplementing, these. This approach, as well as conserving the authentic structural organism (and unlike the other approach), makes full use of the proven strengths of that organism.

Nearly all masonry spanning elements, for instance, have large margins of safety if their supports hold firm. This is true not only of arches, domes and vaults, but also of architraves and lintels which, when cracked, are still potentially capable of spanning as flat arches. The risk to them is primarily a risk of excessive movements of the supports, to which the thrusts they exert may, of course, contribute. If they still stand, this will be thanks largely to adequate support for the outward thrusts they have hitherto exerted - whether provided solely by piers or walls, or partly also by buttresses, iron ties across the spans, or circumferential ties - and attention will be best focussed on ensuring its continuing adequacy. Assessing what is needed calls chiefly for estimates of the thrusts - which can usually be best obtained as upper and lower bounds by equilibrium methods - and of the ability of the vertical supports to continue to carry their loads with whatever inclinations and eccentricities arise.

As a further example, a major attribute that has contributed to the ability of some columnar temple structures to survive earthquakes has been the articulated character of their masonry. This character should be preserved and not jeopardised, the emphasis being placed on making good any damage to the individual blocks of stone, as is being done in the present work on the Parthenon.

Most importantly, no analytical model should be taken to justify remedial works if it suggests that a structure hitherto safe should have suffered collapse or other failure, or if it indicates other responses to load that are seriously inconsistent with those that we can observe. The model is then at fault; not the structure. The evidence furnished by the structure itself is always primary.

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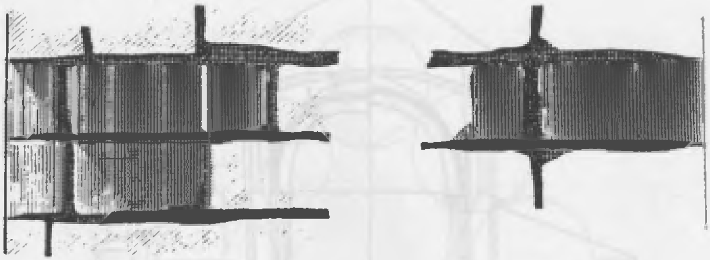

Fig.1 Partial vertical sections through the masonry of a main pier of the Paris Pantheon. (from Rondelet, J, Addition au mémoire historique sur le dome du Pantheon Française, Paris, 1814)

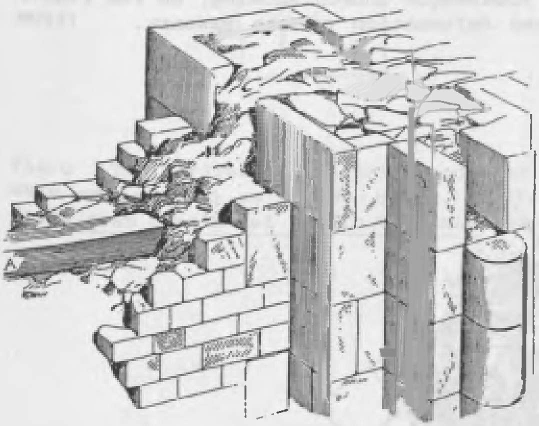

Fig.2 Typical Romanesque construction with a fill of mortared rubble behind ashlar facings and an embedded timber at A. (from reference 12)

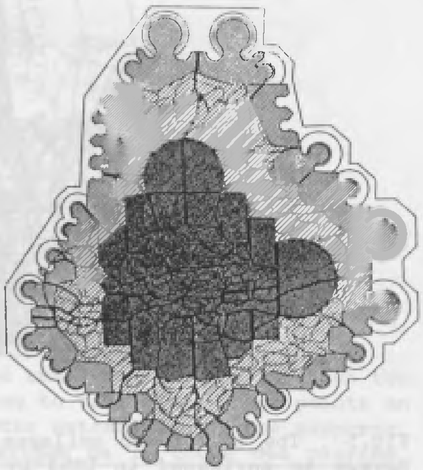

fig.3 Section through a crossing pier of the Cathedral of Bayeux, showing the original Romanesque pier surrounded by later Gothic masonry. (from De Dion, H and Lasvignes, L, Cathedrale de Bayeux: reprise en sous-oeuvre de la tour centrale, Paris, 1861)

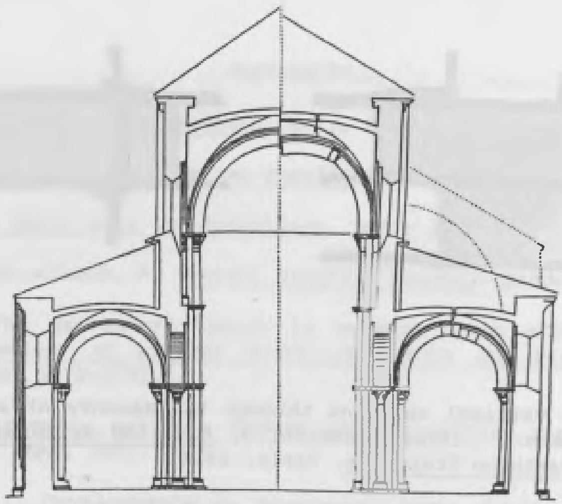

Fig.4 Transverse section of a Romanesque church showing, on the right, typical cracking, articulation and deformation of the masonry. (from reference 12)

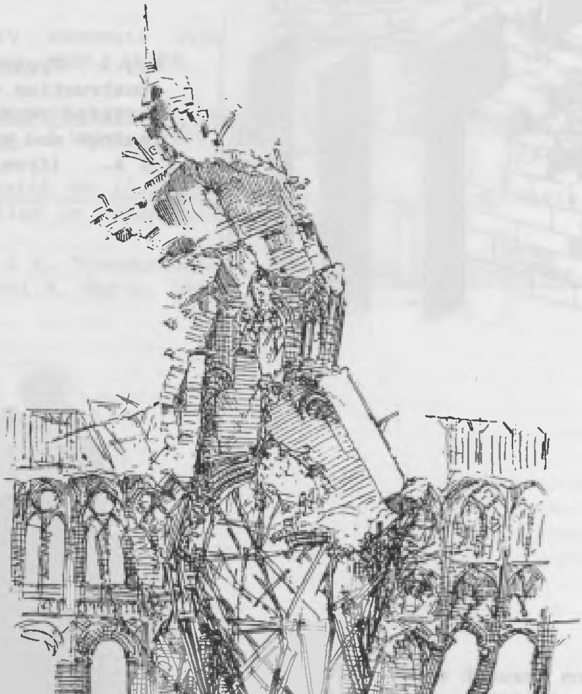

Fig.5 The threatened collapse of the central tower of the Cathedral of Bayeux as envisaged in 1853 by Viollet-le-Duc. (from a drawing in the Archives des Monuments historiques, Paris)

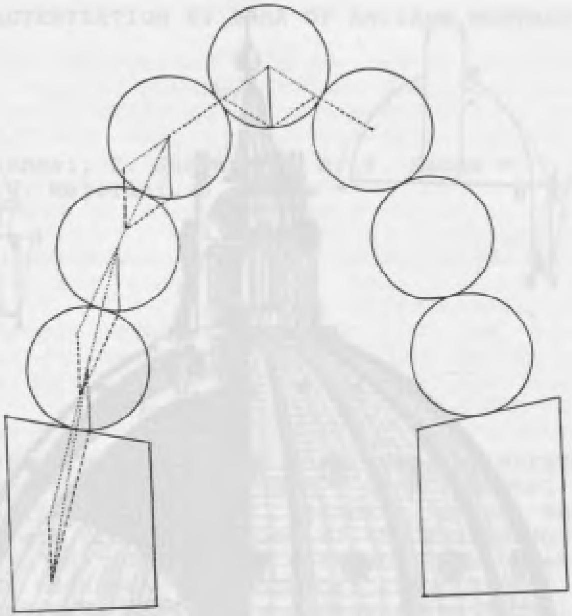

Fig.6 The earliest equilibrium model of the arch, in which the voussoirs were represented as smooth balls. (from Poleni, G, Memorie istoriche della gran cupola del Tempio Vaticano, Padua, 1748)

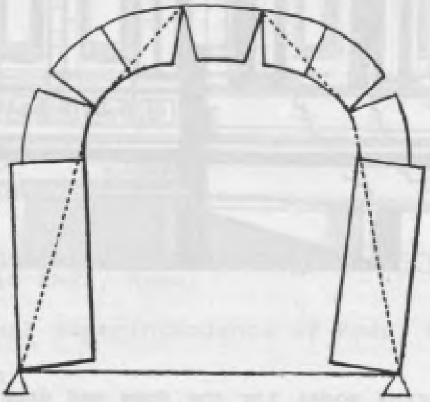

Fig.7 An alternative early model based on small-scale tests in which the low levels of contact stress allowed hinging to take place about points on the intrados and extrados and about the outer extremities of the supports. (from Frezier, A F, La théorie et la pratique de la coupe des pierres, vol.3, Strasbourg, 1739)

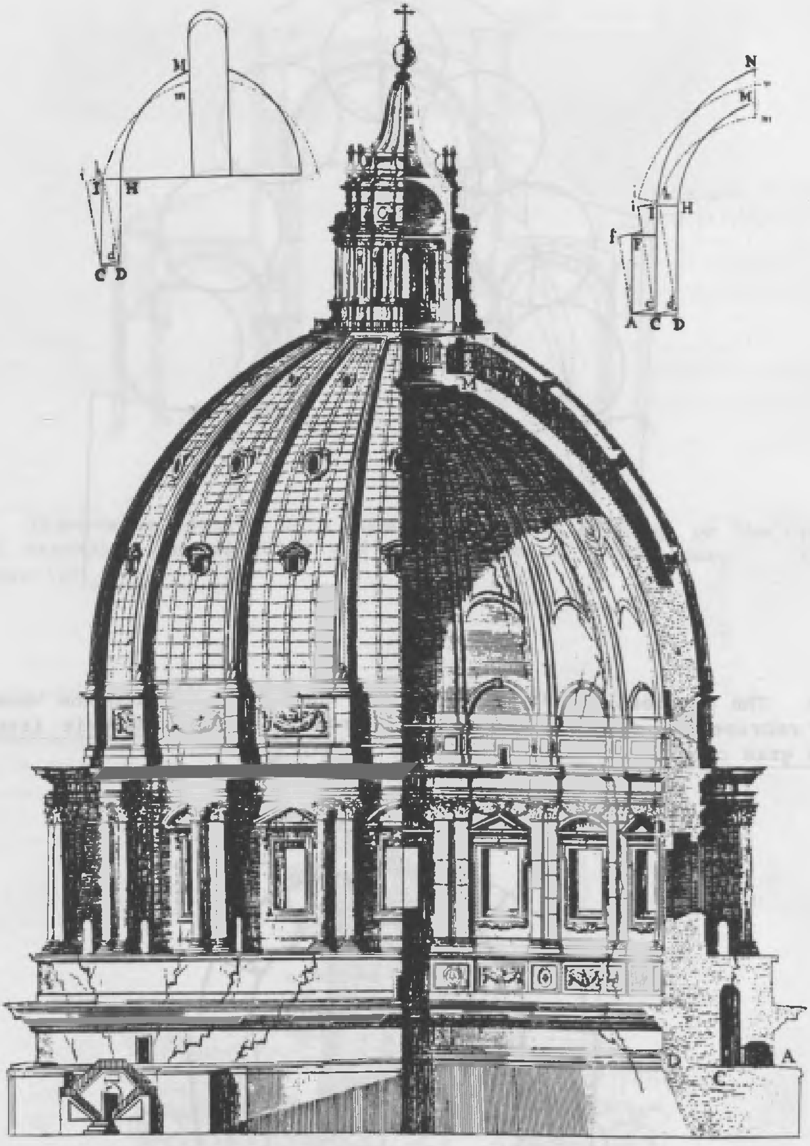

Fig.8 An equilibrium model for the dome and drum of St Peter's, Rome (top right) based on the observed crack pattern (below). (from Le Seur, T, Jacquier, F, and Boscovich, R G, Parere di tre mattematici sopra i danni che si sono trovati nella cupola di San Pietro, Rome, 1743)

CHARACTERIZATION BY INAA OF ANCIENT MORTARS

by

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SUMMARY

In this paper are presented the results obtained from chemical determination by INAA (Instrumental Neutron Activation Analysis) of trace elements on 50 samples of mortar provenient from a building of the archeological site of Gabii(Rome). The determined elements have been used as input for data treatment by Canonical Discriminant Analysis, Principal Components Analysis and Clustering Techniques. The obtained results confirmed the attributions based on archaeological and architectonic observations.

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INTRODUCTION

The samples of mortar studied belong to the hamlet of Castiglione, a country building risen in late medieval age and located inside the archeological site of Gabii (Rome).
This territory, that storically is one of the most significant among the suburb districts, was acquired in 1987 by Ministry for Cultural and Ambiental Goods and surrendered to the Archaeological Superintendence of Rome to form an archaeological park.
The site is located 20 km east of Rome, between the Aniene river and the hills of Albano, along the layout af the ancient "Via Prenestina" (figure 1).
Here, in the late VIII century b.C. grew the Latin town of Gabii, one of the most important centres of "Latium Vetus" in pre-roman age, later become a Roman Municipium (ref. Guaitoli M.-1981).
Recent excavations of the Archaeological Superintendence of Rome have been performed on the Hamlet of Castiglione.The building appears built on the remains of medieval Castrum and presents an L-shaped plan consisting of five works (I - V) carried out in order.
All buildings have traces of numerous remakes and reconstructions, that testify an extended use and complicate constructive phases.
The sample of mortar studied in present work come from the building II and particularly from the inside of the room n. 2d.
In the last use the lower floor was designed as stable and the upper floor (room n. 2d) as warehouse.
Formerly the upper floor was used for lodging and dormitory of seasonal farm-labourers.
For the building II it is possible to distinguish three main constructive phases:

1) The carrying out the building, leaning against the pre-existent construction.

2) The plugging of some windows of the lower floor, of one window of the upper floor and the reduction of the remaining windows of the upper floor.

3) The reconstruction of the roof and the strengthening of the pillars that support it inside the room n. 2d.

Nevertheless it is not possible to define with accuracy the absolute chronology of the constructive phases; yet it is probable that the first phase is placed between the end of the XV century and the first years of the following century, while the third phase is dated around the second half of the XIX century.
The samples of mortar considered come from building works related to all the three constructive phases.

EXPERIMENTAL PROCEDURES

A few specific regions belonging to different architectonic elements of the inner-side perimeter of the room have been chosen and classified by visual examination.

For each region has been withdrawn, after removing the outer crust, a sample of mortar, each about 8-10 cm3 in volume, by using hard wood tools in order to avoid metal contaminations.

A total of 50 drawings has been performed. The samples have been made homogeneous by mild grinding in smooth glass mortars.

An aliquot part of each grinded mortar, each about 0.5 grams, has been put in a polyethilene vial and irradied for about 30 hours in the rotating rack ("Lazy Susan") of the 1 MW TRIGA reactor of the Institute of Casaccia with thermal neutron flux of 2.6 10e12 n cm-2 s-1 and flux homogeneity better than 99.85 %. Primary and secondary standards with the same geometry of the samples were simultaneously irradiated and used for calibration and conversion to concentration units.

After irradiation the samples were allowed to decay some days, and then transferred and measured.

ata collection was performed using an Ortec Ge(I) detector with FWHM of 1.68 keV and efficiency of 29% at 1332 keV, coupled with a Livius (Silena) 8K channels PHA-MCA system and the net area of gamma peaks measured.

The usual corrections (self absorption, interference evaluation) have been performed and the element concentrations evaluated.

A total of 19 elements has been determined.

Soon afterwards every irradiated sample has been treated with 1 M HCl and successively vacuum filtered.

This procedure allowed a separation of the calcareous soluble fraction from the insoluble pozzolana and sand.

Insoluble fractions have been weighed, and of both of them the gamma spectrum has been recorded.

Due to the elapsed time only long half-life isotopes could be determined, for a total of 9 elements per sample.

RESULTS AND DISCUSSION

The determined elements have been used as input in our data treatment programs in order to obtain the main features of the group.

The table 1 summarizes the starting classification by visual examination of our samples, divided by sets according to their provenience and supposed appurtenence to different building phases. Both the elements determined in the HCl soluble and insoluble fractions of the mortars (two groups of 9 elements) and the elements of the untreated mortars non determinable after separation (11 elements) were used in data treatment, so obtaining a new set of 29 variables, which gave the results reported in figure 2.

The separation in the three building phases is quite good and, for instance in CAN1-CAN3 plane, virtually unambiguous. The total discriminant capability is quite satisfactory, especially if we observe the table 2, which reports the mean values of the elements set by set. It is evident that the single elements don't show a clear evidence of differentiation of the groups.
On the other hand if we use the linear combination of their values to form the canonical variables we obtain discrimination capability, even if none of the original variables do.

CONCLUSIONS

In this paper we have shown how some mortars from different ages and building phases can be unambiguously discriminated on the basis of elemental chemical determinations.
INAA (Instrumental Neutron Activation Analysis) appears particularly suitable in this field, due to its unique ability in multielemental, non destructive, analysis coupled with high sensitivity and accuracy.
Canonical Discriminant Analysis, Principal Components Analysis, Clustering Methods were used in data treatment in order to obtain the maximum of information and discrimination capability.
The finally obtained results show a completely consistent confirmation of ipotheses and attributions based on archaeological and architectonic data, so proving the important role of this high-performance analytical tecnique in the problems of the restoration and classification.

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BUILDING PHASE	SET	SAMPLES	ARCHITECTONIC ELEMENT
1st PHASE	A	20	original structure and arches
	G	1	lintel
2nd PHASE	B	10	window post
	C	7	plugging of window
	D	2	portal jambs
	F	2	repairs
	H	1	threshold
	L	1	plugging of brick
3rd PHASE	E	5	stiffening pillars
	I	1	anomalous stiffening pillar sample

Table 1 Starting classification by visual examination of the architectonic structures of the corresponding mortar samples.

SET	A	B	C	D	E	F	G	H	I	L
Ba	1822.67	1927.58	2077.95	1690.10	2628.32	2048.64	1231.88	1474.22	2634.60	1521.58
Ce	239.71	208.37	222.15	205.62	322.37	220.01	243.25	174.14	330.40	172.60
Co	24.01	19.58	19.66	18.69	30.46	20.39	24.23	22.23	32.25	18.11
Cr	57.73	30.68	30.88	35.35	21.36	44.43	61.58	115.52	19.70	24.92
Cs	35.48	29.23	30.33	27.78	25.54	30.03	35.77	30.91	26.37	29.45
Eu	3.64	3.18	3.20	2.98	4.62	3.23	3.56	2.69	4.87	2.70
Fe	50637.50	43018.16	43227.79	40860.73	63408.63	43125.74	51022.49	43471.72	66424.65	40168.21
Hf	10.15	8.58	8.55	8.35	13.21	8.87	10.00	8.20	3.18	9.00
Lu	.002	.002	.001	.002	.000	.002	.002	.001	.002	.001
Nb	0.24	0.23	0.23	0.18	0.39	0.21	0.29	0.20	0.39	0.18
Nd	15.83	13.82	14.52	13.58	22.36	14.31	14.10	15.90	22.11	13.97
Rb	464.27	422.17	450.09	478.37	362.43	453.91	462.11	328.21	379.08	290.36
Sc	15.17	11.18	10.98	11.31	11.20	11.71	16.02	19.44	11.48	13.70
Sr	7160.50	11594.19	11023.81	7432.50	5109.51	10966.26	3241.99	4667.71	5314.38	3590.85
Ta	0.67	0.67	0.53	0.64	0.86	0.66	0.32	0.77	1.03	0.41
Th	67.23	59.47	62.98	60.11	100.68	63.28	69.16	43.52	91.84	45.16
Yb	5.38	4.86	5.04	4.72	6.49	4.97	5.62	3.88	6.80	3.49
Zn	33.81	36.21	83.18	20.16	50.94	49.31	31.17	28.93	25.83	34.17
Zr	5.76	5.29	5.84	4.11	10.25	5.37	5.91	4.51	10.47	4.49
SAMPLES	20	10	7	2	5	2	1	1	1	1

Table 2 Mean values of the determined chemical elements divided by set (see Table 1).

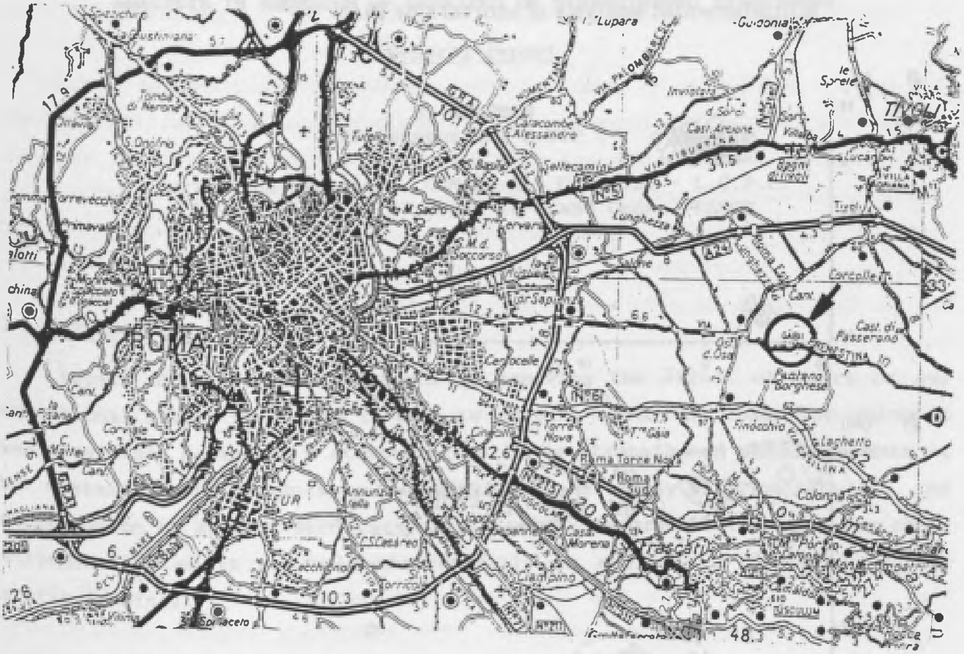

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Fig. 1 Location of Gabii, in which is situated the building studied in the present work.

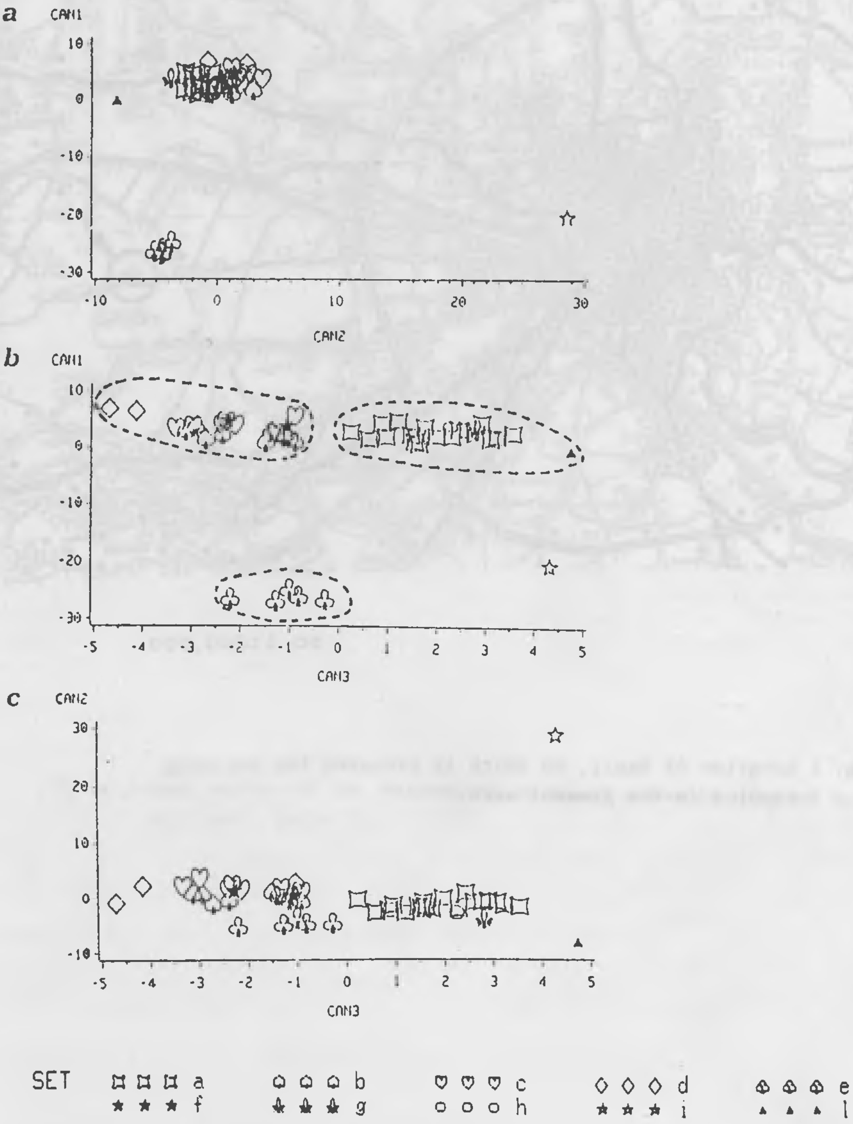

Fig. 2 Canonical Discriminant Analysis on the group of 50 samples by using as input data the determined trace elements. The ellipses point out the clear discrimination in three building phases.

analysis

ANALYSIS IN REDESIGN OF REPAIRED OR STRENGTHENED STRUCTURES

General report

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I - INTRODUCTION

Repair of a structure consists in removing the effect of years or any exceptional action by making it able to sustain again the loads for which it was previously designed. A structure has to be strengthened when one wants it to sustain loads which were not forecast initially. In particular, the conservation of ancient stone masonry in seismic regions often asks for their strengthening as it was only theoretically or empirically designed against static loadings.

So repair or strengthening of a structure aims at changing its mechanical behaviour. To design this change following an appropriate way, it is necessary to know the present mechanical state of the structure, to evaluate the mechanical effect of the repair or strengthening and the new strength of the structure. For those three purposes, numerical analysis is a very useful tool, even if it must be used simultaneoulsly with metrological investigations and if some analytical approaches can sometimes be effectively used.

Owing to the progress of hardware and software in recent years, it is now possible to make static, dynamic, linear or non-linear computations, dynamic non-linear ones remaining highly time-consuming. The greater part of the difference between methods of analysis stays in the physical model that is chosen.

The methods presented to the present conference can be separated in two classes : those which consider masonry as an homogeneous continuous medium, and those which do no ignore the existence of mortar joints between stones.

II - MASONRY AS AN HOMOGENEOUS SOLID

Baggio [1] uses a step by step elastic finite elements analysis, discretizing arches and domes with plate and membrane elements. The program takes care of the non-resistance of masonry against tensile stress ; at each step it modifies the rigidity of elements where tractions are present by reducing their thickness. This procedure is applied to the analysis of the efficiency of repairs made to the dome of S. Pietro in the eigteenth century. The author gives a good look to the difficulty in obtaining correct geometrical and mechanical datas, and he rightly concludes that such an analysis could only represent an estimate of the complex behaviour of real structures. An interesting feature of his code is that it runs on a personnal computer.

Ignatakis, Stylianidis and Stravrakakis [2] present a similar method, but they introduce the possibility of crack closure, which is very interesting when one has to analyse the effect of the repair or strengthening of a structure by adding external efforts. On a particular example they compare this procedure with a computation without cracking criterion and so they show that the account of cracking makes a great difference in the strengthening project.

Barthel [3] uses a non-linear finite element program to study the behaviour of masonry cross vaults under dead load and displacements of the abutments. He adopts a material model for concrete transferred to masonry, which is able to represent the non-linear behaviour in compression and the cracking in traction. His results are very interesting as they reproduce the usual cracks which are found on the studied monuments and enlighten on their mechanical working. As the author himself writes, the three-dimensional non linear analysis used is interesting for research, but probably to expensive for engineers.

As a three dimensional dynamic finite elements computation is expensive and heavy , Wenzel, Frese and Vratsanou [4] prefer to model masonry walls with beams and diagonal struts. It seems to me that by such an analysis it is difficult to take into account simultaneously the in-plane and the out-of-plane rigidity of a wall. Their paper gives a very interesting example of all the problems presented by the modelisation of an important group of buildings. Nevertheless they do not say how they actually check their model.

Moric and Anicic [5] use analogous models to study the earthquake resistance of stone buildings when floors are made of wood or reinforced concrete. Their paper, which suggests the same questions as the preceding one, clearly shows that the in-plane rigidity of floors and the exact nature of links between walls and floors must be included in the model.

As simplified models raise different questions, Karantoni and Fardis [6] compare these with shell and plate finite elements in the static analysis of buildings simulating their seismic behaviour. As could be forecast they find that the F.E. method is more adequate to determine the state of stress of a building. They then apply this method to the assessment of strengthening techniques. But they do not study the influence of the initial state of the structure on the efficiency of strengthening techniques.

III - MASONRY AS JOINTED STRUCTURE

Blasi, La Bianca and Bellini [7], by presenting a particular example, propose a method of determining the static configuration of a damaged masonry arch. Starting from the probable initial state of the arch, they model it as elastic finite elements with joints which do not support tensions. Then they follow the evolution of the arch under loading. If the final state that they find reproduces the present state of the monument, two points are not very clear in their paper : what is the exact criterion of opening of a joint and what are the boundary conditions they used in their computation.

Blasi and Foraboschi [8] model masonry arches as linear elastic finite elements seperated by gap elements following Coulomb's friction law. They apply this method to an arch submitted to horizontal displacements of footing and compare it with success with a continuous analytical method they had previously proposed. They also apply it to a flat arch and compare it with an experiment in photoelasticity. As the model seems now present in numerous finite elements codes, I regret the lack of comparison with an actual masonry structure.

Ignatakis, Stravakakis and Penelis [9] present the computation of masonry walls under plane loadings taking into account the non linear damaging behaviour of bricks and of joints. They introduce the influence of the transverse principal stress, evaluating it from a modification of a simple model of Francis, Horman and Jerrems. As few datas about the behaviour of bricks and mortar are available, the authors use an adjustment of a model for concrete. The mode of failure they find is in accordance with known experimental results.

Miltiadou and Abdunur [10] investigate, through numerical computation, the mechanical role of injection in the behaviour of a repaired masonry wall. As their finite element model reproduce the presence of blocks, mortar in the joints and infill, they had to choose a simple global geometry. They use an elastic constitutive law for blocks, an elastic-plastic one for mortar and infill. The mechanical contribution of the injection is simulated by changing the characteristics of the injected material. One of the conclusions of the computation is that injection has not a systematic positive effect on the mechanical behaviour of the wall.

Musiani [11] studies the dynamic behaviour of slender masonry structures. The first model he presents is made of rigid blocks that do not slide along their joints. The second one introduces the elasticity due to the geometrical shape of bearing surfaces and to the deformations of blocks and joints. As this model gives analogous results when the movement is sufficiently large, the author uses the rigid one to analyse the seismic resistance of obelisks. As no sliding or tensile resistance is present in the analysis, it is difficult to agree with the conclusion of the paper which suggest that the restoration of the joints is generally sufficient to strengthen the monument.

Blasi and Spinelli [12] use a similar model, but they add deformable Winkler monolateral cushions between the blocks, in order to be able to take into account the mechanical state of the stone. They present two applications about a colonnade, in its deteriorated state and in its state after restoration of the joints between blocks. The results clearly show the effect of repairing, but its positive character is not so evident.

IV - <u>COMMENTARIES AND CONCLUSION</u>

As finite elements codes are now available in many civil engineering laboratoires, it seems reasonable to try to use them in the static analysis of masonry structures. Nevertheless the complexity of the constitutive law that is used must be related to the aim of the computation. For example,] believe that the location and even the orientation of maximum tensile stress in geometrically simple structures under monotonic loading is rather independent of the constitutive law. On the contrary if displacements are imposed to the structure or if one is interested in its displacements, the constitutive law can have an outstandind role. These remarks were previously made by Heyman [13] in 1977 about shell structures. So it should be interesting to compare the results presented in certain papers to those of a simpler static analysis.

As far as static analysis is concerned, it seems that no major innovation appears with regard to what was presented in the book published by Sacchi Landriani and Riccioni [14] in 1982. The merit of authors is often essentially in the applications. Certain typical problems of masonry structures are to rarely or not approached. Among those one can quote the account of rubble infills, the transverse behaviour of masonry walls which put particular questions in certain masonry buildings and bridges.

In dynamic analysis, simplified models are generally used. It is certainly right as one is concerned with the behaviour of a column, but, when they are not compared with experimental results, it is difficult to have an opinion about their use for complex masonry buildings, where non-linear phenomena and three-dimensional effects can be important.

As the papers proposed at the present conference display the state of the art in the domain, the preceding remarks show that certain research topics are still opened.

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[3] R. BARTHEL : Computer simulation of the structural behaviour of masonry cross vaults.

[4] F. WENZEL, B. FRESE, V. VRATSANOU : Structural analysis of the Auguste-Victoria Church of the Ascension in Jerusalem. Assessment of its safety by employing response spectrum modal analysis.

[5] D. MORIC, D. ANICIC : Floor stiffness influence in the distribution of horizontal forces in earthquake strengthening of stone masonry buildings.

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STATIC CONFIGURATION OF

THE ARCH OF GALLIENO IN ROME

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SUMMARY

The simple structure of the Arch of Gallieno is neverthless interesting from both a constructional and a structural point of view.

The geometrical rigour of the original concept and its realisation may be seen in the reconstructions drawn here.

Its present precarious state was studied by means of an appropriate numerical model, and the factual evidence is exemplary in its clarity.

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Florence, April 1989

1. INTRODUCTION

The present report describes two important stages in an intervention for structural restoration of a monument that is significant on account of its architectural rigour and where the pattern of cracks is particularly clear. The first stage involves an understanding of the criteria used in the planning and building of the monument, in other words, a study of the monument in its original state.

The second stage is a survey of its present tensional and deformative state. In this particular case it will be shown how the monument was created by the strict application of a precise geometrical plan, not only in the definition of the formal elements (ribs, cornices, etc.) but also in the structural ones (position of the joints, planes between stone blocks, etc.).

In describing the present state of the monument, it will be shown how, by using popular numerical model codes, if appropriate procedures are adopted in the definition of the model, results may be obtained that are significant and congruent even in damaged buildings.

2. DESCRIPTION OF THE MONUMENT

The Arch of Gallieno, built on the ancient Monte Cispio, was crossed by the Clivus Suburranus, which led from the Roman Forum to the site known as Ad Spem Veterem.

Originally it had three arches, the central one being higher and wider than the lateral ones, and has always been closer in character to a Gateway rather than a Triumphal Arch. On account of its sobrity of line and essentiality of form, the Arch should moreover be considered as a restoration, carried out during the Augustan period, of the Esquilinian Gate that formed part of the disused Servian Walls, rather than a monument built in the third century.

Subsequently, in 262 A.D., the Arch was dedicated to the Emperor Gallieno and his wife Solonina by the faithful Aurelius Victor. Certain alterations were made to the frieze and upper cornice on this occasion and the dedication to Gallieno was inscribed on the lower half of the lintel.

GALLIENO CLEMENTISSIMO PRINCIPI, CUIUS INVICTA VIRTUS SOLA PIETATE SUPERATA EST, ET SALONINAE, SANCTISSIMAE AUG(ustae), AURELIUS VICTOR, V(ir) E(gregius) DICATISSIMUS NUMINI MAIESTATIQUE EORUM.

(corpus Inscr. Lat., vol. VI, no. 1106)

Today practically the only remaining part of the original arch is the central vault, which is supported contrasted on one side by the 4th century church of SS. Vito, Modesto e Crescenzia, restored or rebuilt by Sistus IV in 1477; on the other side the Arch is contrasted by a late nineteenth century building. Traces of the lateral arches may be seen clearly enough to be able to reconstruct the original proportions of the complete building.

In 1936 G. Gatti noted the sober proportions of the building, in particular the fact that the height and the width of the intrados of the arch were equal in size: this was not immediately evident, as the street level had sunk by approximately 60-70 cm. This feature is not unique in Roman architecture, another example being provided by the cupola of Hadrian's Pantheon: also comparable are the arches of Aosta, Fano, Rimini and again those built in Trieste and Aquino during the Augustan period.

Even in the earliest stages of its construction the Arch of Gallieno was subject to alterations which are visible today both in the moulded areas and in the breaking off of the lines of the Corinthian ribs.

This, together with the surviving traces of frescoed plaster, would suggest that the Arch was once painted all over and that colour was also part of its architectural design. Support is lent to this thesis by the terminology of medieval times, when the Arch was known as Arcus Pictus. L. Rossini, describing the excavations made below one of the two piers by V. Baltard, a frenche "pencionnaire" at Villa Medici, noted: "... one must also bear in mind that this monument was covered in plaster, as may be seen in those fragments found in the side annexed to the church."

The Arch shows considerable and extensive damage (see Fig. 2) which may be classified as follows:

A. Rotation of the South pier;
B. Lowering of the central part in the crown of the Arch;
C. Slidings and fractures in the blocks constituting the "kidneys" of the Arch;
D. Vertical fractures in the blocks forming the bases of piers.

The pattern of cracks reveals in an exemplary manner the mechanism suffered by the monument, due to rotation of the piers because of lack of an adequate contrast to the thrust of the arch: this is the most frequent type of damage found in arches. The rotation of the piers is more noticeable in the South side where up to the last century the arch was free and hence its stability depended on one pier only (insufficient, as will shortly be shown). From the North side, on the contrary, the facade of the fourth century church of S. Vito has probably provided a contrast from the time of the first signs of damage. The rotation of the piers has clearly caused the "opening" of the arch and hence the sliding of the central blocks and their rotation, until the surfaces of contact between the blocks were completely altered, being reduced in places to extremely limited areas, with subsequent concentration of tensions and fracturing of the marble slabs.

The historical iconography always shows the monument in a similar condition to its present one, and thus the damage probably dates froma a time close to, if not coincident with, the collapse of the lateral arches. However, careful observation of the cracks reveals a gradual though modest increase in lesions also in recent times.

An assessment of the extent of the damage in comparison with the original state of the monument may be made by reconstructing its geometry. The arch is in fact realised by following a geometrical plan in so rigorous a way that the slidings and rotations are clearly legible for each stone (Fig. 2). The reference "number" for the geometrical plan is six: the entire geometry is based on a squared module measuring six Roman feet; the central arch is formed within a square having a side of six modules and the whole monument was probably contained within two squares of equal size or in a rectangle with a height of six modules and a base of twelve modules (see Fig. 3). The internal partitions and even the position of the joints also strictly follow the geometrical lines of the modules and their diagonals.

The geometry thus not only has a formal value, but consitute a fundamental element of the structure, being almost a guarantee of its stability.

3. TENSIONAL AND DEFORMATIVE STATE

In the arch there are numerous fractured stones. Current practice in structural survey therefore necessitated an accurate assessment of the tensional state in order to judge the present static situation and to study what type of consolidation work would be possible. It is not sufficient to ascertain that the pressure curve for the central arch alone falls dangerously near the extremity of the supporting base of the piers.

In this context Figure 4 gives the position of the pressure curve for the central arch only and for the state of the arch as it probably was originally. The state of instability in the central arch without the support of the two minor lateral arches is clearly revealed.

Assessment of the tensional and deformative state of the monument was made by means of an Finite Element calculus code with linear behaviour; in subsequent stages, however, discontinuity corresponding to those areas subjected to tensions that are incompatible with the structural characteristics of the monument were introduced.

Figure 5 shows the final numerical model and the tensional state at the intrados of the arch and in the external part of the columns.

Special care was taken over the definition of the geometry of the model, in order to follow the actual sub-division of the blocks, especially in areas corresponding to the planes where sliding has occurred.

The model describes the mechanism suffered by the monument with a fair degree of accuracy, while, from the point of view of the static situation, the tensions obtained clearly explain the existing fractures.

4. CONCLUSIONS

The model in Figure 5 was the basis for an assessment of the monument and for defining the consolidation project, in that the effects of different possible solutions were studied.
In particular, the project would involve restoring the surfaces of contact between the blocks by means of appropriate sealing of the lesions, and introducing tendons at the abutments of the arch with a definite state of pre-tension. In this simple way the supporting function of the arch would be restored, without altering its original structural scheme.

ACKNOWLEDGEMENTS

We should like to thank Prof. Chiarugi who has been responsible for the organisation of this research.

Fig. 1: The prospect of the arch towards the internal part of the city. (A. Moschetti 1872)

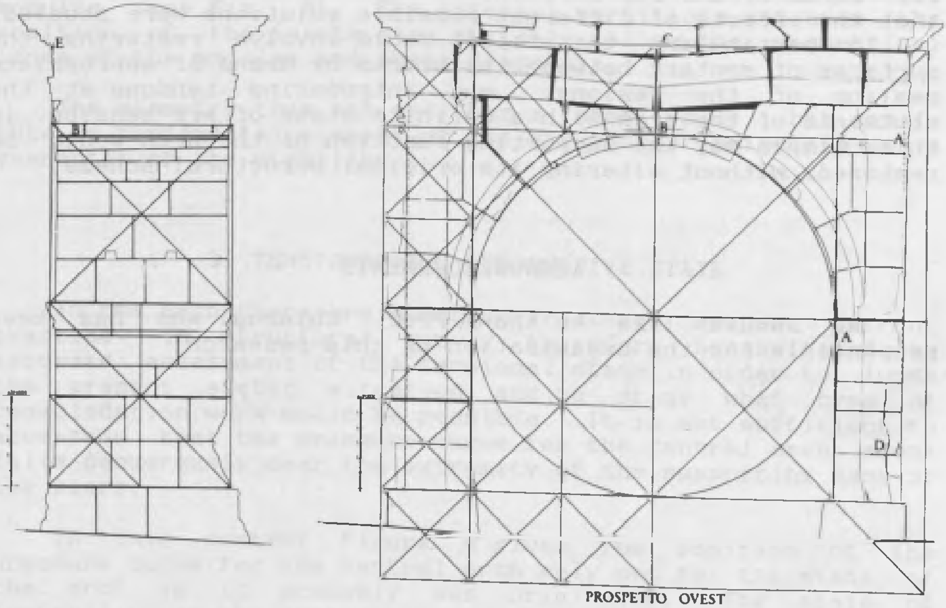


Fig. 2: Constructional geometry of the central arch.

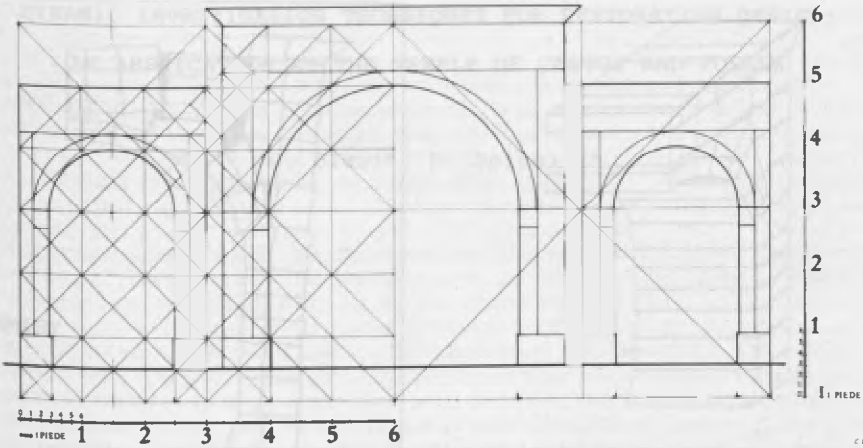


Fig. 3: Hypothesis regarding the geometry of the complete monument.

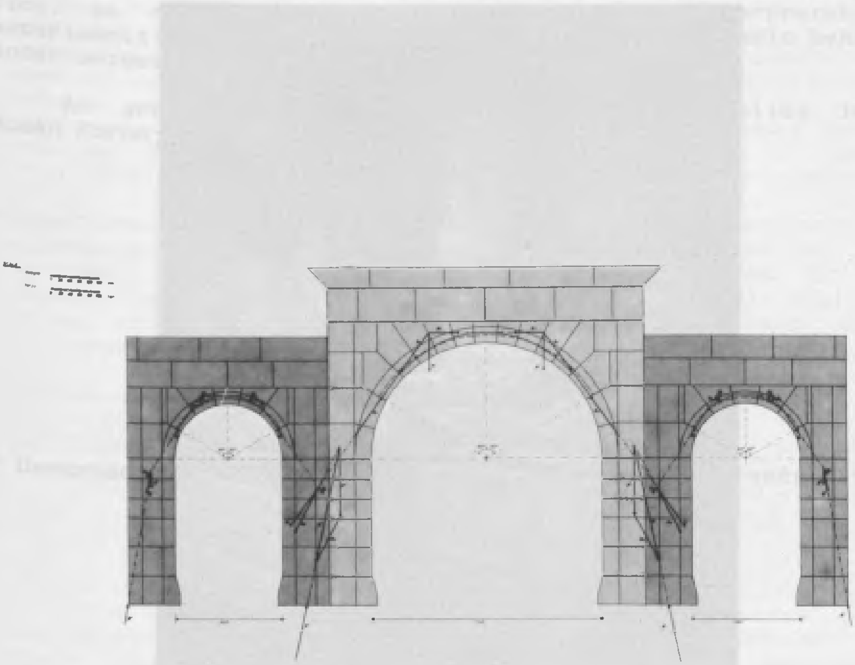
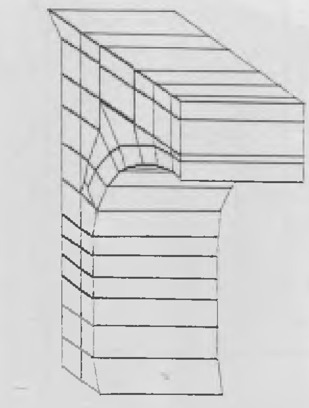


Fig. 4: Pressure curve



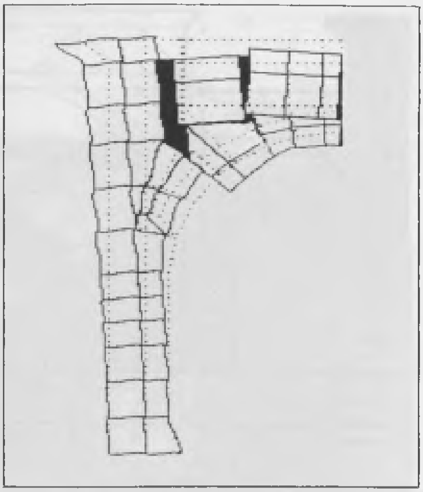


Fig. 5: Numerical model

Fig. 6: Photograph of the lesions"C".

DYNAMIC INVESTIGATION TECHNIQUES FOR RESTORATION DESIGN:

AN APPLICATION AT THE TEMPLE OF CASTOR AND POLLUX

C. Blasi*, P. Spinelli*

SUMMARY

In the present work an investigation procedure for the evaluation of seismic risk of ancient monuments composed by columns and columnades is explained.

The procedure deals with an experimental dynamic investigation, an accurate numerical modelling for the interpretation of experimental data and at last the prediction of dynamic behaviour under seismic excitation.

An application to the Temple of Castor and Pollux in the Roman Forum is at last developed.

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Florence, Aprile 1989

INTRODUCTION

Many monumental buildings expecially constructed by ancient Greek and Roman architects, are constituted by columns and columnades. As mainly they are located in earthquake prove regions, the problem of their preservation can be solved through the evaluation of the seismic risk and the design of possible consolidation works.

The present research arrives after the development by writers' work of numerical procedures for the prediction in the dynamic field and with large displacements of seismic behaviour of structures composed by simply superimposed rigid blocks /1/ /2/ /3/. In those publications the details of numerical techniques are explained. In the present work the complete application of those procedures, partly experimental and partly theoretical is presented, in order to arrive at the evaluation of statical and dynamical characteristics of monumental columnades and for the identification of ultimate states under seismic action.

The procedure starts with an experimental investigation of the structure under dynamic excitation (preferibly with a small vibrator). It is evident that such tests, for the structures peculiarities have to be developed in small displacement field, around the statical equilibrium position. In a subsequent phase through a computer program based on Finite Element techniques, the experimental results are interpretated through an identification procedure which arrive at the definition of mechanical properties of the structure.

In the application example illustrated the computer code ANSYS has been applied. This computer code allows the modelling of gap elements with monolateral behaviour in order to highlight the partializations of contact surfaces between blocks due to the high eccentricity of the load at the top of the columns, for the presences of a cautilevering cornice.

At last the study of the monument response under possible earthquake event, in the large displacement field, is conducted through the modelling of the rigid blocks structure, with the hypothesis of monolateral joints.

2. THE TEMPLE OF CASTOR AND POLLUX : EXPERIMENTAL TESTS AND NUMERICAL MODELLING

The Temple of Castor and Pollux in the Roman forum was built in 484 b.C., but has received at last four reconstructions of which the last executed probably by Adriano Emperor. Of this last reconstruction only three corinthian style columns in marble remain, with a superimposed beam with a highly cautilevering cornice. The structures are characterized by the presence of large lacks in marble material, mainly in the upper part, due perhaps partly to the actions received at the moment of the collapse of the Temple, and also due to the load eccentricity.

Just from XVII century the monument has been subjected to consolidation works and lastly it has been subjected to a series of statical investigations proposed by Dott. Tedone of Archeologica Superintendent Authority of Rome, coordinated by Prof. Chiarugi of the Department of Civil Engineering of the University of Florence.

The stifness of the Temple under horizontal actions is measured, when applied in transversal and longitudinal direction and the column top and the vibration characteristics of the first oscillating modes are recorded. Particularly the transversal motion possesses an oscillating frequency of 1,18 Hz and the longitudinal one possesses a frequency of 1,81 Hz.

It is to be noted that this oscillation corresponds to small amplitude vibration (with not partialized joints between stone blocks). This is confirmed also by the fact that a natural frequency is clearly identified by the spectral analysis of the records, whilst in the case of rocking type motion in large displacement field, a peak shaped spectrum cannot be identified due to the continuous varying of natural frequency (which depends on displacement amplitude). The experiences are then numerically reconstructed through the numerical model of figg. 2 and 3. In fig. 2 the vertical tensional state in a stransversal section due to dead weight is illustrated. The zone with quasi-null tensions, correspondent to sections partializations can be easily noted. In fig. 3 the vertical tensional state in the longitudinal section is also illustrated, under an horizontal force at columns top.

3. EARTHQUAKE RESPONSE PREDICTION

3.1. Computational procedure

The method we have developed and proved to be efficient for predicting the behaviour of stone block columns is based on the following hypotheses:

- The blocks are considered rigid and imagined to have fictitious deformable Winkler monolateral cushions between them.
- The shear resistance of joints is considered infinite. This is not an essential requirement for the theory, as also the case of column's friction force development between blocks has been tackled /4/. However in the following for simplicity it is considered valid.
- Vertical displacements are neglected. This is an hypothesis

that can be dropped but for relatively tall blocks it does not affect the analysis /1/.

The same problem has been studied bu other authors with the analysis of impacts between blocks. The deformable cushion method has a double advantage:

- first of all it can be tuned with experimental analysis, taking into account simply the deteriorated state of the stone;
- secondly it gives easily the value of pressures between blocks.

In figure 4 the most simple problem of only one block is illustrated.

The static problem is characterized by the rotational equilibrium equation, the constitutive law of the cushion (different for partialized case and not partialized case) and finally the link between displacement and rotation (simple in the case of neglecting vertical displacement). With R the stiffness of the cushion in not partialized case is indicated. To be noted that only a univoque relation between φ and F exists (that is to one φ it corresponds only one F, but the contrary is not true). The dynamic problem can be tackled in a similar manner writing dynamic equilibrium equations.

$$-(mh^2+I_G)\ddot{\varphi} = P\left(b-h\varphi-\left(\frac{2P}{9Ks\varphi}\right)^{1/2}\right)+mh\ddot{u}_s \tag{1}$$

$$\ddot{\varphi} = \mathcal{F}(\varphi, \ddot{u}_s)$$

Where: m is the mass of the block
Ig is the rotational moment of inertia of the block
us is the ground acceleration

We have associated to eq. 1 for the time response finding, the Newmark's equation /5/ which supposes linearly varying acceleration during every time step. The solution is obtained through a iteration procedure.

The dynamic solution for a multiple blocks columns is obtainet throught a similar procedure /1/. Of course a non linear system is found with angular accelerations and rotations as unknowns; however it is to be noted that once rotations are assigned, accelerations are found solving a linear system. So the same solving procedure, oscillating between solving equations and Newmark's equations can be applied.

The method has been applied so long for the study and consolidation of various ancient stone monuments.

3.2. Seismic response in actual deteriorated situation and in a restored one.

The behaviour of the temple in the longitudinal direction is rather complicate and cannot be completely solved through our method. Nevertheless it is possible to solve some limit structural schemes to have a feeling for the behaviour of the monument and to have on the other hand some indications on the technique of restoration.

Referring to fig. 5 the solution procedure is the following.

- The fig. 5a scheme is solved simply through a system of eqs. 1.
- The fig. 5b scheme is solved through the same procedure, solving only the columns composed by (blocks) 1 and 2.
  The unknown horizontal force F_1 is determined, for every instant, by stating the following equilibrium equation for the horizontal beam, supposing to neglect the angural acceleration of the beam.
  So the same procedure illustrated in fig. 5 can be applied. to be noted that the eccentricity of the load Q_1 (as for fig. 10a scheme) has an eccentricity $e_1 = e_1(\varphi_1)$ which is known step by step once known φ_1.

The two cases are studied in the actual (deteriorated situation) with only a partyal contact between blocks and in the repaired situation (with all contact surfaces restored).

The figs. 6 - 7 show the displacements and rotations of the mass centers of the two blocks during the seismic excitation of a reference earthquake (the Mercato S. Severino earthquake November 1980 amplified two times has been used).

4. CONCLUSIVE REMARKS

Concluding the proposed method, also with the approximations due to the necessary simplification arising in the passage from the real to the numerical model, can constitute a valuable help for the designer in assisting the design choices for restoration works.

Some suggestions can be derived, as shown in the present work, for restoration design purposes not only of simple columns but also for ancient columnades.

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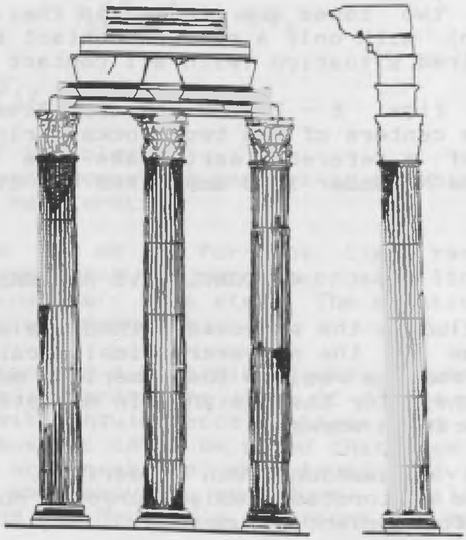


Fig. 1: The Temple of Castor and Pollux with the large lacks in marble material

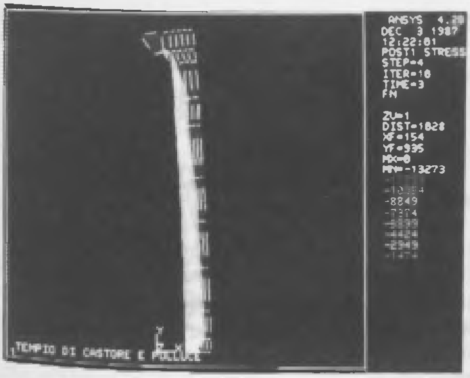


Fig. 2: Numerical model results, transversal section: vertical tensions.

Fig. 3: Numerical model results, longitud section: vertical tensions.

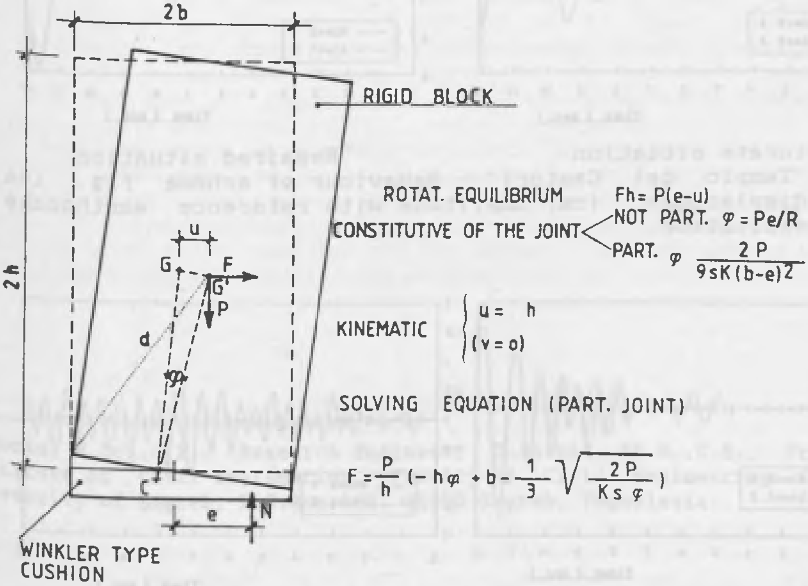


Fig. 4: Single block scheme

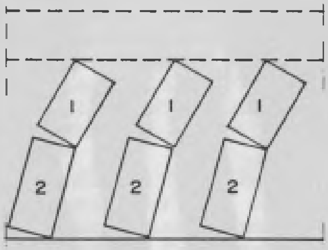


Fig.5a: Infinitely long columnade with equal and equally spaced columnas with equal dynamic behaviour (no horizontal interaction between columns).

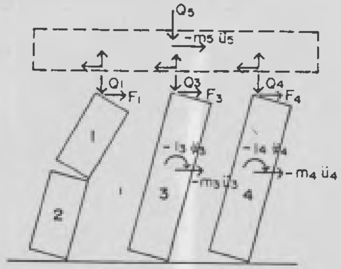
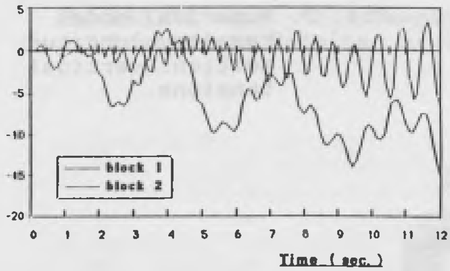
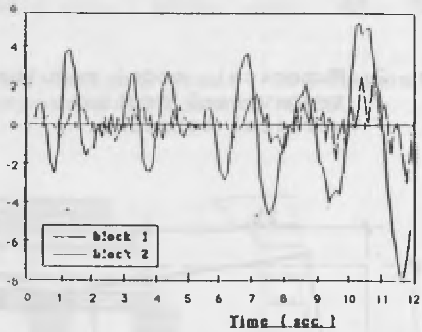


Fig.5b: Not equal dynamic behaviour between columns.

Fig.5: Limit structural schemes of Tempio dei Castori columnade

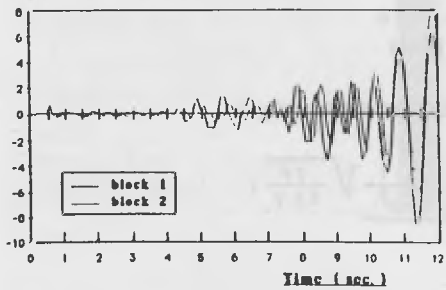


Deteriorate situation

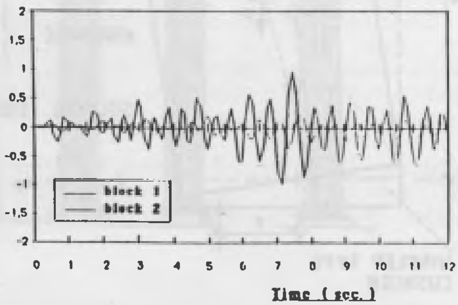


Repaired situation

Fig.6: "Tempio dei Castori" - Behaviour of scheme fig. 10a displacement (cm) amplitude with reference earthquake excitation



Deteriorated situation



Repaire situation

Fig. 7: "Tempio dei Castori" Behaviour of schemes fig.10b: displacement (cm) amplitude with reference earthquake excitation

FLOOR STIFFNESS AND EARTHQUAKE RESISTANCE OF STRENGTHENED STONE BUILDINGS

Dragan Moric and Drazen Anicic

Summary

An idealized four-storey stone building with reinforced concrete and wooden floors is analyzed. Five models involving different combinations of rigid and flexible floors have been studied, and their influence on the dynamic response of the building established. Linear dynamic analyses have been carried out.

As established, no combination of wooden and reinforced concrete floors substantially improves response, and the properties of the building with mixed floors, elevation-wise, resemble those of a building having only wooden floors. This raises doubts as to the efficiency of partial floor strengthening to which the design engineer often has to resort in dealing with stone buildings - cultural monuments.

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INTRODUCTION

In the design of buildings for the action of seismic loads it is customarily assumed that floor structures are infinitely rigid in their own plane and flexible in the perpendicular sense. If monolithic reinforced concrete floors are concerned, the assumption is mainly true, except in the case of very elongated floor plans when the flexibility of the floor structure in its own plane affects the redistribution of forces between vertical load-bearing units (walls, frames, etc.). In Yugoslavia this problem has been studied by Duhovnik (Duhovnik, 1985, 1986). In the case of prefabricated floor structures, stiffness in their own plane is considerably lower as compared with an analogous monolithic structure, as reported in Soviet sources (Korchinsky, 1964). In Yugoslavia the stiffness of prefabricated floors has been studied experimentally by Anicic (Anicic, 1971) and Ibrahimbegovic (Ibrahimbegovic, 1986), and analytically by Verbic, 1987) and Mujcic (Mujcic, 1982). Internationally, the problem has mainly been considered in analytical terms (Unemori et al., 1978; Jain, 1983; Button et al., 1984).

Earthquake strengthening of old stone buildings regularly involves the need to replace the existing wooden floor structures by monolithic reinforced concrete ones. This is a mandatory design operation providing for the ultilization of the full load-bearing capacity of the wall and, hence, entailing minimum strengthening of the vertical load-bearing system of the building. However, when cultural monuments are concerned the design engineer is often torn between the established view - rigid floor structures are a must if the building is to be made earthquake-safe, and the requirements of the preservationists who abhor the replacement of wooden floor structures by reinforced concrete ones as an impermissible devastation of the monument's value.

This paper analyzes an idealized four-storey stone building with reinforced concrete floors (infinitely rigid in their own plane) or wooden floors (flexible in their own plane). Five models have been studied with different combinations of the positions of rigid/flexible floors; their influence on the dynamic response of the building, shape modes and force distribution to the walls has also been established. The performed dynamic analyses are linear and elastic.

BUILDING DESIGN MODELS

Basic Structure

In most cases old stone buildings have wooden joist floors, load-bearing along the shorter span. The basic structural model analyzed in this case is a four-storey symmetrical structure having the same floor plan at all levels, with four transverse and two longitudinal perforated facing walls (Figure 1). All the floors are made of wooden joists which are load-bearing along the span between the transverse walls. The structure itself comprises 20/28 cm wooden joists with 25/5 cm boards (Figure 2). The floor is freely supported by stone cantilevers on the transverse walls; there is no connection between the facing walls across the floor structure. Slip is possible at the supports of the wooden floor structure.

Reinforced Structures

Let us assume that the basic structure is strengtenend by replacing the wooden floor with an d=15 cm reinforced concrete slab anchored to the transverse and facing walls (Figure 3). It is also assumed that the operation is possible at all the four levels.

In this way one has obtained five variants of floor types and positions the earthquake behaviour of which is to be studied. The structural models for these five variants are illustrated in Figure 4, and include:

Model 1 - all floors with wooden joists
Model 2 - all floors with reinforced concrete slabs
Model 3 - reinforced concrete floor of the second storey, others wooden
Model 4 - reinforced concrete floor of the fourth story, others wooden
Model 5 - reinforced concrete floors of the second and fourth storeys, others wooden.

A linear dynamic analysis was run for all the five models listed above by using the Ulcinj, Olympic (1979) N-S earthquake acceleration spectrum with 10% damping (Figure 5). The calculations were run by using the SAP4 Program. Free building oscillations for the first six modes were also calculated.

FLOOR STRUCTURE AND STIFFNESS MODEL

For the sake of simplicity the floor structures were modelled by two perpendicularly positioned vertical members simulating the type of support, i.e., the connection between the walls and the stiffness of the floor structure both in its own plane and perpendicularly to it. The wooden floor structure supported by the transverse walls is modelled by an articulated connection to the transverse walls, while the connecting between the facing walls is modelled - for the sake of model uniformity - by a member of negligible horizontal stiffness and cross section; real stiffness has been assumed in the vertical plane (Figure 6). Such an articulated connection between the wooden joists and the transverse walls ensures joint wall translation. In a real building this would prevent the slip of the joist bearing with the walls. If this is not case in the real building response between the mathematical model and the real structure will differ substantially. The reinforced concrete structure is also modelled by two vertical members rigidly connected to both the transverse and the facing walls (Figure 7).

The stiffness of the floor structure in its own plane was transformed into the stiffness of the equivalent member by calculating the displacement of the floor structure in its own plane and perpendicularly to it; theory of elasticity expressions were used.

RESULTS OF THE ANALYSIS

For all the five models illustrated in Figure 4 (the variations refer exclusively to the floor structures and the respective masses) one has calculated free oscillations for the first six modes, and the response of the structure (by spectral analysis) for the Ulcinj-Olympic (Montenegro, 1979) acceleration spectrum.

The dynamic characteristics will be analyzed and compared as well as the calculated displacements, distribution and magnitude of the forces within the walls, and the earthquake resistance of such idealized building models.

Free Oscillations

Figures 12 through 16 illustrate the forms of the first six modes for all the five models. The from of the first mode is almost identical for all cases. What is valid is the stiffness of the load-bearing walls which is lower in the longitudinal sense. The oscillation period of the first mode is markedly lower for model 2 (all floors being reinforced concrete slabs), which matches data reported for masonry buildings, i.e., T=0.05 . N (N=number of storeys), or T=0.0165 H (H=total building height). The first mode period for the other models is higher by 20% (model 5) to 40% (model 1). See Table 1.

Table 1. Free building oscillation periods in (s)

Model	T1	T2	T3	T4	T5	T6
1	0.36	0.15	0.13	0.13	0.11	0.08
2	0.26	0.10	0.10	0.09	0.05	0.04
3	0.33	0.14	0.12	0.12	0.10	0.08
4	0.35	0.12	0.11	0.11	0.08	0.07
5	0.31	0.11	0.11	0.10	0.07	0.06

The form participation factors indicate the domination of the first mode but also the need to add the contribution of the higher modes, especially in the transverse sense of the structure. The figures clearly show the differece between the oscillation forms of the floors themselves. Rigid reinforced concrete floors display a characteristic translation in two perpendicular sense - rotation for all floor points being identical. Wooden floor structures display horizontal oscillation forms because of their flexibility. As a matter of fact, the form of structures oscillation in cases where the stiffness of the wooden floor prevails is mainly characterized by a uniform line in the vertical plane, and two or more lines in the vertical floor planes with a nodal point (or points) approximately in the plane of symmetry of the building floor plan. Of course, this leads to disagreement with the basic assumption made in the seismic computation, i.e., that the displacements of all points in the structrue at the floor structure level are equal.

The comparison of the actual absolute magnitudes of displacement at the top of the structure for the analytical models under consideration yields an interesting ratio:

$$D_1 : D_2 : D_3 : D_4 : D_5 = 6.53 : 1 : 5.22 : 5.48 : 4.02,$$

which is very illustrative of the horizontal stiffness of all the five model deformability considerably. The essential differences in model behaviour are illustrated very clearly in Figure 8. The figure does not show only the relative displacement differences but also the difference in terms of structural seismic response: only model 2 presents a form typical for a building loaded dominantly by shear, while the other models suggest the considerable influence of bending, which affects to a much greater extent the stability and resistance of the stone building.

Forced Oscillations

In the analysis of the earthquake resistance of a structure, the relative storey displacement as determined by the adjacent storey differential displacement to storey height ratio, d/h) may be more important than the absolute magnitude of displacement. This information is presented in Table 2 for all levels of all models.

Table 2. Relative storey drift

Storey	Model No.				
	1	2	3	4	5
Four	0.023	0.0019	0.0236	0.0089	0.00952
Three	0.0210	0.00295	0.0198	0.0181	0.0175
Two	0.0159	0.0034	0.0983	0.0179	0.0111
One	0.00659	0.00194	0.00256	0.00828	0.00293

Two average relative storey displacement magnitudes for different floor stiffness combinations are reviewed in Table 3. Here the storey displacements are shown in dependence upon the type of the floor structure above and below.

Table 3. Average relative storey displacement magnitudes

Structures above	Structure below	$(d/h)_{av}$	Ratio
fully fixed to foundation	r.c. slab	0.00247	1
fully fixed to foundation	wooden joist	0.00744	3.01
reinforced concrete slab	r.c. slab	0.00274	1.11
wooden joist	wooden joist	0.0196	7.94
reinforced concrete slab	wooden joist	0.0104	4.21
wooden joist	r.c. slab	0.00922	3.73

Design bending moments differ considerably, especially where transverse walls are concerned. Figure 9 illustrates the diagrams for bending moments about axis Y for wall No. 1. Rigid reinforced concrete floor structures prevent the cantilever action of the wall and take over the bending moments. Bending moments can cause the overturning of cantilever-acting walls. (The same will occur with longitudinal facing walls affected by perpendicularly acting earthquake forces).

If the floors are rigid, shear failure of the walls is the only hazard. If floor structures flexible in the horizontal plane are involved, failure due to bending moments is also possible along with shear failure, and bending moments must therefore be taken into account when calculating the ultimate strength of the masonry.

CONCLUSIONS

The analysis of floor stiffness and of its influence on earthquake resistance of stone buildings has produced some new facts.

Rigid reinforced concrete floors simplify building oscillations into lateral and torsional forms; there are no horizontal oscillations in the floor slab plane. Similarly, rigid slabs considerably reduce the horizontal displacement of the structure, ensure an approximately ideal form of transverse force distribution considering the stiffness ratios, and protect the structure from overturning, i.e., from the influence of bending moments. The basic problem is how to provide for shear load-bearing capacity; the improvement, if required, does contribute to the shear capacity of the masonry. The shear capacity of existing masonry can easily be established by tests in situ.

If the replacement of all wooden floors by reinforced concrete ones is not permissible for preservationist reasons, the effects of eartquake strengthening will be dubious.

Judging by the results of this study, combined solutions - provisions of rigid reinforced concrete floor slabs at some levels, and wooden floors at others - do not improve the situation substantially because such structures resemble much more, in terms of their properties, structures having all flexible floors. This is not in agreement with the intuitive conclusion according to which an already rigid floor structure above the last storey should contribute "considerably" to improving the building response to horizontal forces.

Similarly, any improvements aimed at increasing the shear capacity of the masonry (concrete sheathing or stone wall grouting) become meaningless if the floors are flexible because they do not provide safety against moment failure of stone walls. Nevertheless, wall overturning due to the action of bending moments need not be feared in masonry buildings of standard floor plans and moderate elevation (3-5 storeys). In the worst of cases the alternating action of earthquake forces will produce the opening and closing of cracks in mortar joints subject to tension and to local crushing of the compressive edge.

The design engineer will find it very difficult to evaluate the earthquake safety of a building with flexible floors (or of buildings involving semivariants) because the basic principles of standard, regulation-based calculations no longer apply and, therefore, cannot guarantee an accurate determination of safety.

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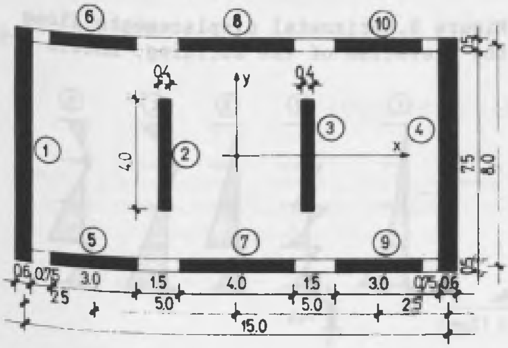

Figure 1. Design model, floor plan

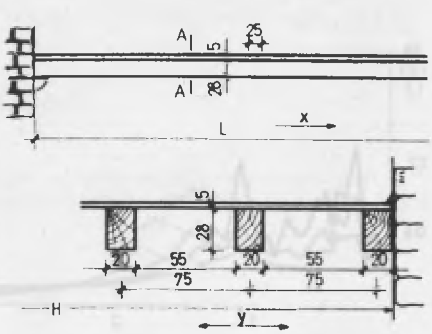

Figure 2. Wooden floor structure

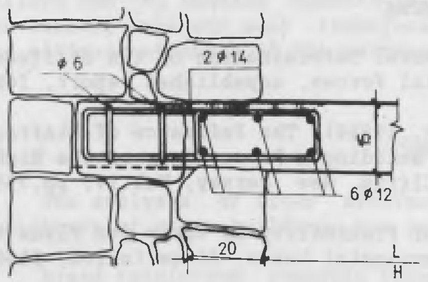

Figure 3. Connection of reinforced concrete floor structure and stone wall, detail

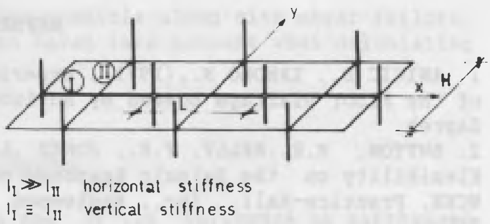

Figure 6.Design model of wooden floor structure

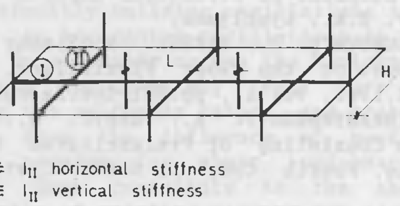

Figure 7.Design model of reinforced concrete floor structure

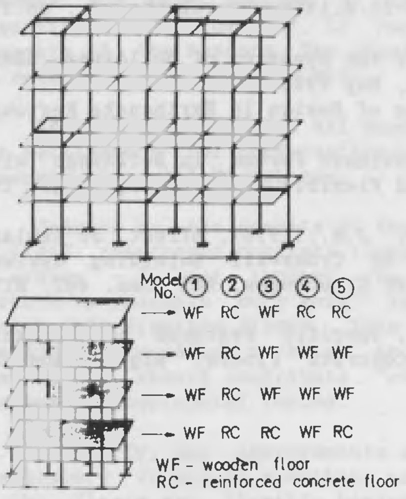

Figure 4. Design scheme of models 1-5

Figure 8.Horizontal displacements along the elevation of the building, models 1-5

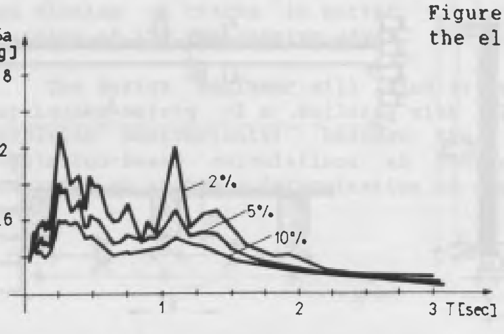

Figure 5. Acceleration spectrum for Ulcinj, Olympic earthquake record

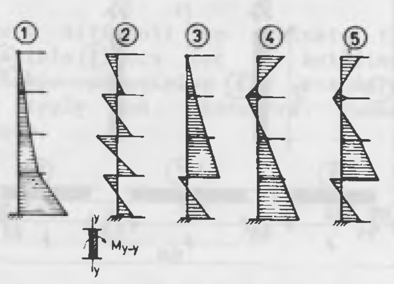

Figure 9.Bending moment diagrams for models 1-5

DYNAMIC EVOLUTION BY EARTHQUAKE EXCITATION OF MULTIBLOCK STRUCTURES

A.SINOPOLI *

SUMMARY

The analysis of the dynamic response of a monolithic stone column or of similar structures to seismic excitation is a very complex problem. For this reason, an extensive scientific literature has been produced on this argument, after the pioneering paper of Housner. Most of the works above have investigated only the stability with respect to the rocking. Perhaps, this is a consequence of the assumption generally made that the structure is monolithic and slender; therefore, the slidings between the block and the ground surface have been neglected.

In former works (Sinopoli 1987, Sinopoli 1989), the author has demonstrated that the impact is a focal point in the dynamic evolution of a rigid structure simply supported on a rigid ground. In fact, depending on the geometrical features of the stucture, the impact is responsible of the coupling between rocking and slidings.

Therefore, a new question arises: are the slidings responsible of a failure due to the loss of the geometrical configuration or they make a multiblock column more stable by means of the dissipation mechanism?

The aim of this paper is to investigate the dynamic behaviour of a multiblock column excited by a given ground motion, taking into account the results given by the impact problem.

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INTRODUCTION

The problem of structural repair and maintenance of historical buildings generally requires: a) a good knowledge of the mechanical behaviour of these structures; b) a sure identification of the causes of the failure; c) a measure of the stability degree with respect to the failure causes.

Although the greek temples have proved to be capable to survive over the centuries, the earthquake seems to be the more probable cause of derangements or failure of these structures. This is due to the fact that, for the temples as for almost all the stone buildings, the stress level due to their own weight is low compared with the strength of the material; therefore, cracks or crisis due to excessive stress values are extremely improbable.

Furthermore, under a ground excitation, the more difficult problem is the choice of the mechanical model in order to simulate as close as possible the dynamical behaviour.

In this respect, two general approaches can be recognized in the scientific literature:

- a) The column, simply supported on a rigid ground, is rigid and monolithic. The dynamic stability is investigated only with respect to the overturning. The column generally rotates around a corner edge and, after any impact, around the other one; the impact preserves the angular momentum and reduces the angular velocity (Housner 1963, Yim, Chopra, Penzien 1980).
- b) The dynamical behaviour of a stone column is described by means of a translational and rotational mass-spring model with dashpot devices, which must take into account the mechanism of energy dissipation (Hanazato, Theofanopoulos, Watabe 1989).

In our opinion, the very large dimensions of the temples, the used material and overall the absence of pinned connections between adjacent blocks suggest as a proper model that of a rigid structure, simply supported on a rigid ground and made of rigid blocks, simply supportd each over the other.

As a consequence, the forces governing the dynamic behaviour, under a ground excitation, are only the gravity and the dry friction.

Nertheless, the analysis is very complex: - for quantitative reasons, due to the large number of the blocks; - for qualitative reasons, due to the non linearity of the problem, emphasized by the impacts.

For the reasons mentioned above, a research program has been undertaken to investigate some particular aspects of the problem, in order to converge to a complete model. The investigated aspects have been:

- a) Analysis of the impact for monolithic and multiblock column;
- b) Sensitivity dynamic analysis of the rocking for a multiblock column, excited by a sine wave ground motion. The impact results have been used only for their consequence on the rocking; the slidings between adjacent blocks, at this step, have been ignored.
- c) Sensitivity analysis of the coupled motion (rocking and sliding) of a column, made of three stocky blocks, in order to investigate both the amount of the slidings and that of the dissipated kinetic energy.
- d) Experimental analysis of the impact problem and experimental evaluation of the dynamic dry friction coefficient. This analysis is performed in parallel to point c).

The present paper show the results obtained from the works mentioned in the points a) and b); and the partial results relative to the points c) and d).

THE IMPACT PROBLEM

The impact problem analysis has been performed (Sinopoli 1987, Sinopoli 1989), for both monolithic and multiblock column, by means of a kinematic approach. This means that, due to the extension of the surfaces involved in the phenomenon, the velocities discontinuities have been evaluated as consequence of unilateral constraints imposed to the system.

The interesting results are that, even if the motion before the impact is a rotation around a corner edge, after it, two motions are possible:
- rototranslations;
- slidings.

The occurence of either depends, for a monolithic column, on the dimensional ratio between the base and the height of the column; for multiblock column, on the same ratio of a single block.

The critical number which separates the two cases is $\sqrt{2}/2$. In any case, the motion after an impact is

characterized by a component of sliding, which depends on the dimensions of the column or of the block, and on the angular velocity before the impact.

Therefore, more slender is a monolithic column, better is the approximation that the column, after an impact, is characterized only by rocking. On the contrary, for a column made of stocky blocks ($b_i/h_i > \sqrt{2}/2$), it is expected that, even if the standard values of dry friction and of seismic accelerations cannot excite relative slidings , they can be excited by the impact and then increased by the earthquake.

A first conclusion suggested by the results above is that a multiblock column seems to be more stable than a monolithic one, at least with respect to the energy dissipation and to the overturning. In fact, more slender is the monolithic rocking column, lower is the amount of the dissipated kinetic energy during an impact.

On the contrary, a multiblock rocking column dissipates, at any impact, all the rotational energy; furthermore, it dissipates, in each contact surface, an amount of the sliding kinetic energy, which depends on the dimensions of the block and on the features of the seismic motion. Nevertheless, if the slidings become excessive, they can cause the failure of the whole structure or of its part. Only a forced dynamic analysis can answer this question.

ROCKING ANALYSIS OF A MULTIBLOCK COLUMN

Let us consider a multiblock column excited by a sine wave ground motion, and let us assume that the dry friction is large enough to prevent slidings between the column and the ground. Furthermore, let us assume that the blocks are stocky ($b_i/h_i > \sqrt{2}/2$) and the relative rotations between adjacent blocks are absent.

The dynamic equations of the motion are in this case only that of the rocking. They are, for positive angle:

$$\ddot{\vartheta} = \beta^2 \left\{ \left[1 + \frac{b}{h}\frac{k_s}{g} \operatorname{sen}(\omega t + \psi) \right] \operatorname{sen}\vartheta + \left[\frac{k_s}{g} \operatorname{sen}(\omega t + \psi) - \frac{b}{h} \right] \cos\vartheta \right\} \quad (1)$$

where:

- $\beta^2 = \frac{3}{2}\frac{g}{h}\left(1 + \frac{b^2}{h^2}\right)$;
- h is the height of the column;
- b is the base length of the column
- g is the gravity acceleration;
- k_s is the sine wave amplitude in g units;
- ω is the angular frequency of the sine wave;
- ψ is the initial phase of the sine wave and
- θ is the rocking angle.

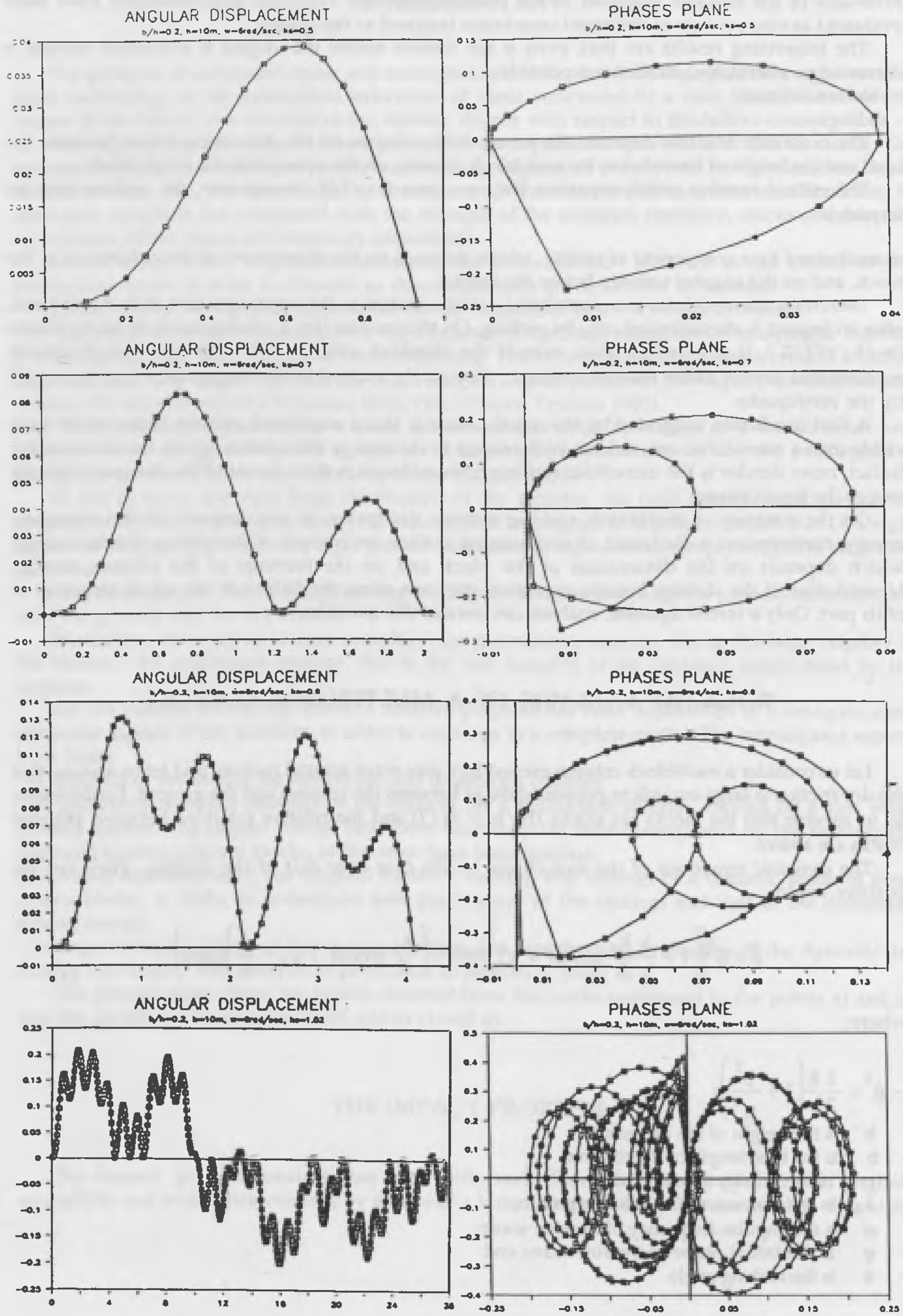

FIG.1. Evolution of the motions of a multiblock column (b=2m., h= 10m.) corresponding to increasing values of the amplitude k_S of the ground motion and for a given angular frequency: $\omega = 6$ rad/sec.

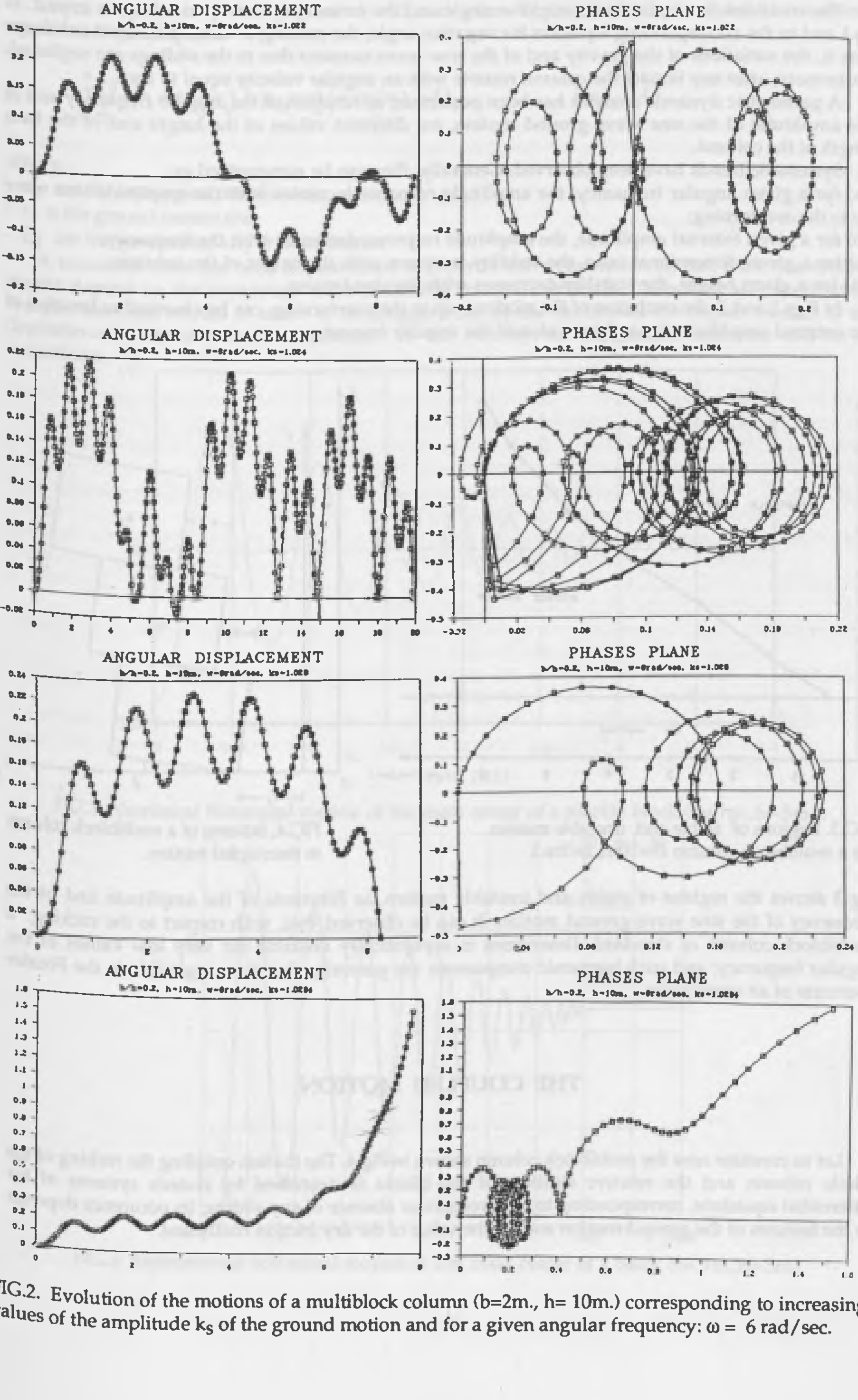

FIG.2. Evolution of the motions of a multiblock column (b=2m., h= 10m.) corresponding to increasing values of the amplitude k_s of the ground motion and for a given angular frequency: $\omega = 6$ rad/sec.

The transition from positive to negative angle and the contrary is characterized by an impact. In eq.1 and in the corresponding equation for negative angle, the rocking is uncoupled by the slidings; that is, the variations of the gravity and of the sine wave moment due to the slidings are neglected. Furtermore, after any impact the column restarts with an angular velocity equal to zero.

A parametric dynamic analysis has been performed as function of the angular frequency and of the amplitude of the sine wave ground motion, for different values of the height and of the base length of the column.

Systematic trends have been observed. Generally, they can be summarized as:

- a) for a given angular frequency, the amplitude response increases with the amplitude sine wave up to the overturning;
- b) for a given external amplitude, the amplitude response decreases with the frequency;
- c) for a given dimensional ratio, the stability increases with the height of the column;
- d) for a given height, the stability decreases with the slenderness.

In Figs.1 and 2 the evolution of the motions, up to the overturning, can be observed as function of the external amplitude, for a given value of the angular frequency.

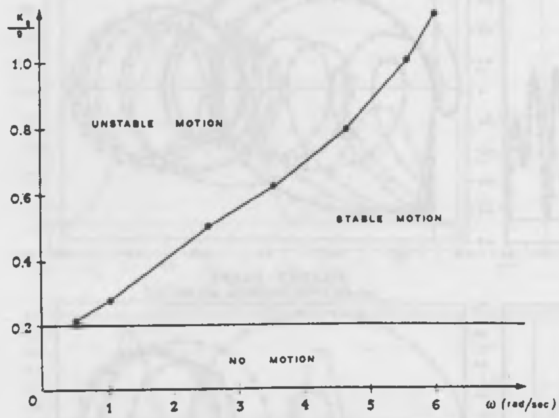

FIG.3. Regions of stable and unstable motion for a multiblock column (h=10m, b=2m.).

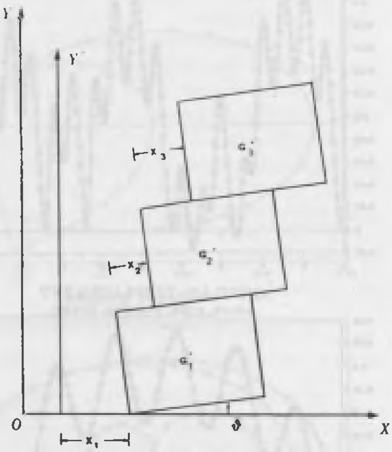

FIG.4. Scheme of a multiblock column in thecoupled motion.

Fig.3 shows the regions of stable and unstable motion, as functions of the amplitude and of the frequency of the sine wave ground motion. It can be observed that, with respect to the rocking, a multiblock column of standard dimensions is asyntotically unstable for very low values of the angular frequency; and such harmonic components are generally absent or negligible in the Fourier spectrum of an earthquake.

THE COUPLED MOTION

Let us consider now the multiblock column shown in Fig.4. The motion coupling the rocking of the whole column and the relative slidings of the blocks is described by sixteen systems of for differential equations, corresponding to the presence or absence of any sliding; its occurence depends on the features of the ground motion and on the value of the dry friction coefficient.

Let us consider only the case, for $\theta > 0$, and when all the three blocks slide. The dynamic equations are in this case:

$$\sum_{j=1}^{4}\left[f_{ij}\ddot{x}_j + g_{ij}(\dot{x}_j) + p_{ij}(x_j)\right] + q_i(x_s) = Q_i \qquad i = 1,..\ 4 \tag{2}$$

where:

- x_i are the lagrangian coordinates;
- x_s is the ground motion and
- Q_i are the generalized dry friction forces.

A numerical code has been implemented in order to solve equations (2); but, their quantitative results depend on the numerical value of the dry friction coefficient. Therefore, an experimental analysis has been started in order to investigate both: a) the validity of the impact model; b) the dependence of the dry friction coefficient on the velocity, on the surface extension and on its smoothness.

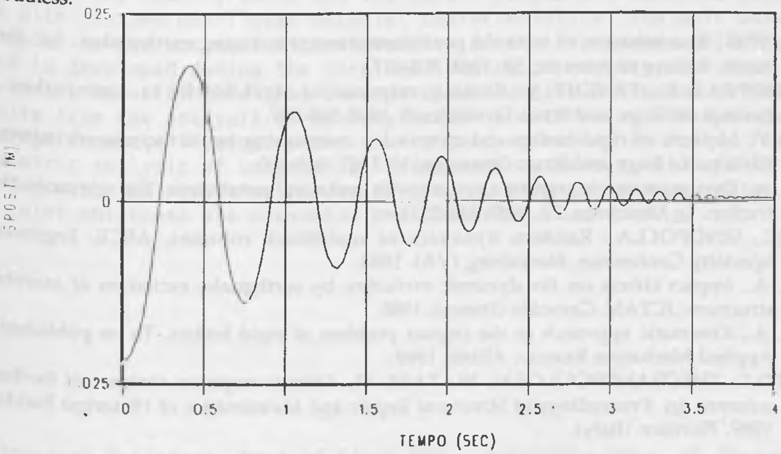

FIG.5. Numerical horizontal motion of the mass center of a marble block (b=.1m.,h=.5m.).

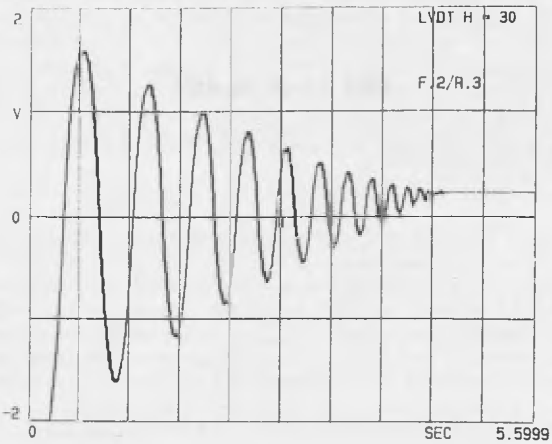

FIG.6. Experimental horizontal motion of the mass center of a block (b=.1m.,h=.5m).

The first experiments performed concern the free motion of a marble smooth block; the obtained results show a good qualitative agreement between the numerical and experimental behaviour. They can be seen, respectively, in Fig.5 and 6, where the horizontal displacements of the mass center are shown (b=0.1m.,h=0.5m.).

Particularly, it can be interesting to observe that the motion is a rototranslation and, after a given time interval, only a sliding; so that the coupling played by the impact is confirmed. The agreement is worse with respect to the time behaviour; in our opinion this depends on the simulated value of the dry friction coefficient.

Nevertheless, the investigations are in progress in order to converge to a complete agreement by means of iterative comparison.

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THE IN PLANE BEHAVIOUR OF MASONRY
AN ANALYTICAL FINETE ELEMENT MODEL

C. Ignatakis*, E. Stavrakakis* and G. Penelis**

SUMMARY

An analytical model for the behaviour of masonry under in-plane loading is presented. The masonry units and the mortar joints are modelled separately each with its own non-linear material characteristics. Tha main advantage of the model is the successful consideration of the transverse principal stress which is developed during the in-plane loading of masonry. For this purpose a triaxial material model has been developed and is presented briefly. The results from the analysis of masonry models are compared with the experimental findings from the literature and a very good agreement is found. Finally a parametric analysis of uniaxially loaded masonry panels until failure is carried out. The resulting curves of masonry strength versus mortar strength and joint thickness are presented and commented.

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15th of April 1989

INTRODUCTION

Masonry is a material which exhibits distinct directional properties because the mortar joints usually act as planes of weakness. The in-plane strength of masonry is mainly affected by the following parameters: a. The strength of both the masonry unit and the mortar. b. The bond between the unit and the mortar. c. The geometry of the wall.

The uniaxial compressive strength of masonry with solid units is always greater than the corresponding strength of the weaker "associate" (mortar or masonry unit)(Hendry,1981). Under biaxial compression, transverse splitting of units leads to masonry failure (Page,1981) at a level of stresses lower than the corresponding in plane biaxial strength of units. At the same time the corresponding biaxial strength of the mortar could be exceeded without joint failure. These inconsistencies are remedied by the introduction of the transverse principal stress (σ_z)(Francis et al.,1970).

With the advent of computer-based numerical techniques, various attempts have been made to model the in-plane behaviour of masonry(Page-Samarasinghe-Hendry,1982). A finite element model capable of predicting joint failure in brick masonry but not a composite failure involving both brick and joint has been developed by Page(1978). The main problem encountered by all investigators has been the lack of a representative material model.

An effort for the development of an analytical model for the in-plane behaviour of masonry until failure, has taken place in the Reinforced Concrete Lab. of Aristotle University of Thessaloniki-Greece. The general structure and some of the basic routines of the RECOFIN program -which has been also developed at the same laboratory and successfully used(Penelis,Stavrakakis and Ignatakis,1975,1988) for the in-plane analysis of reinforced concrete- are incorporated into the program for masonry analysis(MAFEA:MAsonry Finite Element Analysis).

PRESENTATION OF THE ANALYTICAL MODEL

The model is capable of simulating damages in units, mortar or joints and predicting both joint or composite type failure due to in-plane stresses in conjunction with the transverse third principal stress (σ_z). Masonry units and mortar joints are modelled separately and divided automatically into rectangular elements with an internal node. Mortar elements are distincted according to their position(I,II,III see fig.1). External or self weight loading is applied incrementally and monotonically.

Analytical model for the evaluation of transverse stress (σ_z)

Francis et al,(1970) have given a simple model for the evaluation of the transverse stress in uniaxially loaded masonry piers. With a suitable modification for in-plane loading, equations for the transverse stresses of the mortar (σ_{zm}) and of the masonry units (σ_{zb}) have been dirived, in a coordinate system orientated to the directions of joints(see fig.1):

$$\sigma_{zm}=f(\sigma_{xm},\sigma_{ym},\sigma_{xb},\sigma_{yb},\nu_m,\nu_b,\lambda_A,\lambda_E), \quad \sigma_{zb}=-(\sigma_{zm}/\lambda_A) \tag{1}$$

$\sigma_{xm},\sigma_{ym},\sigma_{xb},\sigma_{yb}$: The in-plane normal stresses of mortar and brick in contact.
ν_m,ν_b: The Poisson's ratios of mortar and brick respectively.
$\lambda_A=A_b/A_m$: The ratio of corresponding surfaces of brick(A_b) and mortar joint (A_m) as they are geometrically defined(see fig.1).
$\lambda_E=E_b/E_m$: The unit/mortar Modulus of Elasticity ratio.

The value of λ_E is greatly affected by the stress state of masonry due to the non-linear behaviour of the materials. Consequently, the equations (1) must be introduced into the analytical model in an incremental form. The factor λ_E must now be considered as the unit/mortar ratio of instantaneous values of modulus of Elasticity for the present loading step: $\lambda_{E,inst.}=E_{b,inst.}/E_{m,inst.}$

The transverse stresses (σ_{zm}, σ_{zb}) could influence the analytical process only if a triaxial failure criterion is introduced. On the other hand for a reliable calculation of the factor $\lambda_{E,inst.}$ a triaxial constitutive model is needed.

Failure Surface for Triaxial Stress State

Due to the lack of experimental data mainly for Bricks and secondly for Mortars under biaxial and triaxial stress states, it is assumed that their behaviour is similar to that of a concrete with equal compressive strength. An extensive numerical investigation of experimental data has been carried out and a continuous failure surface for triaxial stress state has been composed. The failure surface is constituted by three families of curves for each one of the triaxial tension, tension-compression and triaxial compression region. In the figures 3a,3b and 3c a hydrostatic section, two deviatoric sections and a section of the failure surface by the plane($\sigma_1=0$), are shown correspondingly. The latest is an analytical representation of the well known biaxial failure envelope of concrete-like materials(Kupfer et al.,1969) and it has been used as a guide curve for the formation of the failure surface.

Triaxial Constitutive Model

Due to lack of experimental data, the development of a rather simple incremental elastic model has been decided. It requires the uniaxial stress-strain curve(see fig.4) and the failure surface of the material. For an element under a given stress state $\bar{\sigma}(\sigma_1,\sigma_2,\sigma_3)$ in principal stresses, the corresponding equivalent stress (σ^e) and the equivalent stress-strain curve are defined as:

a. The ultimate stress vector $\bar{\sigma}_u(\sigma_{1u},\sigma_{2u},\sigma_{3u})$ is defined from the failure surface(figure 3a).

b. The safety factor (c) of the element and a multiplier (K), for the uniaxial ultimate strain (ε_u) of the material, are defined as follows:

$$c=|\bar{\sigma}_u/\bar{\sigma}| \quad (\text{or } c=f_c/|\bar{\sigma}| \text{ if } f_c>|\bar{\sigma}_u|), \quad K=f(\sigma_{1u},\sigma_{2u},\sigma_{3u}, f_c) \tag{2}$$

where f_c is the uniaxial cylindrical strength of the material.

c. Equivalent stress: $\sigma^e=f_c/c$ (3)

d. Equivalent ultimate strain: $\varepsilon_u^e=K*\varepsilon_u$ (4)

By substituting the ε_u with the ε_u^e in the equation of uniaxial stress-strain curve, the equation of the equivalent stress-strain curve is derived. The slope of the latest curve at the value of the equivalent stress σ^e gives the instantaneous modulus of elasticity(E^e_{inst}) for the element under consideration at the present loading increment.

Using the data of Kupfer et al.(1969), a modification function for the Poisson's ratio is developed, depending on the stress state.

A very good matching for both biaxial and triaxial stress states has been observed in comparison with experimental data.

Failure Criterion for Mortar Joints

Every mortar element is checked for the couple of in-plane normal and shear stresses (σ_n,τ) in the direction of joint, using the bilinear failure criterion shown in fig.5(Hamid et al,1982). The joint failure is characterized as unstucking for $\sigma_n>0$ or sliding for $\sigma_n \leq 0$. The post-sliding joint failure criterion is represented by the dashed line of fig.5.

Types of Damaged Elements

An essential part of the MAFEA program is the subroutine "CHECK" where all the elements are checked according to the failure criteria. The elements library of the model includes seven different types of damaged elements with the following features:

Element **a**: Intact brick or mortar element.

Element **b**: Brick or mortar element cracked(smeared model) perpendicularly to the direction of the major principal tensile stress in-plane. The modulus of elasticity normal to the crack and the shear modulus drop to zero as aggregate interlock assumed to be negligible.

Element **c**: Mortar element unstuck from the neighboring masonry unit. The element is treated as a cracked one (type **b**), in the direction of the joint.

Element **d**: Mortar element slided relatively to the masonry unit in contact. The shear stress (τ) drops to the value $\tau_s=\mu*\sigma_n$ (see fig.5).

Element **e**: Brick or mortar element cracked in two perpendicular directions. The stiffness matrix of the element drops to zero from now on.

Element **f**: Brick or mortar element splitted perpendicularly to the transverse principle tensile stress (σ_z).The element retains its stress state with the exception of σ_z, which together with the stiffness matrix drop to zero.

Element **g**: Brick or mortar element crushed. The element retains its stress state but the stiffness matrix drops to zero.

All the phisically possible evolutions of already damaged elements are provided in the cracking procedure and summarized in figure 2.

Computational procedure

The well known technique of step by step loading is used. In each step the additional stresses and strains including the transverse stress (σ_z) of each element are determined by using the stiffness matrices from the stress state of the previous loading step. These stresses and strains are added to those of previous steps and a checking procedure follows. If no changes take place in the state of each element, the next loading step is applied. Otherwise the solution for the present step is repeated after the proper updating of the stiffness matrices of the newly damaged elements and after the addition of the unbalanced forces to the loading vector. Failure is assumed to occur and the computation is terminated when the mathematical procedure becomes unstable.

VERIFICATION-APPLICATION OF THE ANALYTICAL MODEL

One of the most complete experimental works on the in-plane behaviour of brick masonry under biaxial stress state was carried out by Page,(1981,1983). Ten(10) of the 180 experimentally tested panels were modelled and analyzed using the MAFEA computer program. The experimental failure envelopes, on a principal stress system given by Page, are shown in fig.6. The analytically determined points of failure for the 10 models by MAFEA program are also indicated. The agreement is very good.(For the comparison of the damage patterns and the type of failure see: Ignatakis et al., 1989)

Parametric Analysis of Uniaxially Loaded Masonry Panels

The uniaxial compressive strength of masonry (f_{wc}) is greatly affected by the mortar strength (f_{mc}) and the joint thickness. Many experimental parametric studies have been carried out, several analytical models have been developed and a lot of semiempirical formulas have been proposed to express the compressive strength of masonry as a function of the material properties and the geometrical characteristics of masonry(Hendry,1981), (Tasios,1986). Due to the nature of masonry both the experimental findings and the analytical proposals show a wide scatter. The general qualitative conclusions are: a. f_{wc} increases almost linearly with f_{mc} for weak mortars. The rate of increasing, drops rapidly for stronger mortars and finally f_{wc} remains almost constant. b. For a mortar weaker than the masonry unit, f_{wc} decreases rapidly for increasing joint/unit thickness ratio (t_m/t_b). The rate of decreasing drops for thicker joints and f_{wc} tends asymptotivcally to the strength of the mortar (f_{mc}).

A parametric analysis of uniaxially loaded square masonry panels until failure is carried out, using the MAFEA computer program. In the first group, 8 models with variable mortar strength were analysed. The material characteristics, the corresponding strength of masonry and the mode of failure are given in table 1. In the second group, 6 models with the material characteristics of the first group model No 4 and variable joint thickness were analysed. The values of (t_m/t_b), the corresponding f_{wc} and the mode of failure are also given in table 1. In figures 7a and 7b the curves (f_{wc}/f_{bc} vs. f_{mc}) and (f_{wc}/f_{bc} vs. t_m/t_b) are shown correspondingly. The two curves exhibit clearly the qualitative characteristics mentioned above. The type of failure of the models are also in accordance with experimental findings(Hendry,1981), (Page,1981).

Table 1. Material properties, Strength and Failure mode of Masonry Models

GROUP I		f_c (MPa)	E_o (GPa)	σ_t* (MPa)	τ_c* (MPa)	Masonry strength f_{wc}(MPa)	Mode of Failure
Bricks		15.0	8.0	11.5x3.5x5.0cm			
Variable Mortars	1	1.5	0.8	0.06	0.12	3.60	Crushing of bed joints(b.j.)
	2	3.0	1.6	0.12	0.24	7.10	Unstucking of header joints(h.j.)
	3	4.5	2.4	0.18	0.36	10.10	Cracking or splitting of bricks
	4	6.0	3.2	0.24	0.48	13.00	near header joints
	5	9.0	4.8	0.36	0.72	14.30	Cracking or crushing of h.j.
	6	12.0	6.4	0.48	0.96	14.60	Cracking of bed joints
	7	15.0	8.0	0.60	1.20	15.00	Limited cracking of bricks
	8	21.0	11.2	0.84	1.68	15.60	Unstucking of header joints

GROUP II		Material properties	t_m (cm)	t_m/t_b	f_{wc}(MPa)	Mode of Failure
Variable Joint thickness	1	Bricks: see group I	0.35	0.10	13.50	Crushing of bed joints
	2	Mortar: see model No 4 in group I	0.50	0.14	13.00	Limited cracking of bricks
	3		1.05	0.30	9.90	Unstucking of header joints
	4		1.75	0.50	8.40	Cracking of bricks
	5		2.45	0.70	7.40	Cracking of bed joints
	6		3.50	1.00	7.10	Unstucking of header joints

*Strength of unit-mortar interface(see fig.5).Coef.of friction μ=0.75const.
Initial Poisson ratio for all the materials: v_o=0.20

CONCLUSIONS

The analytical model(MAFEA computer program) proved to be capable of predicting the ultimate loads, the damage pattern and the failure mode of masonry subjected to in-plane loading. The main advantage of the model is the successful consideration of the transverse principal stress. The development of a triaxial constitutive material model and the composition of a failure surface in the space of principal stresses were essential for both the correct calculation and the realistic contribution of the transverse stress (σ_Z) to the behaviour of masonry. The model was successfully verificated against experimental data and proved to be suitable for analytical research.

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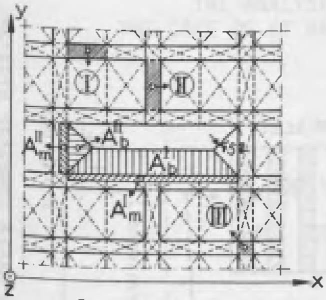
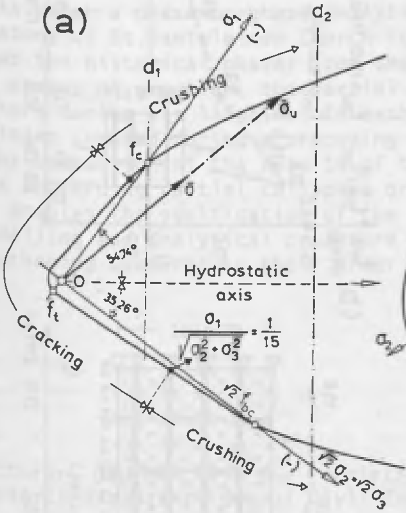


$$\lambda_A^{I} = \frac{A_b^{I}}{A_m^{I}},\quad \lambda_A^{II} = \frac{A_b^{II}}{A_m^{II}},\quad \lambda_A^{III} = \frac{\lambda_A^{I}}{2}$$

Figure 1: F.E.Model of Masonry

Type of new damage	Previous State of Element						
	a	b	c	d	e	f	g
b	b	e	e	b	Fatal failure – No further checking		
c	c	e	c	c			
d	d	b	c	d			
f	f	f	f	f			
g	g	g	g	g			

Figure 2: Damaging process of the elements



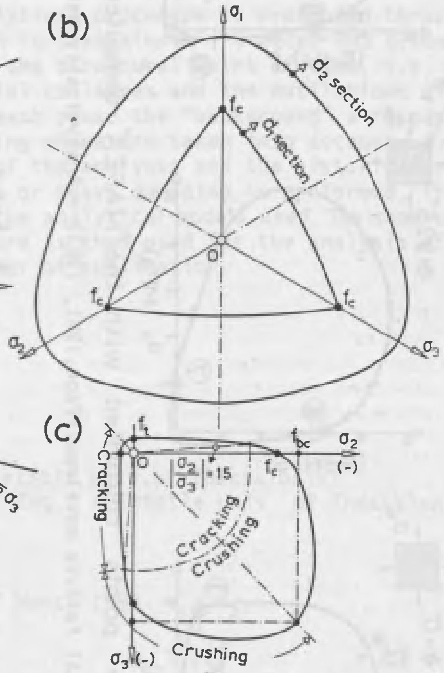


Figure 3: Failure surface
(a): Hydrostatic section
(b): Deviatoric sections
(c): Biaxial section

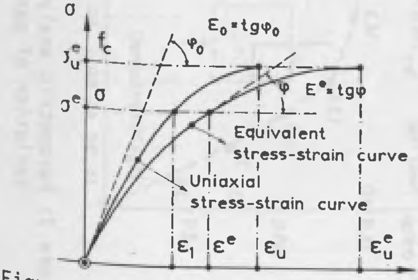


Figure 4: Triaxial Constitutive Model
Equivalent stres-strain curve

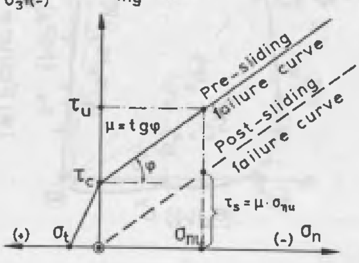


Figure 5: Joint Failure Criterion

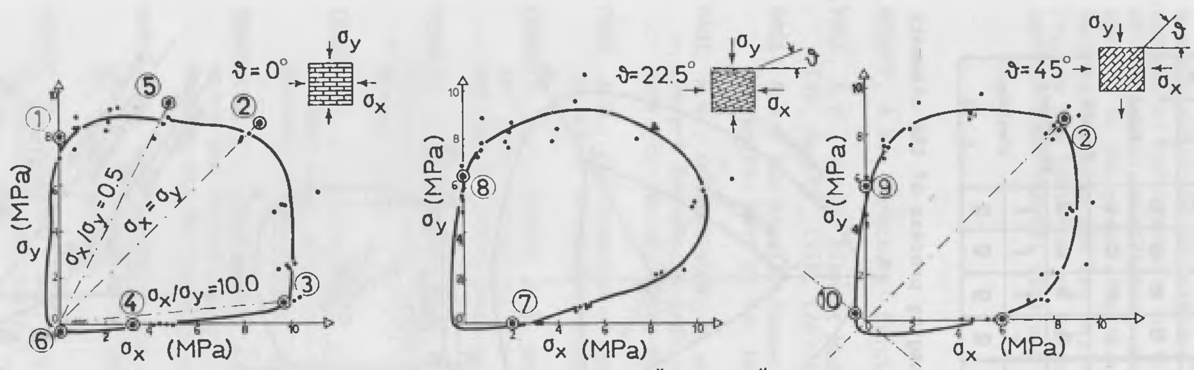


Figure 6: Experimental failure envelopes (Page, 1981, 1983) - Analytically determined points of failure.

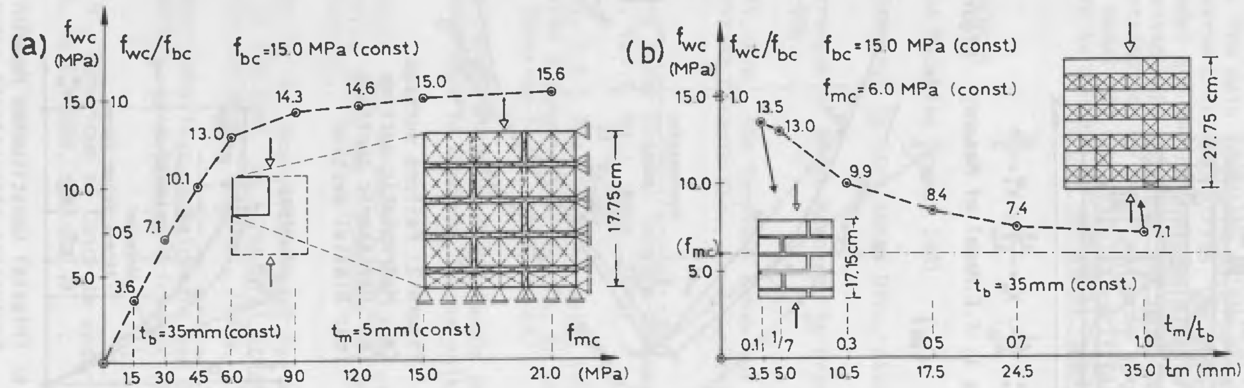


Figure 7: Parametric analysis of masonry panels under uniaxial compression, using the MAFEA computer program. Variation of panel strength with: (a) Mortar compressive strength, (b) Joint/Unit thickness ratio.

THE ANALYSIS OF A BYZANTINE MONUMENT.
THE CASE OF ST.PANTELEIMON CHURCH IN THESSALONIKI

K. Stylianidis*, C. Ignatakis**

SUMMARY

In this paper a phase-by-phase analytical procedure is presented through the case study of St.Panteleimon Church in Thessaloniki, Greece. This procedure follows the historical phases from the structural point of view, i.e. all the basic phases of erection, the partial collapses and the mutilations of the structure during its lifetime. In each phase the "background" stresses and the hinges created during a preceding phase are taken into account. A simultaneous comparison of the results of the analyses and the historical references concerning partial collapses or heavy damaging is performed. This check enables the verification of the analytical models used. The same concept of modelling and analytical procedure is then used for the analysis of the strengthening measures in their order of application.

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ST.PANTELEIMON CHURCH IN THESSALONIKI

Structural form and historical background

The St.Panteleimon Byzantine Church in Thessaloniki was built during the early period of Paleologi(late 13th-early 14th century) (Ioannidou, 1986). In its initial form it was a brick and stone masonry crossed compound four-pillar church with a three-aisled perimetric gallery (Fig.5). It had one central dome, one dome on the narthex and two more domes on the gallery. Wooden tierods resisted the horizontal thrusts developed by the four main arches, the vaults of the narthex and the vaults of the gallery. Wooden beams forming a belting system were found embedded in the surrounding masonry walls and connected with the wooden tierods mentioned above. The walls were resting on deep foundation beams while the pillars were resting on masonry foundation blocks connected with the perimetric foundation beams(Traganou-Deligianni,1986). The historical documentation of the monument reports successively an early collapse of the western aisle of the gallery, a cutting off of the wooden tierods of the narthex and a collapse of the northern and later of the southern dome of the gallery(Ioannidou, 1986). As no attempt has been undertaken to restore the missing architectural and structural elements, all the mutilations and the aging effects are today apparent on the monument (Fig.1,2).

Damage pattern

Damages are more extensive in the higher parts of the monument and tend to diminish in the lower parts. Taking into account the crack pattern and the structural form of byzantine churches, it can be concluded that the initial damages were mainly caused by the self-weight thrusts and the accompanying deformations. The damages were later precipitated because of the gradual weakening of the supporting system due to aging and because of earthquakes. Actually, in situ investigations verified that, at the present time, the wooden tierods can not function as they had been designed to, mainly because of sliding of their supports in the masonry. As a result, the supporting system gives way causing extensive cracking. Some of the cracks can be characterized as hinges since they are very deep and are accompanied by great rotations. In three main arches, out of the four supporting the central dome, the width and the span have values of the same order of magnitude. These arches, having underneath a front tympanum initially built as an infill, are diagonally cracked, having the typical cracking pattern of a plate supported on three edges and loaded on the free edge. In the so called chappels, which are remainders of the collapsed gallery, great inclinations of the walls supporting the vaults are observed. Probably the most dangerous cracks are these of the compound pillars where there is a tendency of formation of inclined sliding surfaces due to the combination of normal and shear stresses. These surfaces tend to separate the compound pillars into their components. On the contrary, the cross-sections of the pillars on the foundation appear to be undamaged. In the part of the foundations, where exploratory soil sections were cut, some cracks appeared on the deep beams, but there is no sign of overall instability.

Remedies

In spite of the structural problems arised during seven centuries of life, it cannot be ignored that the monument is still standing. Consequently, the remedial measures tend to help the existing load transferring system, rather than to change it(Wenzel,1983). As criteria for the proposed interventions, in addition to the reversibility requirement, the concepts of modern design codes were used. For the vertical loads and the design seismic actions, serviceability and strength criteria were taken into account, which allow for limited cracking but no formation of hinges. These criteria led to the proposal of filling the cracks with compatible mortar injections, addition of new metal tierods, lightly prestressed, at the same level as the existing ones and construction of thin doweled shotcrete jackets on the outer surface of the arches, given that the masonry did not have the required strength. Furthermore, the reinforced jackets, combined with the new metal tierods, can give adequate ductility to the strengthened structure for the prevention of collapse under sevére seismic actions.

ANALYTICAL PROCEDURE

Analytical possibilities

Although it is beyond the scope of this paper to discuss the analytical possibilities, it must be mentioned that in such cases a simple type of static analysis is recommended even for seismic loads(WG F of UNIDO,Ch.4,1984). For the time being, detailed dynamic analyses do not seem to be feasible because of the difficulties in assessing ground motion characteristics and dynamic properties of materials and because of their computational cost. The best that can be expected seems to be a limit-state static collapse analysis using appropriately simplified models of the overall structural system(Mainstone,1986).

Analytical procedure proposed

The analytical procedure performed is a limit-state static collapse analysis. This procedure has already been successfully used in a limited extend in the past(WG F of UNIDO, Ch.9,1984).

Model: The structure has been divided in four main plane frames. Each frame consisted of arches, vaults, piers, walls, columns, tierods, struts, foundation beams and soil springs. Domes were analyzed separately in a previous stage. Elastic properties of masonry, timber, marble and soil were used to represent the adjacent structural elements. Regions of practically infinite stiffness were used to transfer the loads from the arches and vaults on the eccentrically loaded piers. Wooden tierods, marble columns and masonry struts were represented as hinged elements in both ends. Masonry struts were used to represent infills when it was assumed that they were activated. Springs with time-depending stiffness represented the foundation soil. Boundary conditions were considered to be satisfied when the vertical displacements of an individual soil spring (belonging in two perpendicular plane frames) were almost equal.

Loading: Dead loads, seismic load and prestressing loads were considered. It was assumed that the loading was applied in a monotonically increasing way.

Failure criteria: A cross-sectional failure criterion was adopted concerning combination of normal and shear stresses. This criterion was derived through

Mohr´s cycles from the corresponding Kupfer´s criterion for low strength concrete. The constitutive law of all the materials used was linearly elastic. A suitable computer program was writen to determine the safety factor of a polygonal cross-section under biaxial bending. Deformation failure criteria were not used to indentify local collapses and this is a shortcoming of the analytical methods like the one used(Mainstone,1986). On the other hand, deformation criteria were used to model the activation of infills as diagonal struts, when arches or vaults tended to rest on the infills underneath.

Application of the procedure: In each phase the background stresses and the hinges created in the preceding phase are taken into account representing the past history effects. A portion of the loading is applied step-by-step, until a new hinge forms. The model is then accordingly modified to take account of the new hinge and another portion of the loading is then applied. The step-by-step analysis continues until the total loading is applied. The hinges formed represent regions of excessive damages during the adjacent phase. A simultaneous comparison of the analyses results and the historical references concerning excessive damages or collapses is then performed. An appraisal method, like part of the one proposed by ICE(Beckmann-Happold,1983) is then applied.

RESULTS

The phases considered and some of the corresponding analytical results are summarised in Table 1. During the phase of construction, under the total dead loads, hinges were formed in the foundation and in the arches and vaults, especially at keystone regions, with the exception that at the gallery vault, hinges were formed at the origins as well(Fig.3). Under feeble seismic loads, the vault of the western gallery was proved to be unstable(Fig.3). The collapse of the gallery arch did not affect the remaining structure. The cutting off of the wooden tierods of the narthex caused many hinges in the corresponding vault while the rest of the structure was not affected(Fig.4). Application of seismic

Table 1. Analyses results (F:Foundation, V:Vaults)

PHASES CONSIDERED		FIG.	NEW HINGES	TOTAL HINGES	ANALYSES
Phases of construction	Erection of piers	3	1F	1F	3
	Arches and vaults	3	1F/5V	2F/5V	5
Seismic loading : 1.4%		3	1V	2F/6V	1
Collapse of gallery vault					
Structural modification		4	-	2F/2V	1
Cutting off of narthex tierods		4	4V	2F/6V	5
Seismic loading: up to 11.1%		4	2F/5V	4F/11V	9
Seismic loading: increasing		5	MANY		PIERS CHECK
Phases of intervention	Roof removal	-	-	2F/6V	3
	50% prestressing	-	-	2F/6V	1
	Mortar injections	-	-	2F	-
	50% prestressing	-	-	2F	1
	Roof reconstruction	-	-	2F	1
Seismic loading:up to 18.1%(W-E)		6	6V	2F/6V	9
Seismic loading:up to 17.9%(E-W)		6	6V	2F/6V	7

loads led to the formation of many hinges in the arches and vaults which tended to rest on the infills underneath, suffering excessive damaging(Fig.4). Consequently, the ultimate seismic load of the individual piers can be considered as the upper limit of the ultimate seismic load of the structure(Fig.5). After the application of the mortar injections, the level of the remedial prestressing forces was calculated using the criterion of no hinges formation under vertical loads. Considerably higher seismic loads were needed to be applied, in relation to the unstrengthened structure, for the formation of the same number of hinges in the masonry arches and vaults. No hinges appeared in concrete jackets under the ultimate seismic loads of the piers mentioned above(Fig.6). From the analytical results presented above it can be concluded that, using the limit-state static collapse analysis, a satisfactory representation of the damage pattern, including partial collapses, can be derived. Consequently, the precedure used is an effective analytical method for the prediction of the future response of the strengthened structure.

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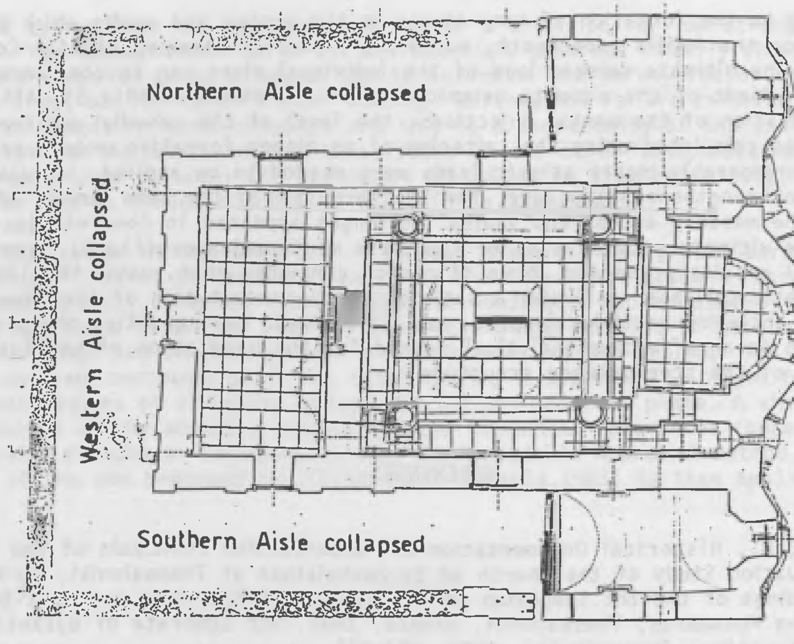

Fig.1 Plan at ground level of St.Panteleimon Church as it is today

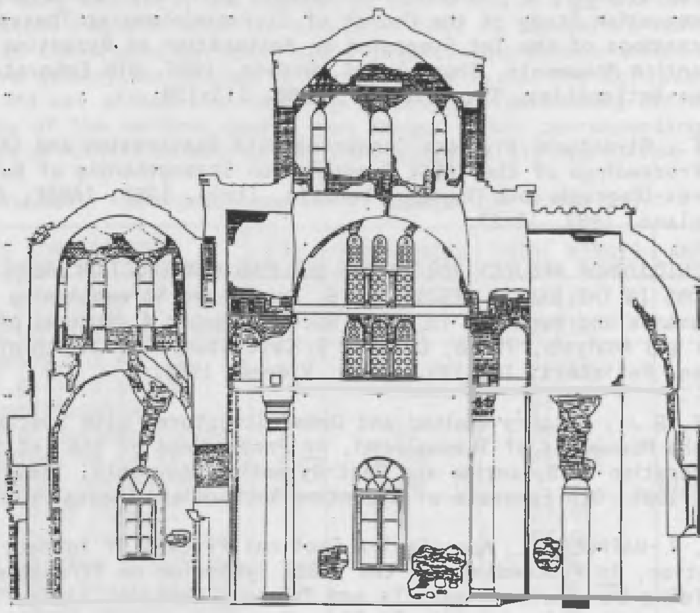

Fig.2 West-East cross section of the Church

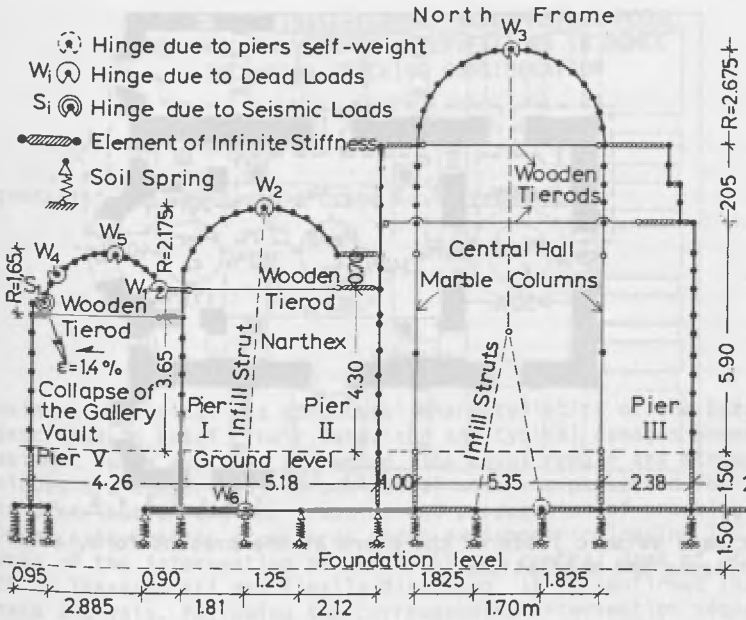

Fig.3 Analytically determined damages during the phases of erection of piers and construction of arches and vaults.

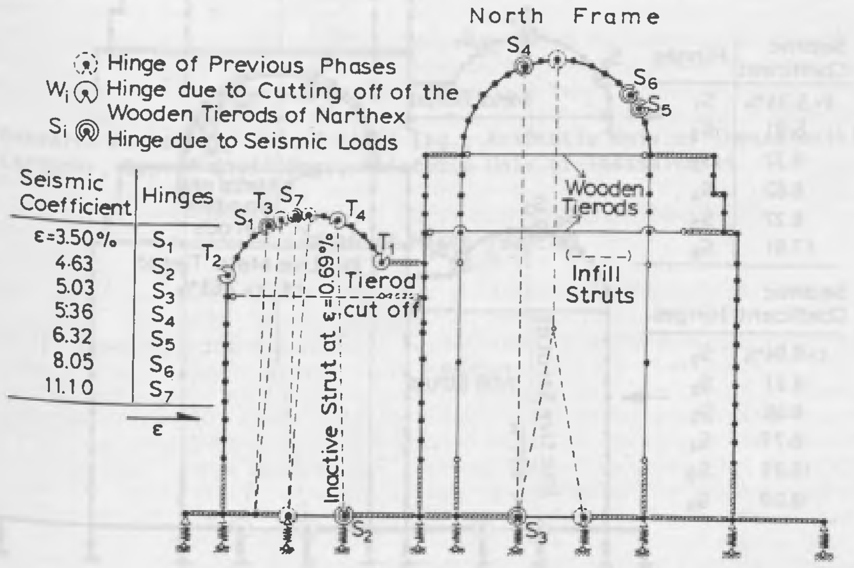

Fig.4 Analytically determined damages of the structure as it is today

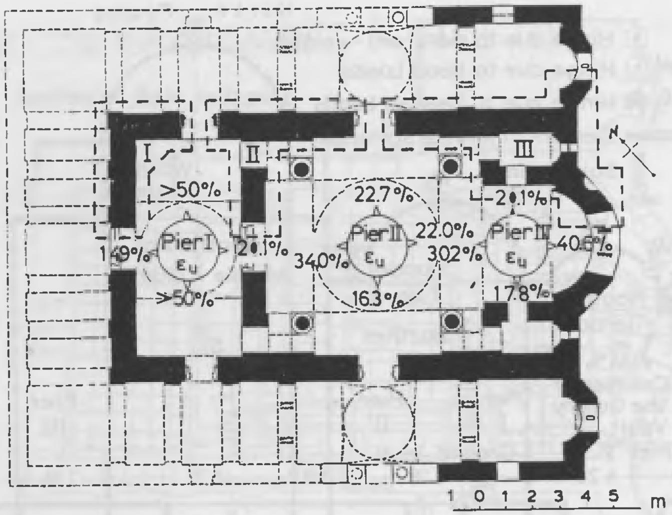

Fig.5 Ultimate seismic loads of the piers at the present form, after the collapses and mutilations.

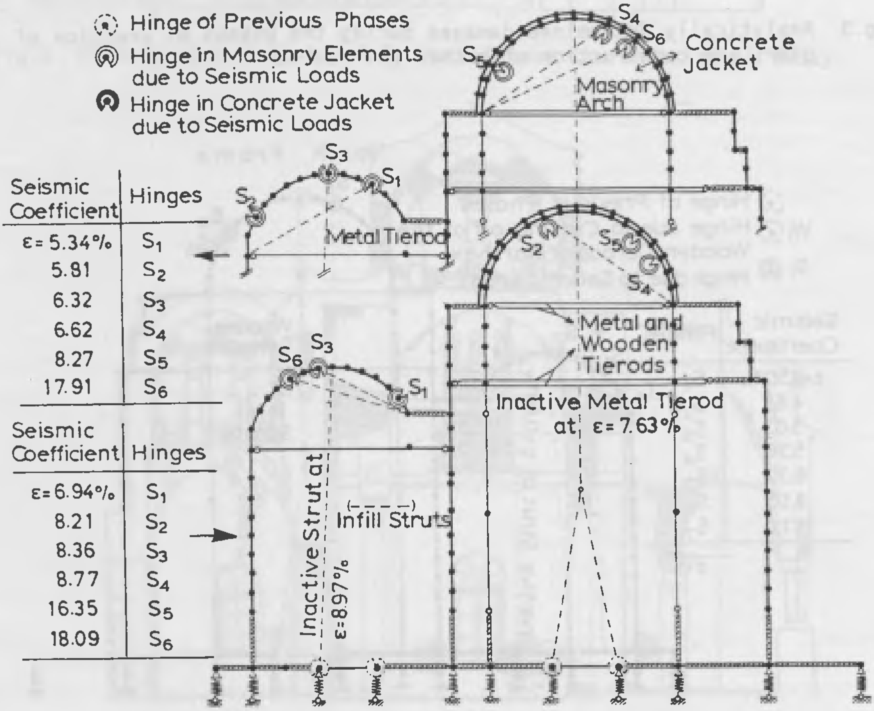

Fig.6 Analytically determined damages of the strengthened structure under West-East seismic loads.

A SPECIAL FINITE ELEMENT AXISYMMETRIC MODEL FOR THE ANALYSES OF INTERVENTIONS IN DOMES INCLUDING CRACKING CONSIDERATION

C. Ignatakis*, K. Stylianidis** and E. Stavrakakis*

SUMMARY

The main morphological and structural characteristics of the Byzantine Domes are described in brief. Their behaviour and typical damages under vertical and seismic loads are also presented. The usual repair and strengthening techniques are summarised. An analytical model is presented for the stress-strain distribution and the creation and propagation of cracking in meridian levels of axisymmetric structures under axisymmetric loading. Two comparative analyses of the intervention procedure in the central dome of St.Panteleimon Church in Thessaloniki are finally discussed. It is confirmed that a phase by phase analysis, following the corresponding intervention sequence, is needed. Cracking consideration seems to be the most significant factor affecting the results.

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20th of March 1989

INTRODUCTION - THE BYZANTINE DOME

Morphological and material characteristics

The Byzantine domes of the period of Paleologi are characterized by a high cylindrical shell usually cut away by eight windows forming strong piers (Fig.1). A wooden tierod runs across the upper part of the openings. The drum is supporting a thick hemispherical-like dome. The heavy roof covering usually consists of ceramic tiles on a lime mortar bed. Masonry consists of shallow solid bricks(4 to 5cm thick) and of equal thickness crushed tile lime mortar joints. The tensile strength of the masonry is very small. The compressive strength is one or two order of magnitude greater than the stresses developed under service loads. Consequently the assumption of a non tension material governed by a linear constitutive law is a realistic consideration(Heyman, 1986).

Behaviour-damages under vertical and seismic loading. Remedies in domes

Under the thrusts of the shell the piers move outwards inducing circumferential tension and meridional cracks appear at the lower zone of the shell (Fig.1,cracks I). Corrosion of wooden tierods accompanied with further spreading of the supporting system, causes additional cracking(Fig.1,cracks II). The dome tends to function as a ring of arches all having the uncracked crown region of the shell as a common keystone(Mainstone,1986). In some cases horizontal cracking at the inner part of the bases(Fig.1,cracks III) and small hinge rotations at the heads of the piers also appear. Under seismic loading, depending on the extent of cracking, the shell behaves as a diaphragm or the piers tend to function like free standing members.

Mortar injection is a typical example of passive type intervention in domes. Concrete skins are not recommended as a normal procedure. Addition of external, prestressed, reversible, corrosion resistant metal ties is an effective active type strengthening measure for a heavily damaged dome.

PRESENTATION OF THE ANALYTICAL FINITE ELEMENT MODEL

An analytical model representing successfully the above mentioned characteristics of the domed structures has been developed. The model does not aim to represent the microscopic behaviour of the masonry but it aims to determine the stress-strain distribution and the cracking propagation of a complete axisymmetric structure. The masonry is considered to be a homogeneous and isotropic material with average linearly elastic behaviour. The element grid is automatically created. Addition of new nodes and elements is possible in any position. The model accepts external loads at the nodes. Self weight or roof cover loads are transformed to nodal loads.

Library of Finite Elements - Constitutive laws

The model includes the following four types of elements to represent the characteristics of each member of the structure. The constitutive law has the general form of equation (1)

$$\{\sigma_z, \sigma_r, \sigma_\theta, \tau_{zr}\} = \bar{D}_{(4x4)} * \{\varepsilon_z, \varepsilon_r, \varepsilon_\theta, \gamma_{zr}\} \quad (1)$$

a. Axisymmetric isotropic element for the representation of the uncracked region of the dome.(Stiffness matrix $\bar{D}_a$:see Zienkiewicz,1971). b. Axisymmetric anisotropic(stratified) element for the representation of embedded wooden tierods. The material has different moduli of elasticity(E) in the circumferential direction and the meridian level (Stiffness matrix $\bar{D}_b$: see Zienkiewicz,1971). c. Plain stress isotropic element for the representation of cracked region of the dome and a cut-off or corroded embedded tierod. The same type of element can also simulate the drum of piers as they are actually almost under plane stress condition in meridian levels. Consequently the piers can be represented by an axisymmetric drum with an equivalent thickness, which can be calculated by equating the stiffnesses of an individual pier and the adjacent part of the drum(Fig.2). The components of the third column and row of the stiffness matrix D_c have zero values as the material is unable to resist circumferential stresses. d. Uniaxially stressed element for the representation of non-embedded external tierods. The stiff.matrix $\bar{D}_d$ has only one non zero component(D_{33}=E) which indicates that only the development of principal circumferential stress is permitted.

Computational procedure - Cracking propagation

Loading. A simple cracking criterion in meridional level has been adopted. A first analysis of the structure is carried out under total loading. All the elements of type (a), having tensile circumferential stress greater than the assumed tensile strength of the material, are modified to type (c) cracked elements and the analysis is repeated until no further crack occurs. Application of the step-by-step loading technique is not necessary due to constitutive linearity. Unloading. A criterion for crack closure has been adopted based on the value of the radial displacement. A first analysis of the cracked structure is carried out under the load remaining after the unloading. The cracked elements which have negative radial displacement(u_r) reveal a tendency to close their cracks. The elements having values $|u_r|>0.95|u_r|max$ are modified to type(a) uncracked elements having from now on a zero circumferential tensile strength. The above procedure is repeated until no further cracking closure occurs. A comparative example is presented in the next chapters.

INTERVENTIONS IN THE CENTRAL DOME OF ST.PANTELEIMON CHURCH OF THESSALONIKI

The St.Panteleimon church has been built during the end of the 13th century A.C. It has been suffered extensive mutilations during its life. The damages have been intensified by the 1978 earthquake sequence. In Fig.1 the main internal cracks existing today in the central dome are shown. A seriously corroded wooden tierod runs across the upper part of the openings. The heavy roof covering accounts for almost 40% of the total load of the dome. The intervention measures in their sequence of application are the following. a. Roof covering removal for the application of the mortar injections and the placing of metal ties. b. Mortar injections and replacement of the tierod by a new inactive wooden one. c. Placing of metal(titanium) ties at the lowest possible level on the external surface of the shell(Fig.2) and prestressing. d. Reconstruction of roof covering.

Analytical procedure

Two comparative analyses for the dimentioning of interventions are presented. In both of them the intervention sequence described above is taken into account. The development of a compressive circumf. stress of 0.3MPa at the base of the shell under total service loading was used as a criterion for the evaluation of the prestressing force. In the first analysis(Case I) no cracking criterion was adopted. In the second one(Case II) it was assumed that the masonry has a circumferential tensile strength $\sigma_{\theta u}$=0.04MPa. In table 1 the material properties of the model are shown.

Table 1. Structural and finite elements-Material properties

Structural element	Element type	$E_z=E_r$ GPa	E_θ GPa	$v_z=v_r$	v_θ	γ KN/m³
Shell of the dome	(a)	1.5	1.5	0.19	0.19	18.0
Wooden tierod	(b)	1.0	10.0	0.20	0.20	6.0
Drum of piers	(c)	1.5	Ø	0.19	Ø	12.4
Meridionally cracked shell	(c)	1.5	Ø	0.19	Ø	18.0
Corroded wooden tierod	(c)	1.0	Ø	0.20	Ø	6.0
Titanium tierod	(d)	Ø	105.0	Ø	0.20	45.1

E: Modulus of elasticity, v: Poisson's ratio, γ: Density

Case I. No cracking consideration. Analysis in four phases:

A_I. An analysis was carried out under self-weight loads (no roof covering loading-inactive wooden tierod). The structural model and the simulation of piers with a cylindrical axisymmetric drum are shown in Fig.2. The distribution of the mean values of the circumf.stresses σ_θ and the tangential normal stresses σ_t are shown in Fig.5a.

B_I. A second analysis was carried out under an arbitrarily chosen prestressing load imposed through the tierods (N_p=20KN). The mean values of σ_θ and σ_t are shown in Fig.4a.

C_I. A third analysis was carried out under the loads of roof reconstruction. Additional finite elements of type(d) were used to represent the titanium tierods (2 bars of 16mm in diameter). The mean values σ_θ and σ_t are shown in Fig.4b. The stresses of the tierods are very small because the tierods have been put at a level of almost zero radial displacement.

D_I. Having in mind that the final circumf.compressive stress in the base of the shell must be equal to 0.3MPa, a combination of the analyses above gives a value of N_I=58.1KN for the prestressing force. In Fig.5b the mean stresses σ_θ and σ_t are shown.

Case II. Cracking consideration. Analysis in four phases:
A_{II}. In this case the analytical procedure is more complicated. It must not only consider the intervention sequence but it must also take into account the historical background and the cracking patterns created progressively. At first, at the initial stage of construction(active wooden tierod), a cracking pattern was considered to be created in meridian levels due to circumferential tensile stresses exceeding the assumed tensile strength of the material $\sigma_{\theta u}$=0.04MPa. The progressive corrosion of the wooden tieriod resulted to further cracking. Consequently from there on the tierod was considered to be inactive. In Fig.3 the progressive cracking pattern and the relevant stresses finally developed under total dead weight are shown. As the first intervention measure is the roof covering removal, the unloading procedure of the program was then activated to define any cracking closure, but no closure resulted. In Fig.6a the mean stresses σ_θ and σ_t are shown representing the state after the roof removal.
B_{II}. The second intervention measure is the mortar injection procedure, after which the material continuity is reestablished and compressive stresses can be developed under future loads. The cracked model resulted from the analyses of stage(A_{II}) is appropriately modified to the relevant model of stage(B_I). Consequently the stresses already shown in Fig.4a represent again the influence of prestressing load.
C_{II}. The roof reconstruction results in the stress distribution shown in Fig.4b.
D_{II}. A combination of the analyses above gives a value of N_{II}=37.0KN using the criterion adopted in the corresponding stage of case I. In Fig.6b the mean stresses σ_θ and σ_t are shown.

CONCLUDING REMARKS

The analysis of domes, including future interventions is a highly nonlinear procedure mainly because of the structural alterations due to progressive cracking, mortar injections and new members placing. A phase by phase analysis, following the corresponding structural alterations and the future intervention sequence, can lead to more reliable results compared with the results of a conventional analysis. In the comparative analysis presented above the differences observed are significant. For example the requered prestressing force of a conventional analysis was found to be 57% greater than that of a phase-by-phase analysis which included cracking consideration. In the second analysis the stresses are distributed more uniformly.

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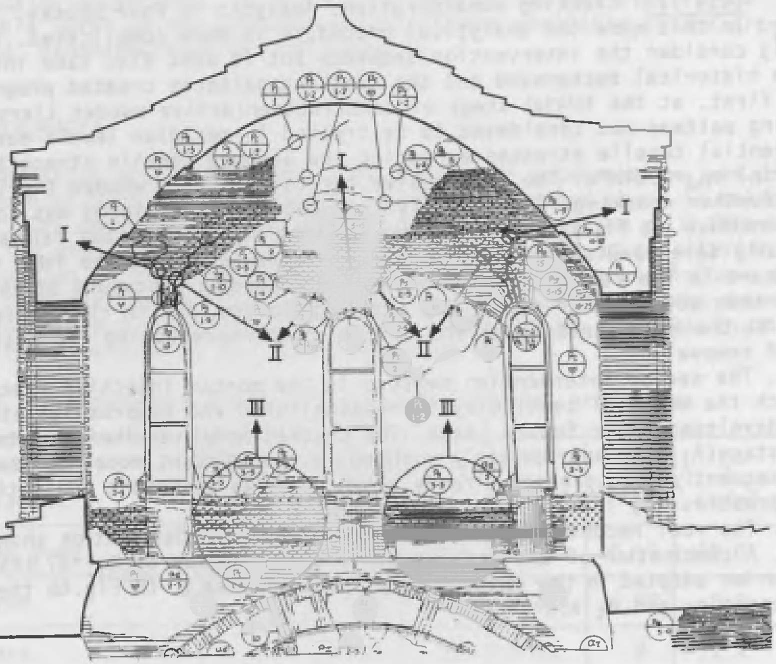

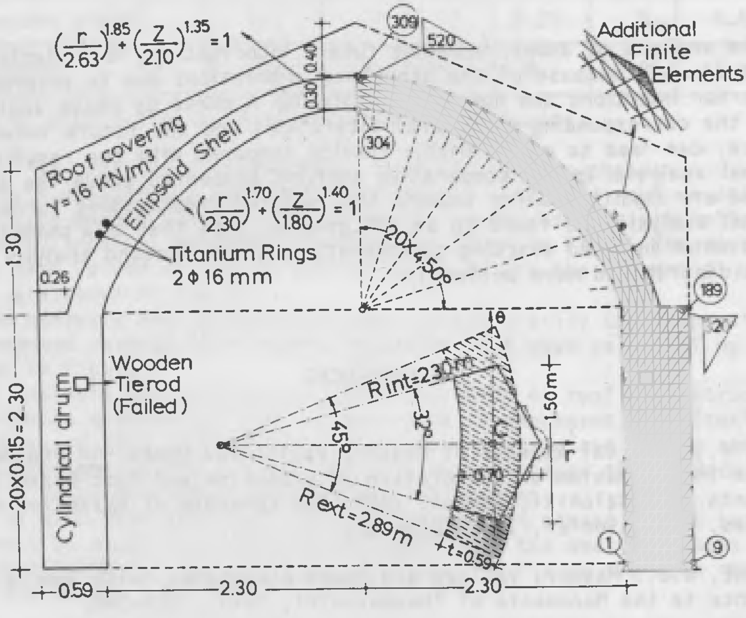

Figure 2. Finite element model of Central Dome

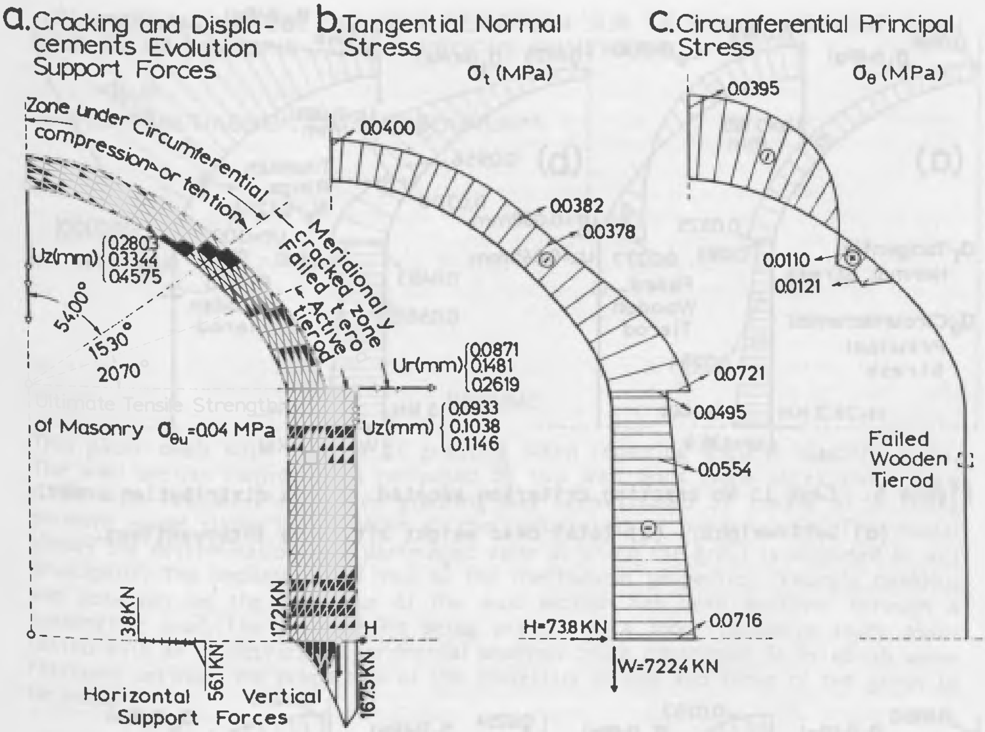

Figure 3. Case II: Cracking criterion: $\sigma_{\theta u}$=0.04 MPa, failed wooden tierod. Stress distribution under total dead weight

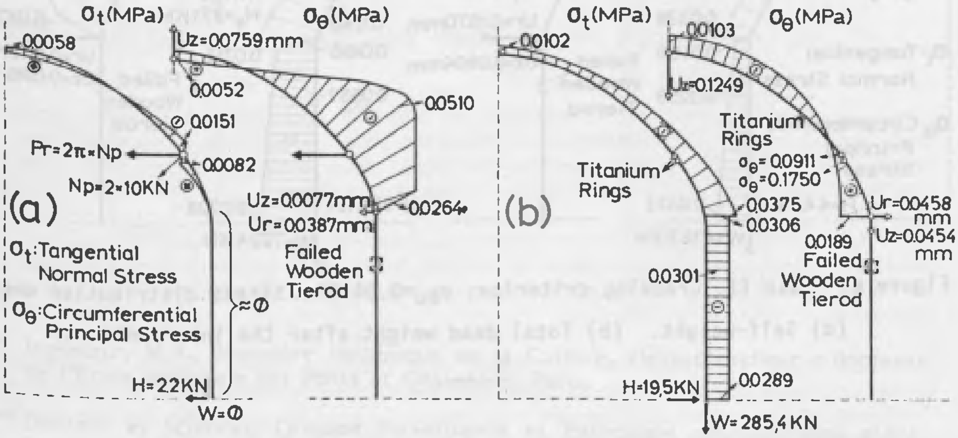

Figure 4. Stress distribution under:

(a) Prestressing force N=20KN.   (b) Roof cover weight

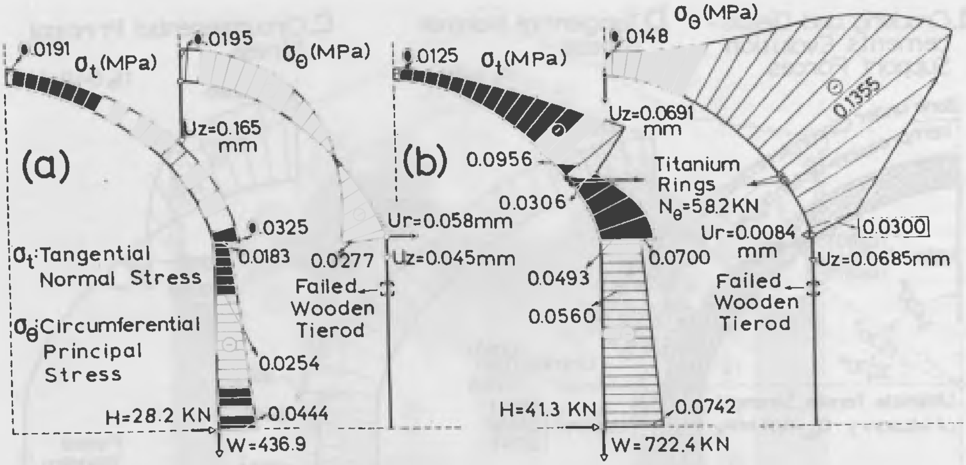

Figure 5. Case I: No cracking criterion adopted. Stress distribution under: (a) Self-weight. (b) Total dead weight after the interventions.

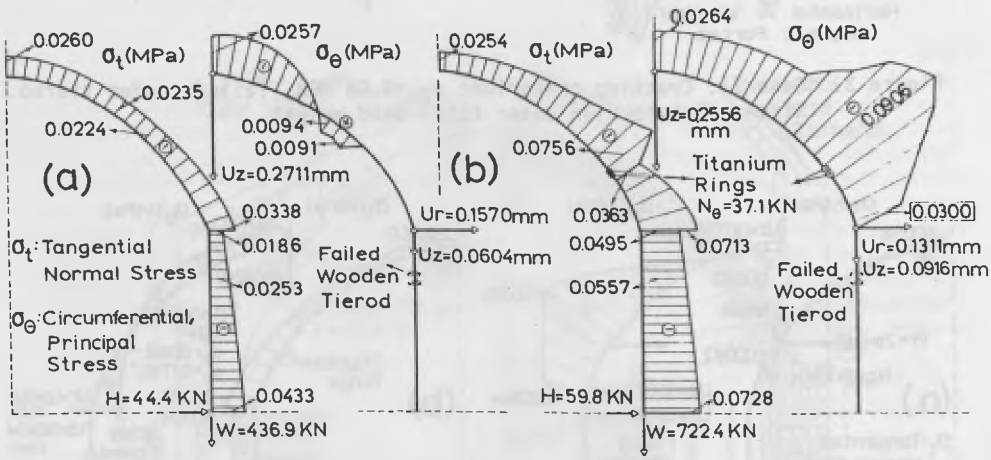

Figure 6. Case II. Cracking criterion: $\sigma_{\theta u}$=0.04 MPa. Stress distribution under: (a) Self-weight. (b) Total dead weight after the interventions.

RÔLE MÉCANIQUE DES COULIS D'INJECTION SUR LE COMPORTEMENT D'UN MUR EN MAÇONNERIE - UNE APPROCHE NUMÉRIQUE.

Androniki MILTIADOU*, Charles ABDUNUR**

RÉSUMÉ

This paper deals with the role of grouting when repairing ancient masonry walls. The wall section considered is composed by two well built stone faces and a core filled with rubble. The role of grouting has been studied by means of a finite element model taking into account all the constituants of the structure. This model allows the determination of a dammaged zone in which the grout is supposed to act principally. The importance of two of the mechanical properties, Young's modulus and cohesion, on the behaviour of the wall section has been analysed through a parametric study.The first results being promising, a more complete study associated with an appropriate experimental analysis could contribute to establish some relations between the properties of the materials in situ and those of the grout to be used.

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1 - INTRODUCTION

Cette communication présente une modélisation numérique permettant d'apprécier le comportement mécanique d'un mur en maçonnerie ainsi que le rôle joué par l'injection d'un coulis de réparation.

Dans ce domaine, aussi étendu que complexe, l'étude s'est limitée à un seul type de construction en maçonnerie choisi sur les critères suivants : configuration particulièrement fréquente, vulnérabilité aux actions extérieures diverses, aptitude à recevoir des injections de coulis. Le modèle physique que nous avons adopté est un mur de section composite comprenant deux parements extérieurs séparés par un remplissage de cailloux et de mortier.

S'il existe de nombreux articles portant sur des constructions récentes de type "mur en briques" [Page, 1978 ; Samarasinghe, Page, Hendry, 1982 ; Sacchi Landriani, Riccioni, 1984 ; Ali, Page, 1987], le comportement des structures en maçonnerie ancienne a été relativement peu étudié. La plupart des propositions théoriques se limitent à la prise en compte de lois de comportement globales imposant une homogénéisation de la structure. Les modèles obtenus sont ainsi souvent inadaptés à une analyse fine du rôle mécanique du coulis. Afin d'étudier ce rôle, nous avons modélisé une section de mur en maçonnerie à l'aide d'éléments finis en tenant compte des propriétés intrinsèques (géométriques et mécaniques) de ses différents matériaux constitutifs.

Dans ces éléments de structures, l'effet des injections de coulis n'est pas uniforme. Il est naturellement plus sensible dans les zones endommagées ou ayant des caractéristiques médiocres (porosité élevée, ...). La présente étude numérique vise d'abord à définir une configuration représentative des zones d'influence privilégiées du coulis, en mettant en évidence une surface endommagée (zones plastiques) caractéristique dans une section de mur. Nous avons ensuite admis que la simulation numérique du rôle mécanique du coulis consiste à affecter à ces zones endommagées de nouvelles propriétés acquises après l'injection.

2 - DÉTERMINATION D'UNE ZONE ENDOMMAGÉE CARACTÉRISTIQUE

Les murs en maçonnerie subissent au cours de leur histoire diverses agressions entraînant des dégradations qui affectent leur capacité de résistance structurelle. Les réparations par injection de coulis sont destinées à restituer, voire améliorer, les caractéristiques initiales de ces structures.

L'effet du coulis est très hétérogène sur l'ensemble de la maçonnerie. Il agit principalement sur le remplisage et le mortier, mais d'une manière privilégiée dans les zones où ces deux constituants sont endommagés sous les diverses sollicitations antérieures.

La **figure 1** montre une section de mur choisie comme support de l'étude. Cette section nous servira tout d'abord à schématiser l'étendue et la nature de l'endommagement sous différentes sollicitations, puis à apprécier l'influence de certains paramètres de l'injection.

L'étude du comportement de cette section a été réalisée à l'aide d'une modélisation par éléments finis. Un maillage fin des zones potentiellement endommagées (remplissage, mortier) a été effectué (**figure 2**).

Une loi de comportement élastoplastique de type Mohr-Coulomb a été retenue pour les parties remplissage et mortier tandis que les pierres ont été considérées élastiques. Le **tableau 1** donne les caractéristiques mécaniques choisies pour les différents composants du mur.

	E	ν	C	φ
Pierre	20 000	0,25	-	-
Mortier	35 00	0,30	0,5	20°
Remplissage	35 00	0,30	0,5	20°

Tableau 1

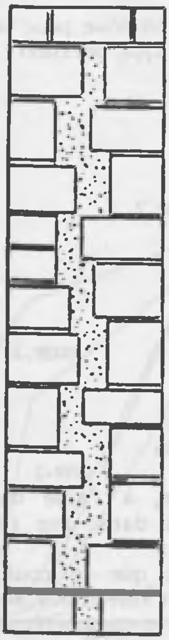

Fig. 1 - Section de mur considérée

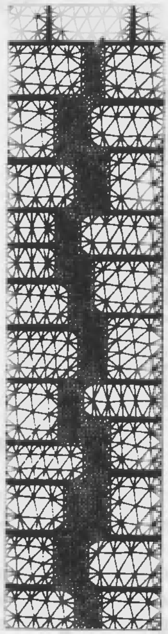

Fig. 2 - Maillage - 2352 noeuds
4424 éléments à 6 noeuds
Programme utilisé : CESAR-LCPC

Ces calculs élastoplastiques nous permettent de mettre en évidence les zones les plus sollicitées de la structure sous un chargement donné. Pour analyser le rôle mécanique du coulis, nous avons ainsi considéré que les zones endommagées correspondaient aux zones plastiques obtenues. Le niveau de chargement a été fixé de manière à ce que cette surface plastique soit égale à la moitié de la surface totale du remplissage et du mortier.

La **figure 3** visualise la distribution de cette zone endommagée sous une pression uniforme appliquée en tête de mur.

Bien que relativement arbitraire, cette caractérisation de la zone endommagée nous semble plus proche de la réalité qu'une distribution homogène.

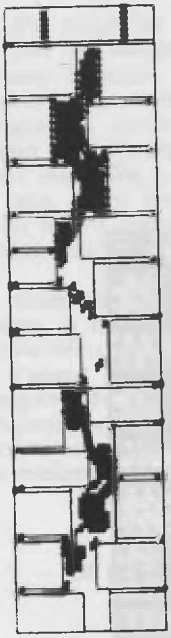

Fig. 3 Zone plastique obtenue pour un chargement de type pression uniforme

3 - RÔLE MÉCANIQUE DU COULIS

L'objet de la présente étude consiste à analyser, à l'aide d'une étude paramétrique, le rôle mécanique joué par le coulis dans une réparation.

En première approximation, nous avons considéré que le coulis agissait uniquement sur les zones endommagées. Pour analyser son rôle, nous avons étudié l'évolution d'un certain nombre de grandeurs caractérisant le comportement du mur en fonction des caractéristiques mécaniques de la zone endommagée. Par ses propriétés intrinsèques, le coulis modifie l'ensemble des caractéristiques mécaniques de la zone injectée et, par conséquent, le comportement global du mur. Dans l'étude préliminaire présentée ici, nous avons analysé de manière privilégiée l'influence des deux paramètres, le module d'Young (E) et la cohésion (C).

Le choix de grandeurs représentatives du comportement du mur est beaucoup plus délicat. Nous savons en effet que la "raideur" et la capacité portante d'un mur sont essentiellement conditionnées par les caractéristiques des deux parements extérieurs. Le coulis agissant de manière préférentielle sur le remplissage n'aura ainsi qu'une influence relative sur ces grandeurs. Il va en revanche contribuer à restituer la cohésion de l'ensemble de la structure en rétablissant la continuité du remplissage et en améliorant l'adhérence entre les divers matériaux aux interfaces.

Dans le type de modélisation que nous avons adopté, l'étendue de la surface plastique peut traduire de manière qualitative le taux d'endommagement du remplissage et donc la cohésion de l'ensemble. Nous avons ainsi choisi d'évaluer le rôle mécanique du coulis à l'aide de ce paramètre surface plastique. La **figure 4** représente l'évolution de cette surface plastique en fonction de la pression (p) appliquée pour différentes valeurs de module d'Young de la zone endommagée. (Sp représente la surface plastique et Sm+r la surface totale couverte par le mortier et le remplissage).

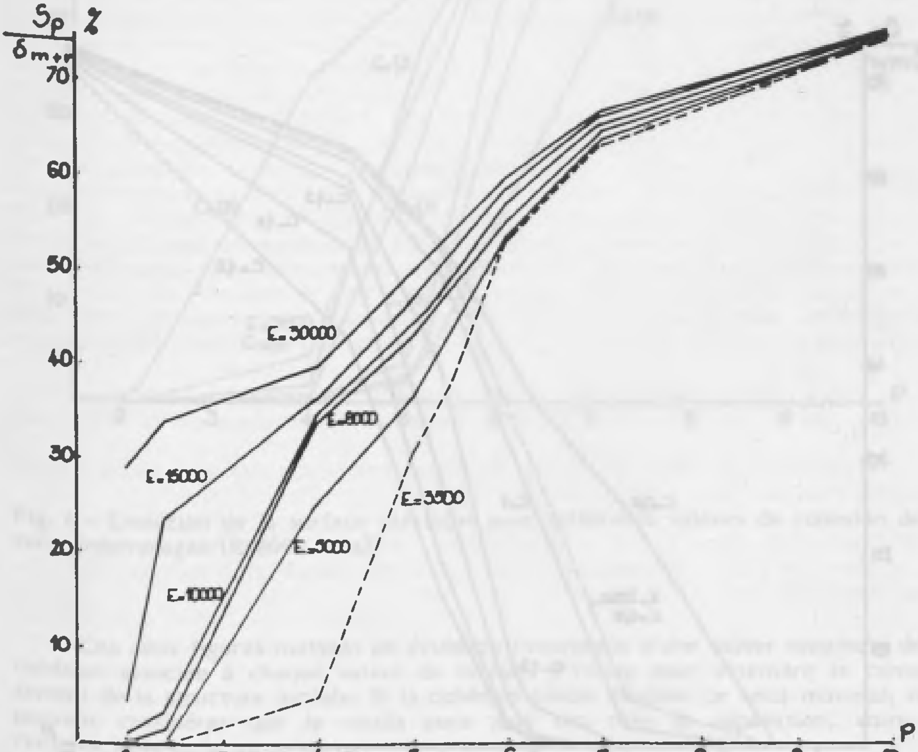

Fig. 4 - Evolution de la surface plastique pour différentes valeurs du module d'Young de la zone endommagée.

Nous constatons que cette surface plastique croît de manière importante avec le module d'Young de la zone injectée. L'emploi de coulis à très fort module n'entraîne donc pas forcément un effet bénéfique sur le comportement de la structure.

Ces résultats doivent toutefois être tempérés par le fait que, d'une manière générale, cette augmentation de module s'accompagne également d'une augmentation de la cohésion.

La **figure** 5 représente l'évolution de la surface plastique pour différentes valeurs de cohésion de la zone injectée. Le module d'Young de cette zone est considéré ici constant et pris égal à 5000 MPa. La courbe caractéristique du remplissage initial (E=3500 MPa, C=0,80 MPa) est reportée à titre de comparaison.

Nous constatons que cette surface plastique décroît de manière significative quand la cohésion de la zone injectée augmente.

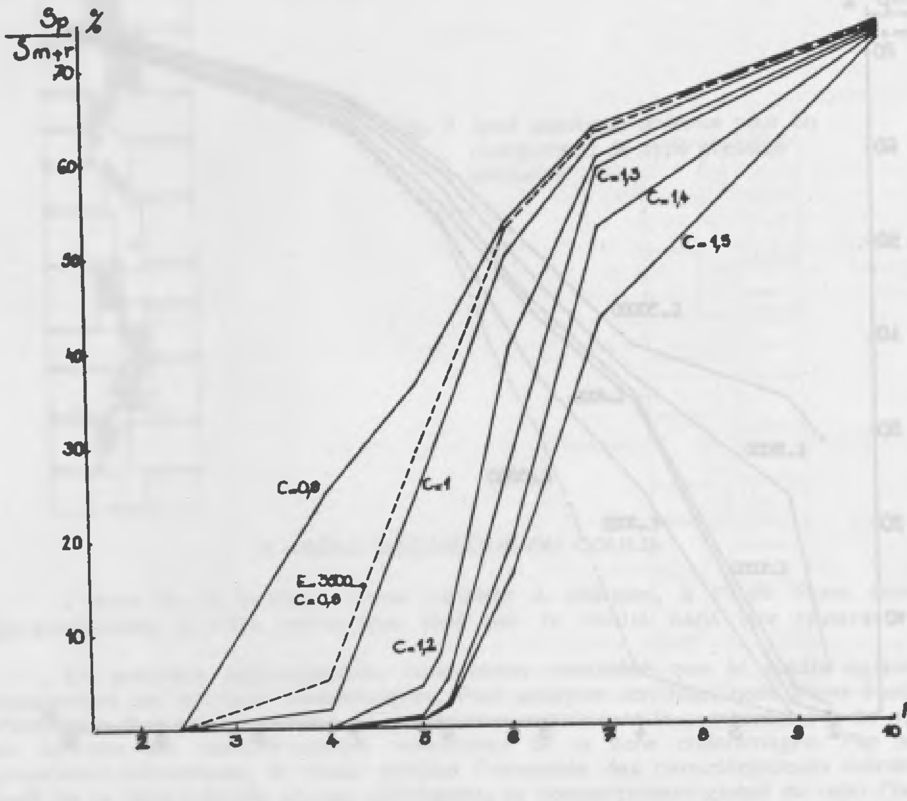

Fig. 5 - Evolution de la surface plastique pour différentes valeurs de cohésion de la zone endommagée (E=5000 MPa).

La **figure 6** montre également le rôle de ce facteur cohésion pour un module d'Young égal à 8000 MPa.

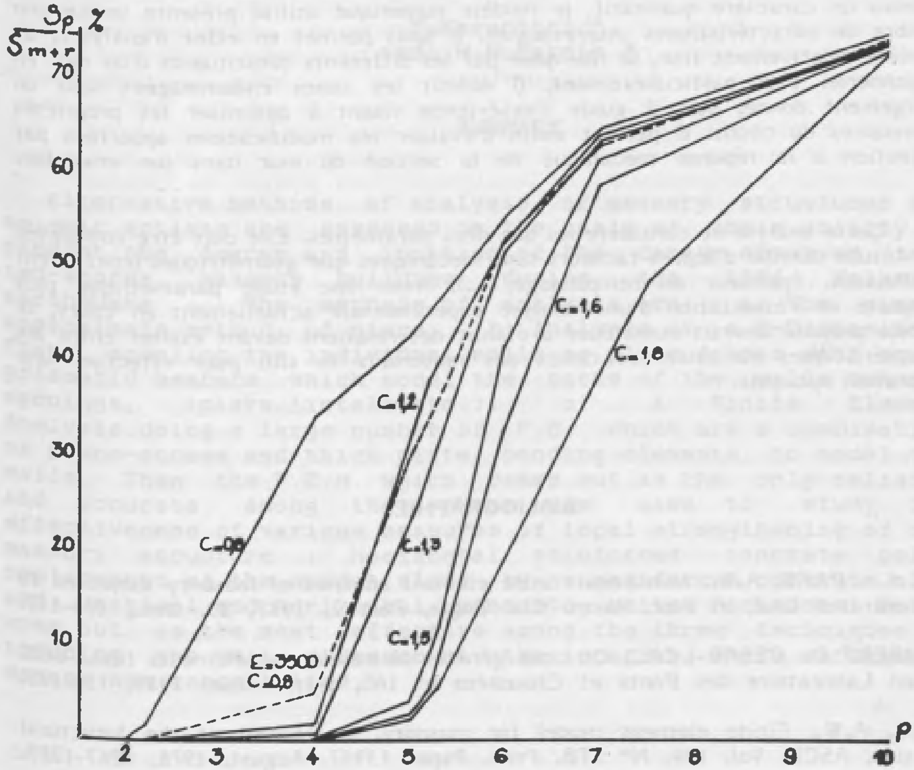

Fig. 6 - Evolution de la surface plastique pour différents valeurs de cohésion de la zone endommagée (E=8000 MPa).

Ces deux figures mettent en évidence l'existence d'une valeur minimale de la cohésion associée à chaque valeur du module d'Young pour atteindre le comportement de la structure initiale. Si la cohésion réelle dépasse ce seuil minimal, nous pouvons considérer que le coulis aura joué son rôle de réparation, voire de renforcement de la structure.

4 - CONCLUSION

Cette étude préliminaire nous a permis de mettre en évidence le rôle mécanique joué par le coulis. Malgré les approximations conférant aux résultats obtenus un caractère qualitatif, le modèle numérique utilisé présente un certain nombre de caractéristiques intéressantes. Il nous permet en effet d'analyser, de manière relativement fine, le rôle joué par les différents constituants d'un mur en maçonnerie. Plus particulièrement, il définit les zones endommagées sous un chargement donné, puis il guide l'expérience visant à optimiser les propriétés nécessaires du coulis. Il permet enfin d'évaluer les modifications apportées par l'injection à la réponse mécanique de la section du mur dans son ensemble.

Cette analyse ne considère ici que deux paramètres. Elle doit être complétée par l'étude du rôle d'autres facteurs tant mécaniques que géométriques (coefficient de Poisson, épaisseur du remplissage, ...). Avec une étude paramétrique plus complète et l'association d'une analyse expérimentale actuellement en cours, le modèle proposé devrait contribuer à établir des relations devant exister entre les caractéristiques du coulis et celles des matériaux in situ pour effectuer une réparation optimale.

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ASSESSMENT OF ANALYSIS METHODS
AND OF STRENGTHENING TECHNIQUES,
FOR EARTHQUAKE RESISTANT MASONRY STRUCTURES

F.Karantoni [1],
and M.N.Fardis [2]

SUMMARY

Alternative methods of analysis of masonry structures for seismic actions are assessed on the basis of their ability to predict the degree and location of the damage observed in a two-storey masonry building during the 1986 Kalamata earthquake . The methods of analysis are: a) The simple approximate method of piers. b) Analysis of a 3-Dimensional Frame, modeling the individual walls as plane frames with squat prismatic members which model the parts of the walls between openings, (piers,lintels,etc.). c) A Finite Element Analysis,using a large number of F.E. which are a combination of plane-stress and thick plate bending elements, to model the walls. Then the F.E.M. which comes out as the only reliable and accurate among the three ,is used to study the effectiveness of various measures of local strengthening of the masonry structure : Horizontal reinforced concrete belts replacement of the wooden floor by a reinforced concrete slab and vertical and horizontal concrete belts. Horizontal belts come out as the most effective among the three techniques in reducing the wall stresses at the location where earthquake damage was concentrated.

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ASSESSMENT OF METHODS OF ANALYSIS FOR EARTHQUAKE LOADING

The strong earthquake that struck Kalamata in Sept. 1986 caused heavy damage to the approximately 2000 stone-masonry buildings in the city. These buildings typically had two storeys and a basement, thick exterior walls of relatively poor quality and wooden floors and roof. Most of these buildings collapsed, but in several others the damage was moderate enough to allow studying their behavior during the earthquake. One of these buildings, situated close to a point where the ground motion was recorded during the earthquake, was subjected to alternative methods of analysis, in order to assess the ability of these methods to predict the distribution and the magnitude of the observed damage. The building and its damage are shown in Fig. 1. The central part of the 2nd floor of the South wall (T3) collapsed, whereas most of the piers and the lintels exhibited serious cracking, esp. at the 2nd floor.

The analysis was done for static horizontal forces resulting from a uniformly distributed peak response acceleration equal to 0,42 g. This acceleration value was obtained from the elastic Response Spectra of the horizontal components of ground motion, for damping equal to 20% of critical. These horizontal loads were applied separately in the NS and EW directions, and their effects were supperimposed to those of the gravity loads.

Three methods of analysis where applied: a) The Finite Element Method (FEM), using a large number of elements (Fig. 2a). The elements used for the walls were a combination of thick plate elements, and plane-stress ones, whereas those for the floors were truss elements (for the floor beams). b) A 3-Dimensional Frame analysis ("3-D Frame"), using one 3-D beam element for each pier or lintel (Fig. 2b), accounting for shear deformations of the elements and considering the "joint" regions at the intersections of piers and lintels as rigid. c) "The Method of Piers" a commonly used approximate hand calculation method, in which the total horizontal force is distributed to the walls parallel to its direction, in proportion to the sum of the lateral stiffness of each wall's piers (with the piers considered as vertical cantilevers, with some degree of fixity to the lintels), and then each wall's horizontal force is distributed to its piers, in proportion to their horizontal cross-sectional area. From the pier shears computed in this way, bending moments at the top and bottom cross section of each pier are computed, taking into account the different degree of end fixity of the pier to the lintels.

Results of the 3 analyses are compared in Fig. 3a-d, in terms of the magnitude and the direction of the principal tensile stress induced by earthquake loading parallel to the wall, at one-quarter points of the end-sections of the piers of the exterior walls. These results, and others not included herein, show clearly that the FEM predictions are in almost perfect agreement with the location and orientation of the observed cracks. On the contrary the other two approaches give almost equally poor predictions. The success of the FEM can be attributed to the fact that it accounts for the bending of the walls due to horizontal loads normal to their plane, which induces large bending stresses in these walls as well as large tensile and bending stresses in the transverse walls (the ones parallel to the load direction) near the corner, due to the transfer of horizontal forces from the first wall to the second. The other two methods do not account properly for this effect, which seems to be a primary cause of the observed damage.

ASSESSMENT OF STRENGTHENING TECHNIQUES

Commonly proposed measures to increase the earthquake resistance of buildings of the type studied herein include the general or local jacketing of the walls, the introduction of continuous horizontal reinforced concrete belts ("chainage") at the level of the lintels, the construction of vertical reinforced concrete belts, in addition to the horizontal, especially at the corners of the building, the replacement of the wooden floor by a reinforced concrete slab, etc. The first of these techniques is certainly very effective, but also very costly. The effectiveness of the other three less costly, techniques is studied herein by computing the reduction in wall principal tensile stresses effected by the application of each of these techniques, to the building studied above. As the FEM turned out as the only reliable method for the prediction of earthquake induced damage, it is used also next for the computation of these wall principal stresses. Since the analysis is linear-elastic, horizontal belts (at the lintels, with depth equal to 0,3m, and width equal to that of the wall) or vertical ones (at the 4 corners, with horizontal dimensions equal to 0,5m), are introduced in the model simply by assigning an appropriately large value to the Elastic Modulus (27 MPa, vs. 2MPa for the rest of the wall) to the corresponding elements of the F.E. mesh of Fig. 2a. The reinforced concrete slabs are introduced by replacing the truss elements used to model the wood floor beams by a diaphragm rigid in its own plane.

The maximum over all load combinations of the principal tensile stresses at the end sections of the piers and lintels of each exterior wall at the locations where damage was observed are depicted in Fig. 4a-d, for the 3 strengthening techniques and for the unstrengthened structure. All 3 techniques reduce significantly these principal tensile stresses, Overall, the introduction of horizontal belts comes out as the most effective among the three strengthening techniques, as maximum principal stresses in the wall are reduced by 50%, on the average. Replacement of the wood floors by reinforced concrete slabs is almost as effective as the introduction of horizontal belts, but only as far as the 1st storey and the basement are concerned. The top storey, which is the most vulnerable among the three, is helped very little by the introduction of floor slabs at the levels below. So, it seems that this strengthening measure should be supplemented with a horizontal belt at the top of the 2nd storey wall. Finally, the introduction of vertical belts at the 4 corners does not reduce wall stresses over those obtained for

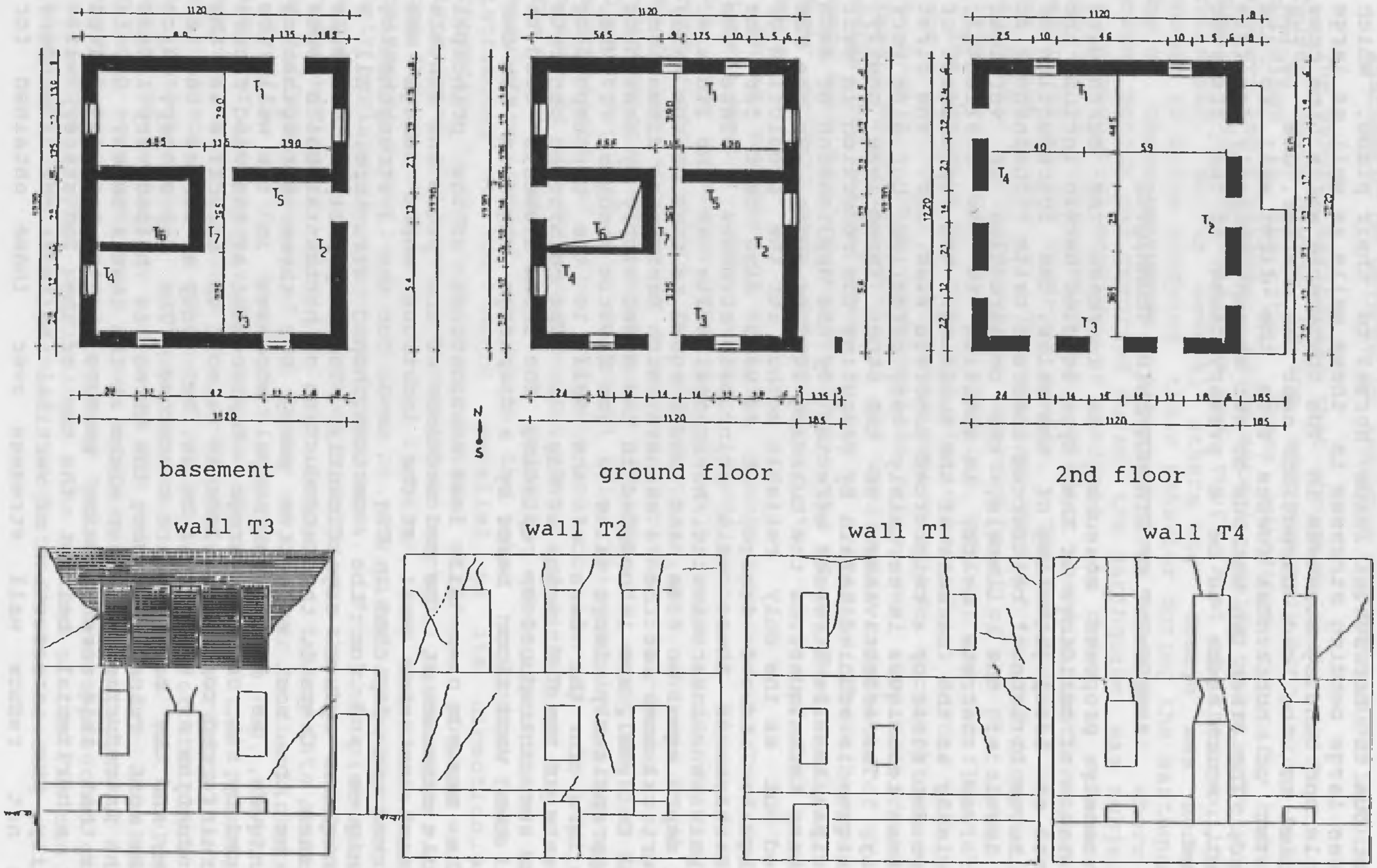
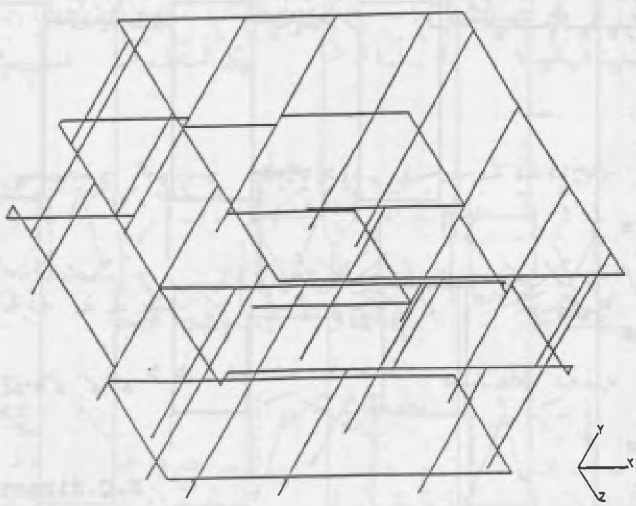


Fig.1 Geometry and damage of the building.

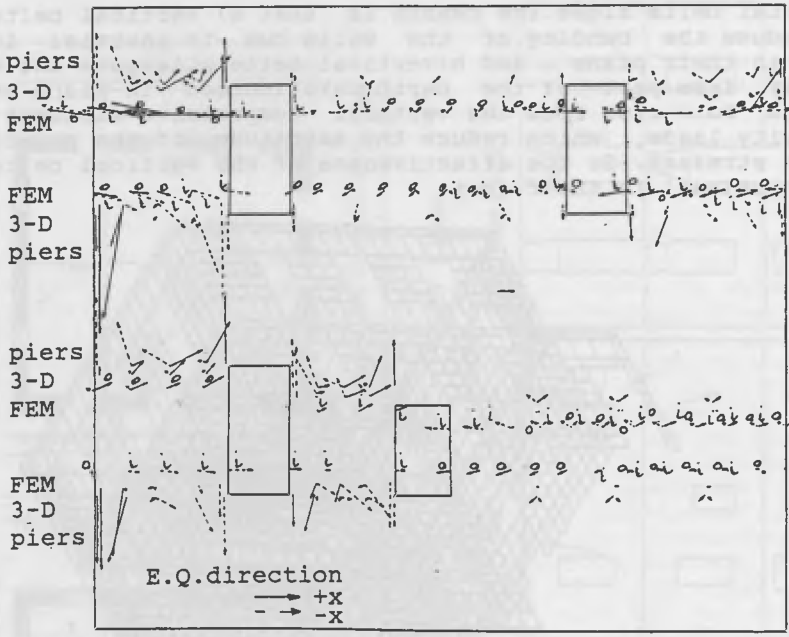
horizontal belts alone.The reason is that a) vertical belts do not reduce the bending of the walls due to inertial loads normal to their plane ; and b)vertical belts alleviate the wall not only from part of the earthquake induced in-plane shear stresses, but also from the vertical compressive stresses due to gravity loads, which reduce the magnitude of the principal tensile stresses. So the effectiveness of the vertical belts is not propotional to their cost.

(a)

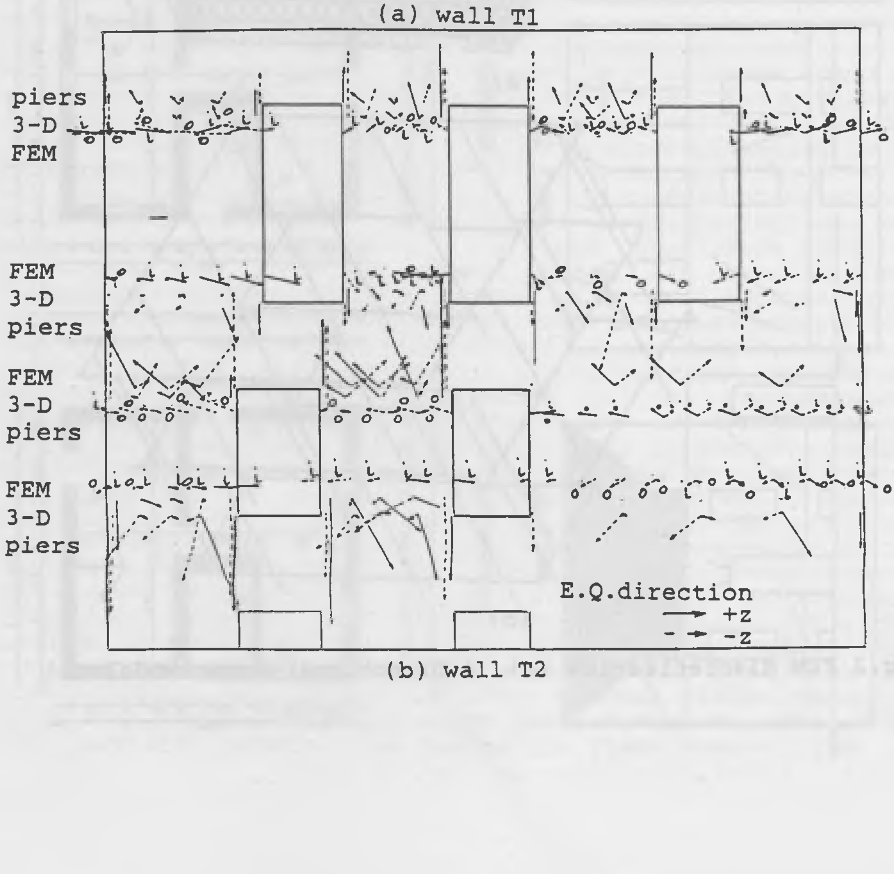


(b)

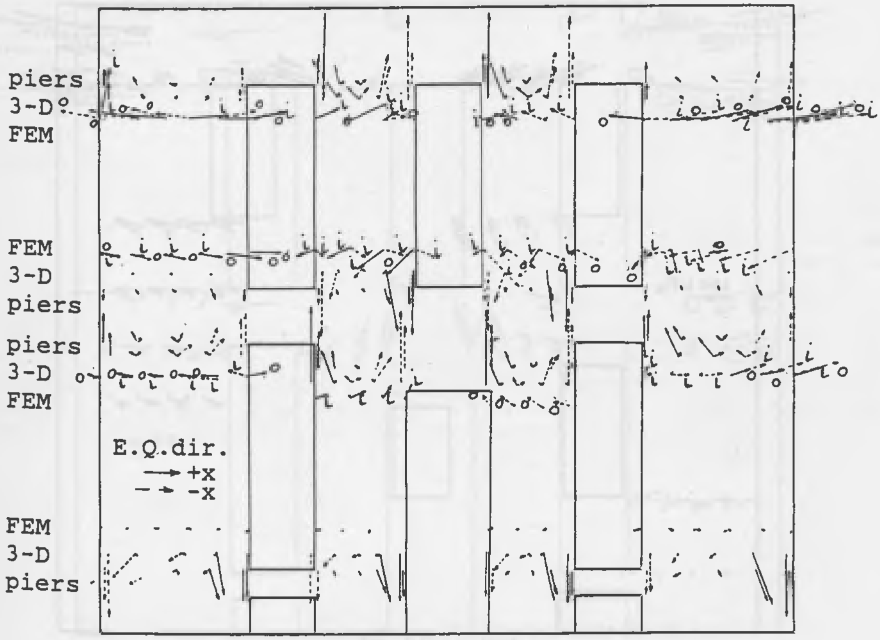
Fig.2 FEM discretization (a), 3-Dimentional Frame model (b)



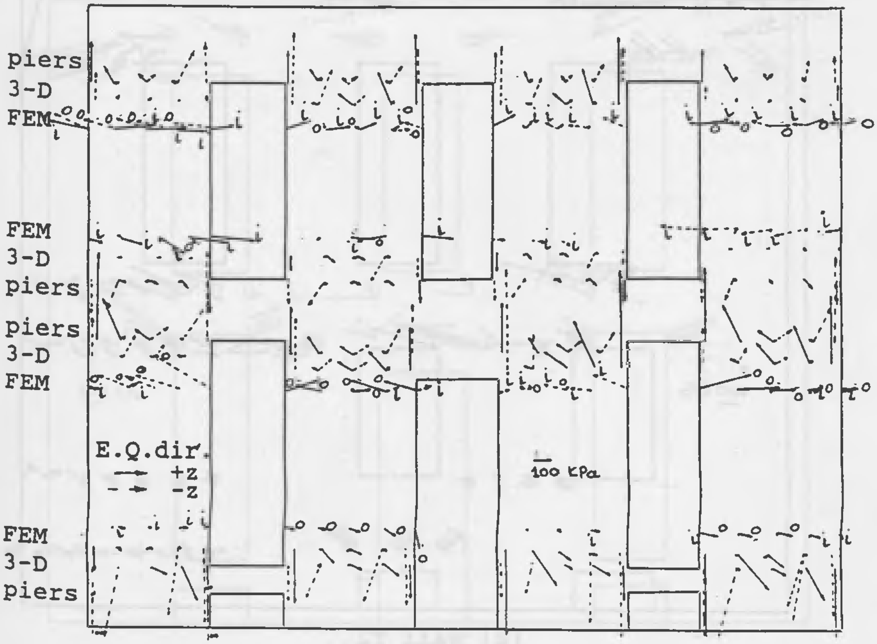
(a) wall T1



(b) wall T2

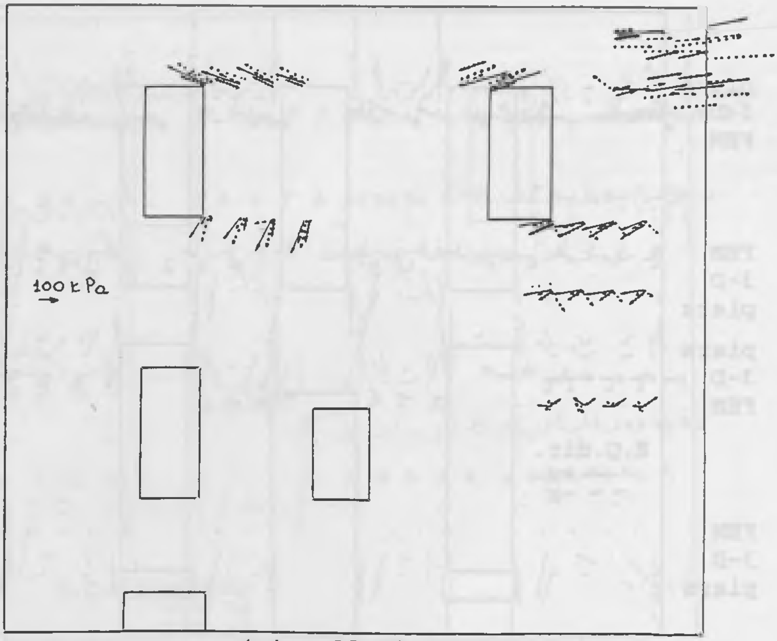


(c) wall T3

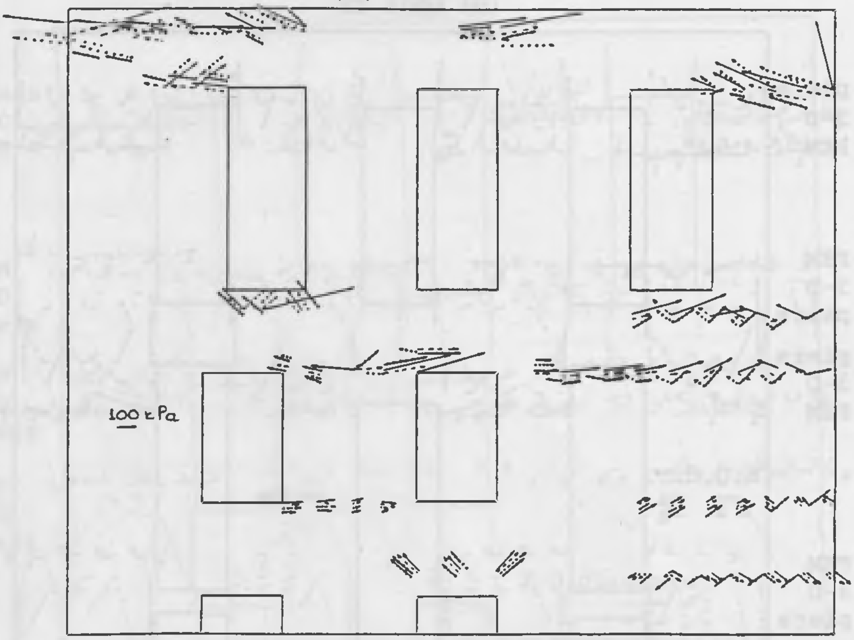


(d) wall T4

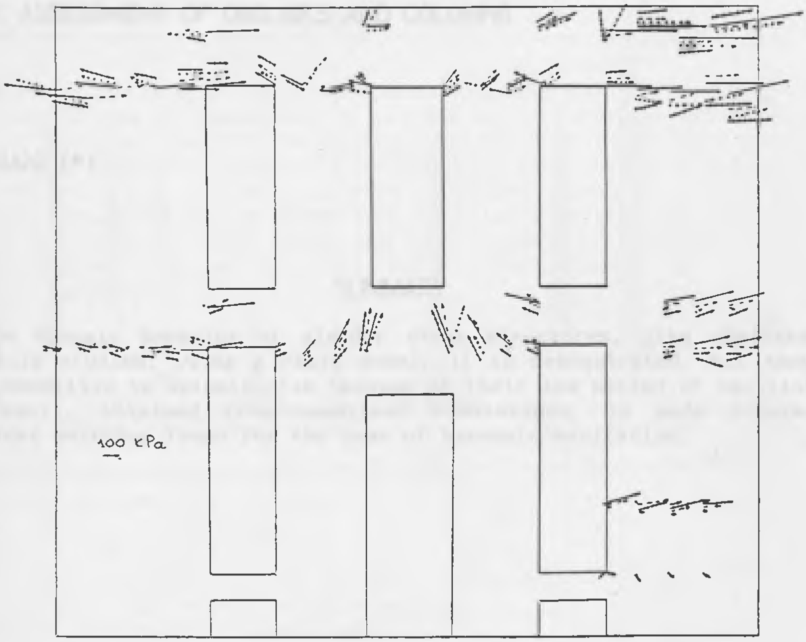
Fig.3 Maximum principal stress directions and magnitudes, according to the 3 analyses, at top and bottom of the piers. (i=inner surface, 0=outer surface of wall)



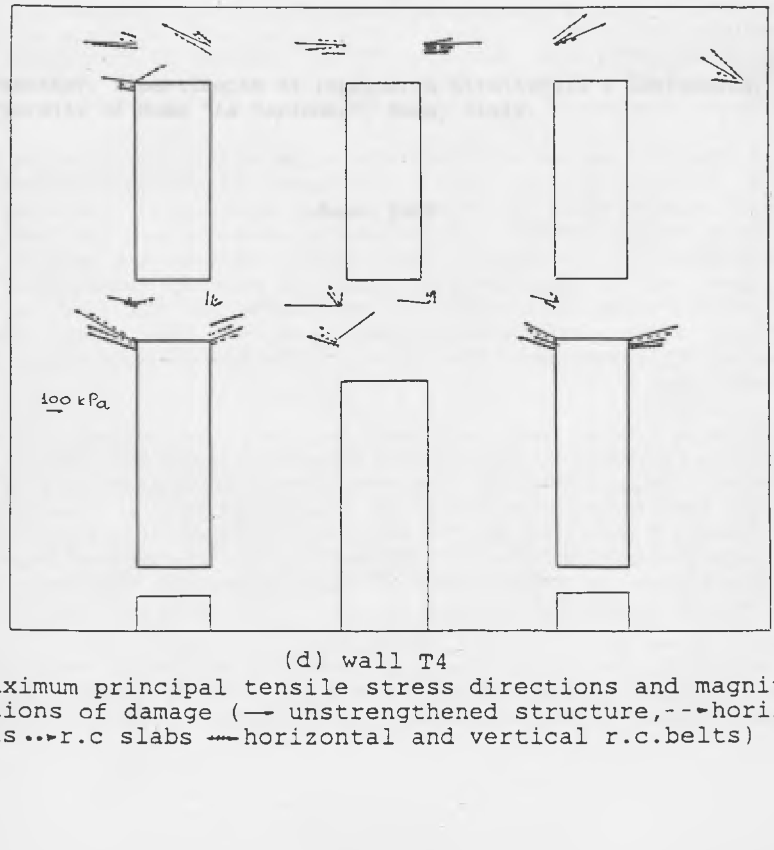
(a) wall T1



(b) wall T2



(c) wall 3



(d) wall T4

Fig.4 Maximum principal tensile stress directions and magnitudes, at locations of damage (—▸ unstrengthened structure,- -▸horizontal r.c.belts ··▸r.c slabs ▸▸horizontal and vertical r.c.belts)

SEISMIC ASSESSMENT OF OBELISKS AND COLUMNS

R. MASIANI (*)

SUMMARY

The dynamic behavior of slender stone structures, like obelisks and columns is studied. Using a rigid model, it is demonstrated that they are little sensitive to seismic risk because of their low period of oscillation. This result, obtained from numerical simulations, is made clearer by analytical solution found for the case of harmonic excitation.

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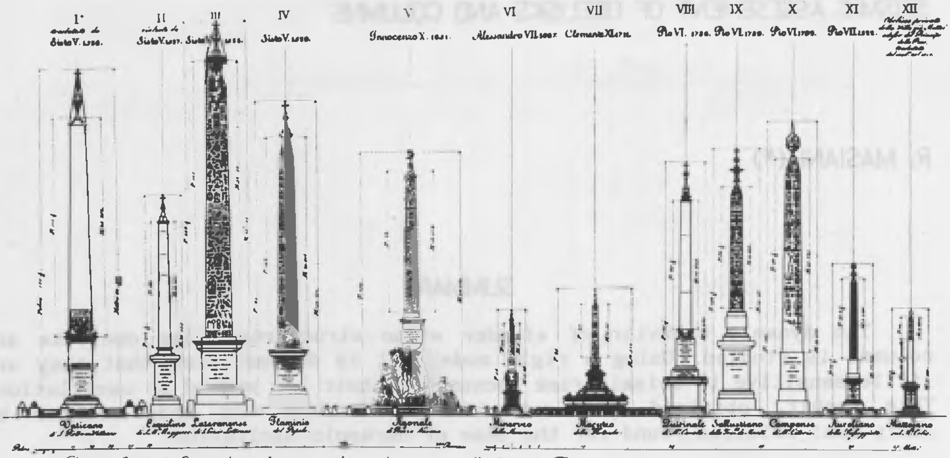
June, 1989



Dodici Obelischi Egizj, che si osservano rialzati ad ornamento della Città di Roma, posti secondo l'ordine della loro rielevazione.

INTRODUCTION

Columns and obelisks are always important historical monuments or parts of them, and so the consequence to study their seismic behavior is obvious. But they are also a special case of structures built of stone masonry with one dimension prevailing over the others; this gives the opportunity to obtain interesting information about other mechanisms like, for example, the out of plane behavior of brick and block walls.

The dynamic response of structures built with blocks standing one upon the other is not a trivial task to investigate though the simplicity of the mechanical models adopted. Because of the monolateral character of the constraints between blocks, the forces-displacements relation are strongly nonlinear (Fig. 1). The usual random vibration methods are hard to apply in these cases, and in this field we are still working; today only the time domain step-by-step integration is applicable to the case of seismic excitation, while for harmonic excitation we have found same very interesting analytical solutions such to give useful indications also for the gèneral case.

Two models have been developed to study the dynamic behavior of columns. The first one is called *rigid* because it makes only use of the laws of the dynamic of rigid bodies .The second one, called *elastic*, accounts for an initial elasticity of mechanical or geometrical nature. Demonstrated the effectiveness of the first one, we shall describe some of its properties and then we shall study the seismic assessment of the obelisks of Rome modeled like rigid blocks. In a next paper we shall generalize this considerations to the case of masonry buildings.

THE RIGID MODEL

This model was adopted until the last century to define the relationship between overturning of tombstones or columns and seismic intensity. Housner (1963) was the first one to take into account the dynamic response analyzing the damages of suspended reservoirs in the Chile earthquakes of 1960. Aslam, Godden and Scalise (1980) made use of a computer program to integrate step by step the motion equations in the case of seismic excitation. They carried out also experimental tests which are well reproduced by their numerical procedure. Muto (1981) and Giannini (1984) have examined and solved the multi-blocks problem.

Formulation

The rigid model is based upon three assumptions:

1) the bodies are indeformable;
2) bearing surfaces are not convex;
3) the friction is sufficient to keep from sliding.

The only allowable motion is the rotation around the edges of bearing surfaces. In the case of plane motion the dynamic equilibrium equation is:

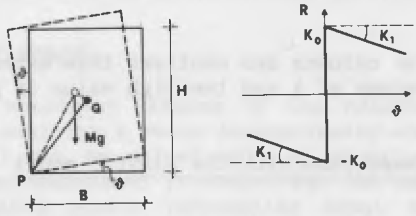


Fig. 1

$$\mathcal{J}_P/M\cdot\ddot{\vartheta} + g\cdot\overline{PG}\cdot\cos(\alpha+\vartheta) = \overline{PG}\cdot\sin(\alpha+\vartheta)\cdot a_t(t) \qquad (1)$$

where $\mathcal{J}_P$ is the polar moment of inertia around P, $\alpha=\arctan(H/B)$ and the other parameters are shown in Fig. 1. Taking into account the alternation of the centers of rotation, the linearized form of (1) is:

$$\ddot{\vartheta} + K_o\cdot\mathrm{sign}(\vartheta) - K_1\cdot\vartheta = f(t) \qquad (2)$$

where, for parallelepiped block:

$$\lambda = \frac{H}{B}$$

$$K_0 = \frac{2\ g}{3\cdot B\cdot(1+\lambda^2)}$$

$$K_1 = \lambda\cdot K_0$$

$$f(t) = K_1 a_t(t)\ /\ g$$

The impacts between blocks dissipate energy; according to the anelastic shock theory, the ratio between the velocity after and before the impact, called restitution coefficient ε, is less than one. A lower bound for ε can be evaluated from the balance of angular momentum (Giannini, 1984); making the hypothesis that the impulsive shock forces shall be applied to the edge, ε depends only on geometrical proportions:

$$\varepsilon = \frac{\lambda^2 - 0.5}{\lambda^2 + 1.0} \qquad (3)$$

Even though real values for ε depend on several parameters, like the stone properties and the shape of bearing surface, experimental tests carried out with concrete blocks of various dimensions (Aslam, 1980), supply values not very far (~2%) from theoretical ones. In any case those kind of structures have a very low dumping; the more they are slender (large value of λ), the less is the dumping. For want of tests we have assumed for ε the expression (3).

About the effectiveness of the model

Obelisks and columns well meet to the assumptions of the above paragraph. In particular the stone blocks are generally well rested one on the other without binder. There was great care taken in the planarity of bearing surfaces, and when there are the astragals, as in the case of the Vatican obelisk in Rome or Teodosio's in Costantinople, the positions of the rotation axes and that of the application points of the impulsive forces are well defined. Moreover, it is easy to calculate the minimum value of the friction coefficient μ in order to avoid sliding in static conditions:

$$\mu \geq \frac{1}{\lambda}$$

For columns and obelisks this expression is always true because of the large values of λ and the high value of μ for the stone surfaces.

Comparison with the *elastic* model

The *elastic* model exhibits an elastic behavior for small rotations accounting for the deformability of stone and binder between blocks or, less obviously, the convexity of bearing surfaces. If these are convex, the axis of rotation moves from an edge to another with continuity; when it reaches the edge, the behavior becomes like that of the rigid model. A *pseudo elastic* model can be formulated (Giannini-Giuffrè-Masiani, 1986): it exhibits an elastic initial behavior the stiffness of which depends not on material but on geometrical properties like the curvature of the surface. This new model, however, is formally identical to the *elastic* one, except that this *geometrical elasticity* depends on the dimensional scale.

It is possible to make an interesting observation about *elastic* (or *pseudo elastic*) model : for the usual values of geometrical proportions, the maximum rotation response is quite independent from initial elastic stiffness. In Fig. 2 response spectra are plotted for the *elastic* model in terms of maximum rotation, computed from mean values over twenty synthetic accelerograms with the characteristics of italian strong motions (Masiani, 1987). The slender ratio (L in the figure) has an influence on the response level, but more relevant is the geometric dimension: higher values of displacements are for small and slender blocks. It is interesting to note that for this type of structures

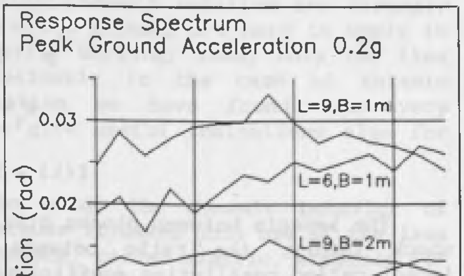


Fig. 2

the response is influenced very little by the elastic stiffness and not very far from the response of the rigid model (the points at zero period on the diagram). For this reason, and also because of the unreliability to evaluate the elastic properties of the stone structures, it seems better to adopt the *rigid* model.

These consideration holds also in other situations, like the out of plane post elastic response of brick or stone masonry structures. The collapse in this mechanisms requires large movements while the elastic stiffness (in any way it is evaluated) is so high that the range of elastic behavior is very small and does not influence the response in terms of displacements; the concept of ductility (the maximum versus the limit elastic displacement) seems meaningless for all cases of non reinforced masonry structures; it is better to refer the response to a limit value of displacement (Giuffrè, 1988).

DYNAMIC RESPONSE

The block oscillator is strongly nonlinear because of the relation between the restoring force and ϑ, which exhibits a sharp discontinuity when $\vartheta=0$. For seismic excitation equation (2) can be solved only by numerical methods. At present, we have formulated an analytical procedure for the case of harmonic dragging motion that can give useful information about the dynamic behavior of masonry structures.

Numerical procedure

A step by step numerical integration using Newmark's method has been implemented (Giannini, 1984) to analyze any kind of excitation, and in particular seismic accelerograms. Dissipative effects are taken into account during numerical integration by the restitution coefficient. This methodology has two disadvantages; the first is the necessity to execute a lot of analyses (in case of non deterministic excitation) with small time steps (one to five thousandth of a second). The second is the difficulty to derive qualitative information about the behavior of these structures because of the strong sensitivity to the shape of the excitation.

Analytical procedure

This innovative way (Giannini-Masiani, 1988) gives an explicit formulation for the response parameters, but at the present state of our studies it is limited to the case of one block subjected to sinusoidal excitation. The block is modeled like a single degree of freedom oscillator, with a viscous dumping equivalent to shock energy dissipation. The equation (2) becomes:

$$\ddot{\vartheta} + d\cdot\dot{\vartheta} + K_0\cdot\mathrm{sgn}(\vartheta) - K_1\cdot\vartheta = F\cdot\sin(\omega t)$$

where $\omega/2\pi$ is the frequency of the dragging motion. It can be solved by the harmonic balance method, choosing the subspace of response functions in the trigonometric functions space.

For each component one balance equation between excitation end response holds. For different initial conditions, the response motion can be harmonic or subharmonic. In Fig. 3 there are the amplification curves, in terms of maximum rotation normalized by limit rotation, for the harmonic response and different values of peak ground acceleration. The curves depend upon the reduced frequency F_r defined as:

$$F_r = \frac{\omega}{2\pi}\sqrt{B}$$

For harmonic excitation the dimensional scale factor for the time is equal to the geometric one raised to 1/2.

This outline, referring to (Giannini-Masiani, 1988) for a more exhaustive treatment, is sufficient for some useful observations. Columns, obelisks and other similar structures don't have periods of resonance: the response function is monotonic. The higher amplification in terms of displacements is for long period: in the range of the frequencies of the seismic action (2÷10 Hz) and for usual dimensions the response is very far from the maximum values. Large amplitude vibrations may occur only at low frequencies, and so, when the ground motion reaches the peak, the column oscillates with a period in correspondence of which the Fourier Spectrum ordinates of ground motion are not significant. For the more, larger structures respond less compared to smaller ones.

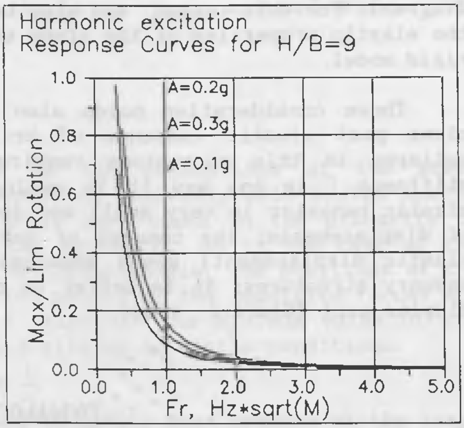


Fig. 3

SEISMIC ASSESSMENT : THE OBELISKS OF ROME

As an example of seismic assessment we have studied six of the twelve obelisks carried in Rome from Egypt during the Classical Age (see Tab. I). The Vatican obelisk was standing on its original position (inside the circus of *Gai et Neronis*) until 1586 when Sisto V moved it to the center of St. Peter's Square; all the other obelisks had fallen down, but already Lanciani (1918) asked himself if this occurred "*by nature phenomenon or by human wickedness*".

In Figure 4 the results obtained by numerical integrations using twenty synthetic accelerograms (Masiani, 1986) are plotted. The diagrams report the probability of not exceeding the rotation normalized by the limit rotation in static conditions (arctg 1/λ). Each curve is for different peak ground accelerations.

	H (m)	B (m)	H/B
Lateranense	32.18	2.97	10.8
Vaticano	25.37	2.70	9.4
Flaminio	23.91	2.40	10.0
Campese	21.79	2.26	9.6
Agonale	16.54	1.79	9.2
Esquilino	14.75	1.34	11.0
Quirinale	14.64	1.34	10.9
Sallustiano	13.91	1.52	9.1
Aureliano	9.25	0.70	13.3
Macuteo	6.34	0.78	8.1
Minerveo	5.47	0.67	8.2
Mattejano	2.68	1.03	2.6

Tab. I

Until the acceleration is less than the value 1/λ·g there is no motion.

It is quite evident that the overturning is not a likely event, especially for the larger monuments. To have 50% of probability to exceed the half of limit displacement the peak ground acceleration must be ≃ 8 m/sec^2 for the Vatican and 2÷3 m/sec^2 for the *Minerveo* obelisk.

CONCLUSION

The effectiveness of the *rigid* model to study the dynamic response of obelisks and columns (and also other slender no-tension material structures) has been proved. Analytical solution holds for the case of harmonic excitation and gives information to have a better insight into the seismic problem. Large amplitude vibrations are always at low frequencies, lower than the predominant ones in ground motion accelerograms: These particular structures are not very sensitive to seismic action. This may outline some criteria for the repair and strengthening against seismic actions. For example, it does not seem a good idea to increase the stiffness with pre-strained steel bars, because often it is an unnecessary intervention that increase remarkably the stresses in the stone. In many cases it is sufficient, and certainly simpler, to take care of the bearing surfaces eliminating the lack of material to restore the monument to its original form.

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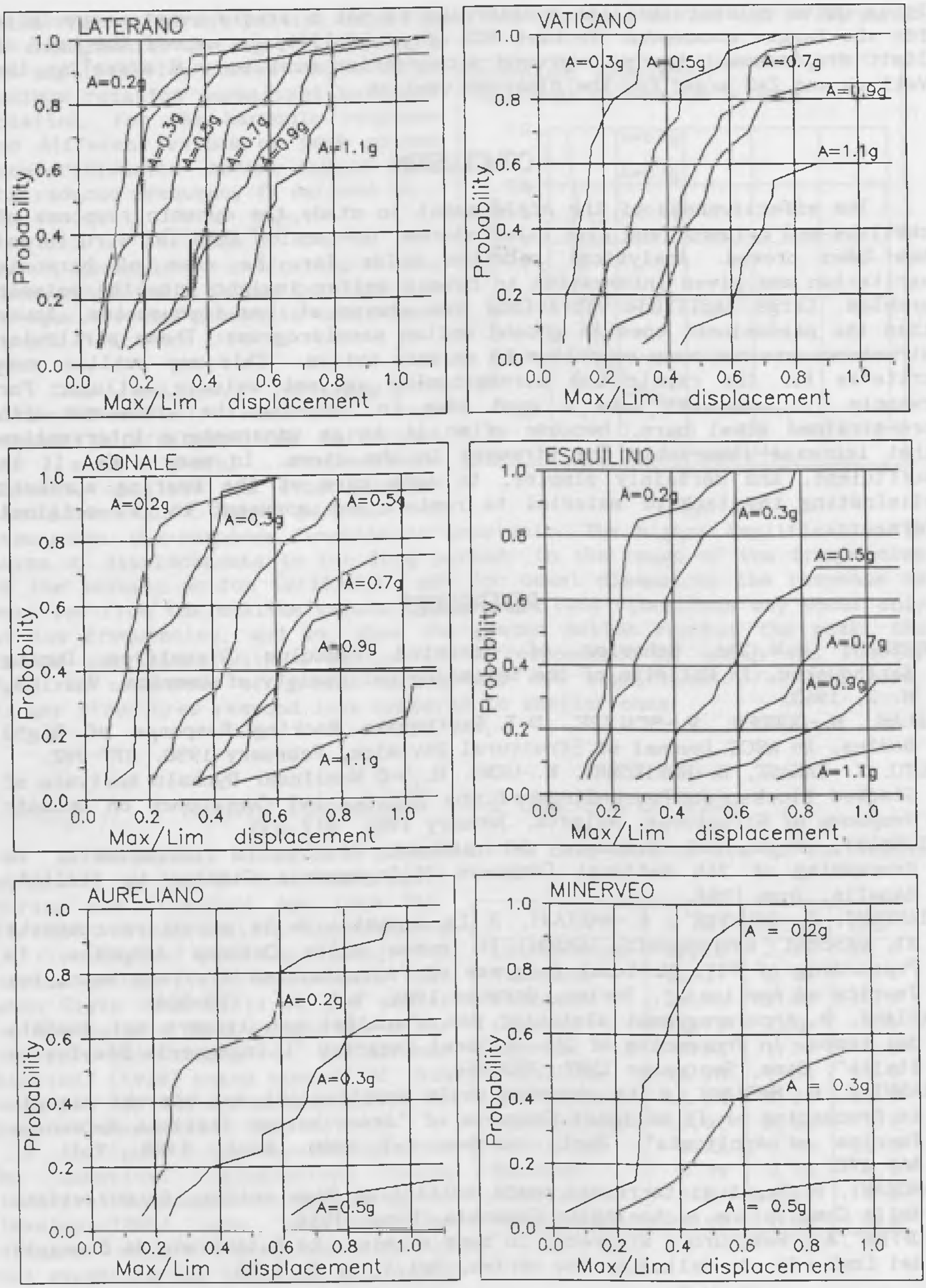
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Probability
Max/Lim displacement

Fig. 4

COMPUTER ANALYSIS OF MASONRY DOMES FOLLOWING CASTIGLIANO'S SUGGESTION. AN EXAMPLE: THE DOME OF S. PIETRO IN VATICANO.

BAGGIO C.*

SUMMARY

A computer based procedure developed to analyze masonry arches on the basis of an iterative method proposed by Castigliano was extended to deal with axisymmetric space structures like classical masonry domes. In the first part of this paper the 'theory' is briefly outlined, and the criteria of the analysis discussed in detail. The second part deals with a complex and intriguing example: St. Peter's Dome and its well known structural problems. Some questions about the breaking of the original iron chains, the effectiveness of the eighteenth century repairs together with more general aspects are tentatively investigated by the developed computer program and some conclusions are drawn.

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INTRODUCTION

In 1879 Castigliano defined '...*imparfaitement elastique*...' bodies which don't recover their original shape, when unloaded. In his definition he includes structural systems made by materials lacking in tensile strength, like masonry arches. He demonstrates that in this case a repeated elastic analysis can determine, by means of successive iterations, the geometric form of the reacting section, and applies this method to the calculation of the 'Mosca' bridge in Turin (Castigliano, 1879).

Upon this basic idea a program running on an AT Personal Computer was developed (see flow-chart, fig. 1), using a common f.e. analysis package program as a 'subroutine'.

GENERAL FEATURES OF THE PROCEDURE

Plane and axisymmetric space structures (arches and domes) are modelled by four node plate and membrane elements, figs. 2a-2b. The first computer run is a simple elastic f.e. analysis, with elements effective in their entire cross section. Computed normal force, N, in the meridian direction and moment around local horizontal axis of the element, M, define for each node the eccentricity, EC, of the resultant force. The element zone under tensile stress has to be eliminated, thus reducing the section as a function of EC (see fig. 2c). For each element connected to the same node a different value of EC is computed; in the chosen procedure we compute, as a function of EC and TH, the displacement S which defines the coordinates of the joint for the successive iteration. Hence a mean value $\underline{S}$ has to be calculated for each node. The updated thickness of the element is finally determined in function of the $\underline{S}$ values of the four nodes. For the successive iteration step then, the program modifies the structure geometry, usually reducing the element thickness TH, varying joint coordinates and nodal loads, automatically re-writing the input mesh.

Some tests of the program (not discussed here) on simple arch-shaped plane systems show reliable results (Trovalusci). Solution convergence is reached soon, in four or five iteration steps.

EXTENSION TO SPACE STRUCTURES

To deal with space structures some refinements are needed.

Only a sector of the dome is usually analyzed. In axisymmetric conditions forces acting across vertical boundary planes are orthogonal to those planes. These hoop actions can be represented by truss elements connected to the structure joints (fig. 3a). In the step-by-step analysis tensile hoop forces are gradually eliminated reducing the Young modulus of the truss. Tensile hoop forces vanish, fig. 3b, and the outward thrust must be counteracted by horizontal reactions at the base given by rigid or elastic radial constraints. The number of required iterations increases, up to 8 or 9.

THE VATICAN DOME

It is well known that the Vatican Dome suffered damages ever since the first half of the seventeenth century. Detailed drawings by Poleni show extensive cracking in the inner shell, in the counterforts and at the drum base, the so-called *zoccolone*. During the repairing works in the 1740's, the upper one of the two original iron chains was found broken and six additional chains were placed by Poleni (Poleni, 1748). The whole subject is still of interest. Cause of the damages and effectiveness of the adopted remedies remain, to a certain extent, unknown. Our analytical model tries to look into these questions.

'True' geometry of the dome is itself a problem: geometric data for the analysis are derived from Ferrabosco drawings, dating back to 1630 and considered the best survey available at present! Lantern weight and self weight of the dome are taken from Poleni.

The model corresponds to a one-sixteenth of the dome (figs. 1a-4b), from the base to the crown, except the lantern. Finite element mesh accounts for relevant discontinuities such as the ring corridor at the base, windows in the drum, passages in the counterforts and in the ribs, staircases which perforate the massive zone at the springing of the dome.

Connections realized by rigid beams are used to link the nodes of the rib to the nodes of the shell elements (fig. 4c), under the hypothesis of plane behaviour of the cross section.

Horizontal truss elements stand for hoop compressive actions across vertical boundary planes.

Nodes at the base are restrained against vertical movements; lateral stiffness at the base depends on the lower supporting structure and it is of uncertain evaluation. Since it highly affects the overall behaviour of the dome, it was taken as a parameter. The breaking of the original chain reported by Poleni, together with the measures of the cracks width, gives useful information in view of its calibration. Introducing boundary truss elements carrying tensile force at the levels where the original chains had been placed (tensile strength from Poleni's tests was used), some complete computer analyses were performed beginning with rigid radial constraints at the base and going on parametrically decreasing lateral stiffness until breaking of the chains was reached in the model. Some aspects of the structural response obtained for this last run are shown in comparison in figs. 5-9.

Let's examine firstly the evolution of the hoop forces (fig. 5): at step 1 hoop compressions are distributed from the haunches to the top; tensile forces prevail in the drum and at the base. At step 7 tension stresses have disappeared, except those representing the action of the iron chains, still active. At the last iteration step chains are broken and compression is concentrated in a restricted upper zone. The radial displacement at the base is 1.5 cm, which corresponds to nearly 10 cm of length variation in the circumference. The total width of the cracks reported by Poleni sum up to 16 cm, but this value contains as well, the amplitude of the diagonal cracks.

The computed structural response in terms of displacements is realistic: the total absence of hoop compressions up to the crown, as shown in fig. 5, justifies the cracks extending all over the dome, as reported in Poleni's drawings.

The analysis demonstrates that a little spreading at the base produces the subdivision of the dome into half lunes, leading to a structure consistent with the mechanical models adopted in the eighteenth century analyses.

The thrust line in the rib, as it comes out from the computer analysis is plotted in fig. 6: it is not very different from the curve obtained experimentally by Poleni by loading a flexible hanging chain. The shape of the dome is apt to contain the entire thrust line and the meridional cracks do not endanger its stability.

In fig. 7 the shaded area shows the zones, in the rib profile, under tensile strain at the last iteration, so determining the reacting section. Compression meridional stresses are plotted in fig. 8: the maximum compressive stress is about 1.4 N/mm^2.

The outward thrust at the base (fig. 9) increases with iteration steps as the tensile hoop forces disappear; after the breaking of the chains the thrust reaches an upper limit of 1.8 MN. Correspondingly each pier (carrying a quarter of the dome) has to withstand to an horizontal force equal to 4.2 MN. Taking into account the vertical downward weight of the dome and self weight of the pier we can conclude that the computed horizontal thrust is about eight times less than the value which would produce tensile cracking (shrinking) at the base section of the pier.

CONCLUSIONS

Must be clearly pointed out that analytical results could represent only an estimate of the complex behaviour of the structure, all the same some useful conclusions were drawn.

1) Breaking of the chains is related with deformation at the base of the dome, probably due to settlement.

2) The observed analytical response could justify the cracking pattern reported by Poleni together with the spreading of the dome at the base.

3) Anyhow the enormous cross piers could sustain an outward thrust up to eight times the computed value, so the overall stability of the structure is unquestionable.

4) A critical zone, weakened by close passages, can be found at the haunches of the dome in the rib, where the thrust line almost touches the intrados (fig. 6). It appears that a further spreading at the base might endanger the structure. Here the chains placed by poleni prove useful: an analysis performed by the program showed that an additional radial displacement around 0.7 cm could take place without breaking of the chains placed in the eighteenth century. Correspondingly the thrust at the base decreases to nearly 0.4 MN but the chains prove effective in keeping the thrust line within the profile of the dome at he haunches.

It can be concluded that eighteenth century structural analyses and remedies by Poleni were substantially correct.

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ACKNOWLEDGEMENTS

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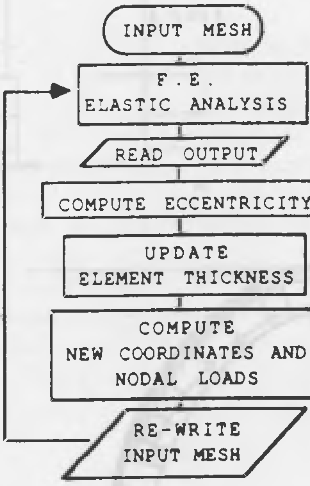

FIG. 1 Program flow-chart

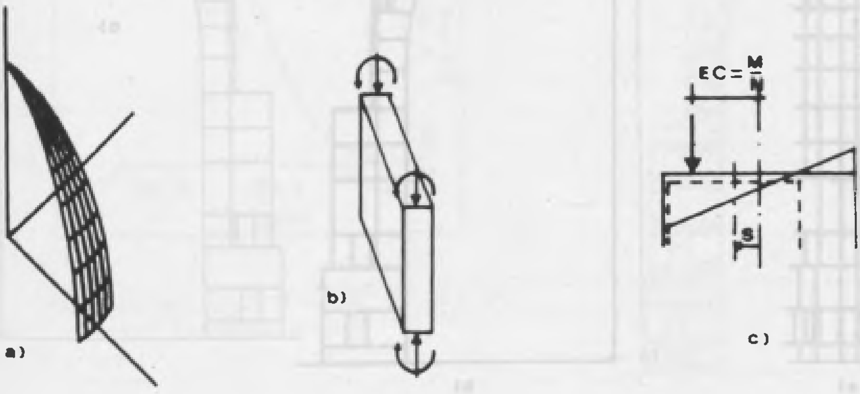

FIG. 2

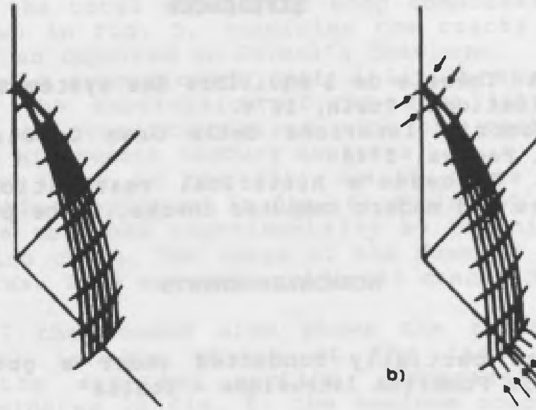

FIG. 3

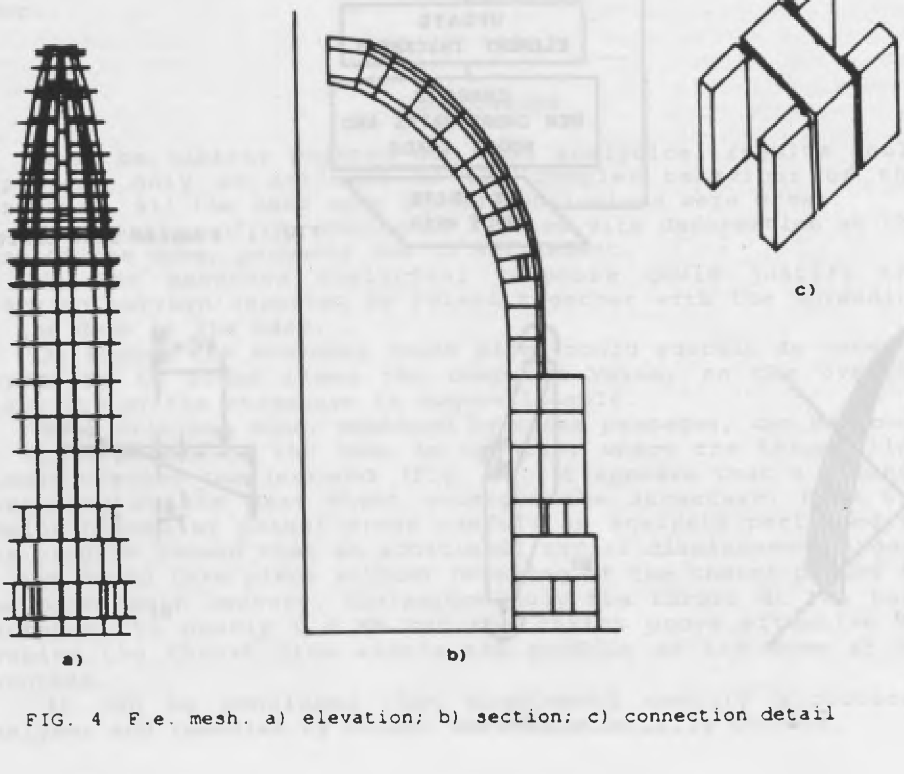

FIG. 4 F.e. mesh. a) elevation; b) section; c) connection detail

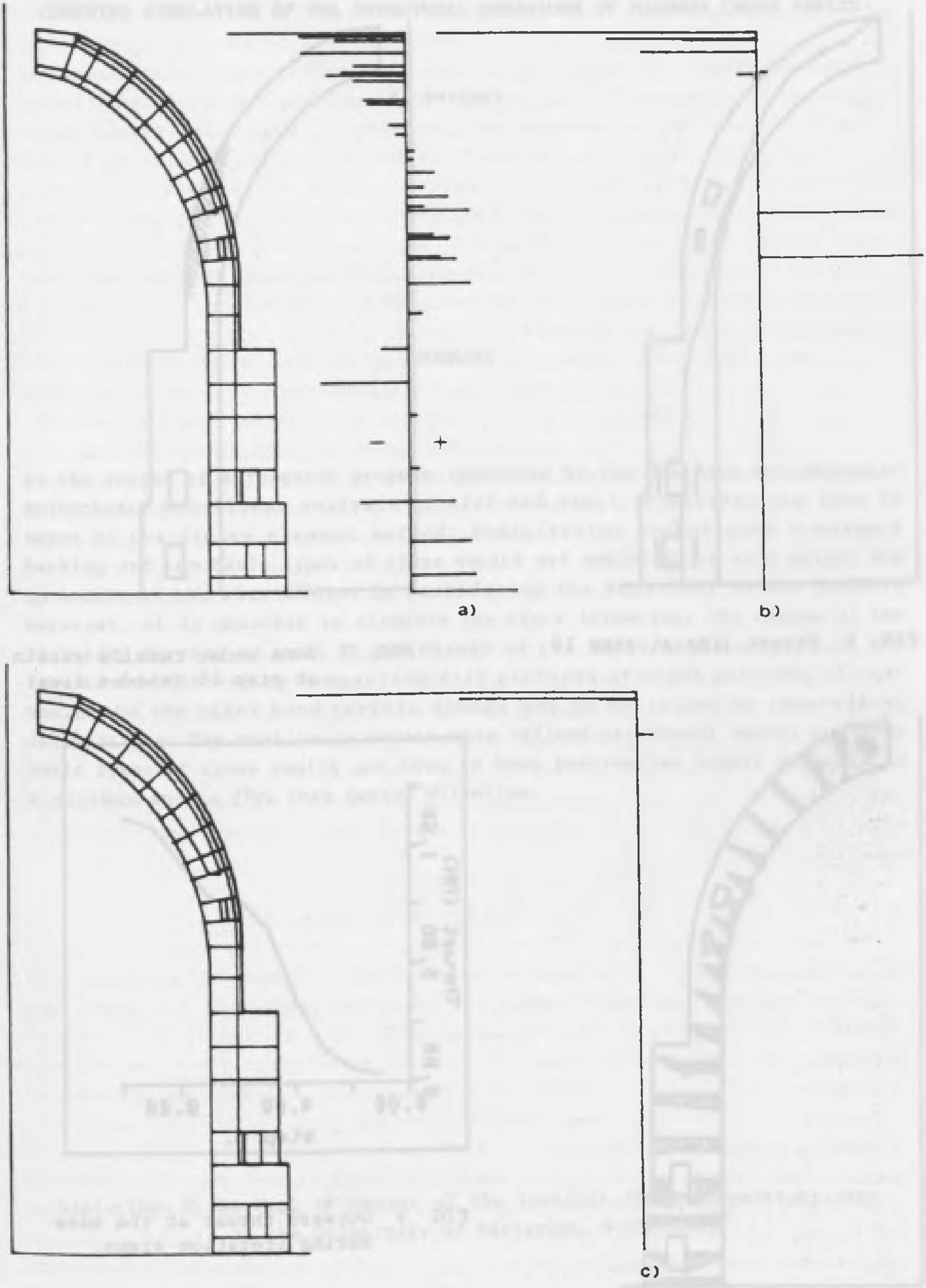

FIG. 5 Hoop forces during iteration steps. a) step 1; b) step 7; c) step 10.

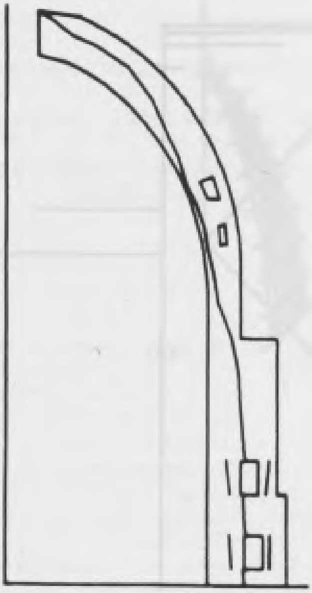

FIG. 6 Thrust line at step 10

FIG. 7 Zone under tensile strain at step 10 (shaded area).

FIG. 8 Meridional compressive stresses at step 10.

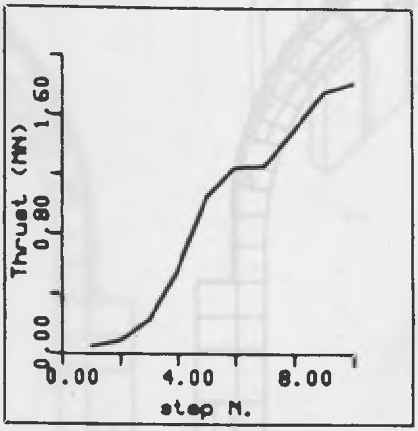

FIG. 9 Outward thrust at the base during iteration steps.

COMPUTER SIMULATION OF THE STRUCTURAL BEHAVIOUR OF MASONRY CROSS VAULTS

R. Barthel

SUMMARY

In the course of a research program sponsored by the Deutsche Forschungsgemeinschaft, non-linear analysis of arch and vault structures are done by means of the finite element method. Semicircular arches with a masonry backing and two basic types of cross vaults are subjected to self weight and spreading of the abut ments. By considering the behaviour of the masonry material, it is possible to simulate the crack formation, the change of the resulting forces, as well as the change of the inner state of stress. The results are checked by comparison with pictures of crack patterns of real vaults. On the other hand certain damage may be explained by theoretical calculations. The goal is to create more refined structural models for some basic types of cross vaults and thus to keep destructive repair measures to a minimum and to give them better direction.

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15. April 1989

INTRODUCTION

In practice there are many sources of uncertainty when examining and analysing historic vault structures. The application of conventional analysing methods is complicated due to the three-dimensional shapes which are often doubly curved, to the material properties of the masonry and to the boundary conditions. This often leads to expensive remedial measures, which are often unnecessarily extensive and which alter and, to some extent, destroy the historic building substance. Therefore the research project aims at a refined knowledge of the structural behaviour. This should create the conditions for a minimisation of the required measures.

The work consists of four steps:

1) The most important types of cross vaults are to be compiled and classified according to their shape, structure and material. The objective is to determine a few idealized basic types from the endless variety existing in order to make them accessible for analysis.
2) Typical sorts of damage occuring to vaults are to be described in relation to the vault shape, structure and the loading case. Records of damage surveys and completed repair operations are to be compiled and evaluated.
3) Analysis is to be conducted by means of the finite element method, examining the influence of various parameters. The theoretical results are to be compared with the actual damage in the vaults; also a theoretical explanation may be given.
4) The work will be concluded by giving advice for the analysis and repair in practice. Which simplifications can be made when calculating? Which parameters are to be considered?

This work has not been completed but first results of the finite element analysis concerning arches and two types of cross vaults are presented below.

BASIC ASSUMPTIONS OF THE FE-ANALYSIS

The program "ADINA" was applied with "PATRAN" acting as pre- and postprocessor. The material model for concrete availible in ADINA was transferred to masonry. So the assumptions are:

1) No or minimal tensile strength
2) Orthotropic behaviour after the formation of cracks. The stiffness perpendicular to the crack level is turned into zero, the parallel stiffness is not changed.
3) Non-linear stress-strain relationship in compression. The stress-strain curve begins with an initial tangent modulus and goes up to a peak value followed by a falling branch. Under low compression the material behaviour is approximately linear-elastic with an E-modulus of 3000 MN/m^2 and ν = 0,25 which corresponds to a brick masonry.
4) A biaxial or triaxial compressive failure envelope

Sliding is not considered. This type of failure isn't relevant in the calculations presented in the following.

Two-dimensional solid elements were used for calculations of arches and three-dimensional solid elements were used for spacial cross vaults. The elements are stacked in two or occasionally four layers. This makes it possible to simulate bending tensile cracks and to get realistic results in the thick areas like the groin or near the abutments.

ARCH

Some simple preliminary studies on the semi circular arch with a masonry backing will be presented here. The masonry backing is assumed just so high that no cracks occur in the arch under self weight. Between the masonry backing and the arch a stiff connection is assumed. The masonry backing is supported horizontally at the connection to the nave wall.

Even for very small displacements of the abutments, the steepest line of thrust forms and a mechanism of movement of a three-hinge arch forms with the corresponding cracks on the top and the bottom face of the arch. In the upper part of the masonry backing vertical cracks form. The resulting horizontal reaction becomes smaller and its position moves downwards. Vertical cracks form in the masonry backing (Fig. 1).

After displacements of support of about 2 mm the crack pattern and the position of the reaction does not change. If further displacements occur the cracks only broaden at the hinges corresponding to the rotation of the arch segments. The cracks in the masonry backing do not change any more. Only after further major displacements does the horizontal force increase. (II. Order Theory).

TWO ARCHES LYING UPON EACH OTHER

The way in which two arches lying upon each other interact, was examined to obtain indications of how ribs and webs without shear connection behave or how double-layered transverse arches work. A contact surface was defined between both arches of two-dimensional solid elements. Pressure can be transferred perpendicularly to the contact surface but no tensile forces at all. Initially, friction was not considered.

There is little interaction between both arches when self weight and small horizontal displacements of the abutments are taken into account only. The arches separate a little at the crown as well as near the fill. There are, though, shifting movements along the contact surface. Figure 3 shows the crack pattern common in many thick transverse arches and between cross ribs and the webs. It corresponds with the analysis.

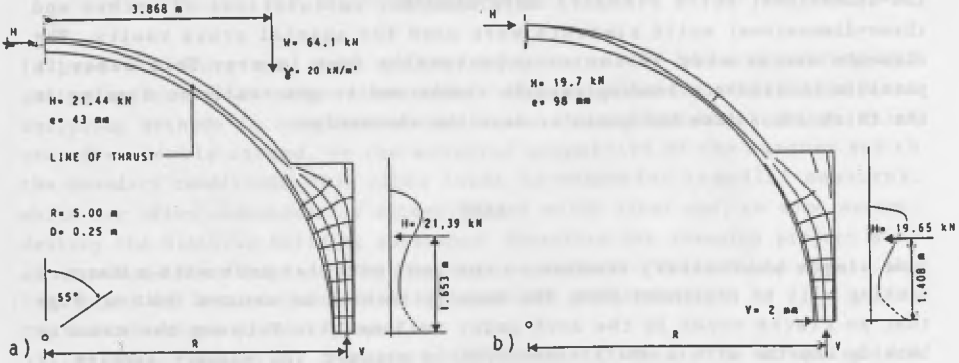


Fig. 1: Arch with masonry backing (width 1m), line of thrust amd stress trajectories, distribution of the horizontal force, resultant forces (for a span of 10 m)
a) self weight b) self weight plus spreading of the abutments

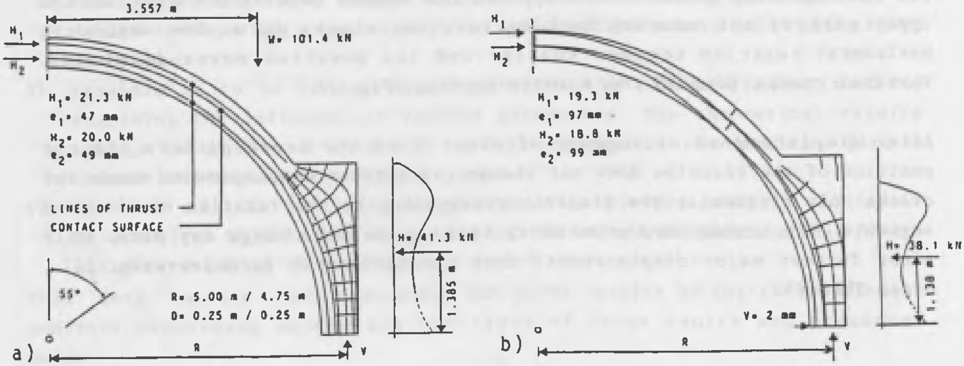


Fig. 2: Two arches lying upon each other with contact surface
a) self weight, b) self weight plus spreading of the abutments

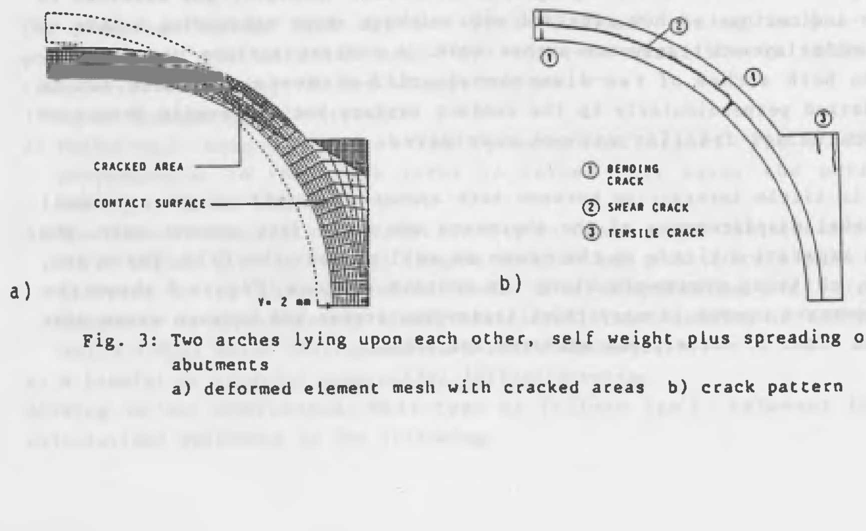


Fig. 3: Two arches lying upon each other, self weight plus spreading of the abutments
a) deformed element mesh with cracked areas b) crack pattern

Beginning with the self weight case, one quarter of a circular cylindrical cross vault was calculated with the corresponding symmetrical conditions along the vertex. In order to consider the neighbouring vault and the horizontal support by the nave wall, symmetry was also assumed along the edges, for the present a rigid support was assumed at approximately the half of the total height.

The influence of the horizontal support along the edges is illustrated by the deformations (Fig. 4). At the vertexes the mesh moves inwards, here cracks occur parallel to the transverse arches and to the nave wall. In the lower area the vault presses against the edge support. The diagonal arch is stabilized in its position by this effect. This forms a relatively stiff area along the groin. Because the edge and the groin deflect differently the web is forced to bend along a contour line.

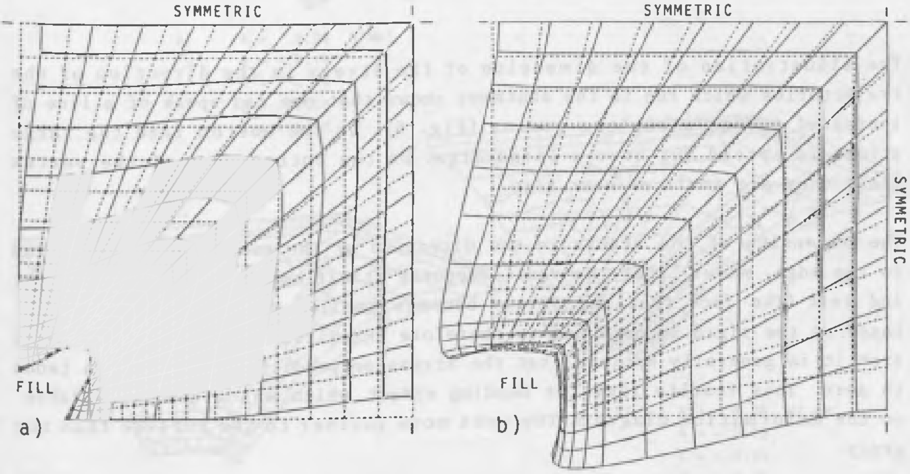
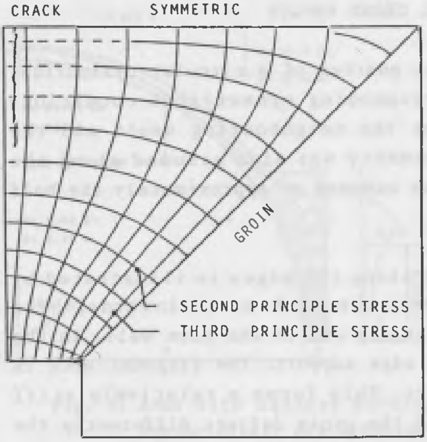
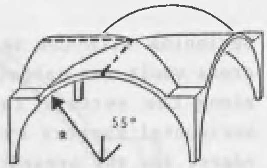


Fig. 4: A quarter of a cylindrical cross vault under self weight, deformed shape (span: 10m, radius: 5m, thickness: 0.25m)
a) with horizontal edge support b) without edge support

Without edge support the deformations are larger. As the diagonal arch has the shape of an ellipse it wants to burst out in the lower part. In this case it is only held by the shear-stiffness of the webs which is a folded plate and shell effect.

The shell-effect can be seen on the course of the stress trajectories (Fig. 5). The forces do not flow straight to the diagonal arch but directly to the abutment. Almost the entire web exhibits a two-dimensional state of stress. Only near the vertex of the transverse arch do normal tensile forces occur which lead to a crack, bending tension does not occur any place in this geometry.





R= 5.00 m
D= 0.25 m
γ= 20 kN/m³

Fig. 5:
Cylindrical cross vault under self weight, stress trajectories

The illustration of the dimension of the stress in the direction of the trajectories which run to the abutment shows that one can speak of a line of increased forces along the groins (Fig. 6). On the bottom face the large stress is spread out over a large area. On the bottom part of the vertex areas they are small or near zero.

The dimension of the stress in the direction of the trajectories which run to the edge, shows, that the vault supports itself horizontally to the right and left (The fact that the stress becomes smaller towards the bottom is based on the rigid support and is therefore unrealistic). Along the diagonal arch it is generally noticed that the stress perpendicular to the arch tends to zero. This results form the bending effect which was already noticable on the deformation diagram. The webs move further to the outside than the groin.

CIRCULAR CYLINDRICAL CROSS VAULTS UNDER DEAD LOAD AND DISPLACEMENT OF ABUTMENTS

The cross vault is substantially stiffer than the arch, as the web acts like a wall between the nave walls. A small enlargement of the distance immediately leads to a decrease of the compression force and to cracks. A mechanism of movement is created. The result is the structural performance of a three-pinned arch with hinge-lines at the vertex and near the abutment.

The schematically drawn crack pattern on the upper side (Fig. 7) shows through cracks and bending cracks. The transverse web is separated into individual arches. In reality this corresponds to the typical tearing off

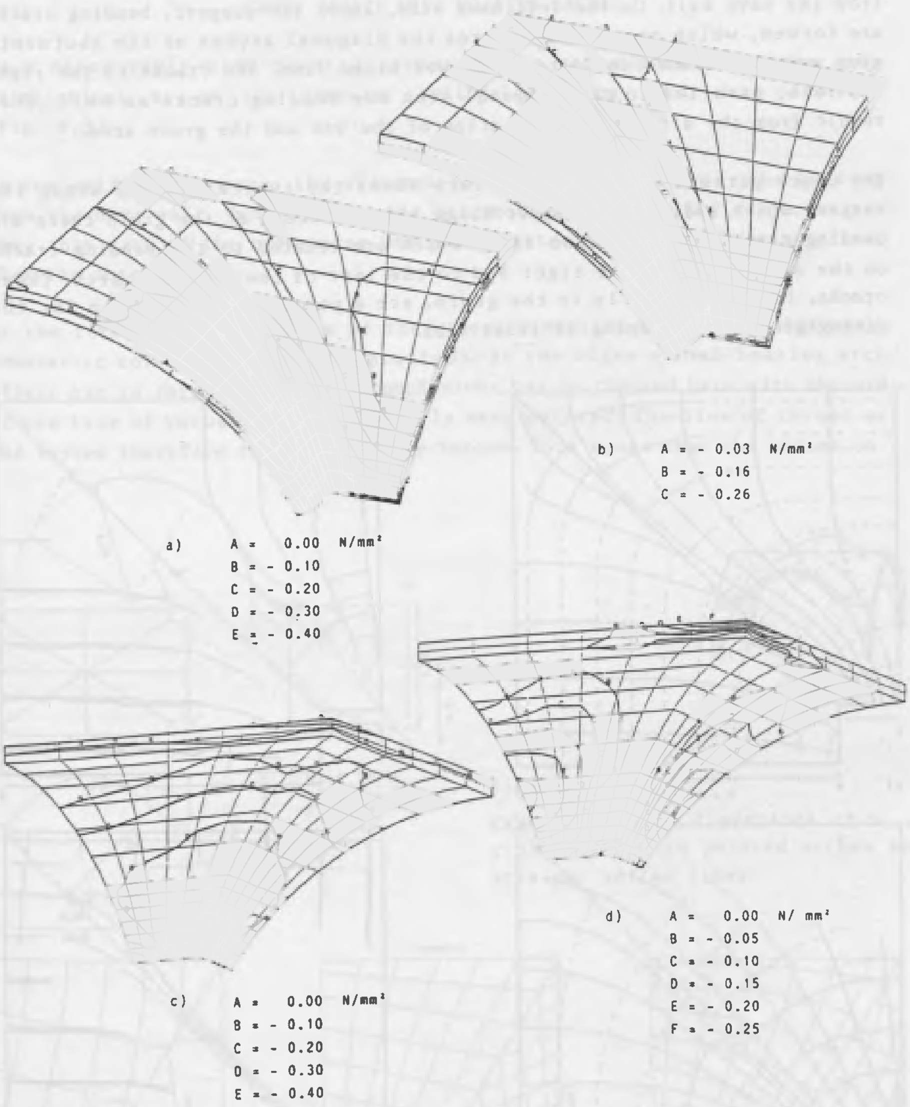


Fig. 6: Cylindrical cross vault, stress distribution and stress dimensions a) top face, third principle stress (in the direction of the trajectories which run to the abutment). b) top face, second principle stress (in the direction of the trajectories which run to the edge). c) bottom face, third principle stress. b) bottom face, second principle stress.

from the nave wall. On the left hand side, above the support, bending cracks are formed, which propagate towards the diagonal arches as the abutments give way. The cracks indicate the lower hinge line. The cracks to the right and left, parallel to the diagonal arch are bending cracks as well. They result from the different deformation of the web and the groin area.

The crack pattern on the bottom face shows the typical cracks along the vertex, which indicate the upper hinge line. Directly at the groin there are bending cracks on the bottom face, which are related to the bending cracks on the upper side to the right and to the left of the diagonal arch. These cracks, located directly in the groin, are a possible explanation for the disengaging and loosening of cross ribs.

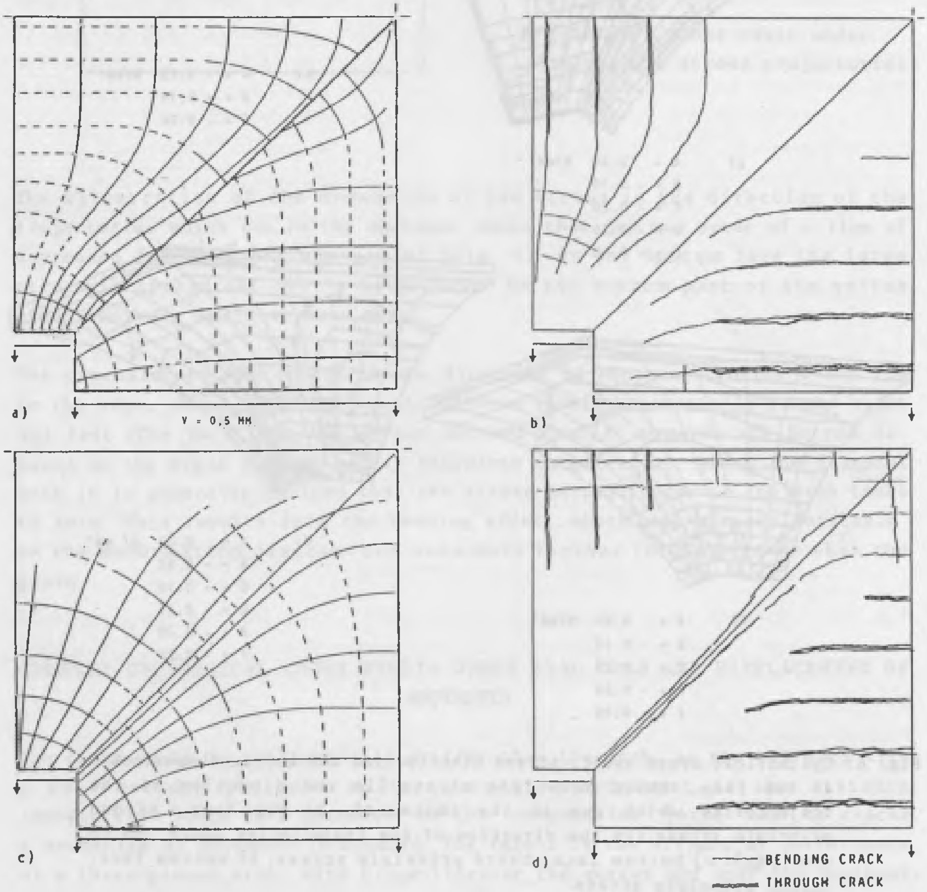


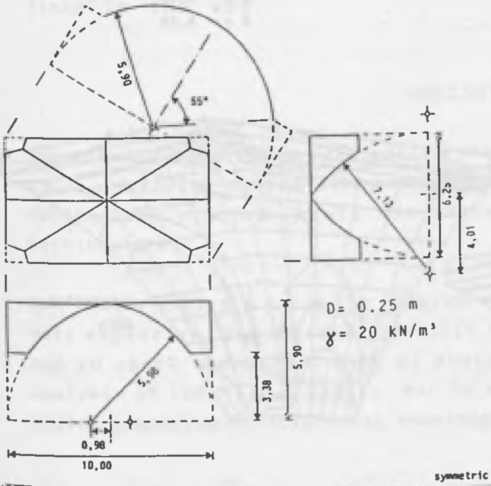
Fig. 7: Cylindrical cross vault under self weight and spreading of the abutments
a) stress trajectories at the top face b) crack pattern at the top face c) stress trajectories at the bottom face d) crack pattern at the bottom face

CROSS VAULTS WITH POINTED ARCHES

On a cross vault with pointed arches over a rectangular base the same calculations as on the circular cylindrical cross vault were performed (Fig. 8,9).

In the self weight case, the deformation picture again shows the tearing from the nave wall, but not from the neighbouring vaults. The trajectories show that part of the transverse web participates in the load-bearing effect over the long-span.

At the first the illustration of the stress dimension (Fig.10) is confusing, however it corresponds to the intuition. At the edges a load-bearing arch effect has to form, so that the conclusions can be checked here with the aid of the line of thrust. The wall arch is very pointed. The line of thrust at the vertex therefore runs at the very bottom. This means that the stress on



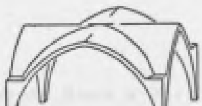


Fig. 8:
Example for the dimensions of a cross vault with pointed arches and straight vertex lines

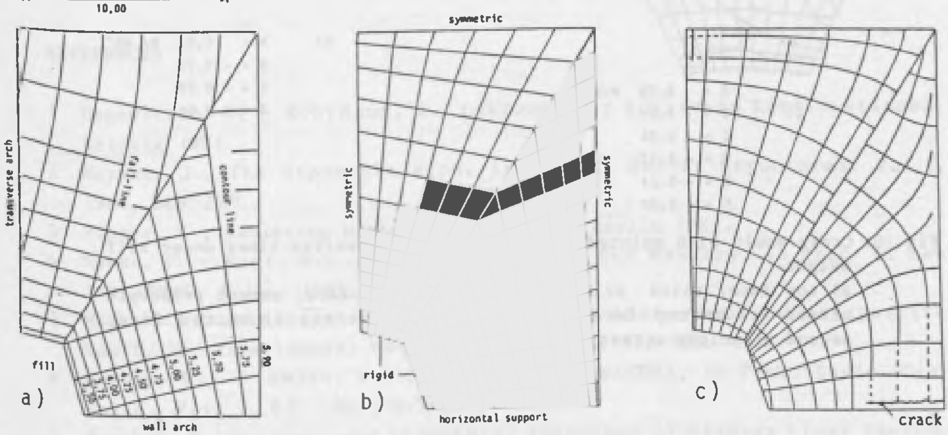


Fig. 9: Cross vault with pointed arches and straight vertex lines
a) contour and fall-lines b) finite-element mesh
c) stress trajectories and cracks under self weight

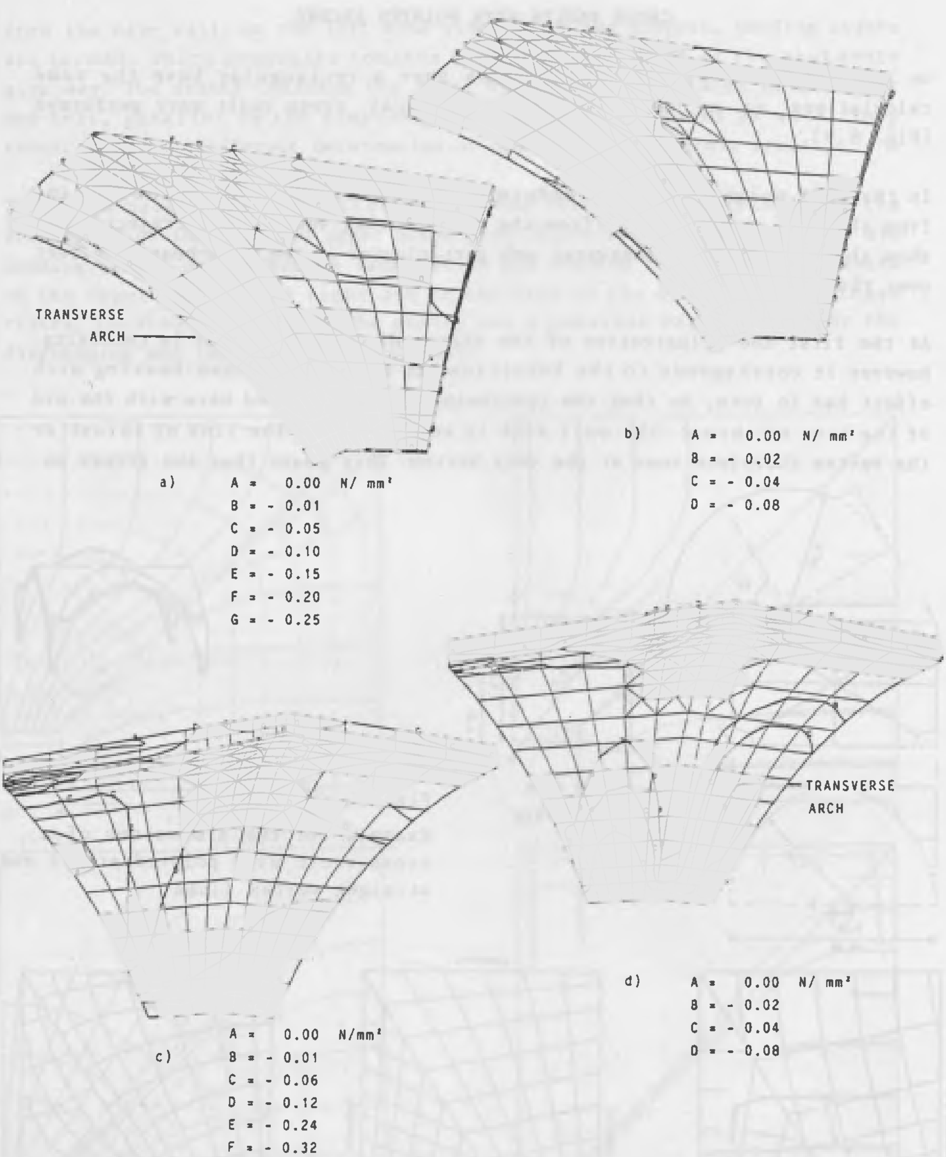


Fig. 10: Cross vault with pointed arches and straight vertex lines under self weight
a) top face, third principle stress b) top face, second principle stress c) bottom face, third principle stress d) bottom face, second principle stress

the bottom is very large and the cross section is crackedat the top. Beside the vertex the support line is located on the upper side. The stress on the bottom is very small, a crack parallel to the vertex line forms.

The corresponding conditions can also be observed on the less pointed transverse arches. In the areas next to the vertex the stress level for dead-load is very small. During a displacement of the abutments bending cracks form here.

In the area of the groin and the transverse web, the stress increases continuously towards the bottom. In the middle vertex an area with relatively little stress forms on the bottom face. This looks quite accidental, but it can be explained. Since the diagonal arch is an arc, a relatively flat area results here. If one imagined a line of thrust here, it would run along the upper side of the cross section. This can be an explanation for keystones that become loose or even fall if they are not fixed in the web.

CONCLUSION

The results show that cross vaults are real shell structures. Even a very small spreading of the abutments cause the typical crack pattern in the vaults. The cracked state can therefore be considered as the normal servicestate.

Non-linear analysis using the program mentioned with this material model is very expensive, and a great effort is required to obtain reliable results and to check them. This kind of analysis is therefore not practical for analysis of individual vaults. But in research it is possible to develop a basic compendium of structural knowledge of cross vaults.

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the bottom is very large and the [illegible] is cracked at the top. Beside the vertex the [illegible] on the upper side. The stress on the [illegible] the vertex line forms.

The [illegible] must be resolved at the last part. [illegible] the [illegible] to the bottom [illegible] dead [illegible] making a [illegible] the [illegible] crack [illegible]

In the [illegible] and the top [illegible]. The [illegible] continues [illegible] the bottom. In [illegible] relatively [illegible] stress forms on the [illegible] quite accidental, but it can be explained [illegible] relatively flat area results [illegible] of thrust lines, it would run along the upper [illegible]. This can be an explanation for keystones that [illegible] fall if they are not fixed in the web.

[illegible]

[illegible] real small structures, [illegible] the typical crack pattern [illegible] considered as the overall [illegible]

[illegible]

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INTRODUCTION TO EUROCODE 6 - STRUCTURAL USE OF MASONRY

B.A. HASELTINE

JENKINS & POTTER - LONDON

INTRODUCTION

As part of the desire to remove barriers to trade between European community countries, the European Commission have been working on the production of a set of structural Eurocodes to ensure that the design of structures is not an artificial barrier to trade when contractors or consulting engineers wish to work in countries other than their own.

The complete programme for the codes, which all use the partial factor approach to limit state design, is shown below:-

EC1 General Unified Rules for different materials and constructions.

EC2 Common Unified Rules for Concrete Structures

EC3 Common Unified Rules for Steel Structures

EC4 Common Unified Rules for Composite Structures

EC5 Common Unified Rules for Timber Structures

EC6 Common Unfified Rules for Masonry Structures

EC7 Common Unified Rules for Foundations

EC8 Common Unified Rules for Seismic Design

EC Actions Common Unified Rules for Actions

This paper concerns EC6, rules for the design of masonry structures, including brickwork, blockwork and stonework. The part of EC6 covering, at the moment, Unreinforced Masonry, has recently been subjected to National Examination, but the results of the examination have not yet been taken into account. Work has been continuing on further clauses to cover reinforced and pre-stressed masonry and these are now available for examination. When all of the comments have been taken into account, there will be a single Part 1 of EC6, dealing with both unreinforced and reinforced masonry. Other Parts are being worked on, or are foreseen.

The Eurocodes are primarily intended for use in the design of new structures but could well be used for analysis of existing masonry buildings where the information would be applicable.

CONTENTS OF EUROCODE 6

Section 1 - Scope, definitions and symbols

Section 2 - Basis of Design

This section is common to all of the Eurocodes so that there is a unified approach to the definitions of limit states and design situations. Actions, characteristic values, design values, material properties and material partial safety factors, are all given in this section. Much of the chapter is material independent but where necessary, material dependent parts have been added, eg. the partial safety factors for material for masonry are different from those for other materials and are given in Section 2. Unreinforced masonry does not usually require the consideration of the Serviceability Limit State, as satisfying the Ultimate Limit State usually avoids cracking and deflection problems.

The extract of EC6, Section 2, given below shows the Partial Safety Factors for loads and materials.

Ultimate Limit State

γ_F	permanent loads	1.35
	imposed loads	1.50

γ_M **Partial coefficients for material properties**

γ_M		Category of construction control		
		A	B	C
Category of manufacturing control	A	2.0	2.3	-
	B	-	2.5	3.5

When verifying the stability in the case of accidental actions, the γ_M shall be taken as $\boxed{1.2}$, $\boxed{1.5}$ and $\boxed{1.8}$ for categories A, B and C respectively.

Partial Safety Factors from EC6

Section 3 - Materials

This section describes the types of masonry unit to which the code applies, the types of mortar and their properties. The strength of masonry in compression, flexure and shear is given. The most important of these strengths is the characteristic compressive strength of masonry, f_k, which is described in relation to the strength of the unit and the mortar. The flexural strength f_{xk} is related to the water absorbtion of clay units, the type of unit and the mortar. The shear strength, f_v, is related to the type of unit, mortar and extent of vertical loading.

Section 4 - Design of Masonry

This section gives the fundamental requirements for design of constituent structural members for structural behaviour under normal loads and under accidental situations other than earthquakes. The reason that earthquake loading is not covered is that this subject is part of EC8 that has material dependent sections to supplement each of EC's 2 to 7, giving the additional design requirements for the earthquake situation.

The design of masonry walls is subdivided into three main sections:

Walls subject to vertical loading under the ultimate limit state

Shear walls under the ultimate limit state

Walls subjected to lateral loads under the ultimate limit state

In each case the relevant verification is given.

Section 5 - Structural Detailing

Guidance on types of wall, minimum thickness, bonding, connections, recesses, chases, damp proof courses, thermal and long term movements and masonry below ground, are all given in this section.

Section 6 - Construction

It is a principle of the Eurocodes that work on site is only covered to the extent necessary to lead to building quality that is adequate for the assumptions made in the design sections, and these are so given.

DISCUSSION ON CONTENTS

In the drafting of Eurocode 6 the Drafting Panel were required to remain within the general layout of Eurocodes agreed by the Co-ordination Group. Similarly, the Principles, especially the safety requirements, were pre-arranged by the Co-ordination Group. There has been some criticism of EC6, largely on the basis of the excess complication indicated in Section 2, which is common to all the Eurocodes. In practise, this Chapter will be little used by the designer and only those parts of it that give the partial safety factors will be of running interest to him. When there is an EC Actions, a large part of the present Section 2 could be removed and put into Part 1 of that document.

In this part of the paper some of the problem areas in EC6 are highlighted for the benefit of readers.

Partial Safety Factors for Materials

Some countries presently differentiate partial safety factors according to the manufacturing control of the masonry units and the control of workmanship on the building site. This approach has been adopted in EC6; in this way it is hoped that the differences in safety level resulting from varying control will be evened out. Because EC6 must cover a wide range of unit manufacture and construction techniques across the Community countries, four partial safety factors have been given, covering two levels of manufacturing control and three of construction control (Note that 2 spaces in the matrix of 6 are not used). The figures range from 2 to 3.5 but it is expected there will be a move to reduce these numbers in the Editorial Group.

Characteristic Compressive Strength of Masonry f_k

EC6 gives a formula for the compressive strength of masonry, f_k, but was unable to give the coefficient or powers in the formula, although these were shown in the preface. Work has been proceeding in ISO and it is now possible to give the complete formula for a number of different situations. It remains to be seen whether this approach will be acceptable across the Community countries for the whole range of masonry types and materials.

A summary of the ISO proposals is given below:-

Characteristic Compressive strength of masonry

$$f_k = K \quad (f_b)^{\alpha} \quad (f_m)^{\beta}$$

Latest suggestion (ISO)

K = 0.65 for solid and equivalent solid units, and no longitudinal mortar joint in the wall

K = 0.60 for perforated, hollow or cellular units, and no longitudinal mortar joint in the wall

K = 0.55 for both above types of units, but when there is a longitudinal mortar joint in the wall.

α = 0.65 β = 0.20

f_b = nomalised mean comrpessive strength of units i.e. equivalent dry strength x δ, the shape factor.

f_m = compressive strength of mortar (mean of 28 day results)

Characteristic Shear Strengthf_{xk}

There is a little dispute over the shear strength section of EC6 which is summarised below:-

The characteristic shear strength of unreinforced masonry, f_{vk}, may be taken as :

$$f_{vk} = f_{vko} + 0.4\,\sigma_d \leq 0.05\,f_b \qquad (3.2)$$

but not greater than the limiting value given in Table 4

where f_{vko} is the shear strength at zero $_d$ as given in Table 4

σ_d is the design vertical compressive stress in the wall at the level under consideration using the load combination giving the least vertical load

f_b is the compressive strength of the unit as defined in clause 3.1.1.1.

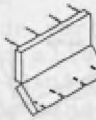
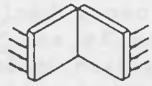
Table 4 : Limiting values of the characteristic shear strength of masonry, f_{vk}, and values of f_{vko}

Masonry Unit	Mortar	f_{vko} N/mm²	Limiting f_{vk} N/mm²
Perforated Hollow & Cellular Concrete Units	M20,M15,M10	0.2	0.8 but not greater than the strength of the unit along its length*
	M5, M2	0.1	
Calcium Silicate Units & CONCRETE UNITS	M20,M15,M10	0.2	0.8
	M5, M2.	0.1	
Clay Units of f_b less than 15/mm²	M20,M15,M10	0.3	1.0
	M5, M2	0.1	
Clay units of f_b greater than 15N/mm²	M20,M15,M10	0.3	1.5
	M5, M2	0.1	

* When the strength of the unit tested along its length can be expected to be greater than 0.15 times the vertical strength perpendicular to the normal direction of laying, by consideration of the pattern of holes in the unit, this limit can be assumed to be satisfied.

Characteristic Flexural Strength of Masonry f_{xk}

The idea of allowing "tension" in masonry, albeit in out of plane flexure, surprises some designers in Community countries, and yet how else does a masonry wall transmit wind forces to the bracing elements of the structure? The suggested table of flexural strengths presently in EC6 is reproduced below. It is recognised that this Table, based as it is largely on British practise, may need some modification in the editorial process.

	Plane of failure parallel to bed joints 			Plane of failure perpendicular to bed joints 		
Mortar type	M15 M20	M10 & M5	M2	M15 M20	M10 & M5	M2
Clay units with a water absorption: less than 7% between 7% and 12% over 12%	 0.7 0.5 0.4	 0.5 0.4 0.3	 0.4 0.35 0.25	 2.0 1.5 1.1	 1.5 1.1 0.9	 1.2 1.0 0.8
Calcium silicate units	0.3		0.2	0.9		0.6
Concrete units or highly perforated units with a characteristic compressive strength ≥ 3.5 N/mm² used in walls of thickness* : up to 100 mm 250 mm	 0.25 0.15		 0.2 0.1	 0.45 0.25		 0.4 0.2

* The thickness should be taken to be the thickness of the wall, for a single-leaf wall, or the thickness of the appropriate leaf, for a cavity wall. Linear interpolation is permitted for thicknesses between 100 and 250mm.

Table of f_{xk} from EC6

Eccentricity

The concept of an accidental eccentricity explicitly used in design is normal in some countries for reinforced concrete but is rarely applied to masonry. If thin walls are traditional, an accidental eccentricity based on a proportion of the effective height, can be very large and can have a disproportionate effect on the design capacity of the wall.

The structural eccentricity to be allowed for may be calculated from first principles or a very simple model may be adopted. Most calculations of eccentricity make use of some sort of frame analysis which inevitably assumes a rigid connection between the floors and walls. In masonry, such a rigid connection is not normally achieved so that the angle of rotation of the floor is not the same as the angle of rotation of the wall. This usually means that the eccentricity calculated by rigid frame analysis is too high. A very simple approach is to assume a hinged arrangement of floor and wall. The three possible arrangements are given in Figure 1.

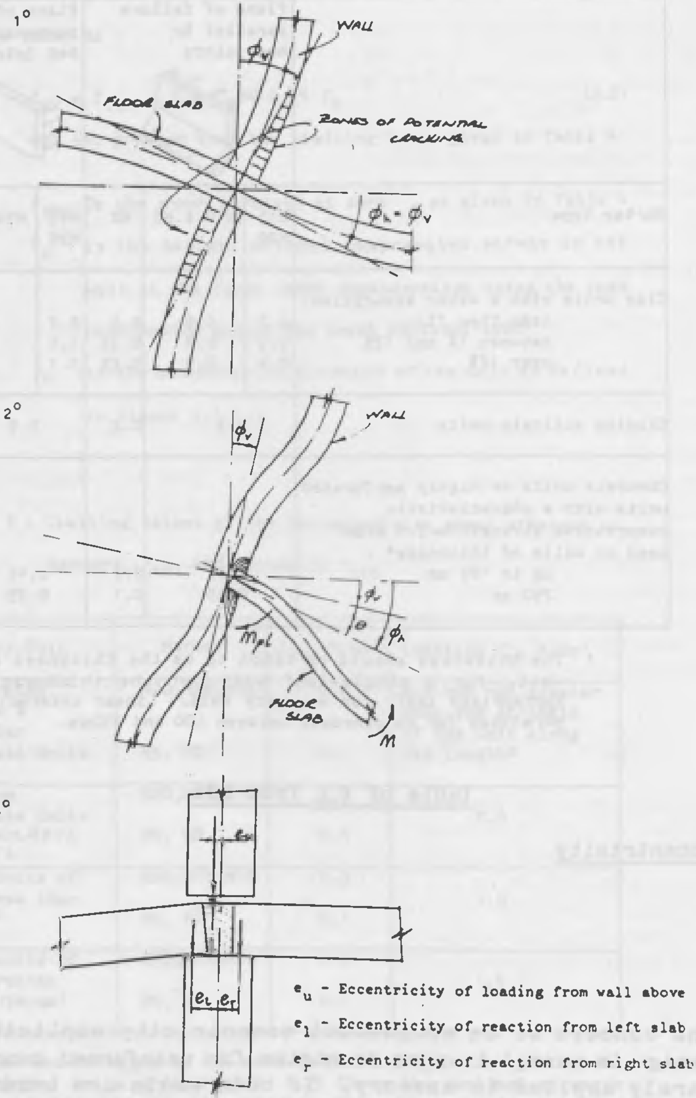


Fig.1 Floor Wall Junctions in Masonry

1. Rigid system
2. Semi-rigid system
3. Hinged system

In the ISO draft code, moments calculated from frame analysis may be reduced by one third before being applied to the wall. Some work has suggested that even then the eccentricity obtained is too high and that, maybe, the reduction should be two thirds of the calculated eccentricity.

Allowance for Slenderness and Eccentricity

In the verification of a wall for vertical load capacity a reduction factor is applied to allow for the effect of slenderness and eccentricity. The slenderness ratio having been calculated from rules given concerning effective height and effective thickness, a capacity reduction factor is given at the top or bottom of the wall and the centre of the wall, as shown below. It will be noticed that an eccentricity due to creep is mentioned in the formula for use at the centre of the wall. This is a controversial subject and one suggestion made in ISO is for a lower partial safety factor for materials to be permitted if a "more rigourous" calculation, including the effects of creep, is carried out.

Φ at top or bottom of wall

$$= 1 - \frac{2e_i}{t}$$

$$e_i = \text{structural eccentricity} + \text{any eccentricity from horizontal loads } e_h + \text{accidental eccentricity } e_a$$

Φ at centre of wall

$$\Phi_m = 1.14\left(1 - 2\frac{e_{mk}}{t}\right) - 0.02\frac{h_{ef}}{t_e}$$

but not greater than $\left(1 - 2\frac{e_{mk}}{t}\right)$

$$e_{mk} = \frac{M_1 + M_2}{2\ N_m} + e_{hm} + e_{creep} \pm e_a$$

Detailing

Many countries in Europe assume that unreinforced masonry walls must be tied together with ring beams or ring anchors at each floor level, but this is not universal. EC6, therefore, recognises that the tying together may be carried out by timber floors or beams as are traditional in eg. the United Kingdom.

A detailed section on chases and recesses has been the subject of detailed comment from some countries.

Work on Site

Having decided that there will be three categories of construction control, these have to be defined in words. As presently drafted the words are probably not sufficiently explicit to differentiate between the categories and in the Editorial process some tightening up will be required.

CONCLUSIONS

Eurocode 6, Common Unified Rules for the design of masonry structures, has been subjected to National Examination and produced a large volume of comment. It is to be hoped that this comment can be incorporated without undue compromise leading to no-one being satisfied. After comment has been made on the section dealing with reinforced masonry, soon to be issued for discussion, a modified version of EC6, covering both unreinforced and reinforced masonry should be available in about three years time for trial use as an ENV, published by CEN.

A NON-LINEAR FINITE ELEMENT APPROACH TO MASONRY ARCH AND MASONRY FLAT ARCH

Carlo Blasi* and Paolo Foraboschi*

Summary

The arrangement of a non-linear finite element approach to the masonry arch and to the masonry flat arch is proposed.
This approach employed a particular coupling of elements; this coupling consists in a «brick element», which is linear elastic and isotropic, and a «gap element», which is a monolateral friction element, capable also of producing separations and slidings between the brick elements.
Nevertheless the non-linear finite element approach employed has to be suited to the particular type of masonry structures analysed. Two of the main masonry structures are analysed: the masonry arch and the masonry flat arch.
To check the proposed arrangement of the method, an analytical approach for the masonry arch, and photoelasticity results for the masonry flat arch, have been employed. The check proved the proposed arrangement of the method to be completely reliable.

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02.05.1989

INTRODUCTION

This work originated in the debate about the employment of finite element methods in the analysis of the structural behaviour of masonry constructions.
The most common numerical approaches usually employed are linear elastic, nevertheless the structural behaviour of the masonry constructions is non-linear also under loads that are very far from collapse.
Furthermore the limit tensile strength is much lower than the limit compressive strength, so that internal fractures are present also under very low loads.
Then the shear force is also transferred by particular mechanisms.
These evaluations show how linear elastic finite method approaches to masonry structures are capable of producing incomplete or unreliable results.
To overcome these problems, a particular coupling of finite elements was proposed by Chiostrini, Foraboschi and Sorace ([1], 1989).
This coupling consists in joining a linear elastic isotropic finite element (brick element) with a monolateral-friction element (gap element). This approach has employed a numerical code of calculus named «Ansys» ([2], 1985).
Nevertheless this approach needs to be suited to the type of masonry structures examined, since the correct model for each type of structure depends on the type of structure itself.
The employing of this approach to the masonry arch and to the masonry flat arch has been developed in this work, to fit the method to these types of structures.
The results of this research are the achievement of a correct non-linear finite element model for these types of structure, employing the method introduced in [1].
The results were checked by a comparison with an analytical approach for the masonry arch ([3] Blasi and Foraboschi, 1989), and with photoelasticity results for the masonry flat arch.
The comparison proved that the proposed method reproduced well the correct results.

NUMERICAL MODEL OF THE MASONRY ARCH

The proposed finite element schematization of the masonry arch consists in dividing the arch into blocks; each block is defined by two transversal sections to the axis of the arch and by the part of the arch comprised inside them. These blocks are hypothesised as being linear elastic isotropic, so that they are schematised with elements having those characteristics, called «brick elements».
The brick elements are joined by elements which are capable of

transferring compressive strength, in their normal direction and shear force limited by the Coulomb friction law, in the tangential direction. So that these elements, called gap elements, being incapable of transferring tensile strength, have a monolateral behaviour.
A gap element is formed by two parallel surfaces, which define their orientation; this element is also capable of producing not only monolateral behaviour, but also fractures and slidings; in fact the two parallel surfaces separate themselves in correspondence with tensile strength and slides in correspondence with shear force superior than that furnished by Coulomb's friction law.
So that this finite element schematization reproduces the main features of not only the masonry mechanical behaviour, but also of the masonry cinematical behaviour.
An example of this proposed schematization is in the figure 1.
This method was first applied to reproduce the cases analysed in an analytical continuum approach to the masonry arch, proposed in [3], in order to make a comparison, for the purpose of checking this proposed method.
The comparison proved the proposed method to be completely reliable; furthermore the method appeared to have no numerical problems: so that the proposed non-linear finite element method can be applied by referring only to structural evaluations, without taking numerical aspects into account.
The comparison was developed for the two limit horizontal thrusts at the abutment and for the mechanisms of collapse.
An example of the comparison of the two limit horizontal thrusts, refering to S=90 cm , is the table 1.
Also the comparison between the mechanisms of collapse showed a complete correspondence between the analytical approach and the proposed numerical approach.
In particular the position of the hinges in the haunches was proved to be the same.
The figure 2 shows the structural behaviour determined by the inferior horizontal limit thrust, and the figure 3 shows the structural behaviour determined by the superior horizontal limit thrust.
Nevertheless this approach produces results not only in the limit situations, as with the analytical approach adopted, but in all situations.
The proposed method supplies the stress-strain distribution, the displacement, the internal fractures and slidings, and the reactions of the restraints, produced by any load.
Furthermore this approach allows one also to execute evolutive analyses, up to collapse.
Thus an ultimate safety factor of the structure is also obtainable; and the ultimate safety factor is very important for masonry structures, being very variable.

NUMERICAL MODEL OF A MASONRY FLAT ARCH

The finite element schematization of a masonry flat arch

is obtained by employing the brick elements and the gap elements employed for the masonry arch; nevertheless the problems to overcome are different.
The figure 4 shows the proposed schematization.
To check the schematization, a comparison with photoelasticity results is reproduced.
The photoelasticity results are obtained by translucent blocks set in such a way as to reproduce a physical model of a masonry flat arch having discontinuity between the blocks.
The photoelasticity results show in particular an «arch effect» of the stress; so that only an ideal arch inside the flat arch has structural importance, being the only part of the structure involved in the stress; the remaining part of the arch instead has no structural importance.
The figure 5 shows the photoelasticity results.
The results of the proposed finite element schematization shows a perfect correspondence with the photoelasticity results; in particular the arch effect is completely reproduced by this method.
The figure 6 shows the principal stress directions, the figure 7 shows the values of the principal stress of compression, and the figure 8 shows the displacements produced by the applied force.
The proposed numerical model shows also the importance of the friction coefficient in this type of structure; in fact if the friction coefficient is too low there is collapse, since the blocks slide down.

CONCLUSIONS

A numerical approach to the masonry arch and to the masonry flat arch has been proposed.
This numerical approach consists in a non-linear finite element method, employing brick elements, which have a linear elastic isotropic mechanical behaviour, and gap elements, which have a monolateral friction mechanical behaviour and which are capable of producing separations and slidings between the brick elements.
This approach has been checked by an analytical method, for the masonry arch, and by photoelasticity results, for the masonry flat arch; the method has been proved to be completely reliable.
The proposed method supplies results of the structural behaviour of the masonry arch and of the masonry flat arch produced by any load; in particular it supplies stress, strain, displacement, internal fractures, internal slidings, and the reactions of the restraints.
Furthermore the proposed method supplies also evolutive analyses, so that the collapse mechanisms and the associated load are obtainable. And also the ultimate factor of safety is obtainable, which is very important for masonry structures, being very variable.

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/3/ C. BLASI and P. FORABOSCHI
« The Masonry Arch: A continuum Approach and a Discrete Approach »
in the Proceedings of the International Conference STREMA 1989
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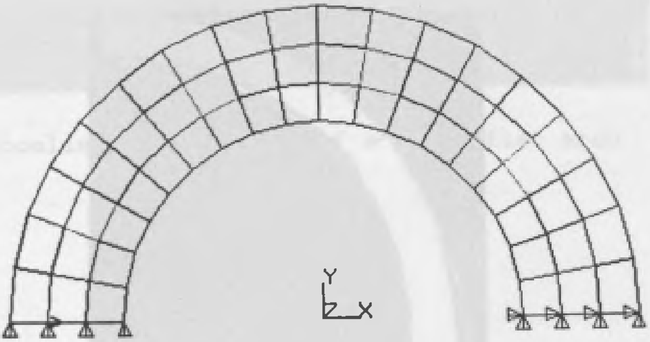

Figure 1: The masonry arch non-linear finite element schematization employed

S=90 cm	L=6 m	L=8 m	L=10 m	L=12 m	L=13 m	L=14 m	L=15 m	L=16 m	L=17 m
H_{min} con	18	29.7	42.2	54.1	59.8	66.4	73.1	78.8	85.2
H_{min} dis	18	29.5	42	54	60	66.5	73	79	85.3
H_{max} con	/	50.4	62	68.8	72.1	75.2	78.2	82.1	85.4
H_{max} dis	/	50.5	62	69	72	75.5	78	82	85.4
H_{min}, H_{max} kg_f/cm ; con ≡ continuum , dis ≡ discrete									

Table 1: Comparison between the proposed numerical approach with an analytical approach

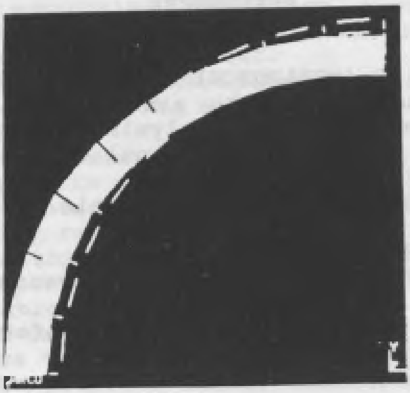

Figure 2: Displacements of the arch loaded by its own weight and by the inferior horizontal limit thrust

Figure 3: Displacements of the arch loaded by its own weight and by the superior horizontal limit thrust

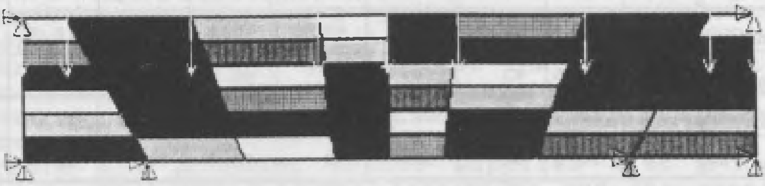

Figure 4: The masonry flat arch non-linear finite element schematization employed

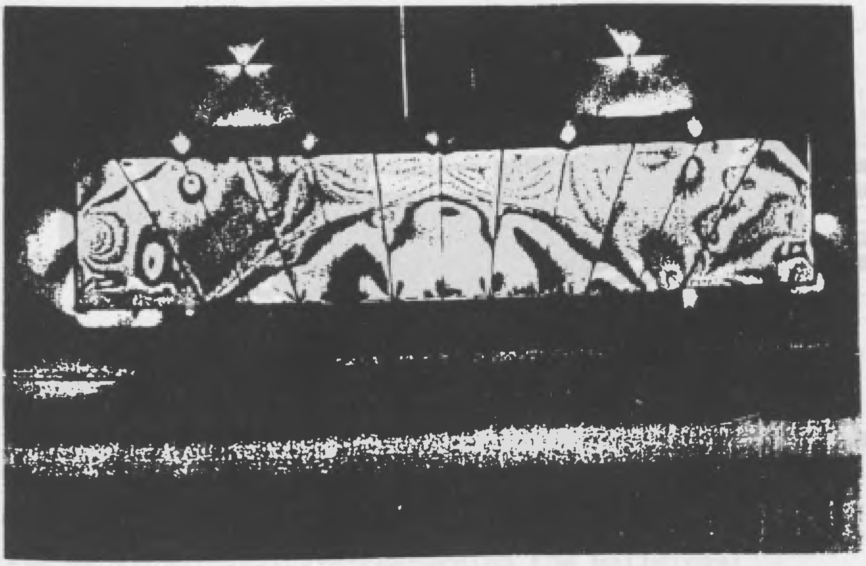

Figure 5: Photoelasticity results of a block flat arch

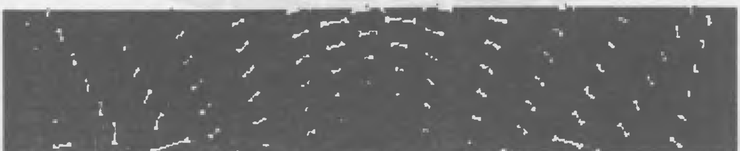

Figure 6: Principal stress directions of the masonry flat arch produced by the proposed schematization

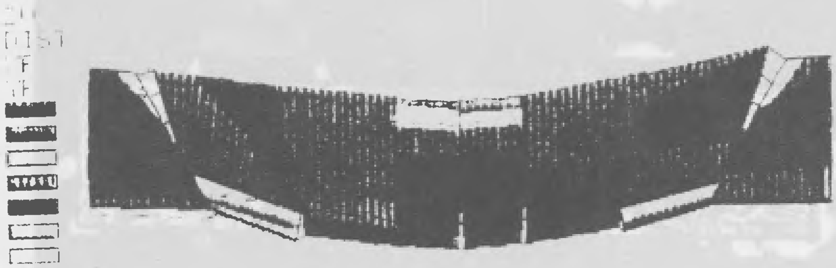

Figure 7: Value of the principal compressive stress obtained

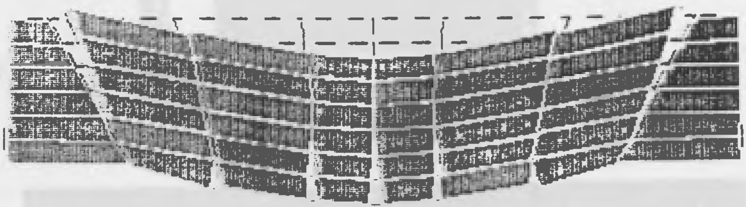

Figure 8: Displacements of the masonry flat arch loaded by the force showed in figure 1, obtained with the model

RENFORCEMENT ET RESTAURATION STATIQUE DES BATIMENTS DE LA MOSQUEE ET DES PRISONS DE LA RESIDENCE DU RECTEUR A LA FORTERESSE DE RETHYMNO.

Tassos Sakellaris - Dimitris Silivridis

SUMMARY

The purpose of this communication is the presentation of the stuctural analysis concerning the buildings: Prisons of the Rector's residence and Mosque situated in the fortress of Rethymno.

The criterion and the adoptation of the appropriate static model as also the method of static analysis constitute the principal points of this communication.

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Athènes, 14 Avril 1989.

1. INTRODUCTION

Le but de cette communication est la présentation de l'analyse statique qui concerne les bâtiments:
a) Bâtiment des Prisons du complexe de la résidence du Recteur.
b) Bâtiment de la Mosquée.
Les deux bâtiments se situent dans la forteresse de Rethymno.

Le bâtiment des prisons de la résidence du Recteur est une construction Vénicienne qui date de la fin du 16ème siècle.

La Mosquée est une construction Ottomane du 17ème siècle.

Les deux bâtiments sont construits en maçonnerie de pierre naturelle.

Les matériaux, le mode de construction ainsi que la géometrie sont illustrés aux figures No1 et No2.

Le sol de la fondation est constitué de roches calcaires.

2. ENREGISTREMENT DE LA SITUATION EXISTANTE

Les deux bâtiments ne présentent pas de degâts importants. Les degâts qui ont été observés sont les suivants :

a) Fissures de caractère local, d'épaisseur 1 a 3 mm environ, se situant surtout au voisinage des ouvertures des bâtiments. (portes, fenêtres etc.).
b) Décollement des joints a l'intrados:
 -des voûtes au bâtiment du recteur
 -de la coupole de la Mosquée.
c) Ecaillage superficiel des pierres d'importance secondaire.

3. TRAVAUX DE RECONNAISSANCE - ESSAIS

A cause des difficultes objectives il a été impossible de réaliser un programme d'essais complet. Les travaux de reconnaissance et les essais effectués sont les suivants:

a) Endoscopie de la maçonnerie.
b) Essais de compression des pierres.
c) Analyse chimique du mortier.
d) Excavations de reconnaissance aux fondations.

4. EVALUATION DES ÉLÉMENTS DISPONIBLES

4.1 Résistance du mortier

Etant donnée l'impossibilité de prendre des carottes, la résistance à la compression du mortier a été estimée a 6Kgr/cm2, en tenant compte de l'analyse chimique, de la bibliographie relative et de l'experience.

4.2 Résistance des pierres

On a effectué au laboratoire des essais de compression simple sur deux catégories de pierre (calcaires-tufs) qui représentent la maçonnerie portante des deux bâtiments.

On a prélevé au total 10 echantillons cubes des dimensions 6X6X6 cm.

Après avoir éliminé les valeurs extrêmes, la résistance à la compression a été determinée comme la moyenne arithmetique des 8 echantillons: $\sigma m = 169$ Kgr/cm2.

4.3 Résistance de la maçonnerie

Les formules semi-empiriques citées ci-dessous ont ete utilisées pour la détermination de la résistance à la compression:

$$fwc = 2/3\sqrt{fbc}\cdot\sqrt[3]{fmc} \quad \text{Kgr/cm2}$$
$$fwc = 2/3\left[\sqrt{fbc} - \alpha\right] + \beta\cdot fmc \quad \text{MPa}$$

Ainsi nous avons obtenu les résistances suivantes:

-Batiment du Recteur: $fwc = 16$ Kgr/cm2 et $\sigma adm = 4$ Kgr/cm2.
-Mosquée : $fwc = 20$ Kgr/cm2 et $\sigma adm = 6$ Kgr/cm2.

La résistance à la traction a été prise égale a 1/10 de la résistance à la compression.

4.4 Module d'Élasticité

4.4.1 Pierres

La formule empirique ci-dessous a été utilisée:

$$E = 600 \div 700\ \sigma e \quad \text{Kgr/cm2.}$$

σe = résistance à la compression des pierres = 169 Kgr/cm2.

$$E = 100.000 \text{ Kgr/cm2.}$$

4.4.2 Mortier

On a utilisé la formule empirique de Wesche :

$$E = 1000\ \sigma e \quad \text{Kgr/cm2}$$

σe=résistance à la compression du mortier = 6 Kgr/cm2

$$E = 6.000 \text{ Kgr/cm2}$$

Cette valeur a été considerée basse et on a adopté comme module d'élasticité du mortier la valeur de 8.000 Kgr/cm2.

4.4.3 Maçonnerie

Le module d'élasticité de la maçonnerie a été déterminé en utilisant les formules empiriques :

-$E_w = \alpha \cdot f_{wc}$ où $\alpha = 1200$ et $f_{wc} = 169$ Kgr/cm2
-$E_w = \beta \cdot E_b$ où $\beta = 0,4$ et $E_b = 100.000$ Kgr/cm2

Ainsi on a obtenu pour E les valeurs suivantes:

-Bâtiment du Recteur : $E_w = 20.000$ Kgr/cm2
-Mosquée : $E_w = 25.000$ Kgr/cm2

5. ANALYSE STATIQUE

5.1 Description du modèle statique

Les criteres utilisés pour la détermination du modèle statique ont été les suivants:

a) L'approximation avec une précision satisfaisante du système porteur de la structure.

b) La compatibilite des résultats de l'analyse statique avec l'etat existant des bâtiments.

c) Les limites financières que le Maître de l'Oeuvre a fixé, sans pour autant compromettre la qualité et l'authenticité du projet.

L'analyse statique a été effectuée sans tenir compte des differences des caractéristiques mécaniques entre la maçonnerie "simple" et celle en pierre de taille.

5.1.1 Bâtiment du Recteur

Le bâtiment du Recteur est constitué de deux "salles" qui ont été etudiées séparement.

La grande salle a été analysée statiquement en utilisant le modèle du cadre plan avec l'hypotèse de base que le système porteur porte à un sens.
Le niveau de la fondation a été pris à 1,50 m environ au dessous du sol naturel, en se basant sur les observations effectuées aux fouilles de reconnaissance. Pour l'analyse statique la fondation a été simulée par un encastrement parfait, tenant compte de la qualité du sol et des dimensions des semelles.

Les murs ont été simulés par des éléments prismatiques de section variable; la voûte par des éléments prismatiques de section constante.La voûte a été subdivisée à huit arcs à fin de obtenir une bonne approximation entre la longueur de l'arc et celle de la corde.

Le modèle statique de la petite salle consiste à une coque cylindrique appuyée aux murs.Le choix de ce modèle a été effectué après avoir essayé divers modèles de cadre plan.Ces modèles ont été exclus parce qu'ils donnaient des contraintes de traction elevées qui ne correspondaient pas à l'état existant du bâtiment.

La voûte a été simulée par une coque mince et a été analysée en utilisant la méthode des éléments finis.(Programme STAAD III).La voûte a été subdivisée à 130 éléments avec un nombre total de noeuds 154 (Fig.3)

Les murs porteurs ont été verifiés comme des portes à faux encastrés au niveau de la fondation.

5.1.2 Bâtiment de la Mosquée

La Mosquée a été analysée statiquement en utilisant le modèle de la coque sphérique (dome),appuyée perimetriquement aux murs.

Le dome a ete analyse pour les charges verticales par la théorie de la membrane en tenant compte des "perturbations" dues aux conditions d'appuis.Les conditions d'appuis ont ete simulées par les 3 systemes statiques suinants:

-Système statique No1:Le bord "C" de la membrane est consideré comme une articulation.
-Système statique No2:Le bord "C" de la membrane est consideré comme un encastrement parfait.
-Système statique No3:La membrane est appuyée sur un anneau sphérique.Dans ce cas nous avons égalisé les déplacements du bord de la memrane avec ceux de l'anneau sphérique.

Pour les murs nous avons adopté deux systèmes statiques:
-Porte à faux encastré au niveau de la fondation et libre à l'autre bord.
-Porte à faux encastré au niveau de la fondation et articulé à l'autre bord.

5.2 Cas de charge

Pour la vérification de la structure aux effets du séisme nous avons adopté les Normes Greques.Le coefficient ε a été pris égal a 0,10 pour tenir compte de l'importance des monuments.Pour le calcul des efforts dus aux effets thermiques nous avons admis une différence de température de ±20° C.

Les différents cas de charge que nous avons adopté pour l'analyse statique sont les suivants:

a) Bâtiment du Recteur - Grande Salle
1. Charges permanentes (P)
2. Charges permanentes (P) + ΔT
3. Charges permanentes (P) - ΔT
4. Charges permanentes (P) + Séisme
5. Charges permanentes (P) + Séisme + ΔT
6. Charges permanentes (P) + Séisme + $P(1+3\varepsilon)$*
7. Charges permanentes (P) + Séisme + $P(1-3\varepsilon)$*

* appliquée a la voûte.

b) Bâtiment du Recteur - Petite Salle
1. Poids propre
2. Poids propre + charges permanentes
3. Poids propre + charges permanentes + Séisme (axe X)
4. Poids propre + charges permanentes + Séisme (axe Y)

c) Mosquée
1. Charges permanentes (coque)
2. Charges permanentes + Séisme (pour les murs)

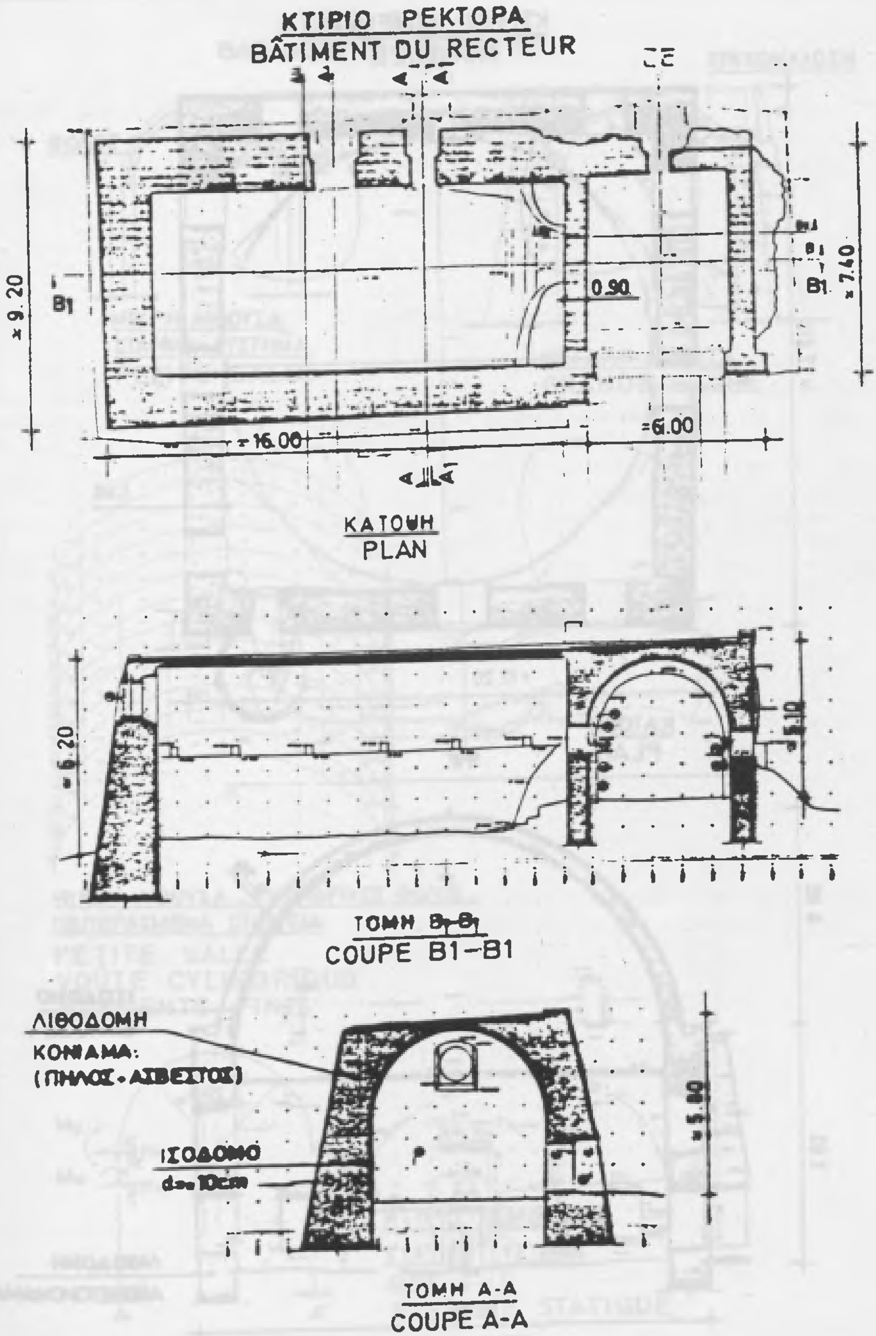
5.3 Résultats de l'Analyse Statique

L'analyse statique nous a donné les efforts M,Q,N. Les contraintes ont été calculées par la formule classique de la Résistance des Matériaux: $\sigma 1,2 = -N/F \pm M/W$.

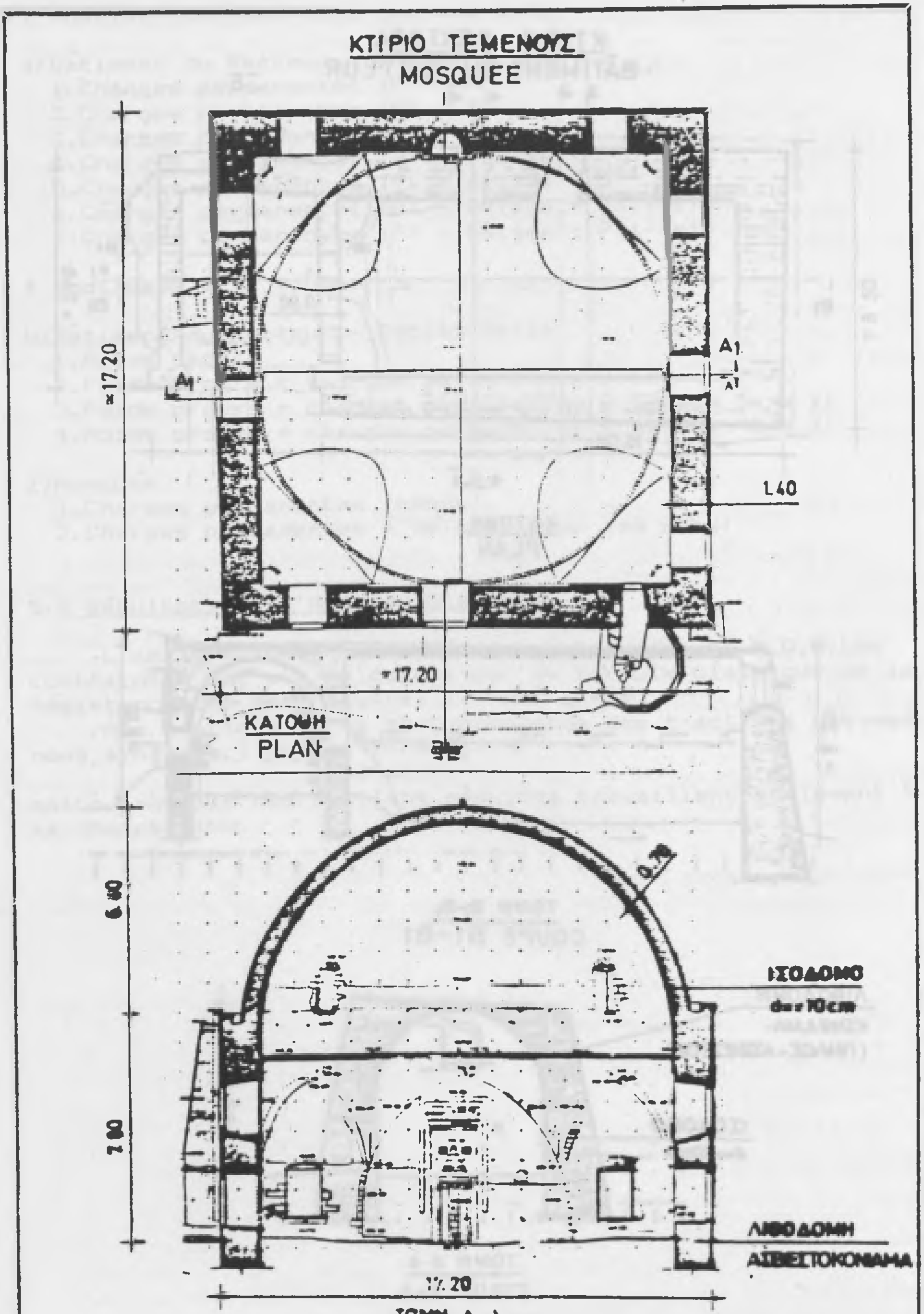
Aux sections où se sont presentés des tractions elevées nous avons appliqué la formule:

$$\sigma = 2N/3cb$$

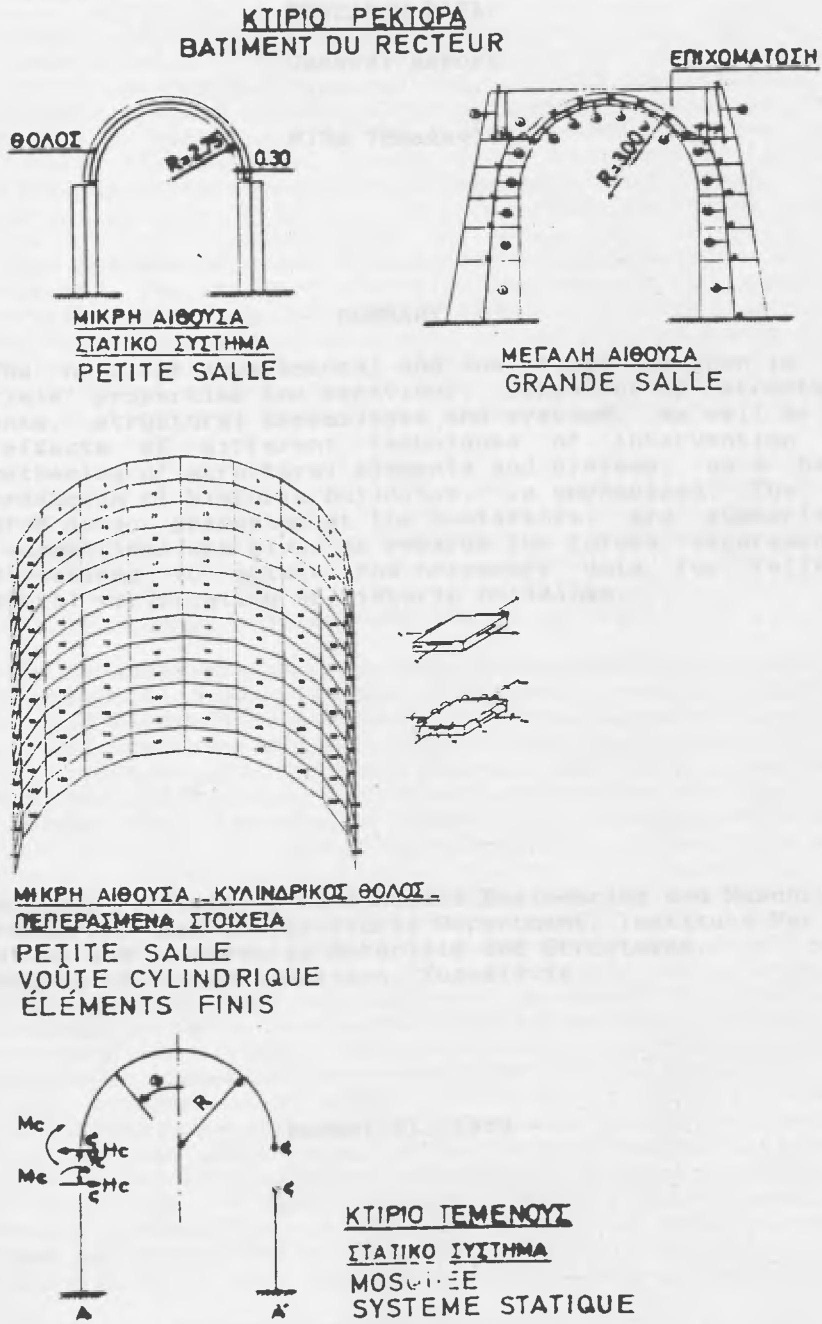
selon laquelle des sections réduites travaillent seulement à la compression.


ΚΤΙΡΙΟ ΡΕΚΤΟΡΑ
BÂTIMENT DU RECTEUR
≈ 9.20
B1
0.90
B1
≈ 7.40
≈ 16.00
≈ 6.00
ΚΑΤΟΨΗ
PLAN
≈ 6.20
5.10
ΤΟΜΗ B1-B1
COUPE B1–B1
ΛΙΘΟΔΟΜΗ
ΚΟΝΙΑΜΑ:
(ΠΗΛΟΣ + ΑΣΒΕΣΤΟΣ)
ΙΣΟΔΟΜΟ
d≈ 10cm
≈ 5.80
ΤΟΜΗ A-A
COUPE A-A

-ΣΧ.1- Fig. 1


ΚΤΙΡΙΟ ΤΕΜΕΝΟΥΣ
MOSQUEE
A1
A1
1.40
≈17.20
≈17.20
ΚΑΤΟΨΗ
PLAN
ΙΣΟΔΟΜΟ
ΛΙΘΟΔΟΜΗ
ΑΣΒΕΣΤΟΚΟΝΙΑΜΑ
17.20
ΤΟΜΗ A1-A1
COUPE A1-A1

-ΣΧ.2 -Fig.2


ΚΤΙΡΙΟ ΡΕΚΤΟΡΑ
BATIMENT DU RECTEUR
ΕΠΙΧΩΜΑΤΩΣΗ
ΘΟΛΟΣ
R=2.75
0.30
R=3.00
ΜΙΚΡΗ ΑΙΘΟΥΣΑ
ΣΤΑΤΙΚΟ ΣΥΣΤΗΜΑ
PETITE SALLE
ΜΕΓΑΛΗ ΑΙΘΟΥΣΑ
GRANDE SALLE
ΜΙΚΡΗ ΑΙΘΟΥΣΑ - ΚΥΛΙΝΔΡΙΚΟΣ ΘΟΛΟΣ -
ΠΕΠΕΡΑΣΜΕΝΑ ΣΤΟΙΧΕΙΑ
PETITE SALLE
VOÛTE CYLINDRIQUE
ÉLÉMENTS FINIS
Mc
Hc
Mc
Hc
R
A
A'
ΚΤΙΡΙΟ ΤΕΜΕΝΟΥΣ
ΣΤΑΤΙΚΟ ΣΥΣΤΗΜΑ
MOSQUEE
SYSTEME STATIQUE

-ΣΧ. 3 - Fig. 3

REDESIGN OF REPAIRED AND STRENGTHENED STRUCTURES: RESEARCH DATA

General Report

Miha Tomaževič*

SUMMARY

The need for experimental and analytical research in the materials' properties and behaviour, behaviour of structural elements, structural assemblages and systems, as well as in the effects of different techniques of intervention and strengthening of structural elements and systems, as a basis for redesign of historic buildings, is emphasized. The new research data, presented at the Conference, are summarized, and suggestions are given as regards the future experimental investigations to obtain the necessary data for reliable structural verification of historic buildings.

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August 11, 1989

INTRODUCTION

Besides wood, masonry is the most important construction material in the history of mankind. Masonry buildings, both residential and public, ordinary and monumental, have been known for several thousands years. Maybe not the eldest, but some very old masonry buildings of all kinds still exist, proving that masonry can successfully resist the effects of time, providing shelter and serving other needs of people for a long period of time.

Old urban and rural nuclei with their monumental buildings represent a cultural and historic heritage of the highest category. Without historic buildings, most modern cities and towns would have no spirit, charm or interest. It is therefore the policy of most countries to keep old nuclei alive by attracting residents to live there, and by protecting the historic monuments from deterioration of any kind.

Since the living conditions in historic buildings are often unsatisfactory, these buildings should be renewed, with the introduction of modern facilities and the rearrangement of individual apartments to meet modern living standards. The renewal of old buildings, which often requires interventions in their structural systems, is the only way to preserve historic buildings for future generations.

Historic buildings must sometimes be strengthened because of deterioration of their structural materials due to the effects of time, which have nowadays accelerated because of bad atmospheric conditions (air pollution), humidity, etc. Sometimes, buildings must be repaired after earthquakes and sometimes strengthening is required to protect them against future earthquakes, for many urban and rural nuclei are located in zones of high seismicity.

The structural strengthening of historic buildings is a part of the complex, multidisciplinary procedure of restoration and conservation of cultural monuments. As such, it cannot be looked at, or solved independently. The structural engineer must be one of the equal members of a team of experts (archaeologists, art historians, conservators, architects, etc.), and his proposals should be incorporated into the process of restoration of buildings. Whenever an intervention in the structural system of an historic building is required, the principles of preservation and restoration of cultural monuments should be taken into account. These principles, however, severely limit the application of many possible technical interventions, and do not permit the reconstruction or replacement of many structural elements typical for the erection period of the building with elements made of modern construction materials.

Speaking of cultural and historical monuments, it can be seen that they include a wide variety of structural types and materials. Although residential, public and monumental historic buildings are all masonry buildings, monumental buildings in general require a specific approach as regards the intervention in their structural system. In many cases, sophisticated technologies must be used for their structural strengthening and complicated mathematical procedures are necessary for structural verification.

Knowledge of different possible techniques of strengthening the existing structural systems and their effects on the improvement of the stability of structural systems of historic buildings, is therefore of relevant importance. This is especially the case of buildings constructed in seismic regions, where, if an earthquake occurs, the structural system will suffer a series of cyclicly acting horizontal loads, which might often exceed the elastic limit of the structure. The structure will be damaged, and, if not properly strengthened, the induced seismic energy will cause the collapse of the building.

RESEARCH AS THE BASIS FOR REDESIGN OF HISTORIC BUILDINGS

General Considerations

Masonry has been used as a handicraft material from old. With exception of monumental buildings, which have been sometimes designed on the basis of experiments and theory of structures, masonry buildings have been constructed on the basis of experience. The walls, in the case of large openings replaced by arches, have been provided to carry vertical loads only, resulting from the walls' dead and live load. The load-bearing capacity of walls has been determined by their thickness, depending on the quality of materials at disposition. In seismic-prone regions, the builders, having in mind the destructive power of earthquakes, sometimes took care for earthquake resistance of buildings by means of using connecting stones, connecting the two bearing layers of stone-masonry walls at regular intervals, or using regular sized coursed stones for better connections of walls at the corners or intersections. Also, the use of iron ties, connecting the walls horizontally at floor levels, is frequently evident.

Since most of the historic buildings are built on the basis of experience, few - if any - data exist about the actual quality of structural materials and the resistance of their structural elements and/or systems to vertical and horizontal loads, which are needed in the decision-making process of intervention in the structural systems of the buildings to be brought to the requirements of modern design.

Moreover, no data exist a priori about the effects of different technical measures developed and proposed for structural strengthening of historic buildings. Until few decades ago, interventions in the structural system of hystoric buildings have been carried out on the basis of the designer's feelings and experience (as was the case of buildings when originally constructed), without exactly knowing what caused the damage to buildings' structures and what will be the effects or the consequences of the applied technical solution on the behaviour of the building. The mechanical properties of the materials were not known, and the effects of the proposed strengthening techniques had not been experimentally verified.

In seismic-prone regions many recent earthquakes have shown that unprofessionally strengthened, renewed or restored historic buildings suffer the most severe damage. Despite that, the approach to the structural strengthening of the buildings from the seismic resistance point of view is still frequently inadequate. Fatal errors are sometimes made by merciless interventions in the buildings' structural systems: load-bearing walls are removed and not replaced with other structural elements, inadequate and ineffective strengthening techniques are used. Also, numerical procedures, not suitable to the structural system under consideration are often used for the redesign of structural elements and systems, giving inaccurate estimates on the actual resistance of the structure.

Since masonry is a non-elastic, anisotropic and non-homogeneous material, it is generally impossible to predict the behaviour of masonry buildings by using the values of mechanical properties of the constituent materials (stone, brick and mortar) as the input parameters in the calculation procedures, based on the assumptions of the theory of elasticity. This is especially true if the ultimate behaviour of historic buildings subjected to seismic loads has to be predicted, although many sophisticated, FEM (Finite Element Method) based computer programs are nowadays available and practically used.

The correct intervention in the structural system requires an understanding of the mechanism of the behaviour of the building subjected to vertical and/or lateral loads, as well as the knowledge of the mechanical properties of its building materials. By understanding the causes of structural damage, and by knowing the actual resistance of the building and its structural elements to the expected loads, adequate technical solutions for strengthening the building can be developed, which of course should be experimentally verified.

In order to evaluate the resistance of the building to vertical and horizontal loads, limit state methods are usually used, and experimentally determined values of the mechanical properties of masonry materials are taken into account as the input parameters. In order to obtain accurate results, the mathematical models used in the calculations must reflect the actual behaviour and failure mechanism of historic buildings when subjected to vertical and horizontal loads.

Structural Observation

It is evident that investigations of different kinds are needed to obtain the necessary data for structural verification and redesign of historic buildings. Some basic informations on the behaviour of their structural system under different loading conditions can be obtained by systematic observations of buildings during a prolonged period of time. For this purpose, instruments are installed at the characteristic points of the building, measuring, for example, the changes of crack width with temperature and time, or displacements and settlements of the building. The measured data are analyzed and conclusions are made as regards the necessary intervention in the structural system.

If the seismic behaviour of historical buildings is studied, the analysis of structural damage to buildings, caused by the previously occurred earthquakes, can clearly define the weak and the good points of different structural systems. On the basis of the analysis of damage, the failure mechanisms of structural elements, as well as of the complete structural systems, can be defined.

However, quantitative data about the resistance of the building cannot be assessed unless additional experimental investigations to establish the material properties and structural characteristics are carried out.

Determination of Mechanical Properties of Existing Masonry Materials, Tests of Walls and Other Structural Elements

As a rule, it is not possible to determine the mechanical properties of the masonry walls by testing their constituent materials in the laboratory. It is also difficult to reproduce the existing masonry walls in the laboratory, even though very thorough chemical and mechanical tests of the mortar and other constituent materials may have been carried out. The only really satisfactory method of determining the load-carrying capacity of old masonry walls involves the carrying out of tests in situ, or the cutting out specimens from the walls and testing them in the laboratory.

Therefore, the mechanical properties of masonry and not of the constituent materials are determined and subsequently used in structural design.

In order to obtain the values of compressive strength and elastic modulus of existing masonry walls, simple compression tests of masonry prisms and walls of different dimensions and shapes are carried out. Usually, specimens are cut out of the buildings and tested either in-situ or in the laboratory by subjecting them to monotonic, increasing vertical loads. Flat-jacks are sometimes used to determine the existing stress state in the critical structural elements of the structural system under observation and different non-destructive methods are used to investigate the structure of the observed masonry wall.

In order to obtain accurate data as regards the lateral resistance of the existing walls, the wall specimens should be tested at the same loading conditions as in the building. Usually, the walls are tested as symmetrically fixed at both ends and are subjected to constant vertical and cyclicly acting horizontal load (programmed horizontal displacements). On the basis of the recorded relationships between the lateral load and horizontal displacements the parameters which define the wall's behaviour, can be determined.

Sometimes, however, simple shearing (shove) tests are carried out. In this case, a single brick is separated laterally from the surrounding masonry, and pushed in one direction by hydraulic actuator.

Unfortunately, different results are obtained by different methods of testing. The differences are due to the different testing procedures used for the determination of the tensile or the shearing strength of the masonry, which cause different stress states in the tested specimens, and, consequently, different failure mechanisms. The data, found in the literature should therefore not be taken for granted, but carefully examined and correctly taken into account in further calculations.

Principal tensile stresses develop in the wall when subjected to a combination of vertical and lateral, seismic loading, whereas pure shear stresses develop in the mortar joints in the case of the shearing strength test of the small, one brick sized specimen. Knowing that principal tensile stresses define the tensile strength of the masonry, and, consequently, the shear resistance of the wall, the latter can be overestimated if the results of the shearing strength tests are directly used in the calculations.

The behaviour of other masonry structural elements, such as arches, vaults, and domes should be also investigated experimentally, by subjecting the tested elements to similar loading conditions as in the buildings.

<u>Determination of Dynamic Properties and Failure Mechanisms of Structural Systems and Assemblages</u>

In order to evaluate the magnitude of the expected seismic forces, developed in the building during an earthquake, the dynamic properties of the building should be known. By knowing the dynamic properties of the building, the expected seismic loads can be evaluated from the known response spectra of the earthquakes, expected at the building's location.

Different experimental procedures have been developed and are available to determine the fundamental dynamic characteristics of the existing structural systems, such as fundamental frequencies and mode shapes. Sometimes, ambient vibrations of the building are measured, and sometimes, the vibrations of the building are caused by vibrators and the response of the building to forced vibrations is measured.

By comparing the measured values of natural frequencies and mode shapes of the building to the calculated values, the assumed mathematical model of the considered structural system can be verified and corrected, when necessary. Since testing is done in the elastic range (small amplitudes of vibrations, low level of dynamic excitation), in this way, the elastic behaviour of buildings can only be determined.

If the failure mechanism of a given building type is not known, shaking-table tests or pseudo-dynamic tests should be carried out by subjecting the models of buildings to simulated seismic ground motion, or by testing the prototype-sized buildings or their structural assemblages by subjecting them to simulated seismic loads.

On the basis of experimental data, adequate mathematical models for structural verification and design of existing historic buildings can be developed and reliable data about their resistance to vertical and seismic loads can be obtained.

Experimental Verification of Strengthening Methods

In order to be widely used, the proposed methods for repair and strengthening the historic buildings must ensure the required degree of load-bearing capacity of structural elements and/or structural systems, but should be, at the same time, also simple and economical to carry out. It is easy to see if the technological solution is simple to carry out, as it is also easy to show its economical effects. However, experiments are the only way to obtain reliable data about the effects of different proposed technological solutions on the improvement of load-bearing capacity and the overall behaviour of the building's structural system. The experiments are the only way also to obtain information on the effects of time on the behaviour of different modern materials used in the process of structural intervention.

Different testing procedures, both destructive and non-destructive are used for the verification of the proposed strenthening methods. Usually, destructive methods are used for the determination of the improvement in load-bearing capacity and overall behaviour of individual structural element or the entire structural system. Non-destructive methods, however, are mainly used to control the workmanship (as is the case of cement-grouting, etc.).

Experimental data on the effets of strengthening methods are used in the process of redesign of historic buildings. Without being supported with experimental data, the results of calculations would have in most cases no practical value.

Analytical Research and Parametric Studies

Frequently, analytical methods which are based on the previous experimental investigations, are used to study the behaviour of historic buildings under different loading conditions.

If, for example, the seismic vulnerability of a group of similar historic buildings has to be assessed, and the decision should be made on the necessary structural intervention, there is no need to investigate all buildings experimentally. Since the experiments are usually very expensive, the material and structural characteristics of only the representative building in the group are investigated in detail. Mathematical model is then formulated, based on the results of experimental investigations, and the resistance of the rest of the buildings is estimated by calculation.

The conclusions are drawn and the consequent technical measures for structural intervention are verified by changing the respective parameters in the calculations. Having the results of the parametric study at disposition, only complementary experimental investigations and tests are needed when the individual buildings are studied and redesigned.

Experiments Versus Analysis in Redesign of Historic Buildings

In the case of structural redesign of historic buildings, the accurate data about the mechanical and other properties of structural materials should be taken into account. Laboratory and in-situ testing of materials is the only way to obtain these data.

It has been already emphasized that strong connection exists between experimental research and analytical procedures developed for the redesign of historic buildings. Because of the complexity and non-homogeneity of structural materials and structural systems used for the construction of historic buildings, it is many times not possible to rely upon the assumptions of classical, elastic theory of structures. The assumptions of the theory of elasticity can give an information about the state of stress in the structural system, if the mechanical properties of structural materials are known, but cannot assess the actual safety of the building. The latter can only be estimated if the analytical models which take into account the possible failure mechanisms are applied in the calculations. These models, however, can be developed only on the basis of experimental research.

When redesigning the building, the correct assumptions should be also taken into account as regards the effects of the proposed strengthening techniques as well as as regards the interaction between the newly applied structural elements or materials and the existing ones. Again, the correct data can be obtained only by experiments.

There is of course no need for experimental investigations if an experimentally-supported analytical method is used in redesign or if data exist about the effects of the applied technical solutions. However, in each specific case, the mechanical properties of existing materials of the building considered in redesign should be verified by appropriate testing the specimens in the laboratory or in-situ.

NEW DATA PRESENTED AT THE CONFERENCE "STRUCTURAL CONSERVATION OF STONE MASONRY"

Recognizing the importance of experimental research in the process of structural conservation of historic buildings, several authors report on their research findings at the Conference "Structural Conservation of Stone Masonry". The contributions submitted to the Conference do not cover all possible aspects of experimental research related to engineering problems of interventions in the structural systems of historic buildings. Nevertheless, the papers presented represent an important contribution to the solution of many unknowns appearing in the rapidly developing field of structural engineering, which is structural conservation and redesign of stone masonry buildings. Therefore, the contributions presented at this Conference will make its Proceedings an important source of information for everyone, working in the field.

In the following, the papers which cover Topics 2.1 a), b), d), and e) of the Conference's program will be summarized. As stated above, in the redesign of historic buildings, it is often not possible to separate the experimental and analytical (Topic 2.1 c) part of the design. Therefore, there are papers which discuss both, experimental and analytical, parts of the redesign process. There are also papers which report experimental data, but discuss the problems of Topics 1 and 3. If not summarized here, all these papers will be mentioned in other General Reports.

Research In Materials' Behaviour

C.Blasi and S.Sorace (Cracks in Marble Due to a Long Term Small Tensile Stress Configuration) investigated the causes of cracks, developed in marble structural elements of ancient Roman monuments. The elements, where the cracks can be observed, are subjected to tensile stresses much smaller than the material's tensile strength. The authors conducted a series of instantaneous and long-term bending tests on specimens, made of the stone material of the remains of an ancient Roman monument and of the Carrara marble of good quality. Long-term load represented 85%, 75% and 55% of the instantaneous flexural capacity of the two different materials (long-term tests of the third series of specimens are still under way). The analysis of test results has shown a very severe effect of the magnitude and duration of load on the material's permanent tensile strenght. It has been found that long-term tensile stresses amounting to 25% of the value of tensile strength of the material, obtained by instantaneous loading, might have caused the collapse of marble structural elements of ancient monuments. Therefore, the authors suggest that, when redesigning the structural systems built in this material, the time effects should be taken into account.

It would be interesting to know, to what extent the dynamic effects (ambient vibrations due to traffic and other disturbances of modern life) accelerate the degradation of permanent tensile strength of marble in structural elements of historic buildings.

R.Egermann (Influence of Manufacturing Process on the Mechanical Properties of Brick) studied the influence of anisotropy in the brick on its basic mechanical properties, compressive and splitting tensile strength. The mechanical behaviour of old hand-moulded and modern extruded bricks has been experimentally investigated on a series of drilled cores, taken out of the bricks in different directions. The mechanical properties of extruded and hand-moulded bricks, burned at different temperatures, have been compared to the mechanical properties of old bricks, taken out of the existing buildings. Interesting results of mechanical tests have been obtained, showing strong effects of modern brick manufacturing process (extrusion) on the anisotropy of brick's mechanical characteristics. Greater compressive strength has been found if the cores were drilled in the direction of extrusion than in the case of orthogonally drilled cores (the differences were less in the case of larger pressure of extrusion). Still greater differences have been obtained as regards the Young's modulus and the Poisson's ratio. In the case of hand-moulded and old bricks, however, the stress-strain diagrams obtained in different directions show almost no difference.

C.Zavliaris (Adhesion Between Stonewall and Polymers) discusses the adhesion between stone and mortar, impregnated with the polymer-based injections. Adhesion between the two constituent part of the masonry is an important parameter which influences the compressive and shear strength of a stone masonry wall: by improving the adhesion between stone and mortar with polymer-based injections, masonry can resist to higher splitting forces developed in the masonry wall at any combination of vertical or vetical and horizontal load. In the paper, a review of existing equations of fracture mechanics to analyze the shear joints is given, and the calculated values of adhesion shear strength are compared to the experimental ones. Good correlation between the theoretical and experimental results as regards the ultimate shear strength is reported.

K. van Balen and D. van Gemert (Long-Term Creep Behaviour of Grouted Stone Masonry. The Experience at the Church of Our Lady at Tongeren, Belgium) investigated the long-term creep effect of epoxy-resin grouted masonry. Creep was measured on two cylindrical cores, bored out of the previosly grouted masonry, and subjected to a permanent compressive load for a period of 8 months. On the basis of test results, the parameters of the Bazant's creep function have been determined and the expected creep of a grouted masonry core of the tower of St.Mary' Church at Tongeren predicted. It was found that the load, carried by the inner core of a composite masonry wall will, after a certain period of time, result into strain and increase in stresses in the outer layers of the wall. Therefore, it has been concluded that an overall strengthening of the composite masonry wall be carryied out.

The paper gives an interesting, experimentally based evidence on the possible redistribution of permanent loads to structural elements, strengthened in a different way. Since the redistribution of loads, if unexpected, may have non-desirable consequences on the behaviour of the strengthened building, the paper is a reminder that the effects of time should be taken into account in the process of redesign, when structural intervention which provides new materials or structural elements to be implemented in the building, is to be carried out.

M.Karaveziroglou, J.Papayianni and G.Penelis (Time Dependent Deformations of Brick Masonry) report their research in creep and shrinkage of masonry which they have carried out in order to obtain data about the time dependent behaviour of the restored masonry buildings. They constructed a series of 24 cylinders, and varied the thickness of bricks and mortar joints, as well as the mortar mixes. After curing for one month at the same conditions, the specimens were subjected to constant compression in spring loaded creep frames at an previously estimated stress/strength ratio of 0.25. The measurements of strain were carried out for a period of 180 days. It has been found that the specimens built with lime- and Santorine-earth-based mortars exhibit larger long-term deformations (more than 1%) than the specimens built with mortars with crushed brick. The creep deformations are also considerably depending on the thickness of mortar joints and the moisture, especially when cement was used as a constituent part of the mortar mix. In the latter case, the prevention of moisture loss can reduce creep deformations by 50%.

Research in Dynamic Properties of Structural Systems

C.Blasi and P.Spinelli (Dynamic Investigation Techniques For Restoration Design: an Application at the Temple of Castore and Polluce) discuss the dynamic behaviour of monumental buildings with structural systems consisting of simply superimposed rigid blocks (columns, colonnades). In order to evaluate the seismic vulnerability, to develop a suitable mathematical model and to propose the adequate solution for aseismic strengthening of this specific structural type, the dynamic characteristics of the remains of the Castor and Pollux Temple in Rome have been determined by means of a small vibrator. The natural frequencies and mode shapes of the columns have been measured in transversal and longitudinal directions. However, excited in the small amplitudes of vibration, no rocking motion has been observed and identified by the spectral analysis of the forced vibration records. The collected data have been used for the formulation and calibration of mathematical model, which is based on the "deformable cushion method". The following assumptions characterize the mathematical model: the stone blocks are considered rigid and are supported by fictitious, so called Winkler cushions; shear resistance of joints is infinite; vertical displacements of blocks are neglected. In the paper, the seismic response of the investigated colonnade to typical earthquakes, recorded in the zone, is also presented. Regrettably, no evidence is given as regards the correlation between the calculated and experimental results.

A.Sinopoli (Dynamic Evolution by Earthquake Excitation of Multiblock Structures) discusses the dynamic response of the same monolithic stone structural systems to seismic ground motion as discussed in the contribution of C.Blasi and P.Spinelli. The approach to the theoretical solution of the problem is similar, but the boundary conditions between the constituent elements are different: the structure is considered as a system of simply superimposed rigid blocks, simply supported on a rigid ground. Consequently, the forces defining the dynamic behaviour of the system subjected to ground motion, are gravity forces and dry friction, resulting into coupled, rocking and sliding motion of the individual parts of the system. Because of the complexity of numerical analysis, the author investigated different simplified cases of the dynamic response of the system to ground excitation: the response of the system to impact; the sensitivity of the rocking response of the system to the sine wave ground motion in the absence of sliding between the blocks; the sensitivity of the coupled response of the system to impact. It is mentioned that experiments have ben carried out to verify the mathematical model. However, no description of tests is given in the paper and only qualitative agreement between the theoretical and experimental free motion of the tested marble block is reported. Further investigations to obtain better correlation between experimental and theoretical results are in progress.

<u>Research in Intervention Techniques</u>

R.Žarnić (<u>Strengthening of Masonry Vaults by Foam Concrete Application</u>) investigated the seismic resistance of a frequent structural element in historic buildings, masonry barrel vault. The behaviour of both original vaults with gravel topping and strengthened vaults with foam-concrete replacing the gravel topping, subjected to monothonic vertical load and to a combination of constant vertical and cyclicly acting horizontal load, has been studied. Vaults were supported by masonry piers. A significant difference in the behaviour and load-bearing capacity of the original and strengthened vaults at both vertical and lateral loading conditions has been observed. The foam-concrete topping, reinforced with steel reinforcing mesh, behaved as a slab and prevented the formation of hinges in the vaults, hence significantly improving the resistance of the vault to both vertical and horizontal load. In the case of strengthened vaults, the shear and flexural resistance of the supporting masonry piers became the critical parameters which determined the load-bearing capacity of the structural system.

Regrettably, besides experimental data obtained on the tested model structures, no suggestion is given as regards the calculation of the lateral and vertical load-bearing capacity and deformability of the tested structure.

CONCLUDING REMARKS

As can be seen, the presenting authors have concentrated their efforts to investigate the material properties, which cannot be determined by using the standardized testing procedures. The experimental investigations include the study of time-dependent phenomena of classical and modern masonry building materials, or a combination of both, the knowledge of which is extremely important for the prediction of the long-term behaviour of the intervention in the structural system of the building.

An attempt has been also made to mathematically model the dynamic behaviour of the specific structural system of many ancient monuments (simply supported, superimposed rigid blocks). Experiments have been carried out to verify the proposed mathematical models. However, not much evidence is shown on the correlation between the experimental and theoretical results.

A recently developed method for strengthening the masonry vaults has been experimentally verified.

Although most of the topics of the Conference's program have been represented by at least one or two contributions, not all open questions arising in the everyday practice of redesign of historic stone masonry buildings have been answered. Although considerable experimental research related to the problems of structural conservation of historic stone masonry buildings has ben carried out in the last decade and presented at different other occasions, there are still many problems left to be solved in the future.

Systematic and coordinated research is needed to develop standardized procedures and methods for the determination of basic mechanical properties of masonry materials and masonry structural elements, subjected to vertical and horizontal loads. It has been already mentioned that different methods of testing give different results: in order to make easier the redesign of historic buildings, the data, accessible in the existing data-banks (literature) should be obtained in the same way.

The correlation studies between the results of destructive and non-destructive methods to test the mechanical properties of masonry should be accelerated. By knowing the correlation factors between the results of the two methods, non-destructive methods could be used for the determination of material properties, what will minimize the interventions (and damage) in the existing structures of historical monuments, and will also make testing less expensive.

Standardized procedures (load histories, size and shape of specimens, boundary conditions) are also needed to establish the parameters of seismic resistance of structural elements and assemblages of historic buildings (walls, columns, arches, vaults, etc.). By knowing the relationships between the effects of different loading histories on the behaviour of tested specimens, the sometimes very complex and expensive experiments could be simplified.

Experimental data on the behaviour of historic buildings and monuments subjected to seismic loads are still scarce. As indicated by contributions to this Conference, the relatively easily obtainable data about the elastic behaviour of existing structures (ambient vibration techniques, forced vibration methods) are not sufficient to establish mathematical models for the prediction of their ultimate behaviour. Therefore, shaking-table tests of models of historic stone masonry buildings should be carried out, and the ultimate behaviour of the models should be studied by subjecting them to simulated earthquake ground motion.

Specifically, the seismic behaviour of old residential and public buildings in historic urban nuclei, which are - at present - in many countries renewed and reconstructed on a large scale, is not yet systematically investigated. The influence of different types of floors (wooden joist, masonry vaults, rigid slabs etc.) and tying of the walls on the seismic resistance of these buildings is still estimated by more or less inaccurate theoretical models.

To conclude with, many old and new methods for strengthening the structural walls and other load-bearing elements of historic buildings, which are widely used, have never been experimentally verified. Many design equations, which are used in the redesign of historic buildings, have little - if any - experimental background. They are simply based on the assumptions of the theory of elasticity. This is also the case of methods, where modern materials are applied: the contributions presented at this Conference have shown the importance of experimental research, whenever composite structural materials or elements are applied in the structural intervention for the first time.

Experimental research, both fundamental, to obtain data necessary for the development of accurate and effective theoretical methods for structural redesign, and applied, to obtain data about the effects of different strengthening techniques and methods, plays an important role in the process of structural strengthening of historic buildings. Therefore, in the last several years - as the contributions submitted to this Conference indicate - experimental research in materials and structures of historic buildings has become recognized as an inevitably part of the multidisciplinary process of conservation and preservation of historic monuments. Hopefully, interesting research data will be also presented at future conferences of this kind.

Specifically, the seismic behaviour of old residential and public buildings in historic areas is crucial, which are - at present - in many countries repaired and reconstructed on a large scale, is not yet sufficiently investigated. The influence of different types of floors (wooden joist, masonry vaults, rigid slabs etc.) and types of the walls on the seismic resistance of these buildings is still estimated by more or less inaccurate theoretical models.

In general, very many old and new methods for strengthening and structural repair of the load-bearing elements of historic buildings, used for many years, have never been experimentally verified. Many design regulations, which are used in the redesign of historic buildings, have little - if any - experimental background. They are simply based on the assumptions of the theory of elasticity. This is also the case of methods, where modern materials are applied. The contributions presented at this Conference have shown the importance of experimental research, whenever composite structural materials or elements are applied in the structural intervention for the first time.

Experimental research, data concerning ... in general, data necessary for the development of accurate and effective theoretical methods for structural redesign, are needed. To obtain data about the effects of different strengthening techniques and methods, plays an important role in the process of structural strengthening of historic buildings. Therefore, in the last several years - as the contributions submitted to this Conference indicate - experimental research in materials and structures of historic buildings has become recognized as an important part of the multidisciplinary process of conservation and preservation of historic monuments. Hopefully, interesting research data will be also presented at future conferences of this kind.

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LONG-TERM CREEP BEHAVIOUR OF GROUTED STONE MASONRY. THE EXPERIENCE AT THE CHURCH OF OUR LADY AT TONGEREN, BELGIUM

VAN BALEN KOENRAAD (*)& VAN GEMERT DIONYS , (**)

ABSTRACT

To evaluate the long term creep of resin grouted masonry an 8 month creep test has been executed on two cylinders of grouted stone masonry, bored out of the St. Mary's Basilica tower at Tongeren, Belgium.

The creep of the grouted masonry has been measured and an extrapolation has been elaborated according to the relationship proposed by Bazant.

Experimentally measured and mathematically predicted creep evolution fit very well. The important creep values after consilidation show the necessity of a good overall consolidation of the masonry.

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INTRODUCTION

The construction of the St. Mary's Basilica at Tongeren started in 1240 (Derricks, 1986, ref.1). The new church had to replace the old Romanesque church, which was burned at the winning of the tower by the army of Duke Henry of Brabant. The tower of the old church was kept in use until 1500. In 1442 the construction of the new Gothic tower was started, and it was finished in 1582 with a wooden spire. In 1598 this spire was hidden a first time by a lightning and completely destroyed by fire. After a complete restoration at the beginning of the 17th century, the church was again destroyed by the army of King Louis XIV of France in 1677. In 1687 the necessity of consolidation works on the tower has been mentioned for the first time. The works were executed at the beginning of the 18th century. In 1845 the church-fabric ordered a new and urgent restoration of the tower, because the 18th century consolidation had revealed to be unsufficient. These works were executed between 1848 and 1858. Finally in 1871 it was decided to replace the wooden spire by a supplementary storey on the tower. Its construction was started in 1877 and was ended in 1882. That situation still exists today, Fig. 1.

Figure 1. St. Mary's Tower today.

Figure 2. Cracks in buttress of SW pillar of tower.

Untill 1877 and with the wooden spire the tower had a height of 73 m of whom 47.5 is stone masonry. After the renovation of 1882 the tower reaches a height of 52.5 m, completely in stone. But the renovation would not be the end of the problems. On the contrary, it was the cause of new damages.

The original stone used was the local marl stone. It is a soft stone, with a strength of 2.5 to 4.5 N/mm, and a density of 1400 kg/m^3. For the construction of the upper storey with a height of 9 m the Gobertange limestone was used, with a density of 2300 kg/m^3. Although in the construction project the lower part of the tower was partly strengthened to take up the loads from the new upper storey, severe cracking of the tower walls occurred, so that the church-fabric was forced already in 1905 to call for a new restoration.

These calls were finally put into a new restoration project in 1951, and the works could start in 1972, under the direction of the architect G. Janssen. But already during the restoration works of the paraments important cracking occurred in the newly restored parts, Fig. 2.

Crumbling of stones and the appearance of new cracks showed that the tower structure was still moving. Therefore the architect called for the assistance of the Engineering Office Groep Swartenbroeckx n.v. at Hasselt. In collaboration with the Reyntjens Laboratory of K.U.Leuven the office has set up a research program to evaluate the actual security situation of the tower, to determine the causes of cracking and to make proposals for an appropriate consolidation treatment. The experiments have run from December 1987 to June 1988, and were conducted by the Reyntjens Laboratory, in collaboration with Geosurvey n.v. for the geo-electric measurements and De Neef Engineering n.v. for the execution of the consolidation injection experiments. The program was supported by the church-fabric of St. Mary's Basilica, the city council of Tongeren and the Ministry of Publics Works of the Flemisch Government.

Research Program

Based on the positive experiments is a previous research program (Van Gemert - Van Mechelen, 1988, ref.2) on consolidation of ancient masonry by means of injection of hydraulic and polymeric grouts, it was decided to study the possibility of strengthening and stabilizing the tower by injections. For that purpose the following program was set up and executed.

- 1.Geo-electric sounding of the masonry before injection;
- 2.Taking cylindrical corings from the masonry to calibrate the measured resistivity maps;
- 3.Test injection with hydraulic and polymeric grouts;
- 4.Geo-electric sounding and physico-mechanical testing of injected masonry.

Injection of hydraulic or polymeric grout is frequently used nowadays for the consolidation of degraded historic masonry. The principle of a consolidation by means of grouting is quite simple (Van Gemert, 1988, ref.3). Masonry is a composite material made of bricks or stones and mortar. The bearing capacity of the masonry is due to the strength of the stones and mortar, and to the adhesion between mortar and stones. This adhesion forces enable the masonry to resist to the splitting forces which arise in the masonry at loading. By injection of a polymeric or hydraulic grout in the pore structure of the masonry one can diminish the splitting forces and at the same time increase the adhesion between stones and mortar. This will result in a strong increase of the compression strength and the durability of the masonry.

Phase 3 of the test program includes the study of the creep behaviour. It was asked for, to know the effect on the loading mechanism when the injected reinforced core should creep under the load of the Tower.

CREEP MEASUREMENTS

A. Test device

Creep was measured on cylindrical cores with a diameter of 105 mm and lengths of 345 and 305 mm bored out of the grouted masonry. The grout used was an epoxy resin. These cores were placed in a testing device under a sustained load corresponding to 1 MPa.

Deformation was measured with three strain gauges, axially symmetrically placed on each sample (Type TML - PL120, length 120 mm). The device takes into account a possible drafting of the gauges by measuring the resistance area and unloaded gauge. No temperature compensation has been build in, as the whole testing device was installed in a room at constant relative humidity (60 %) and temperature 20C. The whole measuring procedure is discribed in ref. 4 (Van Gemert - Overschelde, 1983).

Figure 3. Testing device.

B. Measurements

The measured values of the strains are given in fig. 4.

The samples are called A and B. The deformation at time zero (the moment the full load (N/mm^2) was applied) is the instantaneous deformation; The great difference in deformation between the cores A and B can be explained by the heterogeneity of the 2 samples. In the part of the sample B where the creep is measured (over the length of the gauges), an important part of the masonry is built up with mortar and grouted holes. Sample A has in the mentioned part more stone than mortar - according to what can be seen from the surface.

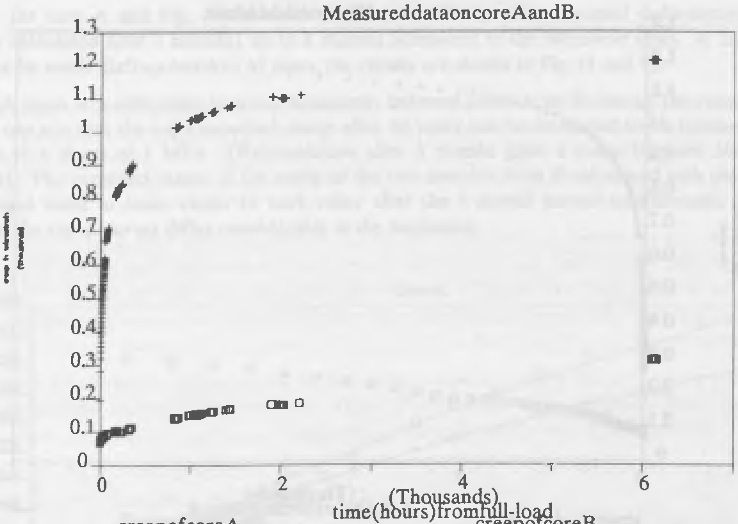

Figure 4. Measured creep in the cylindrical cores A and B. Time in hours from full-loading on . O: core A and +: core B.

C. The creep model

As a mathematical model for the creep function the double power law of BAZANT (Bazant - Osman, 1975, ref.5), has been choosen.

These function seems to fit fairly well with measured data. The hyperbolic equation for predicting creep-strain in concrete of Ross proposed by D. Warren & D. Lenczner (Warren - Lenczner, 1981, ref. 6), for masonry, gave no satifactory results.

As no shrinkage was to be expected due to the elderness of the masonry and as the elactic modulus was expected to be constant in time, the double power law can be simplified in the form :

$$J = (1 + A\,t^n)/E_o$$

J = specific deformation or the deformation per unit of stress

and

A, n, E_o parameters.

D. Analysis of the data

The parameters of the creep function of Bazant have been calculated using a non-lineair least square approach as curve fitting method. To give an equal weight to the data on long as on short term, the integration of the quadratic deviation has been calculated on a logaritmic timescale.

New "smoothen" data had to be calculated on a constant logaritmic time distance, interpolated on the measured data.

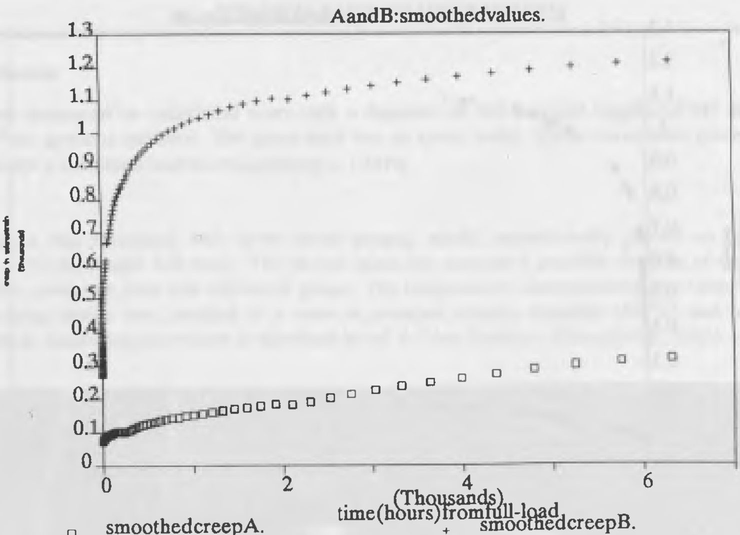

Figure 5. Shows the calculated "smoothen" datapoints with a constant logaritimic time distance. The time scale on the figure is lineair.

Iterating for n the kwadratic deviation integration gives the parameters n, E_o and A in the creep-function of Bazant.

The data after a first 3 months measuring period gave the following parameters.

	n	A	E_o	
core A (3 months)	0,530471	-0,027315	0,013744	1 MPa
core B (3 months)	0,139260	-2,58113	-0,00554	1 MPa

Taking into account the measurements up to 8 months the parameters become

	n	A	E_o	
core A (8 months)	0,63086	0,01274	0,01324	1 MPa
core B (8 months)	0,0955	-1,6170	-0,0021	1 MPa

As figure 5 show there is a good fitting of the creep-function curves with the measure data.

Once the creep-function parameters known, we are able to calculate the expected creep up to, e.g. 10 years. We extrapolated the creep, first after the initeal measuring period of 3 months and later on with the parameters defined after an 8 months measuring time.

Fig. (6) for core A and Fig. (7) for core B show the difference in expected deformation (using the parameters calculated after 3 months) up to 8 months compared to the measured ones. A fairly good estimation can be seen. Extrapolated to 10 years, the results are shown in Fig. (8 and 9).

Although there is a difference in creep behaviour, between cores A en B, due to the reasons we mentioned, we can see that the total expected creep after 10 years can be estimated to be between 1400-1700 microstrain at a stress of 1 MPa. (Extrapolation after 3 months gave a range between 1000 and 2000 microstrain). The expected values of the creep of the two samples A en B calculated with the creep function of Bazant seem to come closer to each other after the 8 month period measurments although the evolution of the creep-curves differ considerably in the beginning.

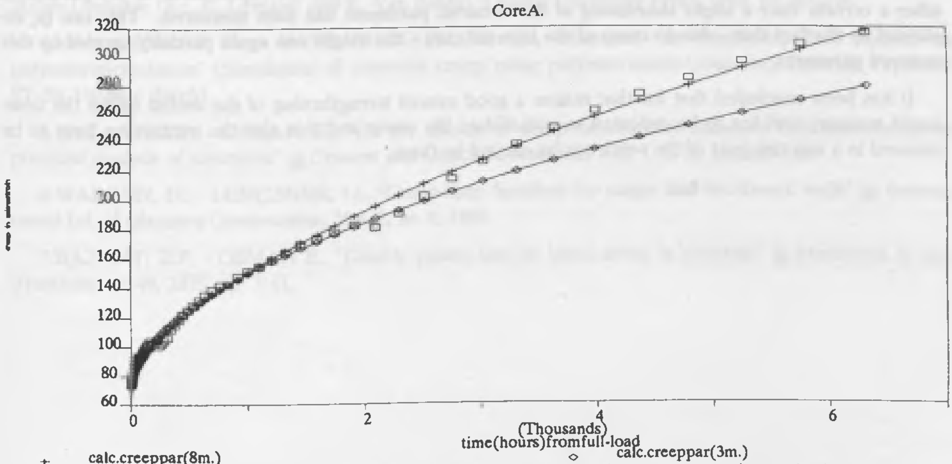

Figure 6. Creep of core A during about 8 months. Smoothed values compared to calculated values

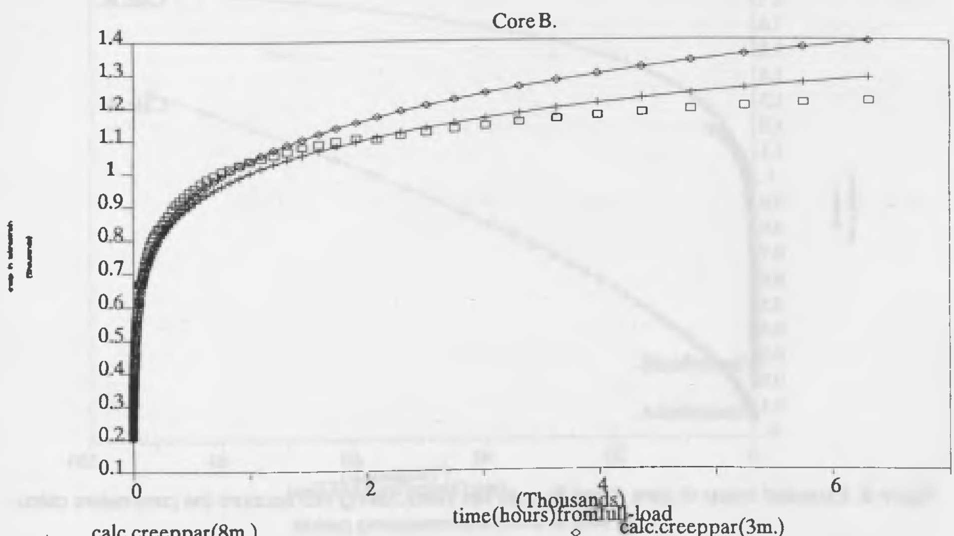

Figure 7. Creep of core B during about 8 months. Smoothed values compared to calculated values taking into account the parameters calculated after 3 months and after 8 months.

CONCLUSIONS

The measurement device enabled us to predict the expected creep of a grouted masonry core of the tower of St. Mary at Tongeren. Due to this expected creep the intervention had to take into account the fact that a sustained load on the inner core of a composed masonry wall will, at the end, result in a strain and a resulting increase of stresses in the outer scales. The higher this sustained load the higher must be the expected creep.

This was already experienced at the consolidation of the pillars of the Cathedral of Antwerp. In this latter case the pillar cores were grouted with an epoxy resin to allow the replacement of the parament. After a certain time a slight shortening of the restored parament has been measured. This can be explained by the fact that - due to creep of the injected core - the weight was again partially carried by this restored parament.

It has been concluded that for that reason a good overall strengthening of the usefull of the the composed masonry wall has to be executed to consolidate the tower and that also the parameters have to be restored in a way that part of the weith can be carried by them.

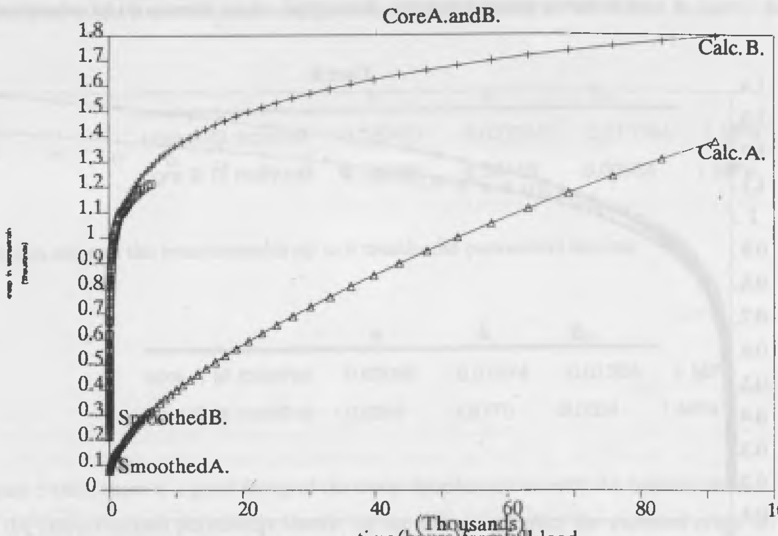

Figure 8. Expected creep of core A and B up to ten years, taking into account the parameters calculated after a 8-months measuring period.

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TIME DEPENDENT DEFORMATIONS OF BRICK MASONRY

M.Karaveziroglou*, J.Papayianni* and G.Penelis**

SUMMARY

From the standpoint of structural performance of the interventions it is essential to have knowledge of time dependent deformations of masonry which are mainly drying shrinkage and creep. As the moisture movement into capillaries is the driving force for both phenomena, the combined effects of exposure to drying and sustained loading on constituents of brick masonry are studied. These constituents are full bricks and mortars made of cement and pozzolanic materials. The parameters investigated in this experimental work are:

1. Mortar consistency
2. Thickness of mortar joint
3. Thickness of bricks
4. Curing conditions.

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INTRODUCTION

Time dependent deformation of masonry structure is a research field which has recently begun to be studied. With regard to repairing and reconstruction of old buildings it is essential to know the time dependent behaviour of restored masonry which is the main structural bearing element of the old construction.

The purpose of this research work was to obtain data on creep and shrinkage of masonry considered as a composite material which consists of full brick and bed of mortar joints made from "traditional" materials. The variables included in this study are considered to be main parameters which influence long-term deformations although there are many others such as the bonding of bricks to mortar joints, the shape of specimens, which effect them.

EXPERIMENTAL PROCEDURE

As there is no established laboratory test of estimating long term deformations of brick masonry, the spring loaded creep frame of ASTM C 512-82 test method was used for measuring creep of brick masonry specimens.

Twenty four cylinders, two for each code number shown in Table I were constructed. Each of them consisted of bricks and at least three mortar joints. They were moist cured for one month and then were subjected to compression. The level of applied stress was analytically calculated to be 25% of their strength taking into account the values given from equations

$$\beta_{D,mw} = 0.8\, \beta_{D,st}^{0.7} \cdot \beta_{D,mo}^{0.2}\ (N/mm^2) \tag{1}$$

$$f_{wc} = (1-0.8\sqrt[3]{\alpha})[\, f_{mc}+0.4(f_{bc}-f_{mc})]\ (N/mm^2) \tag{2}$$

where $\beta_{D,mw}$, f_{wc} : strength of masonry

$\beta_{D,st}$, f_{bc} : strength of brick

$\beta_{D,mo}$, f_{mc} : strength of mortar

α : $\dfrac{\text{thickness of mortar joint}}{\text{depth of brick}}$

(DIN, 1985), (Tassios, 1986).

The compressive strength of used full bricks was 120kg/cm² for 40mm depth and 95kg/cm² for 50mm depth. The proportions and 28 day strength of different used mortars are given in Table II. The constituents of mortars were lime, Santorine Earth, sand(max size 1mm), crushed brick(max size 6mm) and cement(Penelis-Papayianni-Karaveziroglou, 1989). Apart from cement the other constituents are widely used in restoration work and called "traditional" materials. Two kinds of joints were studied of 25mm and 40mm thickness in combination with two types of brick depths 40mm and 50mm.

All the specimens were placed in moisture controlled room (relative humidity between 55% and 65%), while four of them were sealed to prevent moisture loss.

The measurements were carried for up to a period of 180 days. Afterwards, the specimens were unloaded and compressed. Taking the real strength of specimens into account, the stress/strength ratio was found to range from 0.9 to 0.13 instead of estimated 0.25 value. The fact of underestimating brick masonry strength using the above strength equation was the challenge factor for further research in this field.

TEST RESULTS AND DISCUSSION

All the experimental data were plotted in Fig.1 to 8. Looking at them it seems that under this level of stress, curves tend to have a zero inclination after the first days of loading. The greater inclination is that of the first branch of curves where elastic deformations occur. As the time went by the drying shrinkage was the dominant phenomenon. Comparing the parameter of mortar's consistency it can be seen(Fig.1 and 2) that the specimens with the lime and Santorine earth(M2) showed the max.ultimate strain although the rate of strain development was relatively low. Mortars with cement(M8 and M10) were of higher strength and resisted better to the applied stresses. The addition of crushed brick to mortar seems to be beneficial in reducing creep of specimens. A reduction in strain of 65% was found for specimens No1 and No5 (Fig.1) while for specimens No2 and No7 this reduction was 25% (Fig.2).It is clear from Fig.3 and 4, that the larger the mortar joint the higher the strain of specimen is. This of course, was expected as mortar is the source of both phenomena creep and shrinkage. Brick depth seems not to have the same influence on creep deformations as that of mortar joints. It seems that specimens with full bricks of 40mm depth showed greater strain than those of 50mm brick depth(Fig.5 and 6). But this must be seen in combination with the ratio of total volume of mortar to total volume of bricks in masonry specimens which is 0.75 for specimens No6,No10(brick depth 40mm) and 0.60 for specimens No7,No11(brick depth 50mm). Moisture conditions play the most important role in long term deformations. The sealed specimens compared with the unsealed ones showed a reduction in strain and their curves were parallel to axis of time coordinate. This reduction was about 6% for specimens made of mortar M4 and 50% for specimens made of mortar M8. This indicates that considering creep deformations, moisture is a serious factor especially when cement is used as a constituent of the mortar of masonry.

CONCLUSIONS

Specimens built with mortars based on lime and Santorine earth presented the greater long term deformation which exceeds the 1% of their length. The presence of crushed brick in mortars is beneficial regarding the creep behaviour of masonry. The prevention of moisture loss in masonry specimens hinders of creep and drying shrinkage deformations. The use of different depth bricks in construction of masonry may have some significance when considered in combination with the ratio of total volume of mortar to total volume of brick. The thickness of mortar joints influences considerably the creep deformations and structural performance of brick masonry.

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Table I

Specimen Parameters

Specimens No.	Brick depth [cm]	Mortar Label	Mortar Bed joint [cm]	Relative humidity %
1	4.0	M1	2.5	55 to 65
2	5.0	M1	4.0	55 to 65
3	4.0	M4	2.5	55 to 65
4	5.0	M4	4.0	55 to 65
5	4.0	M2	2.5	55 to 65
6	4.0	M2	4.0	55 to 65
7	5.0	M2	4.0	55 to 65
8	5.0	M2	4.0	sealed
9	4.0	M3	2.5	55 to 65
10	4.0	M3	4.0	55 to 65
11	5.0	M3	4.0	55 to 65
12	5.0	M3	4.0	sealed

Table II

Characteristics of Mortars

Mortar No.	Proportions (by weight): Lime	Santorine Earth	Cement	Crushed Brick	Sand	28-days Compressive Strength [kN/mm^2]
M1	1	1	-	-	6	0.59
M2	1	1	-	3	3	1.20
M3	1	0.6	0.4	3	3	2.20
M4	1	0.4	0.6	3	3	5.27

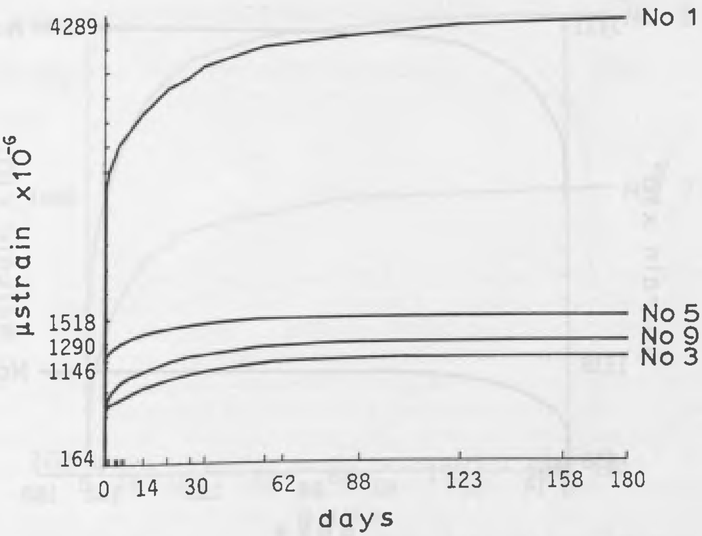

Fig.1 Influence of mortar quality on long term deformations of masonry specimens. Brick depth 40mm.

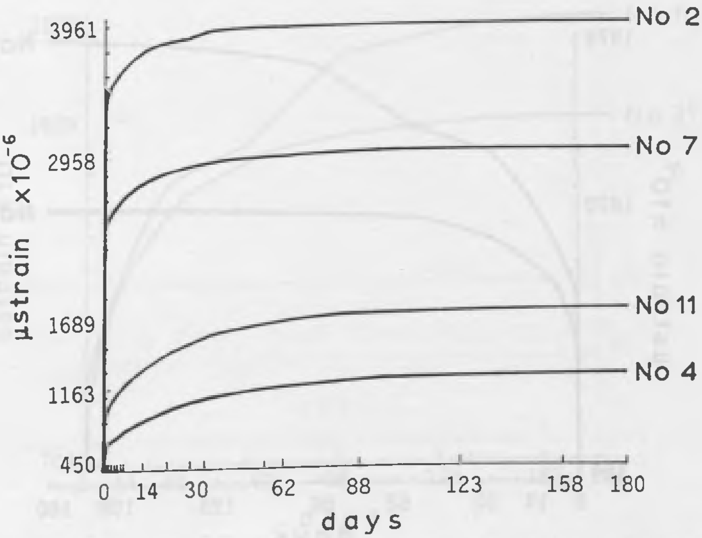

Fig.2 Influence of mortar quality on long term deformations of masonry specimens. Brick depth 50mm.

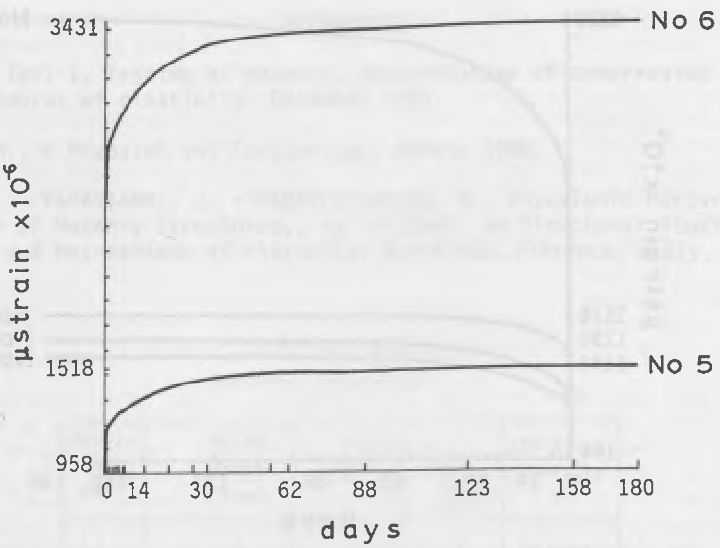

Fig.3 Effect of mortar bed joint thickness on long term deformations. Mortar M4.

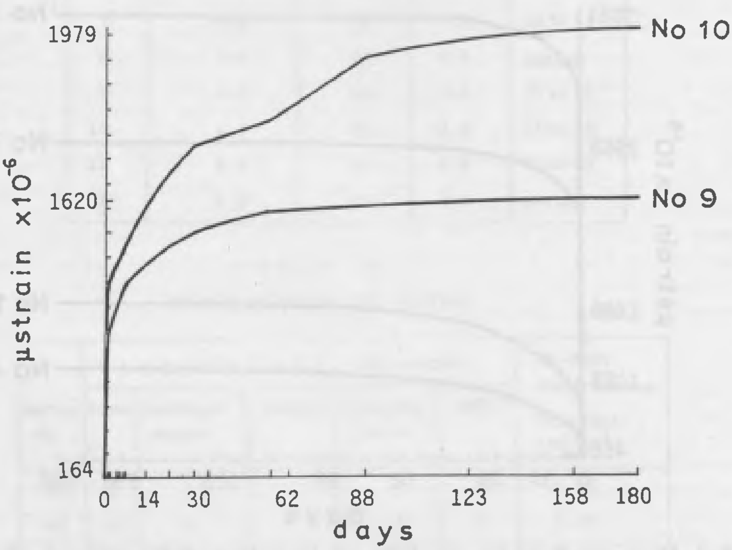

Fig.4 Effect of mortar bed joint thickness on long term deformations. Mortar M8.

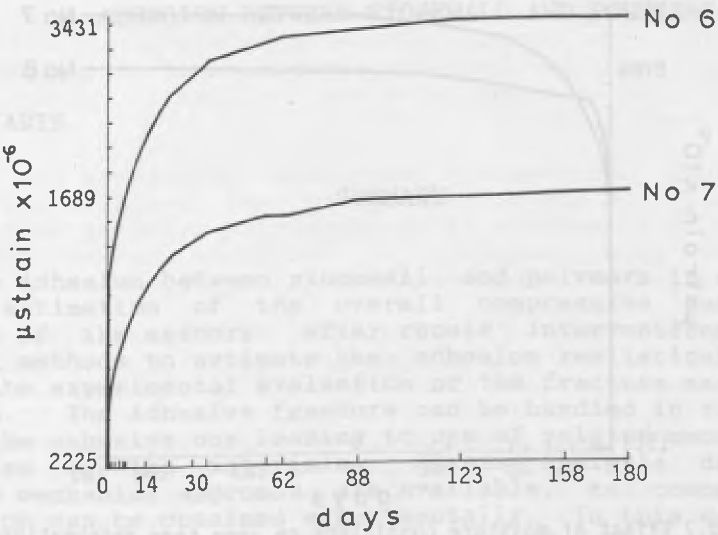

Fig.5 Influence of brick depth on long term deformations. Mortar M4.

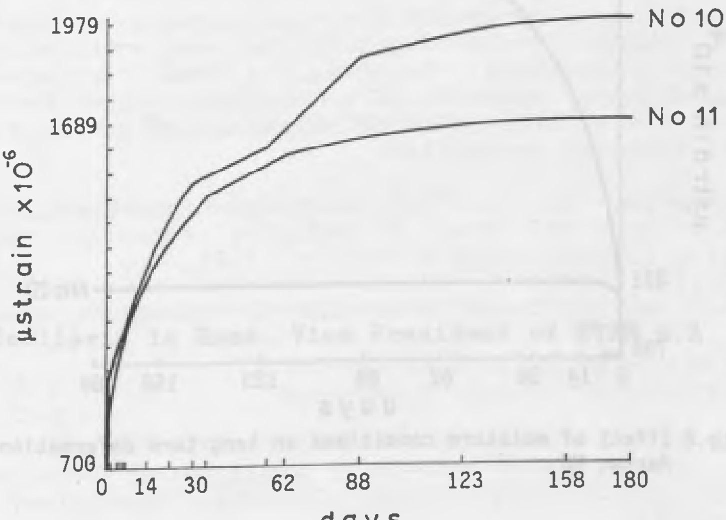

Fig.6 Influence of brick depth on long term deformations. Mortar M8.

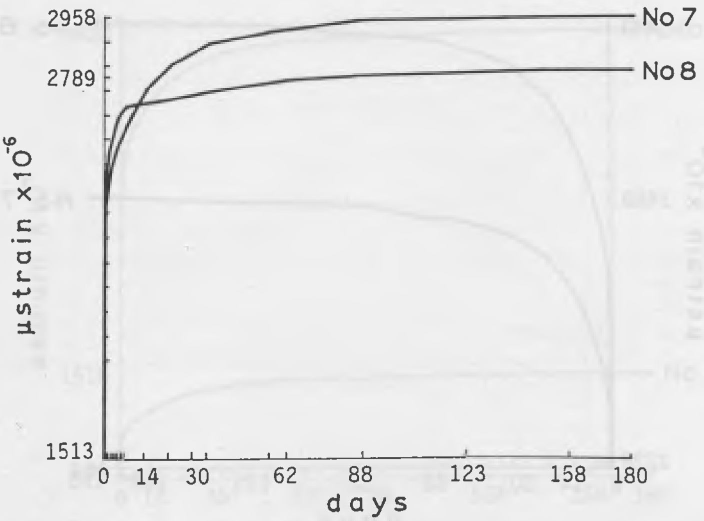

Fig.7 Effect of moisture conditions on long term deformations. Mortar M4.

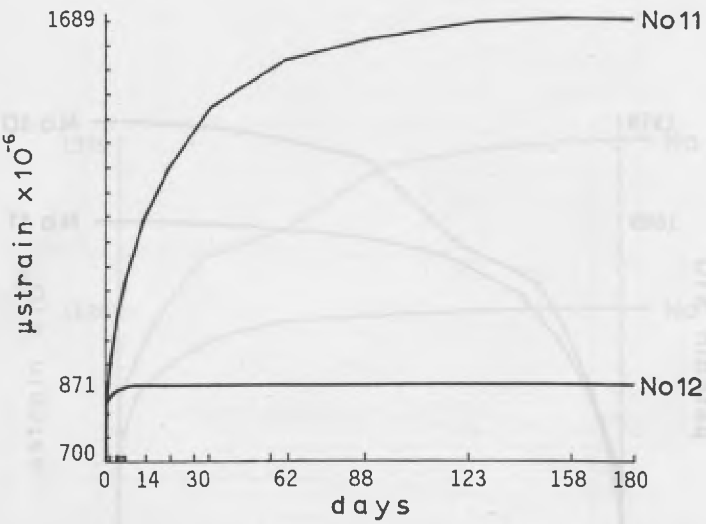

Fig.8 Effect of moisture conditions on long term deformations. Mortar M8.

ADHESION BETWEEN STONEWALL AND POLYMERS

C. ZAVLIARIS

SUMMARY

The Adhesion between stonewall and polymers is crucial for the estimation of the overall compressive and shear strength of the masonry after repair interventions. The existing methods to estimate the adhesion realistically are either the experimental evaluation or the fracture mechanics Approach. The Adhesive fracture can be handled in the same way as the cohesive one leading to use of related mechanical properties of the materials. Stress analysis data and fracture mechanics approach are available, to compare the data which can be obtained experimentally. In this work two different shear adhesive joints were tested. The results of the experimental work are in good agreement with those calculated by using the fracture mechanics approach.

Dr. C. Zavliaris is Exec. Vice President of ETAN S.A.

1. INTRODUCTION

The Adhesion between stonewall and mortar influences both the overall compressive and the lateral shear strength of a stonewall./1/

Among the possible repairing techniques for existing stonewall are the polymer-or polymer modified injections. The aim of injections is to impregnate existing mortars, to fill the gaps (commonly at interface between stone and mortar) and to repair cracks.

In this case the question of the adhesion strength between stone and polymer becomes crucial for evaluating the local compressive and shear strength of stonewall.

However, the estimation of this adhesion strength is possible only experimentally or by fracture mechanics approach, since all the classical theories are not applicable.

2. The Fracture mechanics concept

While in Classical Mechanics all the bodies are regarded as homogeneous continua, the basis of the Fracture mechanics Approach is that in every body flaws or imperfections exist.

The Fracture mechanics concept expresses a quantitative Approach which is a function of the mechanical and geometrical variables involved in the propagation of existing flaws or cracks of a structure, i.e. the applied stresses, the existing flaw or imperfection geometry and the specific material properties.

According to this an existing imperfection can be blocked, provided that a definite function of above quantities is kept below a certain limit.

There are two broad categories of fracture criteria¦

- The energy balance criterion
- The stress intensity approach

The first imposes the criterion, Griffith/1/ that for a thin lamina under uniformly distributed tensile stress with an eliptic crack, the crack is stabilized as far as $\sigma \leqslant \sigma f = \sqrt{\frac{E G}{\pi \cdot a}}$

where: E= modulus of Elast.of the material
G= the critical energy release rate
a= the size of the flaw

The second derived from the stress situation in a lamina under uniformly distributed tensile stress, and states that the condition for crack stabilization is K<Kc where:

$$\begin{vmatrix} \sigma x \\ \sigma y \\ \sigma xy \end{vmatrix} = \frac{K}{\sqrt{2\pi r}} \begin{vmatrix} fx(\vartheta) \\ fy(\vartheta) \\ fxy(\vartheta) \end{vmatrix}$$

$$K = \sigma\sqrt{\pi \cdot \alpha}$$

$$fx(\vartheta) \cos\frac{\vartheta}{2}(1-\sin\frac{\vartheta}{2}\sin\frac{3\vartheta}{2})$$

$$fy(\vartheta) = \cos\frac{\vartheta}{2}(1+\sin\frac{\vartheta}{2}\sin\frac{3\vartheta}{2})$$

$$fxy(\vartheta) = \sin\frac{\vartheta}{2}\cdot\cos\frac{\vartheta}{2}\cos\frac{3\vartheta}{2}$$

The aformentioned aspects apply equally to both the cohesive and adhesive fracture. The only difference is that in the latter instead of the modulus of Elasticity of the material a combined value of moduli of the materials involved in the interface is used.

3. Double Shear Joints

Generally for a crack in a homogeneous continunm it is:

$$Kc = Q.\sigma f\sqrt{a}$$

where Q: a geometric constant, which can be expressed as a non-dimensional function of crack length and structural geometry in the form of finite series. Among the many techniques to obtain it are the direct methods where equations relating the crack tip stresses or displacements to the stress intensity factor are solved by closed form methods /4/.

So for the shear model shown in Fig.1., these expressions are given.

For the experimental set up designed /Fig.4/, the calculation of above quantities gives for Gc= 1,5 KJ/m² (/5/):

Qh= 0,0325
Q = 0,012
Kc= 2,06.10**-2 [N/mm**3/2]
τc= 17,0 MPa

4. Single Shear (Lap) joint

For lap joint with negligible or zero influence of the eccentricity of the load, the stress analysis (cf/2/) is as shown in fig.3 where β is the stress concentration factor.

Gent /3/ examined two extreme forms of sheared adhesive layer. Working with the energy balance approach he assumed that:

$$G = h*Qc$$

$$G = k*\alpha Qc$$

for the thin and the thick adhesive layer correspondingly, with h the thickness of the adhesive, k a numerical factor with value as follows, Qc the strain energy per unit volume and α the length of the flaw (which in this case is a debond).

Combining above equations with those derived from structural analysis of Volkersen he came to the expression of fracture shear stress, for a relatively large and relatively small flaw:

$$\tau c = 1/\beta\ (Ea.Wa/h(1+Va))**1/2$$

$$\tau c = 1/\beta\ (Ea.Wa/\Pi(1+Va).a)**1/2$$

where β: stress concentration factor,

Ea: adhesive tensile modulus of Elasticity
Wa: the work of the adhesion between substrate and adhesive
va: adhesive Poisson's ratio.

And for an epoxy-stone shear (Lαp) joint, with:

E1=E2= 30.000 MPα　　Eα = 5000 MPα

E2 = 30.000 "　　Vα = 030

t1 = t2 = 60 mm

$$D\alpha = \frac{4000}{2(1PV\alpha)} = 1538\ MP\alpha$$

2c = 60 mm, c = 30 mm

a= 0,01 mm　　Wa= 300 mJ/m²

to = 2 mm:

it can be calculated that: τc= 5,0 N/mm²

5. Experimental approach

Two series of tests were carried out In fact the double and direct shear tests. The specimens were slightly modified in relation to standard direct shear specimens proposed by RILEM Technical Committee RAC./s.Fig.6/.

The specimens were cut from typical limestone and loaded in hydraulic testing machine with a rate of Ø.2Ø KN/s and simultaneously the total slip was measured by a Linear Variable Displacement Transducer. An analog to digital converter combined with an amplifier collected the sign from both the load cell of the machine and the LVDT and transferred them to a computer, where they were processed.

The obtained mean curves P - δ or τ - δ are illustrated in Fig.5 and confirm the ultimate τ values calculated by means of the fracture mechanics approach.

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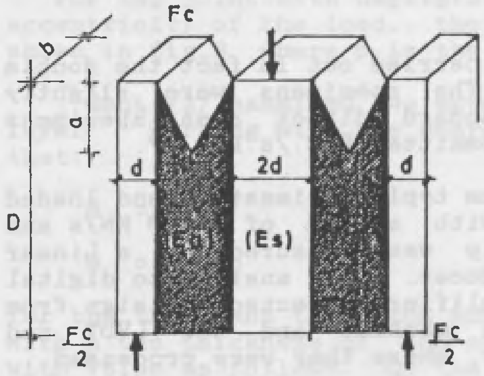


$$K_C = \tau_c \cdot Q \cdot \sqrt{a} \qquad (6)$$

$$\tau_c = \frac{F}{2b \cdot d} \qquad (7)$$

$$Q = Q_h \sqrt{\frac{E_a}{E_s}} \qquad (8)$$

$$Q_h = \frac{d}{D} \cdot \left[0.0325 + 14.63\left(\frac{a}{D}\right) - 46.07\left(\frac{a}{D}\right)^2 + 61.03\left(\frac{a}{D}\right)^3 - 28.86\left(\frac{a}{D}\right)^4 \right] \qquad (9)$$

Fig. 1.

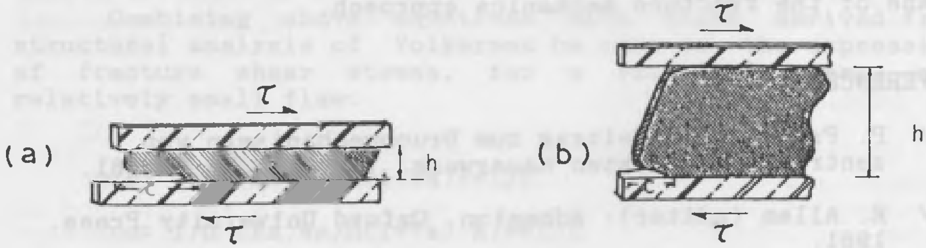
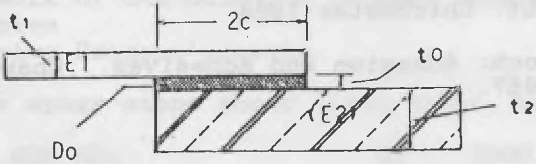


Fig. 2 : Shear loaded joints with (a) thin and (b) thick adhesive layer.

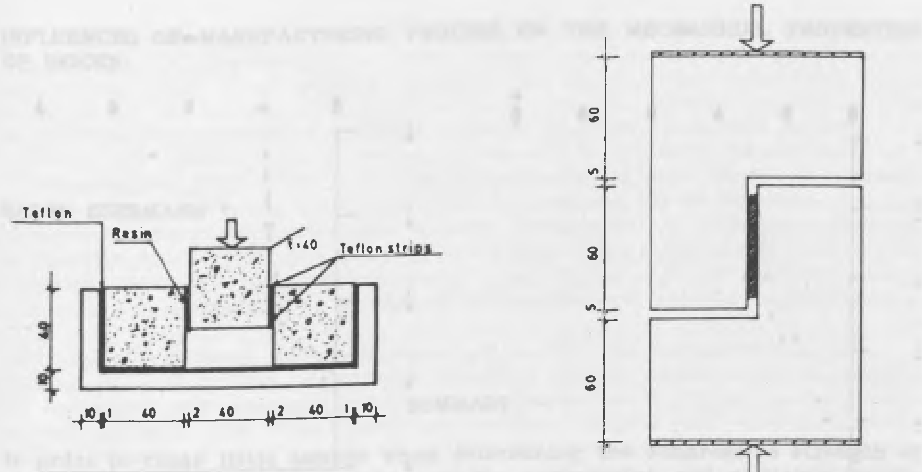


$$\beta = \frac{\tau_{max}}{\tau_m} = \frac{\delta}{\varepsilon} \left[\frac{2\varepsilon^2 - 1 + \cosh(2\varepsilon\delta)}{\sinh(2\varepsilon\delta)} \right] \qquad (11)$$

$$\delta^2 = \frac{2c^2 D_a}{E_2 . t_2 . t_1} \quad , \quad \varepsilon^2 = \frac{E_1 t_1 + E_2 t_2}{2 E_1 t_1} \qquad (12)$$

D_a: adhesive shear modulus. (13)

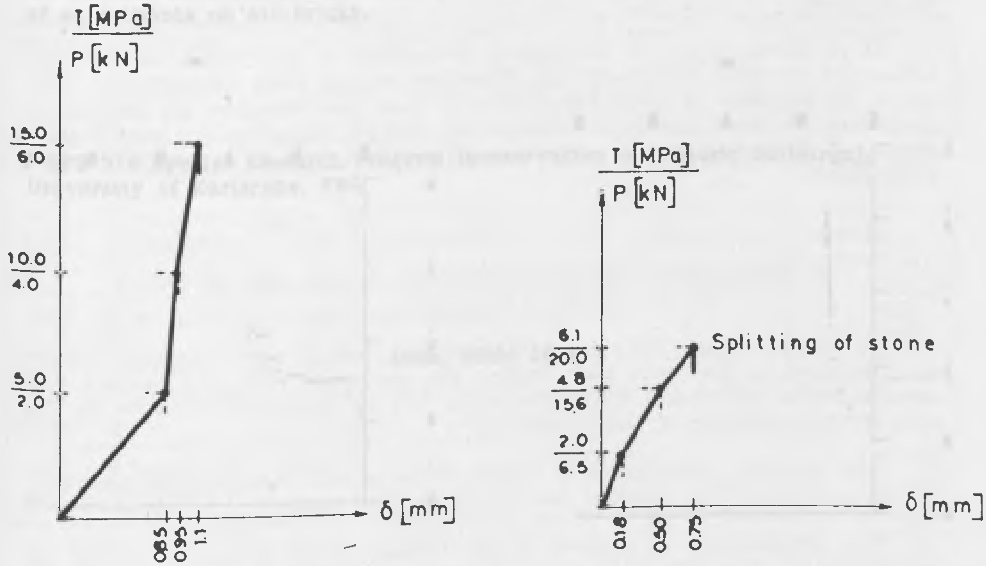
Fig.3 : Single lap joint analyzed by Volkersen.



a. Double shear test.

b. Single shear test.

Fig. 4: Direct shear tests



a. Double shear joint

b. Single shear joint.

Fig. 5: τ = τ(δ) relationship

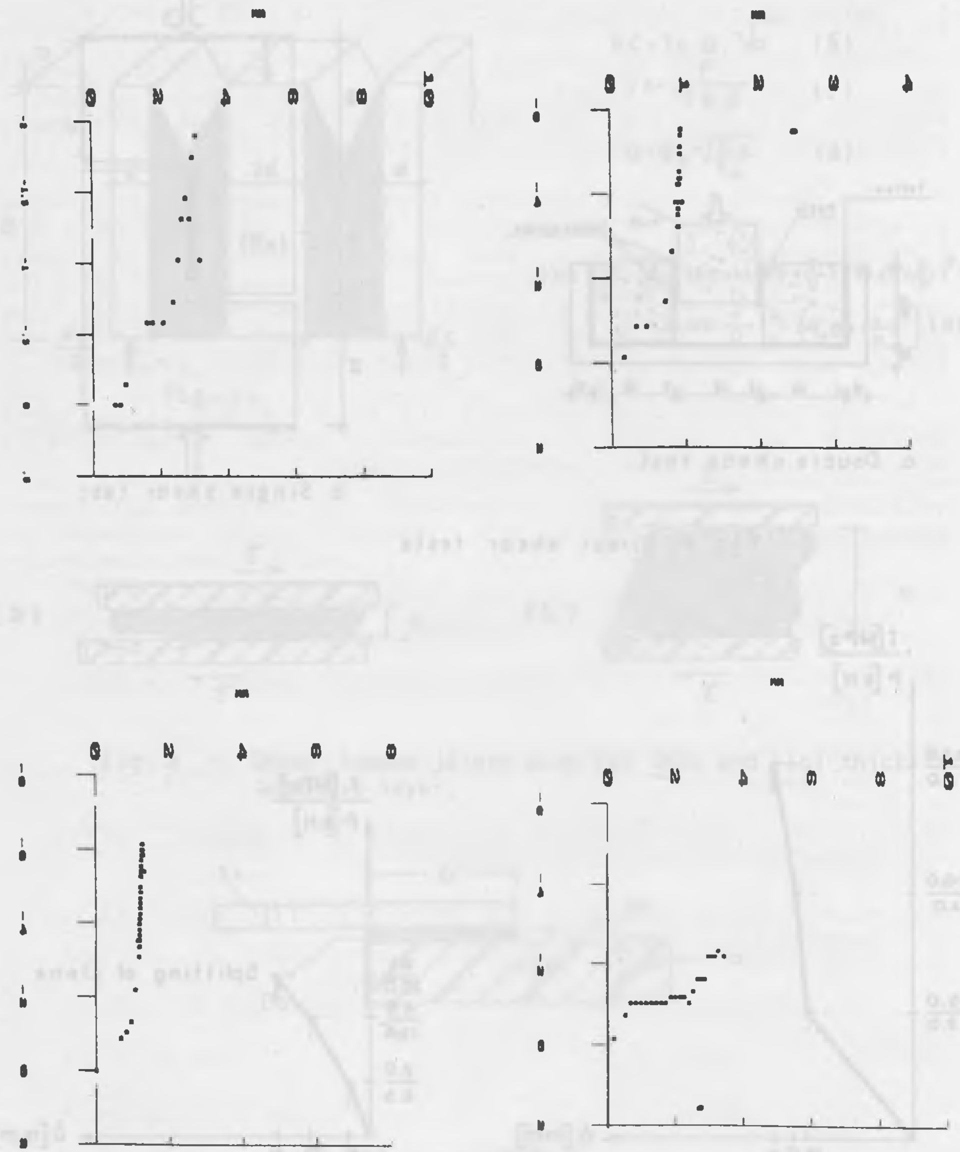


Fig.6: τ= τ(δ) relationship for specific combination stone-epoxy.

INFLUENCES OF MANUFACTORING PROCESS ON THE MECHANICAL PROPERTIES OF BRICKS

RALPH EGERMANN *

SUMMARY

In order to cause little damage when determining the compressive strength of old masonry, the necessary values of the compressive and splitting tensile strength of stone are determined by testing drill cores. A prerequisite is the knowledge of possible anisotropies in the brick that may have appeared during its manufacturing caused by the shaping. In this report differences of the mechanical behaviour between modern extruded and old hand-moulded bricks are explained. The results of experiments which were carried out on bricks that were especially made by these methods were compared with those of experiments on old bricks.

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June, 12th, 1989

INTRODUCTION

During the examination of historic building substance the question of the load-bearing capacity of the existing masonry often arises. The requirement to carry out such an examination with non-destructive methods and to reach a maximum in the accuracy of the examination results can only be met partially according to the present state of technology. Although the success of the first examinations concerning the usage of non-destructive methods is very promising, it can not replace taking samples, but can only valuably support it by reducing the destructive operations to a minimum.

In the course of the SFB 315 Special Research Program at the University of Karlsruhe a special partially destructive method for the subsequent determination of compressive strength of masonry was developed by (Berger, 1986). This method requires the compressive and splitting tensile strength of stone as important parameters, which can only be reliably determined with the help of drill cores. With bricked up bricks one has to consider anisotropies depending on the type and the material. This means that for example a drill core taken out of the side surface shows different mechanical properties than a drill core taken out of the top surface. The knowledge about these anisotropies is important, as the drill cores can only be taken out of the side surfaces of the bricks, while their mechanical properties perpendicular to that, that means in the loading direction, are of greater interest.

It is known that the mechanical properties of the bricks are strongly influenced by their manufacturing. Therefore on one hand the different stages of development of brick manufacturing that have changed the mechanical properties the most had to be determined, and on the other the quantitative impact of this development, especially on shaping had to be proved by own experiments.

THE DEVELOPMENT OF MANUFACTURING TECHNIQUES OF BRICKS

In former times, until the 18th century bricks were basically always manufactured in the same way. During the course of the 19th century new manufacturing techniques developed which had an essential influence on the mechanical properties of the bricks.

The essential manufacturing stages which mainly influence the mechanical properties are the shaping and the firing technique.

The original way of shaping is the forming of the bricks only with the hands without any devices. It was not until the usage of wooden frames and molds that one could actually speak of the soft-mud molding methods which was later performed by machines which did not change the actual technique though. With the invention of the screw conveyor for transporting plastic masses by Karl Schlickeysen in 1854 the essential change in the shaping technology was made. Due to the extruding methods one has to expect more or less textures (Bender-Händle, 1982; Grätz, 1969) as the clay particles are aligned the most when passing through the shaping die. After the basic invention of the de-airing auger this principal was further developed (dry press, vacuum press), but not essentially changed.

The strength of the bricks and its variation in one batch are strongly influenced by the firing technique. As temperatures of 800°C were reached during firing in a charcoal-kiln - one of the oldest firing techniques - the problem is not so much to reach possibly high temperatures but much more to distribute these regularly over the whole batch. It was not before the invention of the Kassel kiln (between 1820 and 1840) that the batch variations were reduced which before had been a result of strongly and weakly fired bricks. After the use of the de-airing auger the further development of kilns was no longer focused on the effort to minimize the weak firing which had already been achieved in the most part, but to increase the productivity.

OWN EXPERIMENTS

Summary of the tested material and the experiments

sample ID	origin	year	preparing	shaping: hand-molding	shaping: extrusion	firing
MZ	brickworks	1988	brickworks		brickworks	brickworks (1000°C)
SM	brickworks	1988	brickworks		brickworks	laboratory (800°C)
HM	brickworks	1988	brickworks	lab		laboratory (800°C)
QU Quern	parsonage	1796	?	x		?
BE Beckh	brewery (PF)	1884	?	?	?	?

All examinations were carried out on drill cores that were taken out of different surfaces of the bricks. Besides the determination of the dry density the compressive strength and the splitting tensile strength are destructively determined. The Young's modulus and the Poisson's ratio are determined using strain gauges.

Discussion of the results

Dry density

In Fig. 1 one can see that there are no significant differences neither concerning the firing temperature (comparison between MZ and SM) nor the shaping (comparison between SM and HM). The expected greater difference in the shaping may not have occured as a result of an ideal combination of raw materials which is influenced by the brickworks. This means that the distribution of the grain sizes was so well adjusted that almost the same dry density could be achieved in the hand molding.

Compared to the dry density of old bricks (QU, BE) the artificially manufactured bricks show higher values (Fig. 2). The determined dry densities (1.4 to 1.6 g/cm^3) are customary for historical bricks. This is also confirmed by (Al-Kass - Hadi - Khalil - Ali, 1989) and (Berger, F.,1987).

Compressive strength

Regarding compressive strength (in the y-direction) significant differences between the bricks MZ, SM and HM arise (Fig. 1). The extruded, self baked brick (SM) only has 73 % of the compressive strength of an industrially manufactured brick (MZ). The reason for this may be the higher firing temperature. It is surprising that the hand made brick (HM) shows only about half the compressive strength of the extruded brick (SM) because almost identical dry densities were determined. This shows that the orientation of the clay particles has a significant influence on the compressive strength.

In comparison to the old bricks (QU, BE) the hand made brick (HM) has nearly the same compressive strength (Fig. 2).

Splitting tensile strength

The splitting tensile strength of the extruded, self baked brick (SM) is only slightly different from the industrially manufactured brick (MZ), while the hand made brick (HM) shows only about half the splitting tensile strength of the extruded brick. This is not very surprising as it is well known that there is a relation between compressive strength and splitting tensile strength.

Variation of the compressive strength

The question of the variation is very important for the subsequent examination of the compressive strength of bricks, as the number of samples taken of a building should be kept as low as possible because of the preservation of monuments. Fig. 1 shows that the variation of compressive strength in the y-direction only differs slightly given a 200°C-difference in the firing temperature (MZ/SM). The variation is expressed by the coefficient of variation. The "self baked" brick (SM) shows an even 10 % smaller variation than the industrially manufactured brick (MZ); the reason may be the obvious cracks in the modern bricks that result from the reduction of internal stress which could have developed during the shaping.

As expected the variation of the hand made bricks (HM) is one third higher than the one of the extruded bricks (SM). Significant differences concerning direction can not be seen. Regarding variations it is therefore not relevant from which side of the brick the drill core is taken.

The comparison with the old bricks (Fig. 2) shows a much higher variation. However it must be taken into consideration that the bricks MZ, SM and HM were manufactured under laboratory conditions and were never used, whereas the drill cores of QU and BE originate from bricks in use which besides the design loads had to bear unplanned stress resulting from the demolition. One has to consider though that taking immediate samples from the building leads to less variation than a subsequent examination of demolition material.

Young's modulus

Considering the Young's modulus in y-direction there are significant differences for different firing temperatures. The industrially manufactured brick (MZ) is about twice as stiff as the self baked extruded brick (SM). The extrusion (SM) approximately leads to a Young's modulus which is twice as high as that of the hand made brick. This knowledge has to be considered especially in reconstruction, because the usage of extruded bricks even if they were fired using a low temperature may cause unintended load distributions as a result of differences in stiffness.

The comparison with old bricks shows that the own hand made brick HM reaches the Young's modulus of the historic bricks, which is approximately 7000 N/mm².

Regarding the stress-strain-diagrams (Fig. 3) the industrially manufactured brick MZ is the only one showing a clear linear elastic material behaviour (up to 2/3 of the crushing strength) whereas the nonlinear behaviour increases from SM (no figure) to HM. This nonlinearity is widely confirmed by old bricks.

Poisson's ratio

It can be seen that the capacity of lateral strain of the industrially manufactured brick MZ is about one third larger than that of SM and about twice as large as that of the hand made brick (Fig. 1).

This shows that the qualitative proportion has turned around compared to the Young's modulus . The brick with the higher Young's modulus also has the largest capacity of lateral strain.

This is also confirmed by the comparison of the old bricks QU and BE (Fig. 2).

Factors of direction (anisotropy)

The question of differences in direction is of special interest regarding values of strength and deformation. From the tables 1 – 3 one can see that there are no significant differences between the drill cores in x- or z-direction. They are therefore joined to an average. Regarding the compressive strength one can see a clear difference between MZ and SM concerning the factor of direction ($\beta_{Dy}/\beta_{Dx,z}$). The self baked brick only shows 84 % of the factor of direction compared to the industrially manufactured brick (Fig. 1).

The comparison with the old bricks (Fig. 2) shows that the factor of direction concerning compressive strength of HM is approximately located between QU and BE. Surprisingly the factors of all 3 objects are smaller than 1.

The own examinations have proved that the properties of strength in the direction of extrusion (= in general the later load direction) are increased by the extrusion process (factor of direction > 1) while hand molding causes an opposite effect (factor of direction < 1). The greater the energy that presses the clay into the mold, the smaller the factor of direction seems to become.

The previous examinations would implicate that the compressive strength of the drill core, which is determined from samples of the old masonry, would have to be decreased by an additional factor of direction (0.75 – 0.9). In following examinations the value of this factor shall be determined more accurately.

When regarding differences in direction of the deformation properties (Young's modulus , Poisson's ratio) it is striking that for the extruded bricks the factors of direction are much larger in quantity than for the compressive strength (Fig. 2). In other words, the texturation influences the deformation properties more than the strength values (Fig. 1).

The factors of direction of hand made bricks (HM) and of old bricks (QU, BE) are almost the same (Fig. 1).

The stress-strain-diagrams (Fig. 3) very clearly show the influence of the shaping on the behaviour of deformation of the bricks. While the modern brick (MZ) doubtless has direction-depending diagrams, the hand made brick shows none.

The stress-strain-diagrams of the old bricks (QU, BE) do not show any differences at all.

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BERGER, F. , Die gemauerten Gewölbe der staatlichen Kunsthalle Karlsruhe, Gutachten zur Standsicherheit, (unpublished), 1987.

sample ID	dry density [g/cm³]				compressive strength in y-direction [N/mm²]				splitting tensile strength [N/mm²]			
	n_K	n_Z	$\bar{x}$	v	n_K	n_Z	$\bar{x}$	v	n_K	n_Z	$\bar{x}$	v
MZ	94	20	1.83	1.6	20	9	43.0	18.1	25	10	3.94	32.4
SM	126	19	1.90	1.2	36	13	31.3	16.3	18	9	3.76	18.0
HM	95	20	1.82	1.4	18	10	15.6	22.2	19	10	1.82	21.7
QU	113	19	1.64	3.4	21	11	8.1	32.1	35	9	0.47	28.4
BE	80	10	1.43	3.7	15	8	9.8	32.5	37	10	2.22	35.0

Table 1 Results of the strengthening tests

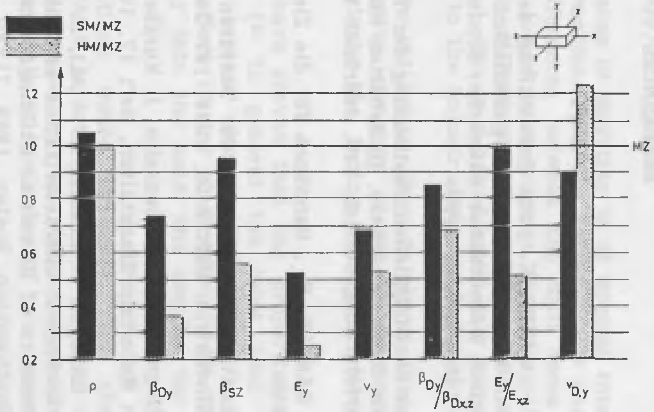
sample ID	Young's modulus in y-direction [N/mm²]				Poisson's ratio in y-direction [-]			
	n_K	n_Z	$\bar{x}$	v	n_K	n_Z	$\bar{x}$	v
MZ	10	5	22669	5.5	9	5	0.19	6.4
SM	5	3	11867	18.7	5	3	0.13	12.5
HM	7	5	5716	21.2	7	5	0.10	33.5
QU	4	2	2726	11.7	4	2	0.14	19.1
BE	3	2	9390	38.9	3	2	0.18	6.3

Table 2 Results of the deformation measurements

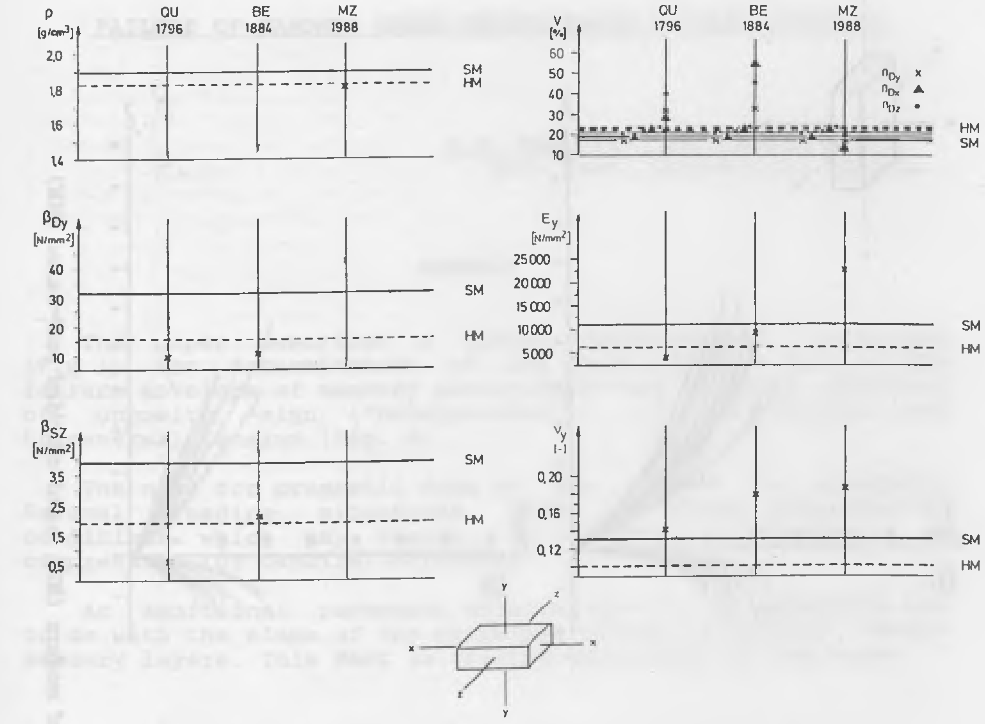
n_K ... number of tested drill cores
n_Z ... number of bricks from which cores were drilled
$\bar{x}$ ... mean of n_K observations
v ... coefficient of variation in [%]

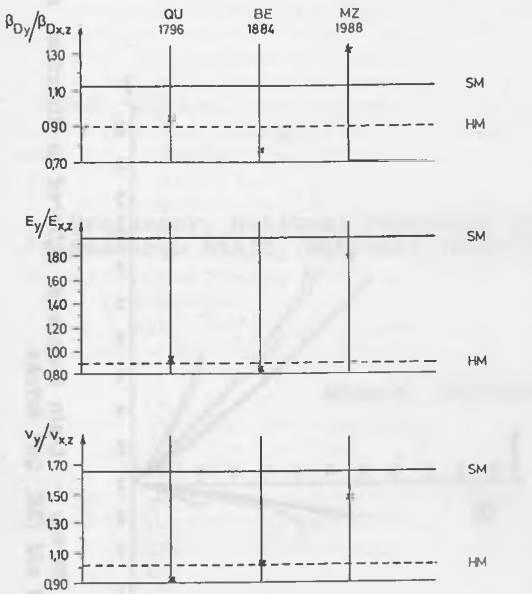
sample ID	β_{Dy}/β_{Dx}	β_{Dy}/β_{Dz}	E_y/E_x	E_y/E_z	v_y/v_x	v_y/v_z
MZ	1.31	1.34	1.69	1.85	1.36	1.58
SM	1.11	1.12	1.68	2.19	1.44	1.86
HM	0.87	0.92	0.88	0.92	0.91	1.11
QU	1.01	0.87	0.86	1.02	0.88	0.93
BE	0.75	0.72	0.82	0.82	1.00	1.13

Table 3 Results of the factors of direction

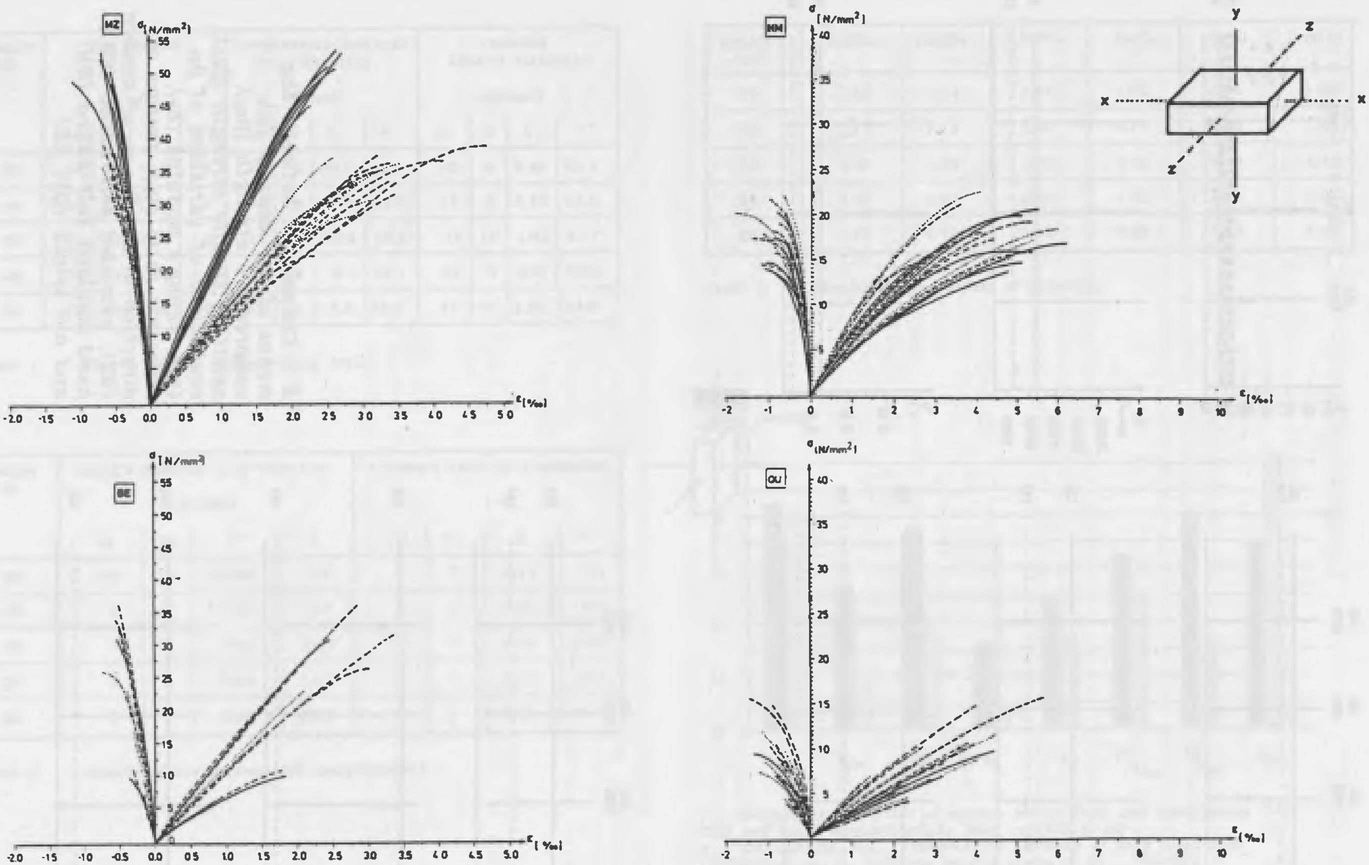


1 Mechanical properties of modern bricks (MZ), low fired bricks (SM) and hand-moulded bricks (HM), related to MZ
ρ ... dry density, β_{Dy} ... compressive strength of drill cores from y-direction, β_{SZ} ... splitting tensile strength, E_y ... Young's modulus, v_{Dy} ... Poisson's ratio, $\beta_{Dy}/\beta_{Dx,z}$ ... factor of direction for the compressive strength, $E_y/E_{x,z}$ ... factor of direction for the Young's modulus, v_{Dy} ... coefficient of variation of the compressive strenght in the y-direction.





2 Comparison between the means of dry density (ρ), compressive strength (β_{Dy}), splitting tensile strength (β_{SZ}), coefficient of variation of β_{Dy} (v_y), Young's modulus (E_y), Poisson's ratio (ν_y), factors of direction of β_D, E, ν, of modern (MZ), extruded self-baked (SM), hand moulded self-baked (HM) and old bricks (QU, BE).



3 Stress - strain diagrams, differed in direction, of modern (MZ), hand-moulded self-baked (HM) and old (BE, QU) bricks.

FAILURE OF MASONRY UNDER HETEROSEMOUS BIAXIAL STRESSES

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SUMMARY

The paper describes a simple exprerimental technique (Fig.1) for determination of the most decisive part of the failure envelope of masonry plates, subjected to plane stresses of opposite -sign ("heterosemous"), i.e. compression and transversal tension (Fig. 2).

The need for pragmatic data on the subject is apparent: Several loading situations lead to these heterosemous conditions which may result in drastic reductions of compressive (or tensile) strengths.

An additional parameter entering the σ_1, σ_2 interplay has to do with the slope of the principal stress direction versus masonry layers. This fact is treated separately in the paper.

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Athens, September 1989.

1._ INTRODUCTION

In-plane loaded masonry walls (e.g. under seismic loading or under differential settlements, etc.) fail under compression /tension ("heterosemous"(*)) plane stress conditions. Under such conditions, the structural characteristics of masonry are dramatically different than under uniaxial compression: A rapid decrease of compressive strength is observed when a small transversal tensile-stress acts simultaneously; besides, a more brittle failure is exhibited.

These facts are not always correctly taken into account when modelling the structural behaviour of masonry, or (more frequently) a linear critical "σ_1/σ_2" model valid for concrete is adopted. However, low strength masonry deviates very much from such a model, as it will be reminded in this paper too.

Therefore, a realistic study of in-plane behaviour of masonry necessitates a thorough study of the biaxial stress condition.

This paper limits its field of study only in compression /tension stress domain. Mainly experimental results are reported.

2._ SHORT LITERATURE SURVEY

Previous research aimed more frequently to the compression /compression domain; however, there is a considerable amount of work related to the heterosemous behaviour as well.

Fig. 1 shows a schematic and only indicative representation of the anticipated failure domains of a flat-blocks' masonry. An intensive anisotropy is apparent, practically unknown in the case of concrete. Besides, unlike concrete, the role of a lot of parameters may drastically modify such failure criteria in the case of masonry: Mortar to block strength ratio, joint to block thickness ratio, efficiency of vertical joints, curing effects, etc; their combination may lead to a large variety of seemingly contradictory results, which render impossible the formulation of a "simple" universally applicable criterion (such as it has been attempted in literature under an abstract "applied-mechanics" inspired approach).

In the more restricted area of heterosemous domain, we have reproduced in Fig. 2 some experimental results, all normalised to the compressive strengths of the respective masonries normal to their bed-joints or, alternatively (Fig. 2c), normalised to the compressive strength under 45° against bed-joints.The basic data of the respective investigations are given in Table 1.

(*) From greek heteros = other, different, and sema = sign

Table 1: Basic data of the investigations reported in Fig.2 (strengths in MPa).

No	Authors	f_{bc}	f_{mc}	$f_{wc}90°$	$f_{wt}90°$
I	Page, 1982	15.41	5.08	7.50	0.40
II	Albanesi et al., 1981	-	-	9.50	0.37
III	Samarasinghe et al., 1980	30.50	9.60	20.00	0.32

From the sketch of Fig. 3 it becomes clear how unrealistic is to follow a "concrete"-like failure criterion. It is not only that errors larger than 100% may frequently happen, but even physical phenomena tend to be misinterpreted: When, under a given small tensile stress, transversal compression develops, then masonry resistance is in fact i n c r e a s i n g instead of the reduction predicted by the linear criterion. Of course, in the case of a strong masonry or a fully jointed cube-blocks' masonry, the linear criterion is a more pragmatic approximation.

This paper is but a small contribution towards a better understanding of these phenomena.

3._ EXPERIMENTAL INVESTIGATION

a) In order to facilitate the experimental study of full scale masonry under plane heterosemous stress conditions, a DOUBLE DIAGONAL compression test (Fig. 4) has been conceived. Thus, biaxial conditions may be easily applied in the central area of the masonry panel tested.

b) In fact, based on the stress expressions for a unidiagonal compression test (Fig. 5), one finds the following expressions regarding the double diagonal test; their low level of precision has to be recognised:

$$\sigma_{1c} = \frac{2}{\pi * d * t_w}(k*P_1 - P_2)$$

$$\sigma_{2t} = \frac{2}{\pi * d * t_w}(P_1 - k*P_2) \qquad \text{(Equ. 1)}$$

Under elastic isotropic conditions, it may be taken $k \doteq 3$ at the central point. However, for the problem we are facing here, it is advisable to use a much lower k-value to account for an average σ_y-distribution (Fig. 5) within a broader area (approx. four bricks lengths) where failure is expected to develop. Thus, elastoplastic redistribution of stresses was roughly considered (only for compressive stresses) by taking in this study $k \cong 1.5$; obviously, this matter is expected to introduce a considerable uncertainty to be studied at a later stage.

If now

$$\lambda = P_2/P_1 \qquad \text{(Equ. 2)}$$

denotes the ratio of bidiagonally acting loads at the moment of failure, Equ. 1 becomes

$$\left.\begin{aligned} \sigma_{1c} &= \sigma_o*(k - \lambda) \\ \sigma_{2t} &= \sigma_o*(1 - k*\lambda) \\ \text{where} \quad \sigma_o &= 2*P_1/\pi*d*t_w \\ \text{and} \quad k &\cong 1.5 \end{aligned}\right\} \qquad \text{(Equ. 3)}$$

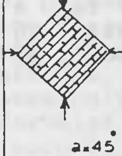
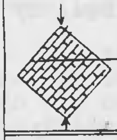
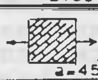
It is apparent that λ-values dictate the stress domain at the central area of the panel:

$\lambda \leqslant \frac{1}{k}$ compression/tension

$\lambda \geqslant \frac{1}{k}$ compression/compression

c) Some of the tests carried out in the Laboratory of Reinforced Concrete NTUA are reported in Table 2.

Table 2: Experimental data and results of the investigation

Loading scheme	No	$\lambda = P_2/P_1$	P_{1cr} (KN)	d_x (mm)	d_y (mm)	t_w (mm)	Experimental results f_{bc} (MPa)	f_{mc} (MPa)	f_{wc}^{90} (MPa)	f_{wt} (MPa)
	1	0.300	120	1100	1100	86	7.66	2.050		
	4	0.150	161	975	975	86	7.66	5.110		
	5	0.150	172	1037	1037	86	7.66	5.110		
	8	0.416	330	1040	1050	86	7.66	4.920		
a=45	10	0.416	333	1025	1024	86	7.66	4.920		
	15	0.637	442	1025	1035	86	7.66	3.880		
	18	0.637	463	1035	1035	86	7.66	3.140		
	19	0.000	122	1022	1018	86	7.66	3.040		
	20	0.000	123	1020	1020	86	7.66	2.290		
a=90°	25		358	955	982	86	7.66	3.140	4.180	
	26		3.5	1035	1040	86	7.66	4.351		0.039
a=45	27		3.0	1035	1030	86	7.66	2.748		0.028

The tests No 25 and No 26, 27 were <u>uniaxially</u> carried out under different bond-joints' angles:

No 25 $\alpha = 90°$ compression ($f_{wc}^{90°}$)

No 26, 27 $\alpha = 45°$ tension ($f_{wt}^{45°}$)

It is worth noting that because of the small compressive strength of the blocks and the mortars used (and indeed of the masonries themselves) the tensile to compression strengths ratios measured in the present investigation were comparatively higher than in the References mentioned in Table 1.

d) In Fig. 6, the experimental results are illustrated on a σ_1, σ_2 critical curve for the angle $\alpha = 45°$ studied (an angle which seems to present the highest practical interest for the verification under seismic and differential settlements' conditions).

Biaxial stresses were calculated by means of Equ. 3. Nevertheless, a slight homogenisation has been made to account for the different materials' strengths in each test (or couples of tests), as shown in Table 3. To this purpose, an empirical formula was used predicting the uniaxial compressive strength of low strength masonries (Tassios, Chronopoulos, 1986).

$$f_{wc}^{90°} = \frac{1}{2} f_{bc} + 0,10 * f_{mc} \qquad \text{(Equ. 4)}$$

Table 3: Stress values [MPa] and corrections (average values for each couple of tests)

Test No	calculated		$f_{wc}^{90°}$ (Equ.4)	factors	corrected	
	σ_1	σ_2			σ_1	σ_2
1	-0.97	+0.44	4.03	0.96	-1.01	+0.46
4,5	-1.58	+0.91	4.34	1.06	-1.49	+0.86
8,10	-2.58	+0.91	4.32	1.03	-2.51	+0.88
15,18	-2.74	+0.16	4.18*	1.00	-2.74	+0.16
19,20	-1.33	+0.88	4.10	0.98	-1.36	+0.90
26	-	+0.039	4.26	1.02	-	+0.038
27	-	+0.028	4.10	0.98	-	+0.029

[(*) reference strength]

4.- DISCUSSION

Although the shape of the critical curve shown in Fig. 6 is quite similar to some of the curves reproduced in Fig. 2c, it has to be noted that the initial assumption of an appropriately "averaging" k-value (3b) may introduce a considerable change to the higher part of the curve. Otherwise, the "double diagonal" compression test seems to be a rather helpful instrument for the study of biaxial behaviour of in-plane stressed masonry panels.

Analytical predictions on the basis of detailed physical modelling reported elsewhere (Tassios, 1989) seem to confirm, roughly though, the findings of the present investigation, supposing however that even vertical joints of the masonry were fully operational.

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TASSIOS T.P., CHRONOPOULOS M.: " Aseismic dimensioning of inter ventions (repairs/ strengthenings on low-strength masonry buildings", Middle East and Mediterranean Regional Confer ence on Earthen and low-Strength Masonry Buildings in seis mic areas, August 31- Sept. 6 , 1986 , Middle East Tech. University, Ankara, Turkey.

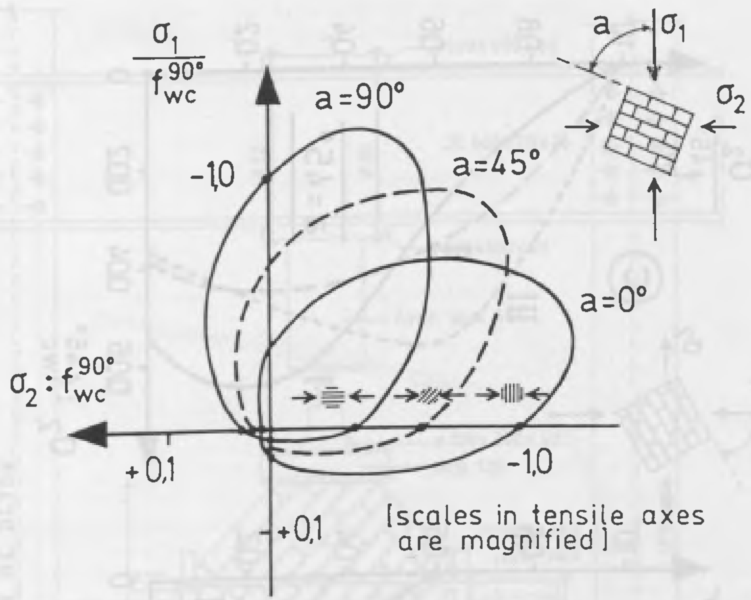

Fig. 1: Schematic representation of critical principal-stress combination of flat-blocks' masonry. Real behaviour may greatly differ of this schematisation.

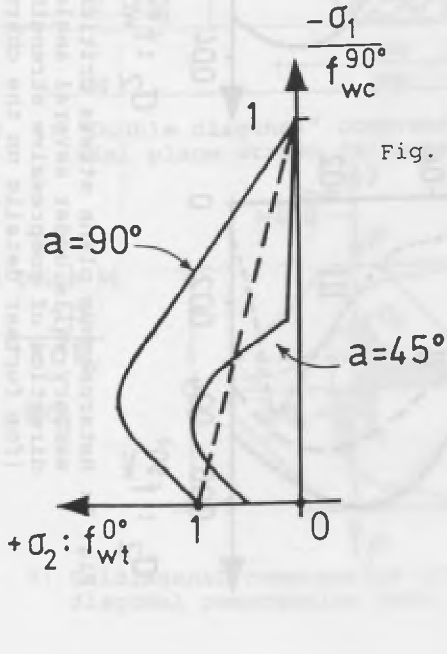

Fig. 3: An unrealistic linear criterion in the heterosemous domain may introduce serious errors.

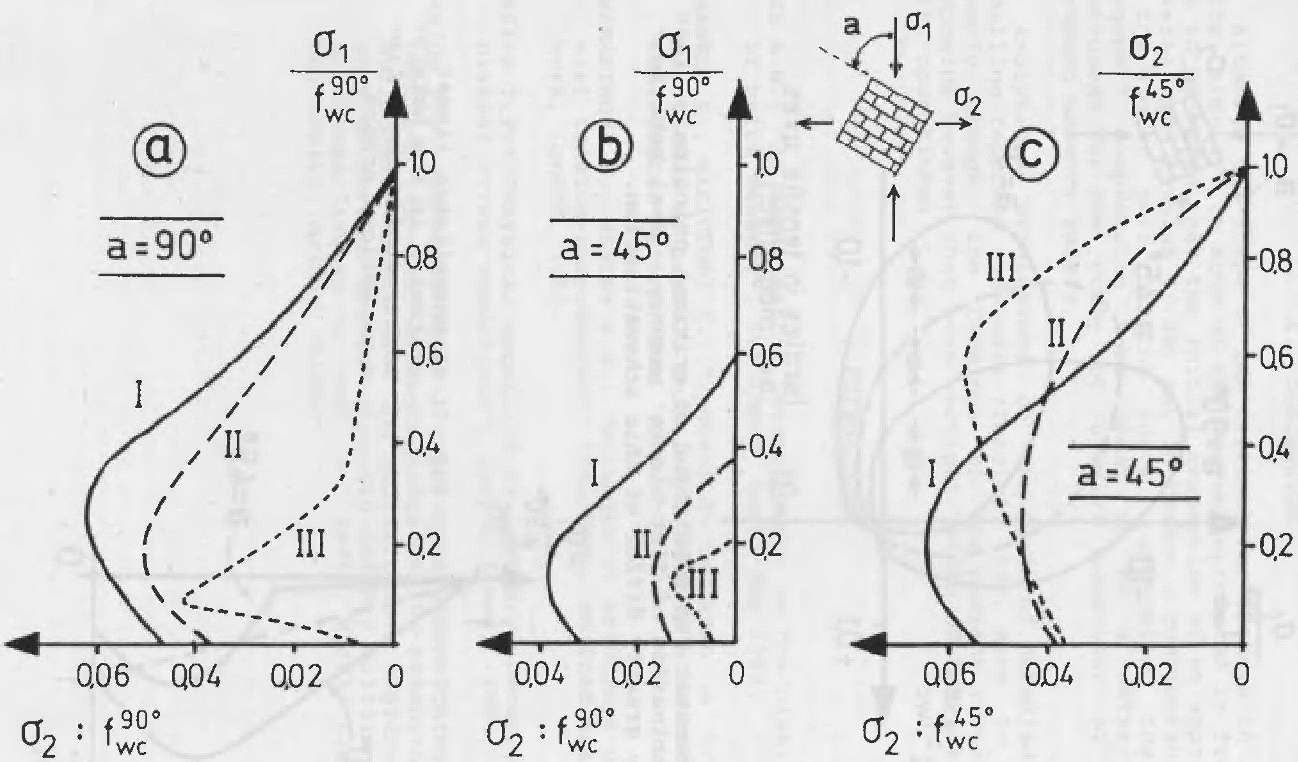

Fig. 2: Heterosemous plane stress critical curves of brick masonry walls under several angles "α" between the direction of compressive strength and the bed-joints; (for further details on the characteristics of masonries, refer to Table 1).

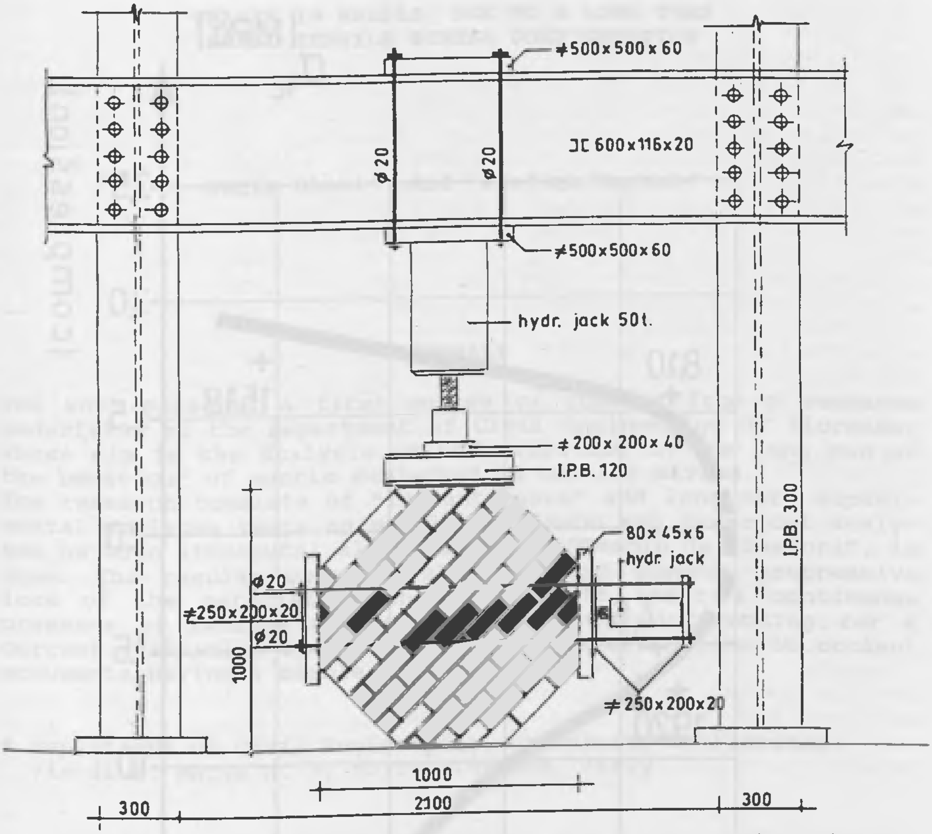

Fig. 4: "Double diagonal" compression test for studying biaxial plane stress failures in masonry.

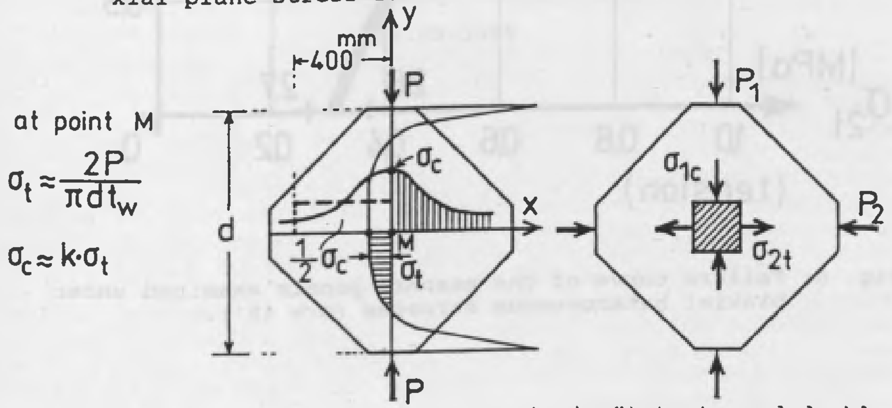

Fig. 5: Unidiagonal compression ("splitting") test, and double-diagonal compression test.

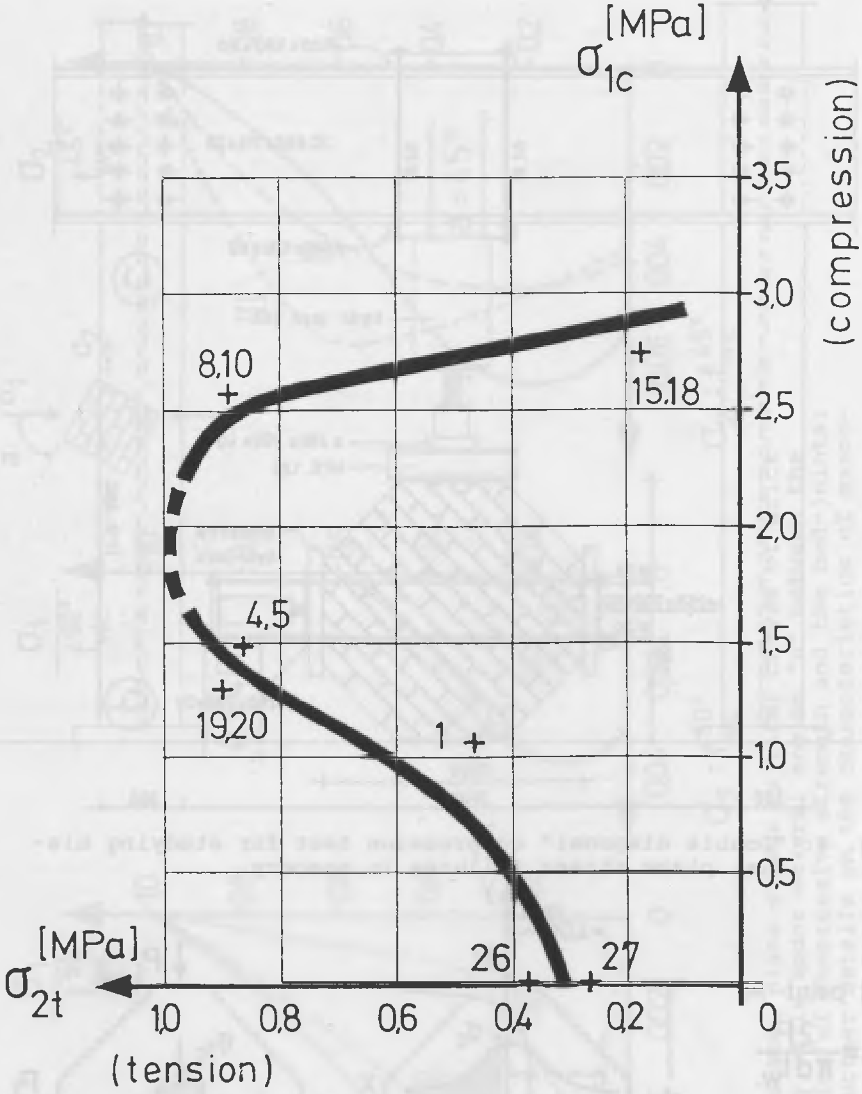

Fig. 6: Failure curve of the masonry panels examined under biaxial heterosemous stresses ($\alpha = 45°$).

CRACKS IN MARBLE DUE TO A LONG TERM SMALL TENSILE STRESS CONFIGURATION

Carlo Blasi* and Stefano Sorace*

SUMMARY

The work presents a first series of results from a research undertaken at the Department of Civil Engineering of Florence, whose aim is the analysis of the evolution in the long run of the behaviour of marble subjected to tensile stress.
The research consists of "instantaneous" and long term experimental collapse tests on marble specimens and numerical analyses on some structural elements of the "Tempio dei Castori", in Rome. The results obtained show a trend towards progressive loss of the material's tensile strength under a continuous presence of tensile stress, which is very interesting for a correct evaluation of possible repair interventions on ancient monuments having a marble structure.

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02.05.1989

INTRODUCTION

It's easy to find cracks in marble structural elements which, regarding to shape and position, are typical of a tensile stress presence. However, many numerical studies conducted on historical buildings, in particular monuments of Ancient Rome, revealed the presence of cracks in elements constantly subjected to tensile stresses decidedly inferior to the tensile strength usually attributed to the material.
This obsevation induced us to make an accurate study of the phenomenon, which is very important not only in order to explain the various types of cracked states that may be found in ancient monuments, but also for an opportune evaluation of repair techniques. In fact, some failures registered in repair and strengthening interventions which make use of steel elements (bars, plates, etc.) can be related to an insufficient knowledge of the stress state induced by these interventions as well as of the actual tensile strength capacities of the materials.

The research was divided into two parallel sections.

1. Experimental laboratory analyses for determining the tensile strength of marble specimens, with "instantaneous" and long term tests.
2. Theoretical and numerical analyses of the stress states which have produced fractures in ancient structural elements.

For the second section, in particular, three structural elements of the "Tempio dei Castori", in the Roman Forum, have been studied: two architraves with clear shear fractures and a column drum subjected to combined axial and bending stress.

EXPERIMENTAL LABORATORY TESTS

The laboratory tests conducted consist of compressive and bending collapse proofs on marble specimens according to the scheme represented in Fig.1.
Two kinds of marble specimens have been examined: the first obtained from ruined parts of the Tempio dei Castori, the second from blocks of good quality Carrara marble.
The first kind of specimen has been subjected only to bending instantaneous collapse tests that have given an average tensile strength of 2.83 N/mm^2 with a standard deviation of 0.878.
Some of the Carrara marble specimens have been subjected to compressive or tensile (in this case due to bending action) instantaneous collapse proofs; the average strengths found are equal to 151.20 N/mm^2 for the compressive tests, and to 16.48 N/mm^2 for the tensile ones (standard deviation of 1.41, calculated for 30 specimens).

The average load employed in the bending proofs, collocated in the middle of the horizontally placed specimens, is equal to 144.4 N. The remaining specimens, divided in three groups, have been subjected to permanent loads equal to 122.2 N, 108.3 N, and 80.2 N corresponding, respectively, to 85%, 75% and 55% of the collapse load.

The diagram of Fig.2 reproduces the average times in which it has showed the collapse for the first two values of the load while, at present, the specimens subjected to the third value have not yet collapsed.
The diagram is drawn on the hypothesis of linear regression on a time logarithmic base; this hypothesis describes with good approximation the behaviour of the Carrara marble specimens.
For the Castori marble specimens the cracks are thought to have been produced in recent times; if they had arisen in remote times the straigth lines for the two kinds of marble would tend to be superimposed. In this case the residual strength would be reduced to about 25% of the instantaneous one.
The diagram is certainly very approximate, being realized with an insufficient number of data, but it showes the trend towards loss of tensile strength in marble interested by a continuous presence of tensile stress.

ANALYSIS OF THE CRACKED STATE OF THE ARCHITRAVES AND A COLUMN DRUM OF THE TEMPIO DEI CASTORI

The remains of the Tempio dei Castori, rebuilt in its final form under the Emperor Hadrian, consist of three Corinthian columns and two architraves surmounted by a frieze.

The architraves present an almost identical cracked state, defined by two fractures with a slope of 45° in correspondence with the bearings and a perfectly horizontal and rectilinear fracture as showed in Figures 3 and 4.
The architraves, actually strengthened with large horizontal steel bars, have been studied by means of a numerical analysis using the finite element calculus program ANSYS (AA.VV., 1986, /1/). On the basis of this code we developed a model, including monolateral elements, which was applied with good results in previous works (Blasi-Sorace, 1989, /2/; Chiostrini-Foraboschi-Sorace, 1989,/3/) to the study of masonry or stone structural elements.
In the case under study the code has been applied in the linear elastic version because the aim of the analyses was the evaluation of the stress state originally present in the uncracked architraves, which caused the subsequent formation of the fractures described above.
Fig.5 reproduces the tensile and compressive isostresses obtained from the solution.
We can observe that the maximum values of the tensile stresses (about 0.5 N/mm^2) are present in the zones near the bearings, exactly where the first cracks had arisen.

The compressive isostress shape clearly showes an "arch effect" typical of these structural elements under the load transmitted by the frieze, whose presence in Roman temples had both an aesthetic and a structural function.
In fact, from a structural point of view, the introduction of a very rigid element over the architraves induces quite considerable shear stresses and small bending stresses.
So, the actual scheme of the architraves is typical of a shear collapse, which is confirmed by the shape of the compressive isostress indicated with letter I in Fig.5.b, that exactly repeats the inferior fractures' surfaces present in the elements.

In Figures 6 and 7 the cracks present in a drum of a column of the temple are shown. The column is subjected to an eccentric axial load due to the ruin of adjoining parts of the building. A numerical analysis analogous to the one conducted for the architraves has given a value of about 0.9 N/mm² for the tensile stress which caused the fracture.

COMPARISONS BETWEEN NUMERICAL AND EXPERIMENTAL ANALYSES AND CONCLUSIONS

The numerical analyses conducted on some marble structural elements of the Tempio dei Castori have given very small values of the tensile stresses in the zones actually cracked.
The average tensile strength shown by these elements subjected to long term loads represents only about 25% of the average strength experimentally found with instantaneous collapse tests conducted on specimens of the same material.
These evaluations are confirmed by a first series of results of experimental analyses developed on another kind of marble, of much better quality, that show a progressive loss of material tensile strength in the long run, under the continuous presence of load. Other results of these analyses will be given in further studies but, at present, we can still underline the importance of the "time" factor in marble failure and, for this, in the design stress evaluation of structures built in these materials.

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Fig.1 : Bending collapse tests on the marble specimens

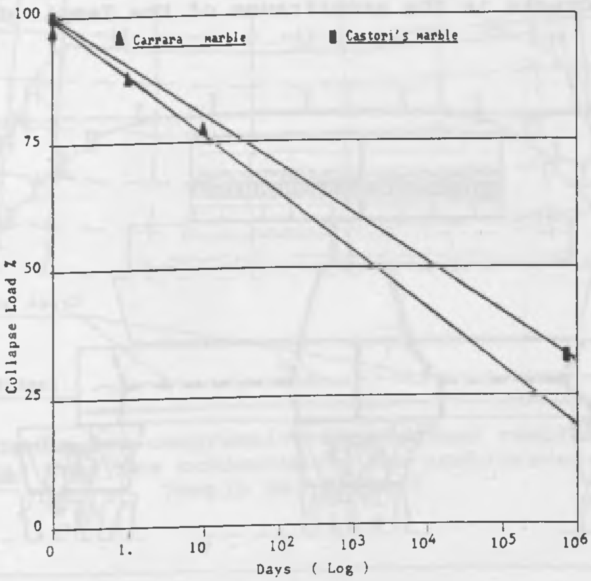


Fig.2 : Loss of tensile strength of the marble specimens subjected to time constant loads

Fig.3 : Cracks in the architraves of the Tempio dei Castori

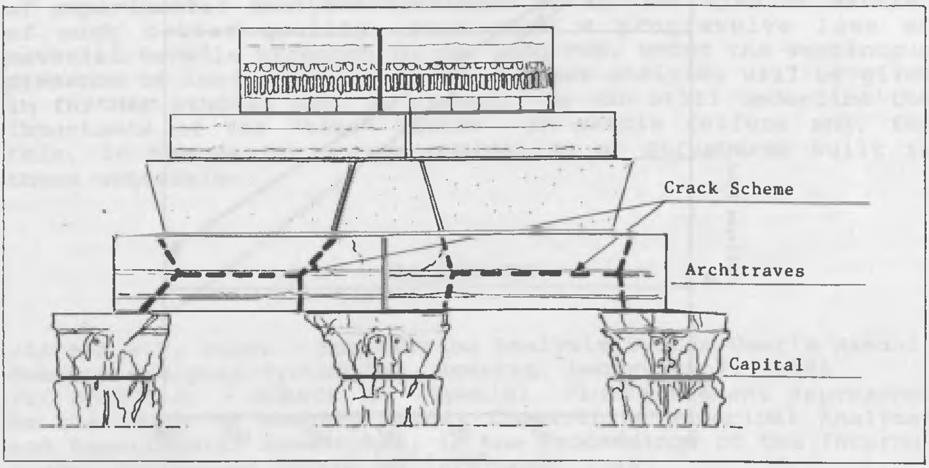
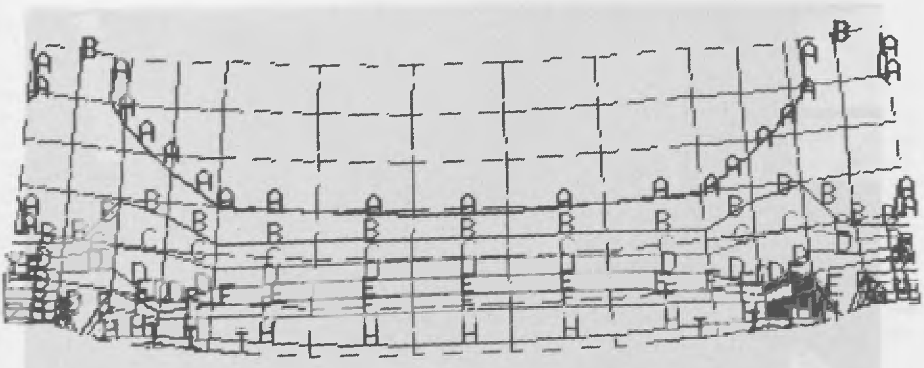
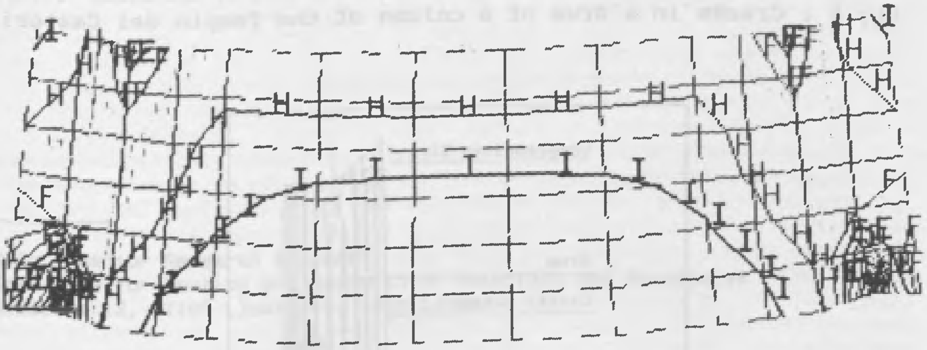


Fig.4 : Schematic representation of the actual cracked configuration of the architraves



a.



b.

Fig. 5 : Tensile and compressive isostresses resulting from the numerical analyses conducted on the architraves of the Tempio dei Castori

Fig.6 : Cracks in a drum of a column of the Tempio dei Castori

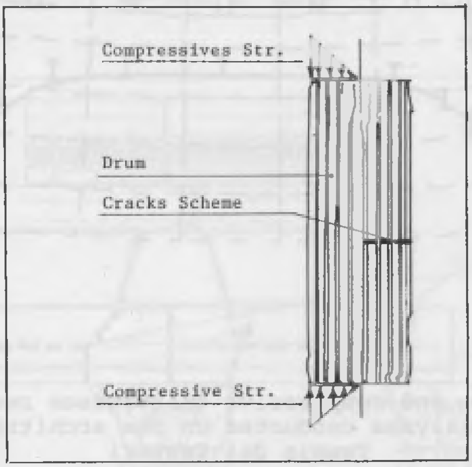


Fig.7 : Schematic representation of the actual cracked configuration of the drum

ACKNOWLEDGEMENTS: We should like to thank Arch. Tedone of Archeological Superintendent Authoriti of Rome.

STRENGTHENING OF MASONRY VAULTS BY FOAM CONCRETE APPLICATION

Roko Žarnić

SUMMARY

The main idea of the proposed method is to increase the load-bearing capacity of vaults as well as their seismic resistance by replacing heavy gravel topping with a reinforced, light-weight foam-concrete topping, connected to the surrounding walls by steel anchors. Two of vaults with a gravel and foam concrete topping have been tested by applying increasing vertical loading and two of them by applying cyclic horizontal loading. A significant increase in load-bearing capacity and stiffness has been observed as well as improvement in the mechanism of behaviour of masonry vaults subjected to the vertical and horizontal loading.

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INTRODUCTION

The revitalization of old monuments and urban nuclei leads to improvements in the living conditions of their inhabitants. At the same time the monumental value of buildings is preserved, and parts become accessible to the public use. Special attention has to be paid to the repair and strengthening of existing structures in order to achieve adequate load-bearing capacity and earthquake resistance. For developing methods for repair and strengthening, experience obtained from the design practice in the solving of problems connected with the restoration of old structures and their behaviour during past earthquakes is of the great value. In the present paper an original method, which was developed for design practice and has been verified by laboratory investigations, is proposed.

Several methods and techniques for the strengthening of masonry vaults have been developed in past. Two of them are often used in practice: the fastening of vaults by r.c. horizontal ties and diagonal steel ties (Aničić, 1983) and the applying of an additional r.c. shell above the masonry vault (Gregorian, 1986). In the both cases, gravel or some other topping has to be placed over the vault after strengthening, which increases the mass and loading of the original structure. In the case of the proposed method, the foam-concrete topping replaces the original gravel topping. It acts as an integral part of of the entire structure, having the equal or even lower specific weight than the original gravel topping.

THE STRENGTHENING OF VAULTS

Masonry vaults in old buildings have in many cases, to be repaired due to damage caused by displacements of the bearing walls or decay of materials. The load-bearing capacity of existing vaults is sometimes insufficient, and because of the inadequate fastening of the surrounding walls the seismic resistance of the entire masonry structural system is inadequate. During the design of the renewal of old buildings in the historic urban centre of Ljubljana, an original method for the strengthening of masonry vaults has been introduced into practice. By applying this method, good results have been obtained, especially because of the simple technology used for producing of foam-concrete on site.

The technique is based on the application of lightweight foam concrete, called PENOBETON, which has been developed at the Institute for Testing and Research in Materials and Structures in Ljubljana. This lightweight concrete can be produced at the construction site by mixing organic foam with concrete of usual characteristics. The mechanical characteristics of foam concrete depend on the quantity of foam added to the concrete. The recommended average specific weight of foam concrete for masonry vaults application is about 1600 kg/m^3, and it has a compressive strength of 20 MPa. The recommended weight of foam concrete is about the same as that of masonry walls, and less than that of the gravel which is usually used for the topping of vaults. The advantages of this material are: workability, a low specific weight with sufficient strength, and a low modulus of elasticity (3 to 4 times less than the modulus of elasticity for normal concrete).

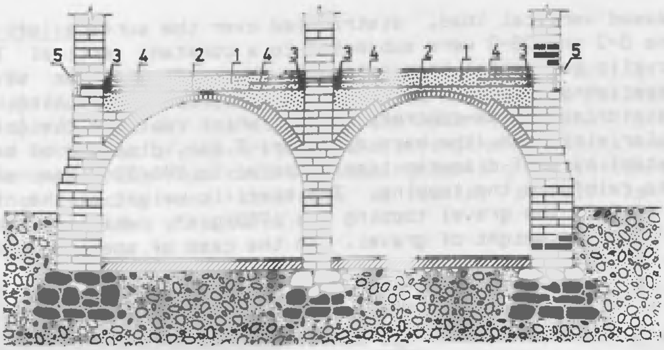

Fig. 1: The strengthening of masonry vaults

Before the foam concrete is applied, the gravel topping has to be removed, the upper surface of the vault cleaned, and cracks and other defects repaired by the usual techniques. The foam concrete topping should be reinforced by wire mesh; minimal reinforcement is sufficient. The thickness of the topping above the highest point of vault may vary from the thickness of the vault to the thickness of the removed gravel topping. The wire mash has to be placed about 30 mm above the vault, and fixed to the surrounding walls by steel anchors. It is recommended that steel bars of diameter 16mm to 20mm be used on each 1.50 to 2.0 m of surrounding wall lengths anchored to the wall by means of rectangular steel plates and nuts. The usual length of the steel bars in the foam-concrete topping is 1.0 m. It is recommended that additional wire mesh, about 1.0 m wide, be placed above the middle of vault, below the upper surface of foam-concrete topping, to prevent the cracks which can develop because of the shrinkage of foam concrete.

INVESTIGATION OF VAULTS

Description of tests

The assumed behaviour of strengthened vaults needed experimental verification. The main intention of the investigations was to study the difference between the behaviour of vaults with the usual type of gravel topping, and that of strengthened ones. Two one-bay masonry vaults, in 1:2 reduced scale, have been constructed using materials with prototype characteristics (Tab.1). The main dimensions of the specimens are shown in Fig.3. Clay bricks (250/125/25 mm) and lime mortar have been used for construction of the walls, and smaller bricks (125/65/35) for the construction of the vaults. The thickness of the vaults was 65 mm in the middle part, and 135 mm in the side parts. Gravel with a specific weight of 1920 kg/m^3 has been used as a topping up to 120 mm over the top of the vaults.

The specimens were fixed onto the testing floor. Specimens O-1 and OS-1 were subjected to a constant vertical load of 50 kN acting on the walls and a gradually increased vertical load, distributed over the surface of the topping. Specimens O-2 and OS-2 were subjected to a constant vertical load of 50 kN and a cyclic horizontal load acting on the walls. After severe cracks occurred, specimens O-1 and O-2 were repaired by cement grouting, and strengthened by reinforced a foam-concrete topping which replaced the gravel topping. Rectangular wire mash (the bars diameter: 3.2mm, distance of bars: 50mm) and smooth steel bars of diameter 16mm anchored to 200/200/12mm steel plates were used to reinforce the topping. The specific weight of the foam concrete used to replace the gravel topping was 1750kg/m^3, which is about 10% less than the specific weight of gravel. In the case of specimen OS-2, the difference of vertical loading was made up for by lead.

Table 1: Mechanical Properties of the Constituent Materials (MPa)

	MORTAR	MASONRY	FOAM-CONCRETE
Compressive strength of cube	0.62	-	24.1
Compressive strength of prism	1.23	6.9	19.3
Bending strength of prism	0.38	-	4.8
Modulus of elasticity	-	600	7390

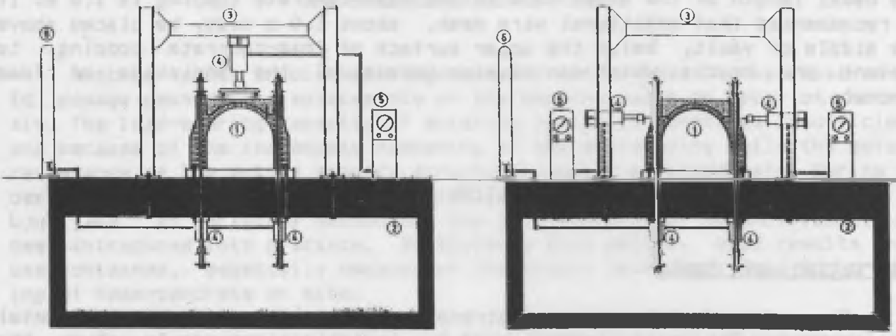

Fig.2: Set-up for the testing of vaults, with gradually increasing vertical load (a) and cyclic horizontal load (b)

During the tests, the forces acting were measured by means of electric dynamometers, the displacements of the specimens and strains in the masonry were measured by means of LVDT-s and electric dilatometers, and the strain in the reinforcement was measured by strain-gauges, as can be seen in Figs.3 and 4.

Test results

By studying the relationship between the acting loads and the deformation of the tested specimens, as well as the relationships between the lateral load and the strain of the masonry and the reinforcement, the failure mechanism of the original and strengthened vaults was analysed. It was found that the foam concrete topping has a great influence on the load-carrying capacity and stiffness of vaults, as well as on the entire behaviour of these structures.

Fig.4: Specimen O-1

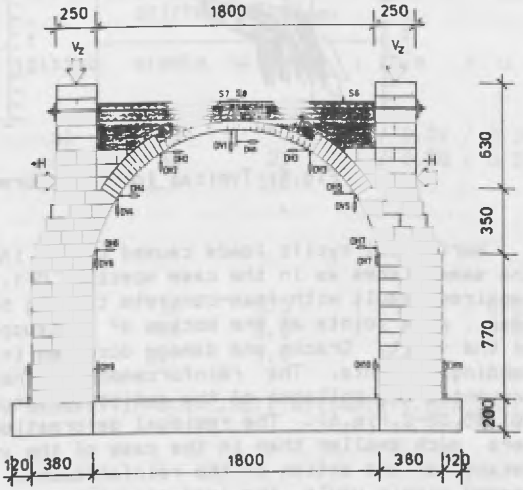

Fig.3: The istrumentation of specimen OS-2

The mechanism of behaviour of the strengthened vaults was entirely different from that of the virgin vaults with gravel topping. The first cracks in the vault with gravel topping which was subjected to the vertical load developed at the top of the vault. With increasing load, additional cracks developed at quarters-span, where the cross-section of the vault changes, and at the supports of the vault (O-1,Fig.7). No cracks or damage occurred in the supporting walls. In the case of the repaired and strengthened specimen, a much higher ultimate load, as well as a different pattern of cracks, was achieved. The foam-concrete topping behaved as a slab with a changing cross-section, fixed to the supporting walls. The first cracks developed at the midspan of the vault due to bending, as can be seen from the measured strains of a wire mesh (OS-1,S5 Fig.6). The internal forces at the fixed supports of the "foam concrete slab" have been partly taken up by the steel anchors (OS-1,S1,Fig.6). Approaching ultimate load, horizontal cracks developed in the supporting walls, due to the action of bending moments. Reaching ultimate load, shear cracks caused by the reaction forces developed in one of the supporting walls (OS-1,Fig.7). In the case of the virgin vault, the load-bearing capacity of the structure depended on the load capacity of the vault, whereas the shear strength of the supporting wall has been the critical parameter of load bearing capacity of the repaired and strengthened structure

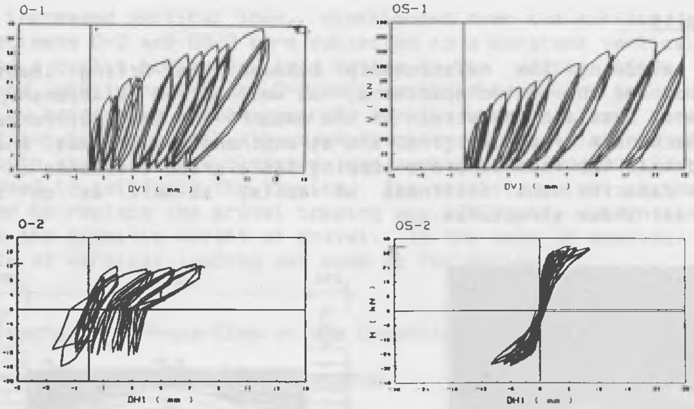

Fig.5: Typical load - deformation loops

Horizontal cyclic loads caused cracks in the virgin vault O-2 nearly at the same places as in the case specimen O-1, as can be seen from Fig.7. The repaired vault with foam-concrete topping behaved as a frame with a stiff beam, with joints at the bottom of the supporting walls and at the supports of the vault. Cracks and damage occurred in the joints due to the action of bending moments. The reinforcement of the topping and the steel anchors prevented the collapse of the entire structure in large deformations range (S1,S5,OS-2,Fig.6). The residual deformations of the strengthened specimen were much smaller than in the case of the virgin specimen (O-2,OS-2,Fig.5) because of the action of the reinforcement. In the case of the horizontally loaded virgin vault, the load-carrying capacity of the structure depended on that of the vault, whereas the flexural strength of the supporting masonry walls was the critical parameter of the load-bearing capacity of the strengthened structure.

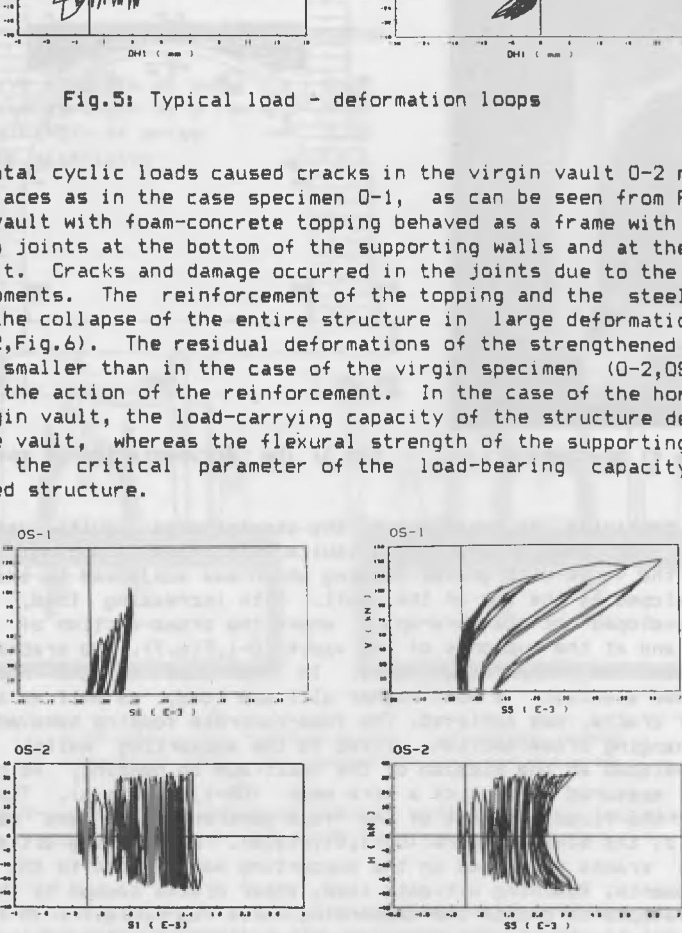

Fig.6: Load - strain loops, measured at the mid-span of the wire mesh (S5) and on the steel anchor bars (S1)

In Table 2 the load capacity and stiffness of the tested specimens is compared. The stiffness of the virgin and strengthened vaults has been compared at the same ratio of the acting loads in comparison to ultimate loading. A comparison of stiffness at equal loads will result in higher differences which will be in about the same range as the differences between the ultimate loads.

Table 2 : Load Capacity and Stiffness of the Vaults

	LOAD CAPACITY (kN)	STIFFNESS (kN/mm)			
	Vertical load V_u	initial	middle	ultimate	i / m / u
O-1	39.2	61	19	6	1 / 0.31 / 0.10
OS-1	134.5	82	48	21	1 / 0.58 / 0.26
OS-1/O-1	3.4	1.3	2.5	3.5	
	Horizontal load H_u				
O-2	14.3	17	12	5.5	1 / 0.71 / 0.32
OS-2	33.2	65	45	5.5	1 / 0.70 / 0.08
OS-2/O-2	2.3	3.8	3.7	1.0	

Stiffness: initial ($V,H=25\%V_u,H_u$), middle ($V,H=50\%V_u,H_u$), ultimate ($V,H=V_u,H_u$)

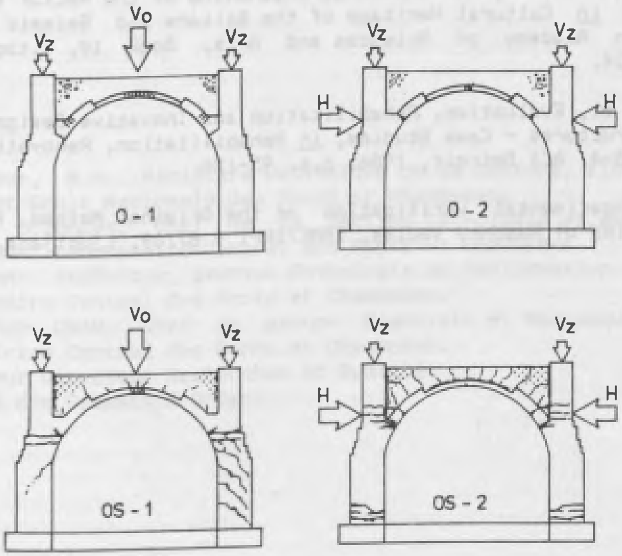

Fig.7: Crack patterns at ultimate load

CONCLUSIONS

Masonry vaults can be successfully repaired and strengthened by replacing gravel topping with reinforced, lightweight foam-concrete, anchored to the surrounding walls. Significant increases in the load-bearing capacity and stiffness of the vaults, as well as in the earthquake resistance of masonry structures, can be achieved using the proposed method of strengthening. In the case of strengthened structures, the shear and flexural strength of the supporting walls are the critical parameters of the load capacity of the entire structure. Strengthened vaults can be easily repaired again, using the methods which are well known in practice.

ACKNOWLEDGEMENTS

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FORMULATIONS DE COULIS HYDRAULIQUES POUR L'INJECTION DES FISSURES ET CAVITES DES STRUCTURES EN MACONNERIE DEGRADEES

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A. DENIS****, N. MUSIKAS*****

RESUME

Hydraulic grouts, although used for a long time for injecting sufficiently large cracks, cannot penetrate into narrow spaces, such as millimetric cracks, because of clogging. This paper presents the results of research carried out by LCPC, on the possibility of changing certain properties of cement grouts by adding ultrafines (silica fumes, lime, earth of Santorin), together with superplasticizers to their formulations. A thorough study of gradings and penetrability characteristics has enabled the formation of hydraulic grouts for injection of fine cracks. Durability tests have shown their behaviour when subjected to an aggresive environment. The mechanical strength values of such grouts have been measured and the role of the association of various types of ultrafines has been examined. Finally, the bond strength of hydraulic grouts to different types of limestone has been studied by means of direct tensile test.

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INTRODUCTION

Les techniques d'injection de coulis sont aujourd'hui couramment employées pour la réparation des structures dégradées. Leur utilisation dans les anciens ouvrages en maçonnerie doit cependant tenir compte d'un certain nombre de particularités liées à la spécificité de ces constructions, à la diversité des désordres constatés (fissures, dégradation des joints ou des interfaces, présence de vides...) et à l'importance des volumes à injecter (Miltiadou, 1985). Par ailleurs, la réparation des monuments historiques impose aussi l'emploi de matériaux d'injection préservant leur qualité esthétique et dont le comportement à long terme soit connu et maîtrisé.

Deux grandes familles de coulis peuvent être retenues : les coulis à base de liants hydrauliques et les polymères (résines thermoplastiques ou thermodurcissables). Ces derniers présentent deux inconvénients importants, leurs caractéristiques mécaniques et physico-chimiques sont très différentes de celles des matériaux constitutifs des structures considérées, de plus, leur utilisation est relativement récente et de ce fait leur durabilité est encore mal connue.

Les coulis hydrauliques ne présentent pas ces inconvénients et sont donc potentiellement très intéressants. Malheureusement, une des difficultés majeures rencontrées lors de leur emploi dans l'injection des fissures ou des cavités est leur essorage dès que la largeur de la fissure ou l'ouverture de la cavité est inférieure à 3 mm. Cependant, compte tenu de leur intérêt, des travaux de recherche, concernant les caractérisques influençant l'injectabilité de cette catégorie de coulis, ont été effectués au LCPC.

Les premiers résultats de ces travaux (Paillère - Guinez, 1984) ont permis de montrer l'importance de la finesse et de la granularité du ciment sur l'injectabilité. Il est apparu ainsi que les ciments injectables ne doivent pas posséder d'éléments supérieurs à 80 µm et que le maximum de grains de dimensions supérieures à 32 µm est de 12 %. Par ailleurs, l'utilisation en proportions variables d'ultrafines telles que la fumée de silice ou la chaux permet de formuler, à partir de ciments commercialisés, des coulis injectables. (Aitcin et al, 1984 ; Paillère et al, 1986 ; Paillère et al, 1989).

Le présent article s'inscrit dans la continuité de ces travaux. En complétant les résultats déjà obtenus, nous avons étudié des formulations de coulis, à partir de ciment associé à des éléments ultrafins et à un superplastifiant, afin d'obtenir des coulis stables et injectables dans de fines fissures. Avec l'amélioration de l'injectabilité, un autre but recherché a été l'obtention d'une gamme de valeurs de résistances assez étendue, afin de pouvoir disposer de plusieurs formulations de coulis hydrauliques pouvant être utilisés selon les performances des matériaux de la structure dégradée. Parallèlement, nous avons abordé l'étude des caractéristiques d'adhérence de ces coulis, vis à vis d'un matériau support constitué par des pierres calcaires de porosité variable. De même, afin de connaître la durabilité, nous avons procédé à l'étude des résistances aux eaux agressives et des caractéristiques physico-chimiques (retrait, porosité...) des formulations de coulis hydrauliques les plus performantes.

1. ETUDE DES PROPRIETES INTRINSEQUES DES COULIS HYDRAULIQUES

1.1. Matériaux utilisés et méthodes d'essais

Les coulis hydrauliques étudiés ont été confectionnés avec les matériaux suivants : ciment, éléments ultrafins, adjuvant, eau. La figure 1 indique les courbes granulométriques des trois ciments utilisés (CPA 55, CPA 55 PM, CLK 45 PM). Trois types d'éléments ultrafins ont été considérés dans cette étude ; la chaux éteinte (Cx), la fumée de silice densifiée (FSD) et la terre de Santorin (TS). La figure 2 indique les courbes granulométriques respectives. Un seul superplastifiant a été utilisé ; le produit G, de fabrication française, à base de résine mélamine formaldéhyde, dont la teneur en extrait sec est de 33 %. Cet adjuvant ne possède, vis à vis des ciments, aucun effet secondaire de retard de prise. Compte tenu des recherches précédentes (Paillère et al, 1989), nous n'avons utilisé dans cette étude qu'un type de malaxage : la dispersion par ultra-sons pendant une durée de 4 minutes. Les caractéristiques de l'appareil à ultra-sons étaient les suivantes : Puissance 250 W, fréquence 20 KHz.

L'injectabilité a été mesurée à l'aide de l'essai d'injectabilité à la colonne de sable (Paillère - Rizoulières, 1978 ; Ferrari - Malliet et al 1981). Cet essai est défini dans les recommandations RILEM du TC 52 RAC (Resine adherence to concrete) et dans la norme française NF-P 18-891. La figure 3 représente l'appareillage utilisé. Le principe de cet essai consiste à injecter, sous pression constante, le coulis dans un tube de plastique transparent (A) gardé en position verticale et rempli d'un sable de granularité donnée. La colonne est injectée à partir de sa partie inférieure et l'on mesure le temps mis par le produit pour atteindre les différents repères de la colonne. La mesure du temps nécessaire au coulis pour atteindre le niveau supérieur de la colonne (36 cm) caractérise son injectabilité. La pression d'injection utilisée a été de 0,075 MPa et la granularité du sable remplissant la colonne était de 0,63/1,25 mm ; ce qui correspond à une ouverture de fissure de 0,2 à 0,4 mm.

Pour la mesure des résistances mécaniques, on a procédé à l'essai brésilien, ou de fendage, conformément à la norme française NF P 18 892, sur des éprouvettes cylindriques obtenues par sciage de la colonne de sable préalablement injectée et durcie. Les essais ont été effectués après 28 jours de conservation à 20° c + 2° et dans une humidité relative de 95 %.

1.2. Principaux résultats obtenus

1.2.1. Etude de l'injectabilité

Influence de la granularité

Conformément aux travaux précédents, les trois ciments étudiés dont la courbe granulométrique ne remplit pas les exigences granulaires (fig. 1) (0 % d'éléments > à 80 µm et moins de 12 % > à 32 µm) conduisent à des formulations de coulis non injectables (tableau 1). De même, l'exsudation ou la sédimentation sont inacceptables.

Association ciment - éléments ultra-fins

Les tableaux 2a, 2b, 2c présentent les résultats obtenus pour des formulations dans lesquelles les ciments considérés sont associés aux ultrafines de l'étude (fumées de silice, chaux, terre de Santorin). La figure 4 montre l'évolution des courbes granulométriques de l'association ciment CPA 55 PM - Terre de Santorin, en fonction du pourcentage de terre de Santorin. Cet ajout d'éléments ultrafins permet l'obtention de coulis possédant de bonnes caractéristiques d'injectabilité, comparables ou supérieures à celles des polymères (fig. 5).

Influence des paramètres teneur en eau et pourcentage d'adjuvant

Les paramètres teneur en eau et quantité de superplastifiant jouent un rôle important sur l'injectabilité. Les résultats obtenus montrent, en effet, qu'il existe une teneur en eau minimale associée à chaque composition de la phase solide. La figure 6 fait apparaître que cette teneur en eau minimale augmente avec la finesse de la phase solide. Il faut noter également que si l'augmentation de la teneur en eau permet une diminution du temps de passage à la colonne, elle s'accompagne généralement d'une élévation du pourcentage d'exsudation (fig 7). La figure 8 présente le rôle joué par le couple adjuvant-teneur en eau. A teneur en eau constante, les temps d'injection diminuent avec l'augmentation en superplastifiant.

Nous avons approfondi notre recherche en essayant d'obtenir pour chaque formulation étudiée, un coulis injectable (temps de pénétration de la colonne inférieur ou égal à 1 min), ayant une teneur en eau minimale, ne présentant pas de phénomène de sédimentation et ayant une exsudation inférieure à 5 %. La figure 9 donne le pourcentage de superplastifiant en fonction du rapport E/C. Elle permet de mettre en évidence une "plage d'injectabilité" limitée par des formulations non-injectables, ou avec des exsudations et sédimentations non acceptables. Nous avons ainsi obtenu pour chacune des associations considérées, de ciment-ultrafines, des formulations de coulis stables et injectables avec un rapport E/C variant entre 0,75 et 1,25 et un pourcentage de superplastifiant variant entre 0 et 2. Il faut attirer l'attention sur le fait que les teneurs en eau et en adjuvant sont d'autant plus élevées que la finesse de la phase solide (ou le pourcentage d'ultrafines) du coulis augmente.

1.2.2. **Influence de la composition sur les caractéristiques mécaniques**

Un autre des objectifs de cette recherche, a porté sur la détermination des paramètres influençant les caractérisques mécaniques des coulis. Les résultats du tableau 3 montrent l'influence respective des éléments ultrafins, chaux et fumées de silice, sur la résistance au fendage des éprouvettes confectionnées à partir de la colonne de sable injectée.

Il apparaît ainsi, que cette résistance diminue quand la teneur en chaux augmente, tandis qu'elle croît avec la teneur en fumée de silice. Ces résultats sont tout à fait cohérents compte-tenu de la diminution de liant actif dans le premier cas et de l'augmentation du liant pouzzolanique, constitué par les fumées de silice, dans le second. Dans ce dernier cas il faudrait dépasser les pourcentages en FSD étudiés, pour atteindre les quantités de ciment ne libérant pas le pourcentage de chaux nécessaire à la réaction totale des FSD.

Les valeurs figurant dans les tableaux 2a, 2b, 2c, font apparaître deux points importants : d'une part il est possible d'obtenir une gamme relativement étendue de résistances, d'autre part les formulations injectées permettent, à partir d'un milieu pulvérulent sans cohésion, d'obtenir un matériau consolidé et résistant.

1.2.3. <u>Résistance aux milieux agressifs</u>

L'étude a été effectuée sur des éprouvettes prismatiques 20 x 20 x 160 mm immergées, conformément à la norme NF P 18-837, dans des milieux agressifs constitués par de l'eau de mer et de l'eau à forte teneur en sulfates. Les principaux résultats sont donnés dans le tableau 4.

Cette étude montre dans les limites de notre expérience tout d'abord, que les coulis réalisés avec les différents ciments et la fumée de silice ne présentent pas de gonflement excessif à 90 jours quelle que soit l'eau dans laquelle ils sont conservés. L'utilisation de chaux entraîne comme c'était à prévoir, un mauvais comportement des coulis, en particulier dans l'eau à forte teneur en sulfates. Les éprouvettes sont dans ce cas rapidement détruites, à l'exception toutefois de celles confectionnées avec le ciment CLK et un faible dosage en chaux. On peut noter par ailleurs que pour des dosages en chaux équivalents (CPA 55 PM - 75 %, Cx 25 % ; CPA 55 PM 50 %, FSD - 22,5 %, Cx 27,5 %), la présence de fumées de silice diminue les réactions de gonflement liées à l'excès de chaux.

2. ETUDE DES CARACTERISTIQUES D'ADHERENCE DES COULIS HYDRAULIQUES SUR DES PIERRES CALCAIRES

2.1. <u>Matériaux utilisés et méthodes d'essai</u>

Les caractéristiques d'adhérence sont conditionnées par les propriétés intrinsèques des coulis et par les caractéristiques des matériaux supports.

Dans cette partie de l'étude, nous avons considéré trois coulis représentatifs dont les compositions sont données dans le tableau 5. Les principales caractéristiques mécaniques de ces coulis sont reportées dans le tableau 6. Les essais de compression et traction par fendage ont été réalisés à 28 jours sur des éprouvettes de diamètre 40 mm, de longueur respectivement 100 mm et 40 mm, conservées à température constante et 95 % d'humidité relative.

Trois types de pierres calcaires couvrant une gamme de porosité assez large (de quelques % à 30 %), ont été considérés comme matériau support. Le tableau 7 donne leurs principales caractéristiques physico-mécaniques.

Pour caractériser les propriétés d'adhérence entre coulis et pierres calcaires, nous avons fait varier les facteurs suivants :

- Nature du coulis (C1, C2, C3) (tableaux 5 et 6)
- Caractéristiques des pierres calcaires (E, L, SM) (tableau 7)
- Epaisseur du joint de coulis simulant une fissure (1 mm - 3 mm)
- Etat de surface (sec, humide (pierres saturées d'eau)).

Une étude préliminaire (dimensions des éprouvettes, conditions d'injection du coulis...), a permis de fixer les modalités de l'essai en traction directe réalisé pour apprécier les capacités d'adhérence des coulis. La figure 10 schématise le principe de l'essai. Les éprouvettes support en roche étaient des cylindres de diamètre 40 mm et de longeur 40 mm. Leurs extrémités ont été rectifiées pour permettre un réglage précis de l'épaisseur du joint de coulis et obtenir un état de surface constant pour toutes les éprouvettes. Après injection et conservation pendant 28 jours à 20° C et 95 % HR, des têtes de traction, en acier, ont été collées sur les éprouvettes pour permettre la fixation d'un système de traction à double rotule. Les essais ont été effectués à vitesse de déformation faible et constante (0,05 mm/min) pour éviter des effets dynamiques aux interfaces.

2.2. <u>Principaux résultats obtenus</u> (tableaux 8, 9, 10)

Outre les valeurs des résistances en traction, les tableaux 8, 9 et 10 indiquent les modes de rupture des joints. Les grands types de ruptures constatés ont été : La rupture adhésive qui est caractérisée par une rupture à l'interface coulis-pierre ; le coulis conserve ici l'essentiel de son intégrité. La rupture cohésive qui a lieu au milieu du coulis injecté et traduit une décohésion du coulis lui-même. La rupture mixte qui est une combinaison des deux ruptures précédentes. Enfin la rupture dans la pierre elle-même.

Une première remarque globale s'impose à l'analyse de ces résultats. Les résistances en traction obtenues, comprises entre 0,5 et 3,9 Mpa, montrent que les coulis hydrauliques étudiés possèdent de très bonnes propriétés d'adhérence. Conformément à ce que nous avons indiqué dans le paragraphe 1-3, nous pouvons également constater que les résistances obtenues avec le coulis C1 (ciment + fumées de silice) sont dans la plupart des cas supérieures à celles obtenues avec le coulis C2 (ciment + chaux). Par ailleurs, nous pouvons noter que les caractéristiques d'adhérence obtenues pour les surfaces sèches sont supérieures à celles trouvées avec les pierres calcaires saturées d'eau.

Nous pouvons ensuite remarquer que la porosité des pierres joue un rôle important sur cette caractéristique. Dans tous les cas, les résistances les plus faibles ont été obtenues pour la pierre la moins poreuse (E). Ce phénomène peut s'expliquer comme suit : la résistance des coulis est d'autant plus élevée que leur teneur en eau est faible (tableaux 6 - 11) mais le rapport E/C doit être cependant supérieur à une valeur limite de l'ordre de 0,3 pour permettre une bonne hydratation du ciment. Dans cette étude, les trois coulis utilisés ont un rapport E/C compris entre 0,75 et 1 pour assurer une injectabilité satisfaisante. Du fait de leur porosité, les pierres absorbent une partie de l'eau contenue dans le coulis, cette absorption a un effet favorable sur les propriétés mécaniques. Nous pouvons ainsi constater que les pierres calcaires L et SM (porosité 11 % et 28 %) conduisent à de meilleures caractéristiques d'adhérence que la pierre E (porosité 4 %). Une étude approfondie portant sur l'influence des propriétés d'absorption des pierres est actuellement en cours. Les premiers résultats montrent que dans le cas particulier de la pierre très poreuse SM, la quantité d'eau restant dans le coulis avoisine la valeur limite nécessaire à l'hydratation du ciment. Ceci peut expliquer le fait que les résistances les plus fortes sont obtenues avec la pierre L de porosité moyenne.

CONCLUSION

L'emploi d'éléments ultra-fins, tels que fumées de silice, chaux éteinte et terre de Santorin permet d'obtenir, à partir de ciments courants, des mélanges dont les courbes granulaires répondent aux exigences d'injectabilité dans de fines fissures. Pour chacune des associations ciment - ultrafines considérées, des formulations de coulis stables et injectables ont été obtenues avec un rapport E/C variant entre 0,75 et 1,25 et un pourcentage de superplastifiant variant entre 0 et 2.

L'association, dans des proportions variables de ciment, de fumée de silice et de chaux permet d'obtenir une gamme de résistances étendue. La résistance diminue en effet quand la teneur en chaux augmente, tandis qu'elle croît avec la teneur en fumée de silice.

Les coulis réalisés à partir de ciment et de fumées de silice ont une bonne résistance aux eaux agressives étudiées. La fumée de silice améliore la tenue aux eaux agressives des coulis contenant de la chaux qui normalement n'ont pas un bon comportement en présence d'eau à forte teneur en sulfates.

Les essais de traction réalisés sur des assemblages coulis-pierre calcaires montrent que les coulis hydrauliques étudiés possèdent de très bonnes propriétés d'adhérence. Ces essais indiquent également le rôle important joué par la porosité des pierres.

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Norme Française NF 15-301. Définition, classification et spécification des ciments.

Norme Française NF 18-891. Produits spéciaux destinés aux constructions en béton hydraulique : Essai d'injectabilité à la colonne de sable en milieu sec et humide.

Norme Française NF P 18-892. Produits spéciaux destinés aux constructions en béton hydraulique : Essai de fendage d'éprouvettes cylindriques de mortier provenant de l'injection d'une colonne de sable.

Norme Française NF P 18-837. Produits spéciaux destinés aux constructions en béton hydraulique : Essai de tenue à l'eau de mer et/ou à l'eau à haute teneur en sulfates.

Abréviations utilisées dans les tableaux et figures :
C : % ciment (en masse)
FSD : % fumée de silice densifiée (en masse)
Cx : % chaux (en masse)
TS : % terre de Santorin (en masse)
SP : % super-plastifiant (extrait sec) par rapport à la phase solide (en masse).
E/C : eau / phase solide
Temps : Temps mis par le coulis pour atteindre le sommet de la colonne (36 cm)
Rf : Résistance en traction par fendage.
Rc : Résistance en compression.
Vl : Vitesse de propagation des ondes longitudinales.

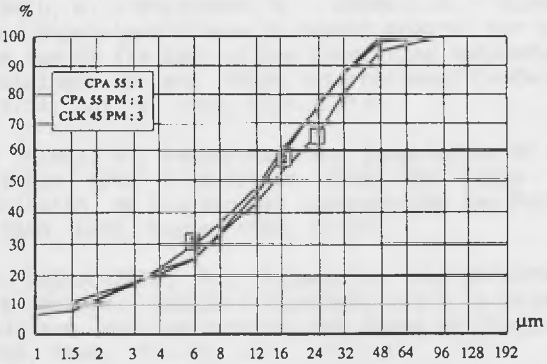

Fig. 1 : *Courbes granulométriques des ciments utilisés.*

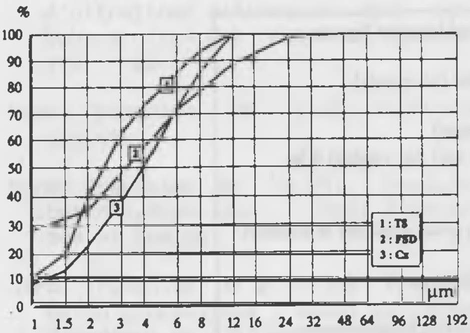

Fig. 2 : *Courbes granulométriques des éléments ultra-fins.*

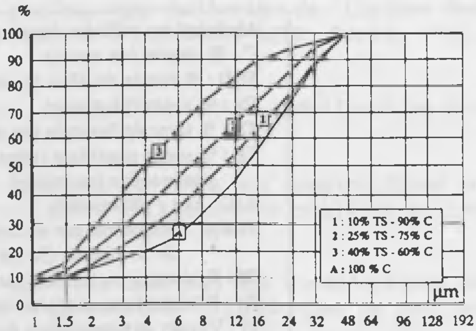

Fig. 4 : *Courbes granulométriques des associations ciment CPA55PM-terre de Santorin.*

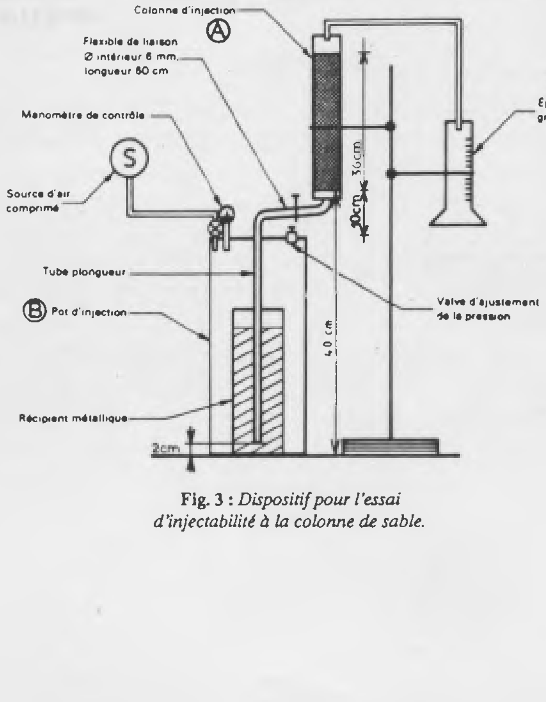

Fig. 3 : *Dispositif pour l'essai d'injectabilité à la colonne de sable.*

Type ciment	E/C	SP	Temps (s)	% Exsud.	Observations
A	1.25	0	14	34	Stable
	1.	0	Infini		
	1.25	0.33	12	36	Sédimentation
	1.	0.66	6	30	Sédimentation
	0.75	0.66	7	20	Sédimentation
	0.5	0.66	29	6	Sédimentation
B	1.5	0	4	40	Sédimentation
	1.25	0	Infini		
	1.25	0.33	34	42	Sédimentation
	1.	0.33	Infini		
	1.	0.66	5	35	Sédimentation
C	1.	0	10	17	Stable
	0.85	0	14	16	Stable
	0.75	0	Infini		
	0.75	0.166	31	30	Sédimentation
	0.5	0.66	Infini		

Tableau 1 : *Injectabilité des coulis à base de ciment (A:CPA55PM,B:CPA55,C:CLK45PM)*

C	FSD	Cx	TS	E/C	SP	Temps (s)	% Exsud.	Rf (Mpa)
90	10			0.75	0.66	16	2	1.33
75	25			0.75	1.33	20	0	2.45
75	25			1.	1.	13	2	1.82
70	30			0.75	1.33	35	0	2.73
60	40			1.	1.66	20	0	1.86
90		10		0.75	0.66	16	5	1.05
75		25		0.75	0.66	40	4	1.02
70		30		1.	0.	30	2	0.62
50		50		1.	0.33	25	5	0.38
70	13.5	16.5		1.	0.66	22	4	1.38
70	13.5	16.5		0.75	1.33	42	0	1.85
50	22.5	27.5		1.	1.66	17	2	1.63
90			10	0.65	0.166	48	3	1.74
90			10	0.65	0.33	28	5	1.70
75			25	0.75	0.166	24	3	1.50
75			25	0.65	0.33	30	2	1.70
60			40	0.75	0.33	21	4	1.50
50			50	0.75	0.33	19	3	1.46
70		16.5	13.5	0.9	0.166	19	4	1.38
50		27.5	22.5	1.	0.33	19	4	1.28

Tableau 2a : *Formulations de coulis (Ciment CPA55PM)*

C	FSD	Cx	E/C	SP	Temps (s)	% Exsud.	Rf (Mpa)
90	10		1.	0.66	17	8	1.25
75	25		1.	1.33	28	0	1.59
60	40		1.25	2.	13	1	1.39
75		25	1.	0.66	11	13	0.47
60		40	1.	1.	22	3	0.28
50		50	1.	1.	25	4	0.34
50		50	1.25	1.	12	12	0.12
34	30	36	1.	2.33	37	2	1.25
50	22.5	27.5	1.	1.83	17	0	0.87

Tableau 2b : *Formulations de coulis (Ciment CPA55)*

C	FSD	Cx	E/C	SP	Temps (s)	% Exsud.	Rf (Mpa)
90	10		0.75	0.66	22	4	2.30
75	25		0.75	1.	25	0	1.60
75	25		1.	1.	10	1	1.59
60	40		0.75	1.66	33	0	1.41
60	40		1.	1.33	18	0	0.98
90		10	0.75	0.33	21	9	1.2
75		25	0.75	0.83	36	7	0.99
75		25	1.	0.	21	10	0.74
60		40	1.	0.	40	5	0.52
50		50	1.	0.66	34	9	0.48
50		50	1.25	0.	20	9	0.32
70	13.5	16.5	1.	0.66	12	3	1.04
70	13.5	16.5	0.9	0.66	16	4	1.03
50	22.5	27.5	1.	1.	33	2	1.15

Tableau 2c : *Formulations de coulis (Ciment CLK45PM)*

C	Cx	E/C	SP	Rf(Mpa)
90	10	0.75	0.66	1.05
75	25	0.75	1.	1.02
70	30	1.	0.	0.62
50	50	1.	0.33	0.38

C	FSD	E/C	SP	Rf(Mpa)
90	10	0.75	0.66	1.33
75	25	0.75	1.33	2.45
70	30	0.75	1.	2.73

Tableau 3 : *Influence du % de chaux et du % de fumées de silice (Ciment : CPA55PM)*

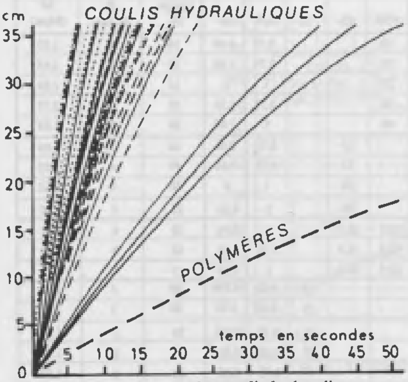

Fig. 5 : *Injectabilité des coulis hydrauliques et des polymères.*

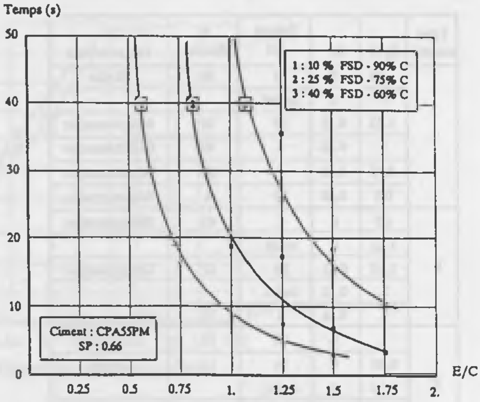

Fig. 6 : *Influence du % de fines sur la teneur en eau*

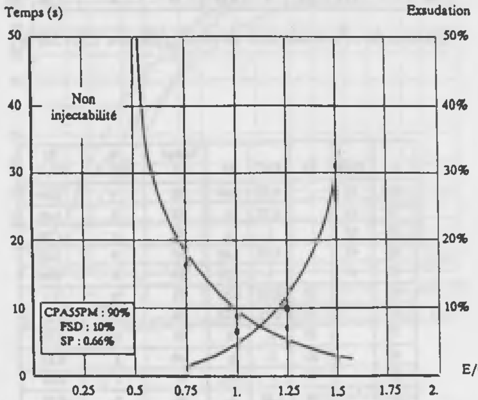

Fig. 7 : *Influence de la teneur en eau*

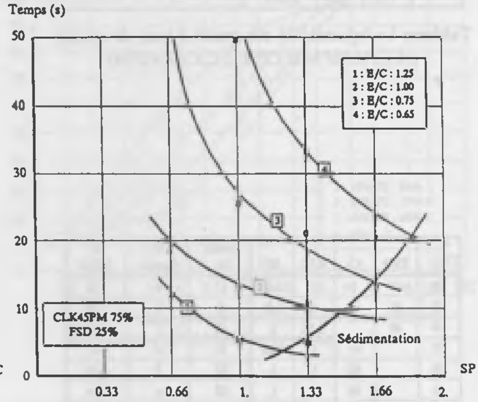

Fig. 8 : *Influence du pourcentage de super-plastifiant.*

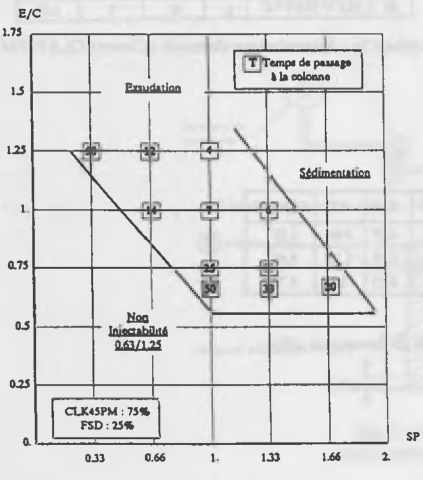

Fig. 9 : *Choix d'une formulation de coulis*

	Formulations de coulis					Retrait (Microns/m)		Gonflement (Microns/m)					
						HR 70%	HR 50%	Eau potable		Eau de mer		Eau sulfatée	
Ciment	C	FSD	Cx	E/C	SP	28 j	28 j	28 j	90 j	28 j	90 j	28 j	90 j
CPA55PM	90	10		0.75	0.66	1800	5690	510	590	540	670	850	880
	75	25		0.75	1.33	2180	6720	760	830	580	790	570	810
	60	40		1.	1.66	2660	7080	850	930	640	750	860	890
CLK45PM	90	10		0.75	0.66	2260	8600	770	750	640	630	630	570
	75	25		1.	1.	2950	+	330	380	260	310	260	330
CPA55	90	10		1.	0.66	1340	4590	460	520	490	510	550	550
	60	40		1.25	2.	2970	+	240	360	190	290	170	190
CPA55PM	90		10	0.75	0.66	1420	4850	310	350	440	520	+	+
	75		25	0.75	0.66	1290	4570	220	240	490	550	+	+
	50		50	1.	0.33	1300	4620	250	310	500	520	+	+
CLK45PM	75		25	0.75	0.83	3020	+	300	370	550	680	530	760
	60		40	1.	0.	3200	+	320	390	200	290	350	+
	50		50	1.25	0	3350	+	+	+	110	250	+	+
CPA55	75		25	1.	0.66	1040	4020	90	130	170	210	310	480
	60		40	1.	1.	810	3540	20	0	320	360	1370	+
CPA55PM	70	13.5	16.5	1.	0.66	1840	5850	680	740	610	740	910	930
	50	22.5	27.5	1.	1.66	2860	+	430	570	730	910	1080	1200

+ : Désagrégation des éprouvettes.

Tableau 4 : *Caractéristiques de retrait et de gonflement des formulations de coulis les plus représentatives.*

Coulis	C	FSD	Cx	SP	E/C
C1	75	25	0	1.33	0.75
C2	75	0	25	0.66	0.75
C3	50	22.5	27.5	1.66	1.

Tableau 5 : *Composition des coulis considérés pour l'étude de l'adhérence (C : CPA55PM).*

Coulis	E (Mpa)	Coef. Poisson	Rc (Mpa)	Rf (Mpa)
C1	9600	0.27	30.4	1.2
C2	5900	0.23	15.4	2.
C3	6200	0.16	14.4	0.75

Tableau 6 : *Caractéristiques mécaniques des coulis C1,C2 et C3.*

Type de pierre calcaire	Masse Vol. (g/cm3)	Porosité (%)	Vl (m/s)	Rc (Mpa)	Rf (Mpa)	Edyn (Mpa)	Coef. Poisson
E	2.65	4	5125	96	7.8	66300	0.14
L	2.42	11	5100	111	10.8	46000	0.21
SM	1.95	28	3400	43	3.9	-	0.23

Tableau 7 : *Caractéristiques physiques et mécaniques des pierres calcaires.*

	E		L		SM	
Coulis	Rt (MPa)	Mode de rupture	Rt (MPa)	Mode de rupture	Rt (MPa)	Mode de rupture
C1	1.8	Adhésif	3.9	Adhésif	2.1	Cohésif
C2	1.3	Adhésif	2.9	Adhésif	1.8	Adhésif
C3	0.9	Adhésif	3.0	Adhésif	2.0	Mixte

Tableau 8 : *Essais de traction. Epaisseur de joint de coulis : 1mm. Surface sèche*

	E		L		SM	
Coulis	Rt (MPa)	Mode de rupture	Rt (MPa)	Mode de rupture	Rt (MPa)	Mode de rupture
C1	1.3	Mixte	2.5	Adhésif	2.3	Mixte
C2	1.1	Adhésif	2.9	Adhésif	1.5	Adhésif
C3	0.7	Adhésif	1.6	Mixte	1.0	Mixte

Tableau 9 : *Essais de traction. Epaisseur de joint de coulis : 3mm. Surface sèche*

	E		L		SM	
Coulis	Rt (MPa)	Mode de rupture	Rt (MPa)	Mode de rupture	Rt (MPa)	Mode de rupture
C1	1.	Mixte	2.4	Cohésif	1.3	Adhésif
C2	0.4	Adhésif	1.7	Adhésif	1.0	Adhésif
C3	0.5	Adhésif	2.0	Cohésif	0.5	Adhésif

Tableau 10 : *Essais de traction. Epaisseur de joint de coulis : 1mm. Surface humide.*

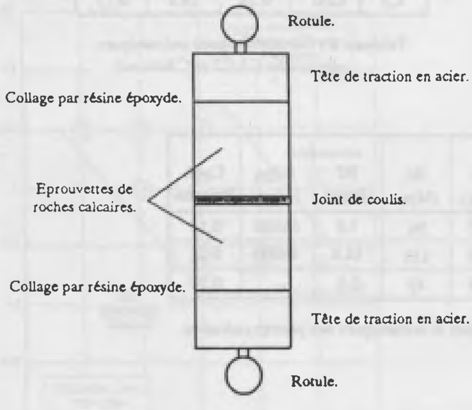

Fig. 10 : *Essai de traction directe.*

Coulis	E/C	E (MPa)	Coef. Poisson	Rc (MPa)	Rt (MPa)
C1	0.35	20100	0.26	93	3.4
C2	0.35	18300	0.26	64	2.4
C3	0.65	16800	0.11	41	2.3

Tableau 11 : *Caractéristiques mécaniques des coulis avec teneur en eau faible.*

SOME PRELIMINARY REMARKS ON AN EXPERIMENTAL RESEARCH PROJECT ON THE BEHAVIOUR OF STONE MASONRY PANELS SUBJECTED TO SEISMIC FORCES

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ABSTRACT

A program of experimental researches on the seismic behaviour of full-scale stone masonry panels is explained. This program was developed at the University of Florence, Department of civil Engineering, as part of the "C.N.R. G.N.D.T.[1] - Subproject 2: Prevention of Building Damage - Working Group 2.2: Experimental Evaluation of the Seismic Behaviour of Structures " coordinated research program.

The main objective of this part of the project is to investigate the seismic behaviour of commonly used masonry typologies in the Florence area. Both cyclic and monotonic tests are designed to assess the mechanical characteristics of stone masonry and its strenght under cyclic actions. Evaluation of flexural and shear capacity, load degradation, ductility and energy dissipation was also previded. This paper presents the setup of two different experimental tests and describes the used laboratory equipment.

(1) National Council of Researches – National Group for Earthquake Defense

1. INTRODUCTION

Since a majority of Italy's architectural heritage consists of masonry buildings, there is a real need to assess the safety and behaviour of such structures with respect to seismic events in an earthquake prone area like Italy. Despite the need, experimental data on the response of masonry structures to repeated sollecitations are generally not avalaible even for the most common Italian typologies. In order to contribute to address this lack of experimental data, the present research is aimed at collecting and supplying information about the behaviour of stone masonry panels — constructed to reproduce a common Florentine typology — in simulated earthquake conditions.

Wall panels are the major seismic-load resisting elements in masonry structures. In general, masonry wall panels under seismic excitation can be subjected to simultaneus in-plane lateral loads, axial loads and overturning moments, depending on their slenderness ratio. The experimental program is developed to evaluate both global mechanical characteristic of stone masonry panels (for instance the G modulus) and the global response to a simultaneus application of lateral and axial loads.

The experimental project is conducted over a two-years period. The first year was concerned with testing procedures setup and with specimen construction. The second year tests will be carried out and results will be evaluated and used to improve avalaible analytical aproaches.

The work is developed as a part of a coordinate research program (see note 1) dealing with a general investigation and evaluation of the seismic behaviour of most common Italian building typologies. The Department of Civil Engineering of the University of Florence first concern was to study masonry characteristic of buildings in the Florence area; thereby performing a serie of experimental tests using standardized testing metodologies in which results from different working groups could be compared [1].

The presentation mainly deals with the most common testing procedures to investigate the shear failure behaviour under cyclic or monotonic loads [2]. This topic has been chosen for these preliminary remarks because of the general interest to a better unerstanding of the shear cracking mechanism of stone masonry walls [3]. The research was also developed considering the need of specific experimental results concerning big size stone masonry specimens.

Two different testing models (both referred to a 120 x 120 x 40 cm stone masonry specimen — about an half story-height wall panel —) were considered of a particular interest because of the large thickness of the specimens (1:1 scale with real stone masonry walls).

Together with the testing procedures showed in the following, the research program comprends also testings of both mortars and stone units and compression test on small bricks walls, not reported in the present paper.

Specimens were constructed using sandstones and limestones according to the usual composition of stone masonry in the Florence area and two different types of mortar, characterized by volumetric ratio of hydraulic lime, cement and sand of 1, 1, 6 and 2, 1, 9. Stones from demolished buildings were used in order to reproduce actual real situations.

2. LABORATORY SHEAR-COMPRESSION TEST

This test determinates the in-plane response including peak strength and ductility of masonry wall specimens under lateral, fully reversed cyclic loading conditions [2]. Six specimens with same stone components and two different types of mortar were constructed. Each specimen had reinforced concrete top and base beams and two brick bed joints, characteristic of a common situation in the Florence area. A description of the laboratory shear-compression test specimen is depicted in Fig. 1. The instrumentation scheme of the specimens is shown in Fig. 2; displacement transducers were applied to invar wires to measure diagonal and vertical overall deformations and horizontal displacements of the specimen versus horizontal testing load. Others transducers (which are not shown in the Figure) are applied to measure the base slip.

A model of the test setup is shown in Fig. 3. Three servo-controlled hydraulic actuators (MTS) were used to apply the vertical and lateral loads to the specimen. The lateral deflection of the specimen was controlled by the horizontal actuator, employed under displacement control.

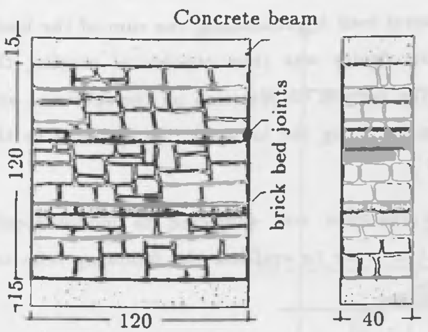

Fig.1: Laboratory Shear-Compression Test Specimen (cm)

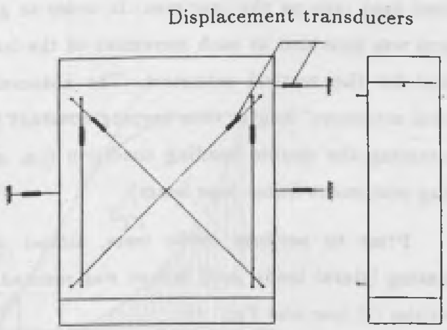

Fig. 2: Specimen Instrumentation

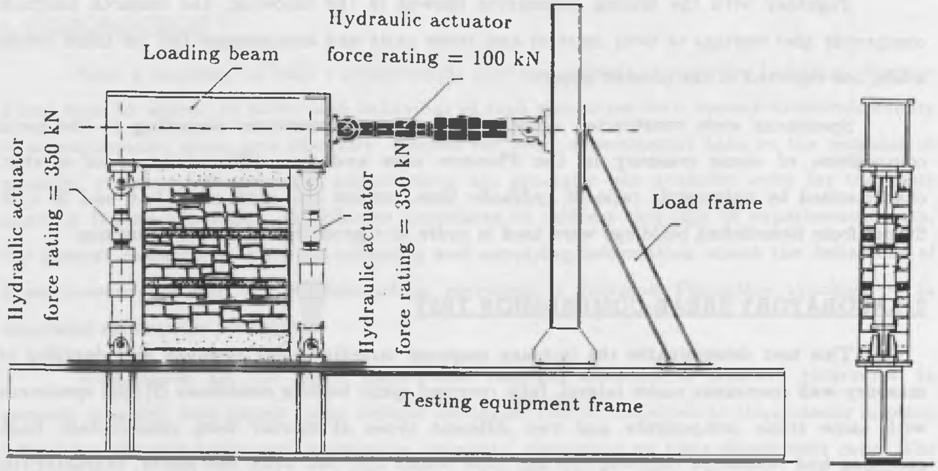

Fig. 3: Test Setup

The two vertical actuators were initially under load control until the exercise load condition of a masonry wall was reached. The vertical actuators were then capable of maintaining a constant axial length under displacement control, simulating double-bending conditions. Lateral supports were also provided (which are not shown in Fig. 3) to prevent the out-of-plane deflection of the specimen.

The test was performed maintaining the double bending condition and a constant vertical load rate on the specimen. In order to guarantee the above conditions, an automatic control was provided at each increment of the lateral load by evaluating the sum of the loads applied by the vertical actuators. The automatic device was then capable of varying the vertical actuators' length, thus keeping constant the vertical compression on the specimen and maintaining the double bending condition (i.e. maintaining the loading beam parallel to the testing equipment frame base beam).

Prior to perform cyclic tests, almost a specimen was subjected to monotonically increasing lateral loads until failure was reached, in order to evaluate the following referring quantities [1] (see also Fig. 4):

V_u ultimate lateral load

v_u ultimate displacement (correspondent to 0.70 V_u)

v_{cr} cracking displacement (see Fig. 4)

E_r referring energy rate ($E_r = \frac{V_u\ v_u}{2}$)

The cyclic test load history was determined according to the one showed in Fig. 5, where time (t) represents a general order parameter. In order to simulate the seismic response induced by a strong earthquake, fully reversed lateral displacements cycles of increasing amplitude were applied to the specimen by the horizontal actuator.

Small amplitude cycles were also introduced into the displacement sequence to check the stiffness degradation after each amplitude increment.

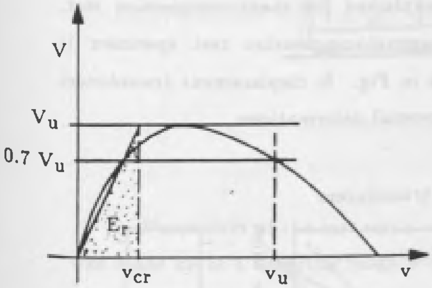

Fig. 4: Referring Quantities in a Monotonic Test

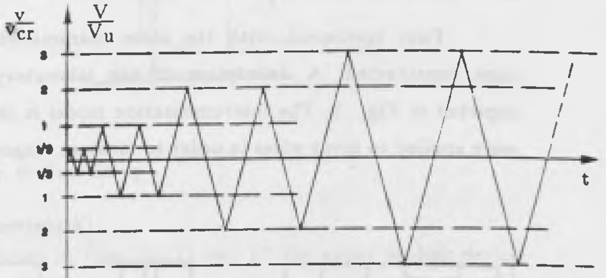

Fig. 5: Cyclic Test Load History

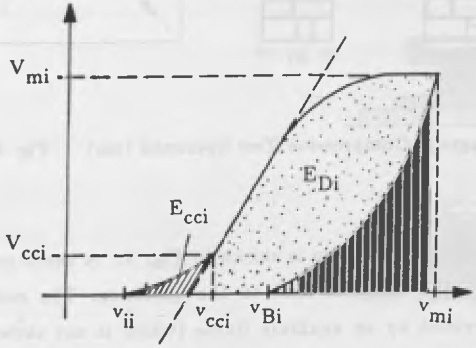

Fig. 6: Referring Quantities in a Cyclic Test

Test results consisted of the hystories of all the displacements measured by the transducers shown in Fig. 3. All the following quantities, referred to the i-th cycle are also considered [1] (see Fig. 6):

v_{ii} initial displacement;
v_{cci} crack closure displacement;
v_{Bi} residual displacement;
v_{mi} maximum displacement;
V_{cci} horizontal crack closure force;
V_{mi} horizontal maximum force;
E_{cci} crack closure energy;
E_{Di} dissipation energy.

3. LABORATORY DIAGONAL COMPRESSION TEST

The test determinates the tensile strength of the specimen, containing a representative selection of stone units and mortar joints [2].

Four specimens, with the same characteristics explained for shear-compression test, were constructed. A description of the laboratory diagonal-compression test specimen is depicted in Fig. 7. The instrumentation model is shown in Fig. 8; displacement transducers were applied to invar wires in order to measure diagonal overall deformations.

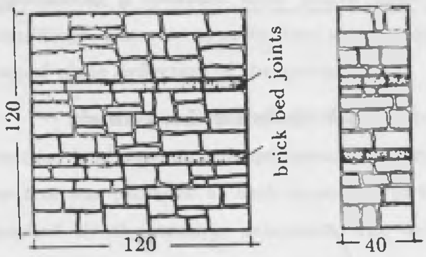

Fig.7: Laboratory Diagonal-Compression Test Specimen (cm)

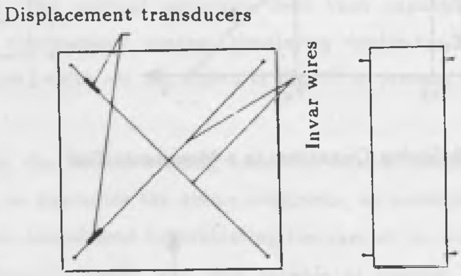

Fig. 8: Specimen Instrumentation

The model of the test setup is shown in Fig. 9. A servo-controlled hydraulic actuator was used to apply the diagonal load to the specimen. The out-of-plane deflection of the specimen was prevented by an auxiliary frame (which is not shown in the Figure) positioned between the test machine columns and the specimen.

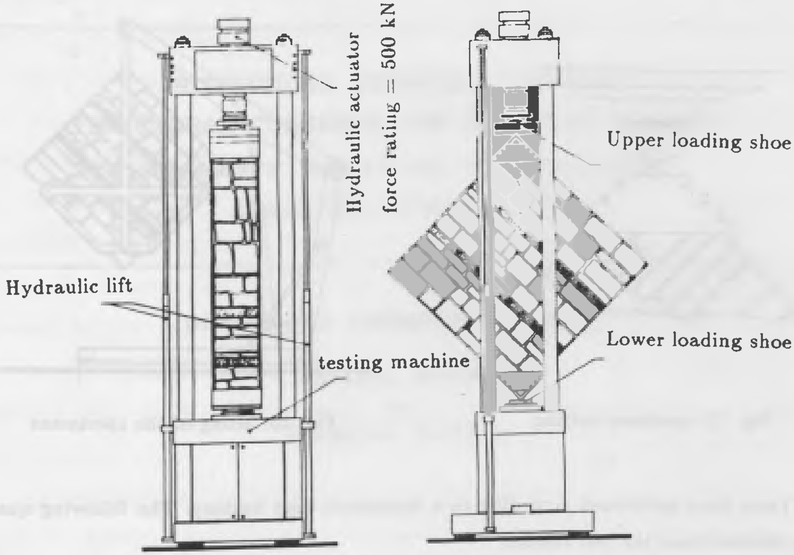

Fig. 9: Test Setup

Particulars of the test setup are shown in Figg. 10, 11 and 12: the upper loading device was made up of a spherical hinge while the lower loading device of a cylindrical hinge. Special plates were devised to contain the specimen corners with a mortar layer to reduce stress concentration in the zones of load application. Special steel frame and the apparatus for lifting and transportation of the specimens are shown in Fig. 13.

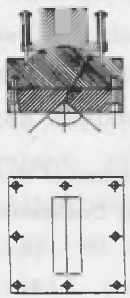

Fig. 10: Upper Loading Shoe

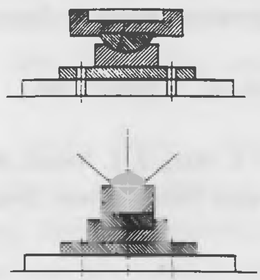

Fig. 11: Lower Loading Shoe

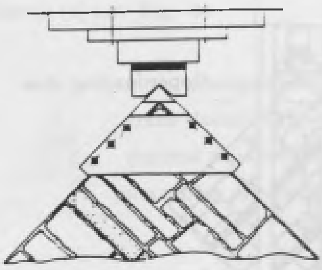

Fig. 12: specimen setting

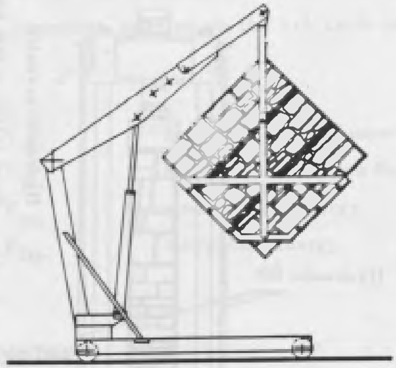

Fig. 13: lifting of the specimens

Tests were performed according to a monotonic load hystory. The following quantities [1] were derived from the test results:

P	ultimate strength;	G_t	shear deformation ($G_t = \frac{D_v + D_h}{g}$);
S_{pt}	shear failure tension;	g	vertical measure base;
D_v	vertical shortening;	G	shear modulus ($G = \frac{S_{pt}}{G_t}$);
D_h	horizontal shortening.		

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[3] P. B. Shing, J. L. Noland, H. Spaeh, E. Klamerus, Inelastic Behaviour of Masonry Walls Panels Under In-Plane Cyclic Loads, IV North American Masonry Conference, 1987

INTERNATIONAL TECHNICAL CONFERENCE
STRUCTURAL CONSERVATION OF STONE MASONRY
(Diagnosis, Repair and Strengthening)
Athens,Oct.31-Nov.3,1989

practical redesign method

GENERAL REPORT

by
Antonino Giuffrè

Introduction to the session.

The nine papers presented to this session offer, with wealth of colours, a painting of the international culture in the field of the scientific and engineered studies of the masonry works.

I can immediately say that today the culture is at a turning-point: the researchers tried very hard to face the structural analysis of such works using numerical models derived by the elastic theory, and the engineers adopted the modern techniques in the strengthening interventions; but it is always more evident that the logic involved by the historical masonry structures requires a different approach.

The apparent rigour of the way gone until here, applying sophisticated models, disappears as soon as it is compared with the reality; on the other side, the turning that insistently many authors claim, seems to lead towards a road not yet marked out.

I would like to asset, on the contrary, that a new road begins to be outlined and can be taken and covered.

The nine papers that are going to be presented contain all the process and prove my assessment.

Please, let us to listen them, and then my general report will present a comprehensive view in order to understand how they depict the whole problem.

Topic 2.2: *PRACTICAL REDESIGN METHODS*

1) C.Ignatakis- K.Stylianidis-E.Stvrakakis *A special finite element axisimmetric model for the analysis of interventions in domes including cracking consideration.*
2) A.Cappi-A.Fontana: *Strengthening of old masonry bridges.*
3) J.P.Adam: *Proposition de confortement parasismique dans la maçonnerie ancienne.*
4) F.S. Diaz Berrio: *Criteres des choix pour des mesures d'urgences.*
5) R. Langenbach: *Learning from the past; traditional and contemporary unreinforced masonry in seismic areas.*
6) R.Meli-O.L.Batiz: *Structural safety of historic buildings in Mexico City.*
7) M.Karaveziroglou-J.Papayianni-G.Penelis: *Criteria for selecting materials for repair brick masonry.*
8) V.Ceradini-A.Giuffrè-A.Pugliano: *Philology and safety in vulnerability mitigation of historical towns in seismic areas: the practice code for "Castelvetere sul Calore".*
9) A.Giuffrè: *Principles for the philological restoration of historical buildings in seismic areas.*

The first paper which would have been presented to this section is that of Ignatakis - Stylianidis - Stavsakakis. It was presented this morning. In it three researchers by the University of Thessaloniki have shown the analysis of the central dome of the St.Panteleimon Church.
They carried out an interesting computer program, using anisotropic elastic finite elements, with a procedure which enables to discover and introduce cracks, changing the stiffness matrix of the constitutive law during a step by step analysis.
Before starting with the analysis they examined the structure, pointed out the actual cracks, understood the mechanism of the

displacement, and decided to introduce a titanium tie-ring in order to control its safety.

But they wanted to reproduce the physical behavior with a numerical model in order to quantify the amount of prestressing to be given to the chain.

They had to introduce a large number of elastic parameters, in order to fill up the matrix of the constitutive law for every type of element, and it is evident that it isn't easy to evaluate them; they had to state a limit to the compression produced by the prestressed chain-rings, without an objective criterion.

The sophistication of the approach presents some intrinsic uncertainties.

They carried out two analyses, linear and non-linear, and got very different results.

Of course the result obtained accounting for the progressive cracking should be more reliable, but nobody can evaluate how much reliable.

So intelligent and laborious work leads only to a qualitative result, with the only useful conclusion that two chain-rings, in some way prestressed, represent a reasonable intervention: this is the same result obtained in the XVIII century, by Poleni, when five steel rings have been added to the Vatican Dome in Rome.

No prestressing on 1748, but we know that prestressing on the masonry disappears very quickly because of the creep and the closing of microcracks.

In my opinion what is good in their intervention is to have avoided concrete skins, to have adopted titanium less affected by the thermic variations, instead of stainless-steel, to have injected the cracks before tying the dome (but I would like to know if they adopted a compatible mortar or, as usual, cement mortar). What is good in their intervention doesn't depend on the numerical analysis.

Nevertheless a numerical analysis is desirable; I would like to propose a different approach.

Rather than subdividing the dome with an automatic mesh, it would have been better to point out its actual state of

partialization.It is not easy: because it is not always possible to understand if a crack goes through the whole thickness or not. We really need a different kind of instrumental observation, not devoted to evaluate the parameters characteristic of the elastic and continuous materials, but addressed to explore the objective consistence of the masonry blocks which constitute the structure. The result of such an analysis, if it is possible to succeed in such a careful survey, would present the masonry dome as a structural system of rigid bodies, which goes to the collapse by transforming itself in a kinematic chain.
The mathematical modeling could be more effective, and effectively based on the real condition of the structure. It should evaluate which collapse mechanism is more probable, and which actions would be able to prime it; which portion is overloaded with regard to the condition of its masonry.
As you see I'm speaking about future way of the research, I'm not criticizing the clever authors from Tessaloniki, but eventually proposing them an other essay of their skill.

But let's go on.
From the second paper, which presents an anchanting structure (the railway masonry bridges have been the glory of the engineering, in the last century), I would like to comment the analysis of the arches and their strengthening.
The authors demonstrate to know perfectly the mechanics of the XVIII century, as presented by Heimann, and they checked the arches through their possible mechanisms. I perfectly agree with this kind of analysis and I think reliable their results.
Nevertheless I can't understand why those arches couldn't bee strengthened using masonry.
In fact the authors built new arches in reinforced concrete under the original ones, and I think that it is impossible to analyze correctly the mixed structure made by the over imposition of an arch in masonry and one in reinforced concrete.
If the masonry arches have supported the load for one hundred years, and a correct way to check their safety exists, why we need

to leave the original feature of the structure for introducing a new and not correctly modeled one?
It is evident that, to the present culture, the modern technology seems reliable in itself, and the ancient one, beautiful to the look, but unreliable.
How more satisfactory, I would say, to study a glorious building, to understand its structural reasons, and to learn speaking its language.

The third paper was presented by Adam. Who doesn't know the books written by Adam? I'm happy to declare that my lectures at the university of Rome are full of slides taken by the books of Adam, and my students know where to find an interesting documentation on the historical masonry techniques.
In this paper Adam tries to derive, by the observation of the damages produced by an earthquake, the mechanical behaviour of the masonry buildings. The archaeologist, used to observe the objects of his science, is more similar to the experimenter then to the analyzer engineer, and this aptitude is precious in the field we are speaking about
The collapse mechanisms that he presents are perfectly correct. It is possible to recognize them among the ruins of Pompei or in Friuli and in Irpinia.
In addition to the mechanisms listed by Adams for the usual houses I would like to report an example as useful in Italy as in Greece, were many ancient temples are yet standing or have been rebuilt.
(Fig.1) Looking at this survey of the temple G in Selinunte, collapsed during a strong earthquake in the eighth or ninth century. It was presented here in Athens by Furio Fasolo, in 1969 at the XVI International Conference on the History of Architecture. We can observe that each column fell overturning in a different direction.
(Fig.2) Looking at this maquette of the temple F, made before the anastilosis, we see that the same earthquake pushed down an analogous structure, standing about fifty meters far from de first one, moving synchronically all the columns.

(Fig 3) The earthquake of Pompei in 62 damaged a temple too. This carved bas-relief (the slide came by one of the Adam's books), related to that event, seems to indicate that a synchronous motion was induced to this structure too.
Since we know that the temple G have never been finished, we can draw the suggestion that the completeness of this type of structure is enough to make it behaving as a whole.
It is an experimental observation, derived by tests carried out by a strong and effective shaking table.
The engineers shouldn't neglect the information which can be derived by the history, managing them through the filter of the science as the archaeologists do; it is evident how useful it would be in the field of the studies of the historical architecture.
But this obvious advice can't be followed if we don't introduce in our cultural frame the correct technique of historical research and historical analysis.

Well, I have to go-on towards the turning-point of the historical buildings restoration.
In the paper of Adam we can find the moment of the present contradiction.
After the correct analysis of the failure mechanisms, which excludes the use of elastic theory in the mathematical modeling of those structure, he presents a list of the most habitual techniques of strengthening interventions.
I have to express some disagreement to such techniques, though, unfortunately, they are terribly spread all over Europe , and in spite of their irrationality, are commonly and acritically applied.

Starting by the foundations I have to point out how irrational and far from the nature of the masonry is the use of the micro-piles.
The characteristic of the masonry, starting by the *opus quadratum*, is represented by the horizontal layers, recommended by all the ancient authors, present in all the well constructed buildings.

The horizontal layer is the best support of the vertical actions and the way to allow the possible rocking motion of the wall around horizontal cylindrical hinges with the minimum effect of breaking.

(Fig.4) The micro-piles provide discontinuous and eccentric supports. If the original base ground settles the masonry wall become precariously supported by not interlocked lateral stones. The alternation of the micro-piles on the two side is not enough to avoid the irrational locally sliding support.

Of course it would be different if a reinforced concrete base beam is present at the foundation as in modern building: it could be able to distribute the punctual action of the piles, but if there isn't such a behaviour and that concentrate action risks to break up the masonry.

How better the traditional under-foundation. It could be made in cement-concrete, taking care to avoid air-bubbles on the contact with the inferior edge of the masonry.

An other usual technique pretends to introduce connections between orthogonal walls. It is suggested by the vertical cracks that often appear, after the earthquake, along the corners of the buildings.

(Fig.5) In XVIII century the architects knew this problem, and they used, in the most important building, to put steel keys on the corner, as they did in the Cathedral of S.Angelo dei Lombardi.

(Fig.6) But the earthquake of 1980 demonstrate that that is not enough. As you see the crack is only translated at the end of the key.

The only effective way was suggested by Rondelet in its treatise: chains as long as the walls, well turned and connected at the corners.

(Fig.7,8) Today it is not difficult to implement this idea inserting steel in suitable holes made with rotative machines horizontally along the walls at the middle of their thickness, and then filling the void by injection.

(Fig.9,10,11) At the top of the wall the suggestion of Rondelet can be applied, rebuilding the last part of the masonry and inserting in it a steel chain as it has been made in the Cathedral of S.Angelo dei Lombardi.

The last observation regards the use of covering the interior of the houses with a thin plate of reinforced concrete.
(Fig.12) We had in Italy a code, promulgated by the Region Umbria as a regional law, which contains this detail.
In this way the ancient transpiring room is transformed in an impermeable concrete bunker. The quality of the life inside is completely changed: it would be better to go to live in a precast cement flat.
In addition we need to notice that a thin rigid shell never will be able to confine the thick masonry walls, and during an earthquake it will be separated making evident its ineffectiveness.

The next two papers with convincing eloquence claimed the need to have a changing if we want avoid, after the damage of the earthquake, the culpable destruction of every historic significance of the architectonic heritage.
The knowledge of the structural characteristics of each typology of building, first of all those which constitute the basic texture of the historic urban areas, is the first step for a correct choose of the strengthening techniques.
It was claimed by Diaz-Berrio with reference to the preventive analyses needed by the correct organization of the urgency measures; the same was asserted by Langenbach, illustrating several different and extremely interesting historical technologies.
The first principle I enunciate in my paper can be considered directly derived by these two contributions:
(Fig.13)
FIRST PRINCIPLE
THE TECHNIQUES AND THE STATICS OF THE HISTORICAL BUILDINGS ARE IMPORTANT TESTIMONY OF THE CULTURE.

first requirement
FOR INTERVENTION ON HISTORICAL BUILDINGS THEIR TECHNOLOGY AND THEIR STRUCTURAL FEATURE MUST BE CLOSELY ANALYZED REFERRING TO THE CULTURAL CONTEXT AT THE TIME OF THEIR CONSTRUCTION.

This stage of the analysis can't be avoided, if we want scientifically recognize the historical nature of the object.
It is evident that this principle point-out a turning in the usual approach to the existing architectures, and requires a new competence to the operator of the static restoration.
What seems not yet clear is how to apply the scientific methods of the safety verification.
Many uncertainties are yet evident, but several answers are coming.

The paper presented by Meli is just at the crossroads, but the paper presented by the second group from the Tessalonichi University, and the paper presented by the roman group are already on the new road.
To conclude my general report I will refer to the mexican work, which is full of interesting observations.
The authors are carrying out experience on historical buildings in Mexico city. It's a pity that they have not yet faced the comparison of the building technique applied in Mexico from the XVI century and the coeval Occidental one. It would be interesting too the comparison with the occidental housing typologies, because from those the interaxes between the walls derive, the number of story, the presence of an inner court,...
What, of the constructive technique, comes from the Occident and what was introduced by the local culture has a relevant cultural interest, and only the engineers or the architects can face this subject; but such information is strictly concerning too with the mechanical features to which we are more directly interested.
I would like to try an explanation.

In the usual housing buildings the stresses produced by the vertical load are quite negligible; it follows that the compression strength of the masonry is an irrelevant parameter.
In addition, if we want to refer to it, we find that it is an extremely fuzzy parameter, which contain some meaning only if related to vertical axial load.
As we know the seismic action produces eccentricity out of the plane of the wall; the state of compression is limited to a portion of the whole thickness, and vertical shear stresses arise on the interior surfaces, with the trend of breaking up the wall.
For this reason the texture of the stones plays the most important role in the behaviour of the masonry, and the compression strength measured with reference to the simply axial force loses its meaning.
An other parameter often wanted, and with heavy efforts measured, is the elastic modulus. The same curiosity, when more financing is available, leads to measure the first vibration period.
So did the authors of the interesting paper I'm referred about.
(Fig.14) Well, I demonstrated at the XV° Regional Seminar of Ravello last September, and then at the Italian Seismic Conference in Milan last October, that the non linear dynamic response of these kind of structure is not affected by the initial period, that is to say by the initial elastic modulus.
In this diagram you see as the dynamic response in terms of maximum displacement is flat, changing the vibration period which depends by the first elastic brunch of the force-displacement relationship.
The response is only affected by the limit horizontal force and by the ultimate available displacement; but in this diagram you see an other important result.
The numerical analysis was carried out parametrically decreasing the ultimate displacement, that is to say increasing the slope of the second branch of the bilinear diagram. As you see the results are always the same until the reduction reaches one third of the geometrical displacement. That allows to affirm that we only need to know if the masonry quality is near or far from the correct

construction.
So, the two most important parameters affecting the seismic resistance of the masonry buildings, are the limit horizontal force and the ultimate displacement,and both of them are related to the possible failure mechanisms and to the quality of the masonry.

For this reason I have formulated the second principle:
(Fig.15)

SECOND PRINCIPLE

THE EARTHQUAKE GIVES RISE,WITHIN THE STRUCTURAL FRAMEWORK OF THE BUILDING, TO SEVERAL KINEMATIC MECHANISMS, NORMALLY INVOLVING ONLY PARTS OF THE WHOLE BUILDING. ONE OR MORE OF THEM CAN MOVE AS FAR AS CAUSING COLLAPSE.

SECOND REQUIREMENT

ALL THE POSSIBLE COLLAPSE MECHANISMS THAT CAN BE SET IN MOTION BY THE EARTHQUAKE MUST BE INVESTIGATED BY TAKING INTO ACCOUNT THE ACTUAL CONNECTIONS. THEY MUST BE POINTED OUT AND, IF POSSIBLE, ANALYZED BY NUMERICAL PROCEDURES.

Well, in my opinion and in the conclusions presented by Meli, the practical way to analyze the seismic resistance of a masonry structure is the following:

(Fig.16)i) EXAMINE EACH TECHNOLOGIC DETAIL OF THE CONSTRUCTION IN ORDER TO LIST THE POSSIBLE KINEMATIC MOTIONS;

ii) FOR EACH OF THEM EVALUATE THE FORCE ABLE TO ACTIVATE THE MOTION, AND THE MAXIMUM POSSIBLE DISPLACEMENT THAT THE GEOMETRY ALLOWS;

iii) LOOK AT THE QUALITY OF THE MASONRY IN ORDER TO EVALUATE THE REDUCTION OF THE LIMIT FORCE AND OF THE ULTIMATE DISPLACEMENT TO BE REASONABLY TAKEN INTO ACCOUNT;

iv) READ THE DYNAMIC RESPONSE IN TERMS OF THE MAXIMUM REQUIRED

DISPLACEMENT INTO THE AVAILABLE RESPONSE SPECTRA.

(Fig.17) Here you see the response spectrum in terms of maximum displacement as function of the limit force and the ultimate displacement.
The check will regard the displacement, and that gives information on the collapse, and on the damage too.

The new direction of the research has been shown in the papers presented by Penelis and by Ceradini: it is the study of the original technologies and the original structural performances.
I would like to underline that the modern analysis carried out on historical materials, in order to make known their characteristic at the same level usual for the modern ones, as Penelis and the other authors have done, is the condition for the design of compatible interventions.
(Fig.18,19,20,21) I could show an experimental test carried out on a masonry vault, built in Irpinia with the aim to get familiarity with the construction, to test the seismic behaviour consequent to a motion of the supports, and to experiment the techniques of strengthening after the damage.
(*you can see the videotape of this interesting experiment during the friday session*)
(Fig.22,23,24) A similar voult has been already built in the crypt of the Cathedral of S.Angelo dei Lombardi.

Bu it's time to conclude, and I'll report the third principle for the philological restoration of historical buildings in seismic areas.
(Fig.25)
THIRD PRINCIPLE
THE ANCIENT CONSTRUCTIVE TECHNIQUES AND STRUCTURAL TYPOLOGIES, IF CARRIED OUT ACCORDING TO THE CORRECT ORIGINAL TECHNIC RULES, ARE IN THE MOST CASES SEISMIC RESISTANT.
THE XIX CENTURY TECHNIQUE CAN PROVIDE SEISMIC RESISTANT DETAILING, ENOUGH TO ACHIEVE THE REQUESTED SAFETY.

THIRD REQUIREMENT

SEISMIC RESISTANCE WILL BE ACHIEVED BY RESTORING THE ORIGINAL TECHNIQUE AND, IF NECESSARY, BY ADDING NEW DETAILING CONSISTENT WITH THE ORIGINAL CONSTRUCTIVE TECHNIQUE: THEY CAN BE DERIVED FROM THE TECHNICAL LITERATURE OF THE XIX CENTURY AND CAN BE CHECKED WITH MODERN LABORATORY TESTS.

DISPLACEMENT INTO THE AVAILABLE RESPONSE SPECTRA.
[illegible] we use the response spectrum in terms of maximum
[illegible] the limit force and the ultimate
displacement.
The other [illegible] the displacement, and that gives information
on the collapse, and on the damage [illegible].

The new direction of the research has been shown in the papers presented by [illegible] and by Carafini: it is the study of the original technologies and the original structural performances.
I would like to underline that the modern analysis carried out on [illegible] materials, in order to make known their characteristic at the same level known for the modern ones, as Penelis and the other experts have done, is the condition for the design of [illegible] intervention.
[illegible] experimental test carried out on a masonry [illegible] with the aim to get [illegible] with [illegible] the seismic behaviour [illegible] to a motion of the [illegible] the [illegible] of strengthening [illegible].
[illegible] the [illegible] during [illegible] session)
[illegible]) a similar vault has been already built in the crypt of the Cathedral of S.Angelo dei Lombardi.

[illegible] the third principle for the [illegible] buildings in seismic areas.
[illegible]
[illegible]
[illegible] TYPOLOGIES, IF [illegible] TO THE CORRECT ORIGINAL TECHNIC RULES, ARE IN THE MOST CASES SEISMIC RESISTANT.
[illegible] TECHNIQUE CAN PROVIDE SEISMIC RESISTANT DETAILING, [illegible] THE REQUESTED SAFETY.

CRITERIA FOR SELECTING MATERIALS FOR REPAIR BRICK MASONRY

M. Karaveziroglou*, J. Papayianni* and G. Penelis**

SUMMARY

This paper deals with the problems of repairing brick masonry, especially with materials used for repair and accomplishment of damaged bearing walls.

Methods of analysis of the existing old materials in order to prepare the new ones that can be used for repair-rebuilding brick masonry are referred. In this case the word "new" means cement and pozzolanic mortars and also bricks produced in Greece, which may be used for this purpose. The research takes into account the existing experience (Laboratory investigations of the chemical and mechanical properties of building materials such as mortars and bricks) and applicationsin structural restoration of monuments. The authors suggest criteria for selecting appropriate materials for repairing in relation to the composition of mortar and the quality of brick.

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INTRODUCTION

In the restoration field the conservation of a monument or the rehabilitation of a building needs systematic analysis of the following: The static stability of the old structure, the applied loads, the type of damage and the used materials. The age and the influence of past time on the monument must also be taken into account. The extension of the research depends on the importance of the historical buildings and the existing financial conditions. Nevertheless, the quality of materials determines the intervention technique; therefore emphasis is given to material tests and results. Because of the lack of standard procedures for testing old materials in order to find out their properties, the modern specifications for new building materials (with modifications) as well as modern chemical techniques - such as x-ray analysis, scanning election microscopy thermogravimetric analysis e.t.c. - can also be used for the purpose.

Based on the existing experience on testing old materials (laboratory tests for estimation their properties), criteria for selecting new materials are given in this paper. According to the authors opinion, mortars and bricks for repair should be made from the same constituents as the original ones and the same "traditional" technique should also be applied.

MATERIAL TESTS AND SELECTIONS

Bricks

Tests on mechanical and chemical characteristics of natural stones or bricks are quite enough for the estimation of their quality. Strength and homogeneity of the material allow imperturbable samples to be taken that can be tested in laboratory (mechanical and chemical properties, porosity). The results can be used in finding the origin of these materials, which is very important for natural stone(marmor or pore stone). For bricks the knowledge about their origin is not sufficient because the producing technique must also be known.

Repairing the old masonry geometrical and chromatic resemblance between new bricks and the old ones is also required since their mechanical and chemical properties are equivalent. Modern brick industries do not usually produce bricks having the specified shape of the old handmade ones. Ordering bricks in small manufactures results in having bricks of desired dimensions. But in this case the mechanical and chemical properties must be regularly tested. The method of production is near to the old one but it can not fulfill the absolute homogeneity and standard quality of the new bricks. Results of a sonic test on bricks 30x40x3cm are shown in Table 1. These can be accepted as sufficient when they are compared with the results of sonic test on old bricks of the monument under repair(Penelis et al.,1980). Manufactured bricks have a high cost because of the relatively low production. These "traditional" bricks which fortunately can be still found in our country are more preferable than those of modern industry which do not meet the restoration requirements. These bricks are also better than the imported ones who are strange to the historical building material of the ancient monument.

It is important to point out that during the repair of damaged masonry of a monument the replacement of brick segments(instead of integral pieces) is advisable in case in which the damage is of local nature. Beyond the material worth, this is also imposed by the respect to the genuine material as a proof of originality. This is translated to repairing only the damaged locations of the masonry and not replacing bricks and mortar, when they are in good condition.

Mortars

Core-taking gives samples for determining the mechanical properties (strength, bonding, modulus of elasticity) of mortars as well as the chemical ones, but it is possible only for "high" resistance mortars. The gradation of mortar grains is carried out when a quantity of mortar is crumbled by hand and sieved. Chemical analysis of a "small" piece of mortar accomplishes the research on its constituents. So,the soluble salts content and the type of binder can be found. Modern techniques such as x-ray analysis, thermogravimetric analysis, scanning election microscopy can be used in order to have additional information about the characteristics of a mortar (structure of binder, porosity, hydraulic compounds). These techniques are also useful if cores are impossible to be taken from mortar because of its deterioration.

The old mortars of historic buildings are mainly lime-sand mortars which also have compounds with hydraulic activity such as natural pozzolana and brick powder. Some Roman or Byzantine mortars have high strength and durability even after hundred years(Karaveziroglou,1985). The excellent properties of these ancient mortars are connected to the use of organic additives in the old mixtures such as eggs, blood, milk, wine etc. This can not be proved today; only casein powder has been commonly used as additive in plasters for painting up to now in order to improve the properties of the mortar(reduction of water amount and increase of bonding). The reproduction of the old mortars based on the laboratory analysis of them(binder:aggregate 1:2 to 1:3) with new materials such as lime, recuperable in market, natural pozzolana (from Santorin, Skydra, Mylos etc.) or brick powder does not succeed in reaching the "high" strength of some mortars -as the "opus caementitium" of romans- although the same gradation of grains is also used. It is obvious that the working conditions in the site influence the quality of mortars and plasterings(or even of concrete). Moreover the way of slacking the lime as well as the working of high hydraulity binders in ancient years were different from these ones used nowdays and in the past they led to mortars with excellent properties. So new mortars for restoration reproduced with the old recipies (according the laboratory analysis) develope of low strength.

Cement or resins must be used in order to increase their strength, if necessary. Cement, even being an industrial product is well cooperating with the other "traditional" ingredients when used to replace a part of natural pozzolana. Resins improve also the mechanical properties of mortars but as organic products with a small past of application should not be used when the same effect can be reached by cement. Concerning the choise of natural pozzolana for a new mortar in order to use the same material as that of the monument under conservation the origin of it should be firstly found out. In this case any historical information about it, is useful. Searching for this origin raises the query how far from the monument should one be looking for it. Example given for the cistern of Kameiros in Rhodos(500 a.D.) the pozzolanic material came probably from Santorin or Nisyros, which in the ancient years were known for their pozzolana. The assumption of using Santorin pozzolana in ancient structures could be reality for settlements in the region of aegean sea or collonies of south greeks. For monuments in the inland this seems rather impossible since the material transportation by land during the ancient years was difficult for well known reasons. Table 2 shows new mortars with pozzolana of different origin which have similar properties (Penelis-Papayianni-Karaveziroglou, 1989).

As far as the other ingredients of mortars are regarded, it is pointed out that the aggregates, as natural sand or crushed bricks, must be chosen according to the gradation analysis of the old mortar of the monument under repair, taking into account their quality (laboratory chemical and physical properties). The amount of crushed bricks or brick powder in the new mortar depends not only on the proportions in the old one but also on the desired colour hue of the restoration mortar. The brick powder dominates upon all the other.

CONCLUSIONS

In this paper criteria concerning the choise of basic building materials for reconstruction of damaged old masonry which was built before invention of Portland cement are suggested. The authors believe that "traditional" materials must be used for intervention of old masonry. It is worth bearing in mind that a monument tell us about its history and the old materials as well as the techniques of their manufacture are authentic proofs of history. This principle of restoration of monuments is of course in valid if there are not serious reasons which justify the use of modern materials and methods for their conservation, such as the safety of construction. In some cases "traditional" materials do not succeed in stopping or restraining the source of damage or the high aesthetic quality of monument is threatened by their use while modern materials and technology answer for these requirements. Then, the modern materials and technology are the alternative choise and their use is indispensable.

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Table 1

Sonic test results in bricks of 50mm depth

Bricks		Distance mm	Time μsec	Velocity km/sec	Statistic evaluation of velocity		
					Average val. km/sec	St.Deviation km/sec	Variation %
Repair	1	395	209	1.900	1.930	0.0525	2.7
		395	210	1.881			
		316	158	2.000			
		316	163	1.939			
	2	396	217	1.825	1.823	0.0207	1.1
		396	220	1.800			
		320	176	1.818			
		320	173	1.850			
	3	395	224	1.763	1.956	0.1699	8.7
		395	212	1.863			
		320	153	2.092			
		320	152	2.105			
Christian part of Rotunda	1	382	206	1.854	1.879	0.0741	3.7
		382	209	1.828			
		382	191	2.000			
		286	145	1.972			
		286	167	1.713			
		286	150	1.907			
	2	362	182	1.989	1.999	0.1047	5.6
		363	187	1.941			
		364	170	2.141			
		307	153	2.007			
		307	158	1.943			
		308	156	1.974			

Table 2

Mortars for repair

Mortar		Mixing proportion by weight							28 days test results		
Origin	a/a	Lime	Pozzolana	Cement	S a n d max size		Crushed bricks max size		Strength		Static modulus of elasticity
					2mm	6mm	2mm	6mm	Tensile N/mm²	Compressive N/mm²	N/mm²
Santorine	1	1	1	-	6	-	-	-	0.166	0.410	533
	2	1	1	-	-	6	-	-	0.421	0.697	760
	3	1	0.8	0.2	6	-	-	-	0.311	1.438	1288
	4	1	0.8	0.2	3	-	-	3	0.519	2.605	1686
	5	1	0.2	0.8	3	-	-	3	1.810	5.573	4341
S k y d r a	6	1	1	-	6	-	-	-	0.193	0.336	709
	7	1	1	-	-	6	-	-	0.142	0.652	823
	8	1	0.8	0.2	6	-	-	-	0.303	0.520	1051
	9	1	0.8	0.2	3	-	-	3	0.816	3.066	1973
	10	1	0.2	0.8	3	-	-	3	1.409	5.018	4215

LEARNING FROM THE PAST
TRADITIONAL AND CONTEMPORARY UNREINFORCED MASONRY IN SEISMIC AREAS

RANDOLPH LANGENBACH*

SUMMARY

Many historic masonry buildings have been subjected to major reconstruction in order to reduce the earthquake danger, resulting in a substantial loss of the historic significance of these monuments. In addition, there has been a steady erosion of historic vernacular buildings around the world which have been abandoned or demolished because of the belief that complete rebuilding is necessary.

This paper examines certain examples of traditional low strength masonry construction in Kashmir, Yugoslavia, and Greece which have evolved in earthquake-prone areas prior to the advent of concrete and steel, and which have exhibited varying degrees of seismic resistance. The principle of this aseismic masonry construction, which is discussed, is that the masonry is not restrained from cracking and shifting slightly during a tremor, allowing energy to be absorbed by the walls in a ductile manner.

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LEARNING FROM THE PAST

TRADITIONAL AND CONTEMPORARY UNREINFORCED MASONRY IN SEISMIC AREAS

There was a great earthquake, and the tenth part of the city fell, and in the earthquake were slain of men seven thousand; and the remnant were affrighted...

Revelation, 11:13

The Power of Perceptions

On December 7, 1988 an earthquake struck the central part of Armenia. Although its force, measuring 6.9 on the Richter Scale, did not fall into the strongest category, this earthquake nonetheless caused major destruction and unspeakable human tragedy. The sight of the devastated buildings with thousands of victims crushed or trapped beneath the mountains of debris touched people around the world.

Just after the Armenian earthquake a San Francisco Examiner headline read: "Unreinforced stone, cement collapses." The article followed with: "Most of the buildings that collapsed were unreinforced concrete structures or older brick and stone buildings built before the modern earthquake-resistant codes." In the same article, an official from the California Office of Emergency Services was quoted saying: "many of the buildings constructed before 1940 in California are unreinforced brick and concrete like those reduced to rubble in Wednesday's quake...some 30,000 buildings in the State would be unsafe if a similar earthquake shook California."(Chin,1988)

SRINAGAR, KASHMIR: A building designed for seismic resistance! An example of late 18th or early 19th century Taq construction.

Contrary to these early pronouncements, however, when the American scientists and engineers arrived in Armenia two weeks later they found the truth to be quite different. Indeed, many masonry buildings in the rural villages closest to the fault were damaged or destroyed, but in Leninakan, where the death toll was highest, it was the modern buildings which were subject to the most catastrophic failures. Ironically, many of the survivors are now living in the older unreinforced masonry buildings in the city, most of which survived with little damage. One member of the American team, Fred Krimgold, reported: "It was amazing to see that masonry buildings, while not unscathed, did not cause the major part of the fatalities." (EERI, 1989)

This situation illustrates that the problem of earthquake safety often is as much a problem of perception as it is of engineering knowledge. The profound effects of an earthquake on the collective consciousness of a people is particularly powerful in connection with masonry buildings - the image of heavy blocks of stone upset by the powerful motion of the ground is almost biblical. Every child has watched toy block models of buildings brought down in an instant by a sharp shake of the table.

As seen by the headlines following the Armenian earthquake, unreinforced masonry is seen by most people as dangerous regardless of the actual potential for earthquake damage. However, there are many different types of masonry, and there differences in the nature of the predicted earthquakes at a given site.[1]

When issues of life safety exist, often it is hard to focus on the problems of the preservation, without running into a great deal of confusion over what is needed and what can safely be left alone. Since masonry constitutes the primary structural and architectural material in a vast number of historic structures in seismically active areas around the globe, to destroy or rebuild these structures in the interests of the perceived risk would lead to a substantial loss of the cultural heritage of these regions.

Once it is decided that strengthening is feasible, the choices with historic unreinforced masonry buildings are remain complex. Does one 1) insert new steel or concrete structures, or 2) rebuild them completely with stronger materials hidden behind the carefully "restored" walls so that the change can be scarcely detected? Indeed, if these are the only choices, what future is there for many historic structures? The seismic system adopted can be of critical importance to the future of the landmark. At some point we are in danger of turning our historic landmarks into Disneyland recreations. One must ask: are there less destructive (and costly) alternatives which can meet life safety objectives with equal effectiveness.

One example in the United States, the **California Capitol** building, was completely gutted in 1976, leaving only the exterior walls and the central drum and dome. All of the interior floors and walls were removed and replaced in reinforced concrete. The remaining masonry was covered with an internal skin of shotcrete and the floors were replaced in reinforced concrete. As a result, while the interior of this building is now genuinely spectacular, with impressive museum rooms, excellent craftsmanship, rich materials, stunning colors and textures, none of it is genuine. On the basis of a 1971 seismic study the building was condemned, and a "heart transplant" was authorized when an "ace bandage" may have been all that was needed (Worsley, 1988.) The California Capitol is located in seismic zone 3, rather than the most severe category #4 which applies to San Francisco and most of the Southern California coast. Earthquake danger exists, but the predicted forces are less. The Capitol most probably needed to be strengthened and repaired, but one asks is whether the risk identified in 1971 could not have been satisfactorily alleviated by less drastic, destructive, and expensive measures.

This gutting of structures for seismic strengthening is not limited to the United States. For example, following the 1979 earthquake in Montenegro, Yugoslavia, many structures in the historic cities of Kotor and Budva have been reconstructed with reinforced concrete floors, replacing the original heavy timber. In some of these structures, reinforced concrete columns have been cut into the masonry, forming completely new reinforced concrete structures, with the historic masonry attached. In addition to the destruction of all of the historic interiors, another tragic consequence of this costly and time consuming reconstruction is that all of the local residents have had to be resettled. Now, a decade after the earthquake, many buildings are still unprotected and rotting, awaiting reconstruction, and massive government investment is needed. In the end, these historic towns will never again be genuinely alive or real. They are now only intended to serve as tourist attractions. The evidence of generations of life and use is being erased to be replaced only by the surface wear caused by the shuffle of countless tourists passing through.

The effect of waving the red flag of "seismic hazard" in front of the public - especially a room full of legislators - cannot be under estimated. Once a property is identified as a seismic

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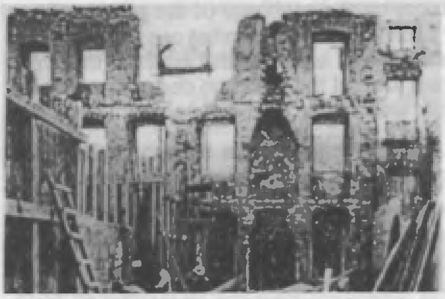
[1]This paper is intended to focus primarily on the problems associated with masonry walled buildings without unreinforced masonry floors or roofs, constructed with regular horizontally-bedded brick or stone (not rubble stone).

hazard, the way is left open for the most extensive and extreme mitigation measures to be proposed. As one engineer said recently: "Anyone can say anything is a seismic hazard. All they have to do is write it down and it is." Any voice of opposition tends to fly in the face of our most basic fears, and following a clear sense of responsibility, the decision makers are understandably inclined to take the safest route.



KOTOR, YUGOSLAVIA: After the 1979 earthquake, the historic interiors of many buildings were removed and reconstructed in reinforced concrete.

KOTOR: Although the exteriors have been restored, the interiors are completely new.

Just such an argument surrounded the strengthening of another large scale historic masonry building in the United States, the **Salt Lake City and County Building**. However, this project is important not because the building was gutted and rebuilt, but because it was the focus of an interesting and provocative debate over the choice between two alternative schemes , both of which promised to preserve most of the interior. These two schemes, however, came from diametrically opposed philosophies on how unreinforced masonry buildings should be reinforced for earthquakes.

The Salt Lake City and County Building project is the first historic preservation project incorporating base isolation. Base isolation has the advantage of reducing the frequency of the shaking as felt by the building, and thus effectively lowering the forces experienced by the component parts of the structure. Instead of what was originally proposed as a "complete gutting and replacement of the interior," which engineers had proposed earlier, the base isolation enabled most of the interior surfaces of the Salt Lake City building to be retained. The base isolation scheme has been completed at a total project cost of over $30 million including the restoration of the building. (BAILEY, et al, 1988)[1]

A simpler and less expensive scheme was proposed by a second engineer hired as a consultant, John Kariotis, co-author of the "ABK Methodology." (Kariotis, 1984) What was most radical about his approach was that Kariotis said that the seismic forces computed to exist using conventional methods of analysis would not in reality occur, even in an earthquake of the same magnitude as that for which the base isolation scheme was designed.[2]

Kariotis' research asserts that masonry buildings actually respond differently than the way the traditional codes and engineering approaches assumed. Rather than amplifying the forces of the earthquake, he states that the heavy masonry walls have the effect of dampening the shaking by acting as a "rigid rocking block on a soft soil base." As a result of the ABK analysis and strengthening methods, the computed force levels in the strengthened building are

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[1]The engineers were E. W. Allen Associates, with Forell/Elsesser as consultants on the base isolation system, The architects were James McElwain, AIA, and Burtch Beall, Jr, FAIA, and The Ehrenkrantz Group.

[2]The ABK Method recommends that the timber frame diaphragms and crosswalls be designed to yield in an earthquake, and thus absorb energy, reducing the likelihood of destructively amplified force levels in the masonry. Randolph Langenbach is currently architectural conservation consultant on the seismic strengthening of the Alameda (California) City Hall following the ABK Methodology.

lower than found under conventional analysis. To explain this, Kariotis reports that the amplified force level of 55% of gravity which was reported that the "City and County Building might be subjected to (BAILEY, et al, 1988)" was computed based on the model of a "single degree of freedom, 5% damped elastic oscillator with a fixed base." In Kariotis' opinion, the expected force levels from the predicted .17G earthquake would be much lower and the base isolation unnecessary. (Kariotis, 1989)

In deciding between the two proposals, the Mayor's Assistant Phil Erickson recently said, "the City Council wanted to satisfy their moral obligation to the public safety, and this [base isolation] was a way to do it." Politically this was important. As Phil Erickson reported: "The building was on the National Register, and we might have been able to find a way around full compliance, but everybody in town comes here for a building permit, and we had to set a good example."

> "You know, I got a sense that the engineers got a real charge out of scaring each other. They would each go around the building and see a crack and , oh God!, its only a matter of minutes and this building is going to collapse around you!....I don't know, maybe the building is perfectly fine, but, when I came in 1984, the City was at the point of nearly vacating it. What are you supposed to do? The engineers - they are credible people, so we just moved ahead and that's what we did [base isolation].

For the City Council, it seemed less risky to accept a system which was shown to lower the earthquake forces dramatically than it was to embrace one based on the observation that those computed forces could not possibly occur. This building, like the California Capitol, is located in an area which is only seismic zone 3, the prediction of extreme shaking, and thus of serious building damage is more remote than for the California coast where many unstrengthened buildings continue to be used.

The trouble with this is that the cost of this project, and the extraordinary amount of engineering work it required, militates against the more general application of this technique. Engineers may argue over whether a less expensive solution, based on a simpler technology, would be less safe, but the real question is what is applicable to the widest range of historic buildings, and thus do the most to mitigate the earthquake hazard for the greatest number of people.

While the high tech, expensive solution is within reach for monumental buildings in the United States, it is not accessible as a solution for most of the world's historic buildings. In justifying the base isolation, it was explained that the building was found to have a weak lime mortar in a deteriorated condition. If every building with soft mortar must either be isolated or rebuilt, most of the most interesting and culturally meaningful landmarks in the world would be lost.

There is, however, another direction which also can be found in vernacular masonry buildings, and that direction deals with the core problem of what happens when masonry walls crack in an earthquake. All of the American retrofit work is focused on preventing masonry walls from yielding in earthquakes. In the present, with strong cement-based mortars, combined with modern materials such as steel and concrete, this is at least conceivable. In historic time, it was impossible. In spite of this fact, not every masonry building collapsed in every major earthquake. Apart from making the masonry walls excessively thick, the only solution was to introduce timbers or iron cramps to help restrain the masonry from breaking apart when it moves. It is the indigenous buildings with timber which are of interest here, not just because of the timber, but because or the weakness of the mortar, and the expectation that cracking of the masonry wall will take place. The buildings found in Kashmir, India, provide an opportunity to examine this approach to masonry construction.

The Aseismic Attributes of Traditional Construction in Kashmir

Earthquakes in Kashmir, India, have occurred with a degree of regularity over the centuries, and the Kashmiri people have had to learn to live with them. Most of the traditional buildings in Srinagar can be divided into two basic systems of construction. The first system, referred to locally as *"Taq,"*[1] consists of load bearing masonry piers and infill walls, with wood "runners" at each floor level used to tie the walls together with the floors. The second system, "brick-nogged" construction, also known as ***"Dhajji-Dwari,"***[2] consists of a braced timber frame with masonry infill. The timber beams in the *Taq* buildings do not constitute complete frames. Instead, large timbers "runners" rest along the load bearing masonry walls, with the floor beams and the "runners" for the cross walls lapping over them. The wood serves to tie the walls of the structure together with the floors. The weight of the masonry serves to "prestress" the wall, contributing to its resistance to lateral forces.

SRINAGAR, KASHMIR: Four-story Dhajji-Dewari *house.*

The construction practices used for these buildings in Kashmir which stand in contrast to the codes and commonly accepted practices today, include 1) the use of mortar of negligible strength, 2) the lack of any bonding between the infill walls and the piers, 3) the weakness of the bond between the wythes of the masonry in the walls, and (4) the frequent (historical) use of heavy sod roofs. It is just such buildings which were observed almost a century earlier by Arthur Neve, a British visitor to Kashmir when he witnessed the 1885 Kashmir earthquake:

> Part of the Palace and some other massive old buildings collapsed...[but] it was remarkable how few houses fell....The general construction in the city of Srinagar is suitable for an earthquake country; wood is freely used, and well jointed; clay is employed instead of mortar, and gives a somewhat elastic bonding to the bricks, which are often arranged in thick square pillars, with thinner filling in. If well built in this style the whole house, even if three or four stories high, sways together, whereas more heavy rigid buildings would split and fall. (Neve, 1913)

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[1]This system, sometimes incorrectly identified as *Dhajji-Dewari*, actually has no specific name in Kashmiri to identify the construction method. The closest name identified by local experts to describe it is "*Taq*". *Taq* refers to the modular layout of the system of piers and window bays. Houses found in parts of Greece affected by earthquakes also have horizontal wood members. The use of horizontal wood ties is also common in seismic areas of Turkey. The bond beams in Turkey are credited with "incorporating ductility to the adobe walls, substantially increasing their earthquake resistant qualities. (Aytun, 1981)

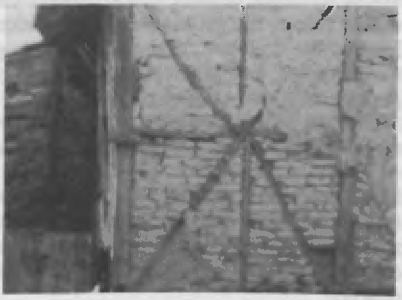
[2]This comes from Persian for "patchquilt wall." Dhajji-Dwari construction can be found in Afganistan and Kashmir, but not in Nepal. (Arya,1988) The brick nogged type of construction also is found in Greece, where it is sometimes used for the upper part of the houses where the stability of the wall is not assisted by the weight of the overburden.

More recently, two Indian engineers, N. Gosain and A. S. Arya ascribed the damage from a 1967 earthquake to the different types of traditional and modern construction in Kashmir. They observed that: "Perhaps the greatest advantage gained from such runners is that they impart *ductility* to an otherwise very brittle structure." (Gosain & Arya, 1967) Gosain and Ayra note that during the 1967 Kashmir earthquake buildings of 3 to 5 stories survived relatively undamaged.

Arya confirmed that his research shows that one of the most important reasons for this is the damping from the friction induced in the masonry of the *Taq* walls. Internal damping "may be in the order of 20%, compared to 4% in uncracked modern masonry (brick with Portland cement mortar) and 6%-7% after the masonry has cracked." His explanation for this is that "there are many more planes of cracking in the *Dhajji-Dewari* [sic: this refers to *Taq*] compared to the modern masonry." It is this distribution of the forces throughout a larger area of the wall, preventing the destructive cracking in one area, which leads to a much greater level of energy absorption than would otherwise be possible. As a result, even though the mortar is extremely weak, causing the wall to yield under a much smaller load, the masonry continues to have a good chance of holding together. The timber runner beams, and floor diaphragms, keep the individual piers from separating which would cause the house to break apart. The more rigid a building is, the stronger it must be in order to avoid fracture. Because of the primitive materials and means of construction in Kashmir, strength was not possible, so flexibility was necessary.

The brick-nogged, or ***Dhajji-Dewari*** type exists side by side with the *Taq* in Kashmir. Half timbered construction continues to provide, an efficient and economical use of materials. The use of wood was kept to a minimum, but it still enabled the thin masonry walls to resist out of plane collapse, while it restrained the in-plane movement of the masonry. This type has also shown a marked resistance to earthquakes when compared to conventional fired masonry or adobe structures. Variations of the *Dhajji-Dewari* type were historically common in many areas not affected by earthquakes, such as medieval England and Europe, and it extended even into North America. However, it has proved especially suitable in seismically active regions such as Yugoslavia, Greece, Turkey, and Kashmir. In some of these areas, earthen or brick building continue to be built in this method and is allowed by the local building codes. (Panayotis, 1981) Arya reports that it has formed the basis for the current Indian Standard Building Code #4326. (Arya, 1988)

In a survey of the damage caused by the 1963 Skopje, Yugoslavia, Earthquake, a London engineer N.N. Ambraseys reports that the "old adobe construction, particularly those with timber bracing, resisted the shock with some damage, but behaved far better than the [modern] brick or the hybrid [reinforced concrete with brick infill] construction." Many of the modern reinforced concrete buildings, which ranged from 3 to 6 stories in height, were seriously damaged or destroyed, while the less substantial adobe buildings survived.



SKOPJE, YUGOSLAVIA: Brick nogged construction.

Although it is difficult to make meaningful comparisons between the performance of the adobe and brick houses and the much larger reinforced concrete buildings in Skopje, the brick nogged type does form an historical precursor to the reinforced concrete frame with unreinforced masonry infill wall construction, now common in many countries. It is striking to note how many buildings affected by the earthquake in Skopje, as well as more recently in Mexico City and San Salvador. The problem is that many of these modern infill wall buildings have performed poorly in earthquakes. If these modern buildings do poorly, how is it possible to claim that the much weaker historic buildings do well?

Traditional and modern masonry: "ductility" versus brittle failure

The infill walls in modern buildings are not designed to be shear walls: they are only meant for enclosure. When masonry is used, it is either brick or hollow tile laid in a cement-based mortar, and these walls are inherently very stiff and brittle. If the lateral loads are great, the deflection of the more flexible frame throws all of the lateral force onto these infill walls which were never intended to carry large loads. As a result, these walls can be shattered. The failure of the walls across one floor of a building can lead to the creation of an unintended "soft story", resulting in the collapse of the structure in subsequent cycles. In engineering terms, research has shown that the behavior of these infill walls is often equivalent to that of a "diagonal compression strut." (Klinger & Bertero, 1976) & (Priestley, 1980) When the frame deflects, it bears upon the infill wall on its upper corner. As a result, until the wall collapses, all of the resisting pressure is delivered to the top of the column just below the intersection with the beam. This can be sufficient to cause the reinforced concrete column to fail in shear, again leading to the progressive collapse of the structure.

If the masonry infill with its equivalent diagonal strut is a danger to the modern reinforced concrete frame, why is it not more hazardous to the weak timber frames found in Kashmir, and Yugoslavia? More research is needed to determine for sure, but the most plausible explanation is that the precompression stress provided by the load bearing weight of the wall, combined with the weak and non-brittle behavior of the mortar, enables the stresses to be spread throughout the wall rather than being concentrated along the diagonal. Instead of one large tension crack, with crushing failure at the corners, the softness and give of the mortar encourages a more wide spread small-scale cracking across the mortar joints of the whole panel. This also allows the building to absorb energy, and thus perform in a ductile rather than a brittle manner.

MEXICO CITY: Collapsed infill with columns damaged by "diagonal compression strut."

While stronger cement-based mortar can provide for a wall with greater strength within its elastic range, the problem in earthquakes is that failure of the bricks or stones themselves can lead to the collapse of the wall. Strength can be important in preventing damage in mild earthquakes, but in severe tremors, even massive strength, especially if it is associated with greater stiffness, can be overwhelmed.

Conservation Technology vs. The Building Codes

Mortar strength and composition has been one of the chief concerns in building conservation technology, and there are few subjects which have received more attention in recent years than mortar and repointing. The discovery of the importance of reducing or eliminating Portland cement from masonry mortars in restoration is one of the cornerstones of recent conservation practice.

> The use of lime-sand mortar ...furnishes a plastic cushion that allows bricks or stones some movement relative to each other. The entire structural system depends upon some flexibility in the masonry components of a building. A cushion of soft mortar furnishes sufficient flexibility to compensate for uneven settlement of foundations, walls, piers and arches: gradual adjustment over a period of months or years is possible. In a structure that lacks flexibility, stones and bricks break, mortar joints open and serious damage results. (McKee, 1980)

This was not meant to refer to masonry in earthquakes, but in light of the Kashmiri experience it is intriguing to ask, whether the notion of a "plastic cushion" is an appropriate concept for walls subjected to earthquake forces. Currently, in the United States, there is a conflict between the Historic Preservation documents which recommend using the weakest and most lime rich standard formula (1 unit cement) to (2 1/4 to 4 units lime) for restoration work, and the Uniform Building Code, which prohibits the use of mortar weaker than the three strongest categories, known as ASTM types M, S & N: (1 cement) to (1/4 to 1 1/4 lime) for any mortar used in structural masonry (which includes most historic masonry walls.)

When unreinforced masonry begins to crack, in terms of engineering analysis, it is usually described as having "failed," even if collapse does not occur. The internal elastic strength of the wall drops, and in repeated cycles, the wall undergoes plastic deformations through movement along the mortar joints (in-plane), or in bending (out of plane). One of the problems with weaker mortars is that of controlling the out of plane failure of walls with a high h/t ratio (height over thickness), but, as found by the researchers of the ABK Method, if the movement of the whole building is dampened by yielding of its internal fabric, the out-of-plane forces are lower. The most important attribute of soft mortar is that, when the mortar strengths are below that of the masonry units, when the wall does crack, it does so along the mortar joints, resulting in greater overall stability.

What can be learned from the buildings in Kashmir? As a material, unreinforced masonry generally performs in a brittle, non-ductile manner, and thus is recognizably dangerous in earthquakes. As a <u>system</u>, however, these examples show that brittle failure does not automatically lead to collapse, and it is the creation of a <u>system</u> which overcomes some of the limitations of the material, which can provide an inspiration for the present.

These insights about building structure must be focused on issues of the locally available construction techniques. It is not engineering "know how", but rather the local economy, labor supply, materials production and delivery, available engineering expertise, and inspection thoroughness which will determine what is actually built. Between the possible and the practical in most earthquake affected cities exists a great gap. The enactment of more stringent engineering regulations is simply not sufficient. In many developing countries, sophisticated engineering and the delivery of materials of uniform quality may not be possible to expect. What is needed is a combination of traditional vernacular construction techniques with modern materials and technology.

Masonry may be properly modeled as a "rigid block on soil springs," or as a "non-ductile, rigid mass on a fixed base," for engineering purposes, but in truth it has life. It moves, it changes color, it ages, and it responds to our own images and dreams of what buildings should be. By "moves", this is not intended to mean falling down in an earthquake, but rather slow and subtle movement, by the heat of the day, by the gradual settlement of the foundations, or by the slow erosion or change of the nature of the mortar bed or of the bricks or stones themselves. This almost organic quality is essential to the aesthetic quality of historic masonry. If we could arrest the effects of time, traditional masonry might lose its magic. Even in earthquake country, it is this essential quality of building which must be preserved.

There has been substantial progress in this direction. In Greece, other parts of Europe, and even in New Zealand, some strengthening projects have been carried out using cables wrapped around the masonry structure. Utilizing the strengthening effect caused by tying the masonry together to create horizontal bands similar in their purpose to the timber runners of the *Taq* system, these buildings continue bear their own weight on the existing masonry.[1] Such

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[1]The effectiveness of horizontal reinforcement (in this case external to the wall) is supported by the observation that horizontal shear reinforcing is more resistive of shear failure displacement than vertical reinforcement. (Priestly, 1980)

systems have the advantage of causing little disruption to the historic masonry surface. The cables are removable, and their condition can be readily inspected. What is radical about this, and other surface mounted strengthening systems is the acceptance of visible changes to a building's interior and exterior appearance. Sometimes it is important to recognize that greater damage may be incurred by changes hidden behind walls which have been rebuilt, than made visible in front of walls which survive intact.

Despite this kind of progress in finding more sympathetic historic preservation techniques, the desire for modernization threatens to result in the eventual demolition of the vernacular masonry buildings in Kashmir- a process which has already showing signs of having begun. As for the new buildings of reinforced concrete, a greater degree of life safety can be promised. As seen in Armenia, promise does not always mean delivery. Perhaps by forgetting the unwritten knowledge of generations of historical evolution, in preference for the seeming certainty of an imported industrialized alternative, a greater risk may result in the end.

The least recognized but most important issue is the tremendous cultural loss which inevitably would accompany the rebuilding of Srinagar and other cities which have similar pre-modern vernacular buildings. When asked the age of the Kashmiri houses, earthquake engineer Anand Arya responded by telling the story of barber who passed his razor on to his son, saying that it had been in the family for 3 generations. "He had replaced the blade, and his father had replaced the handle." The Kashmiri houses were the same. They are ancient, regardless of whether the physical fabric had been replaced in time, simply because they represented the embodiment of a tradition. Total replacement of these structures with buildings of a new reinforced concrete technology destroys that continuity. The work is taken out of the hands of the people whom had traditionally done it and put into the hands of specialists trained in a new way. An alien form makes its appearance on the landscape.

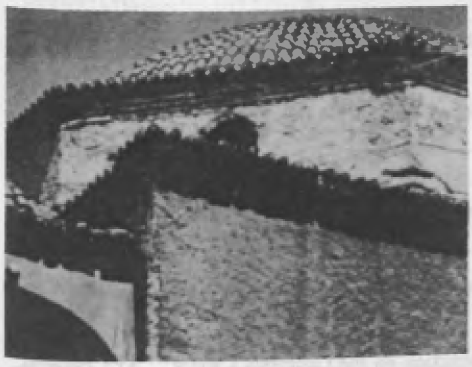
People's perception of the structure of buildings has gone through a transformation in modern times. Traditionally, most major buildings were solid walled structures with the walls bearing directly on the ground. With the current predominance of steel and reinforced concrete as the materials of choice for larger buildings, we are now used to the erection of frames, onto which the enclosure cladding system is attached. As the engineers work hard to convert the highly indeterminate, ambiguous and sometimes uncertain historic masonry buildings into something which can be understood with mathematical certainty, the architects struggle to wrest control of the rigid and unyielding materials of modern day building systems, trying to breathe the kind of subtle life into them that they find at the root of the aesthetic quality of historic structures.

There are many different techniques and systems which might be proposed, but the important point is that historic structures have something to tell us which transcends their formal architectural language. This gift from the past can be erased if the integrity of the original structure is destroyed to meet the demands of hazard mitigation. Understanding both the positive and the negative attributes of masonry construction can guide us towards methods which may be less destructive of original fabric. Some of these methods may even be *more* effective over the long term, not only because they build on strengths which already exist, but also because they are more closely derived from local social, and economic conditions. The purpose of historic preservation is not limited to the static freezing of artifacts. It also has to do with preserving continuity within the slow evolution of building traditions - a continuity which may in the end provide the most effective and lasting defense against earthquakes.

Bernard Fielden has also said that a reinforcing system of placing stainless steel cable in horizontal mortar joints could have great value in conservation work. It would only require the cutting out of a joint, rather than the heavy drilling and coring which has been done for vertical steel placement on some projects. (B. Fielden, 1985)

PELION, GREECE: Traditional stone house showing use of horizontal runner beams in stone walls similar to Taq.

ATHENS, GREECE: Church reinforced with cables.



ATHENS, GREECE: Detail of cable reinforcing at top of wall.

HARRISVILLE, N.H.: Ruined barn showing lateral stability of stone column when under vertical load.

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STRENGTHENING OF OLD MASONRY BRIDGES

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SUMMARY

To find out the extent to which some masonry bridges must be adapted for the new loading conditions, the most important step is concerned with the safety of tall holed piers. This paper describes a mathematical model that succeeds in explaining the causes of the systematic cracking around the openings.
Furthermore, a suitable method to assess the loading capacity of arches is presented.
Finally, design criteria for strengthening and restoration of piers, arches and foundations are given.

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INTRODUCTION

The regional railway network serving the district to the north and west of Milan was first built at the end of the last century. Depending on their locations, the bridges were made either entirely of bricks or else, when stone masonry was used for the piers, with brickwork only for the arches.

Heavier and faster trains made it necessary to modernize and strengthen these lines, and this in turn implied an extensive survey into the needs of improving the bridges so that they could safely cope with the changed service loads.

The taller and, for that period, more advanced bridges (i.e. for these purposes with piers of at least 30 m in height) all contained a platform halfway up as an inspection catwalk (mainly for military purposes, see fig. 1). This means that these piers contain openings that involve stress concentrations.

To find out the extent to which these bridges must be adequated for the new loading condition, and to define how much restoration and/or improvement may be needed, certain steps were taken: (i) data were obtained on the mechanical characteristics and state of degradations of the masonry (whether bricks or stonework [1], [2]), and on the properties of the foundation soils; (ii) the causes of systematic cracking around the openings of the piers were established, in order to prevent this occurring in the future, after restoration; (iii) structural analysis was carried out on the various component members (arches, piers, abutments, foundations) to determine their safety level and to assess the need for any additional measures to increase the strength of the masonry or the cross-sections of the piers.

It is above all the structural analysis of the holed piers that required the most attention. This was developed in three steps: (i) evaluation of the stress concentration around the openings (as an aspect of local safety to prevent cracking, see fig. 2); (ii) assessment of overall stability; (iii) dynamic response of the system to hunting or earthquake induced excitation.

Work on each of these steps can be greatly facilitated by the use of proper idealizations of the stress-strain relationship and by the choice of suitable techniques to convert the actual 3-D reality of the piers into 2-D computer codes.

This leads to considerable saving in computer time, and seems worth further explanation to underline its operational simplicity.

ANALYSIS AND STRENGTHENING OF PIERS

The systematic presence of cracking around the opening makes it reasonable to presume that there is a strict causal relationship between the cracks, the openings in the piers and the constraints exerted by the catwalk.

A first clue on the cause of cracking is to be found in the well known studies on stress concentrations around a hole due to plane states of stress in the linear range [3], [4], [5]. To be precise, the results of these studies cannot be directly extended to own particular case, because of the form of the opening and the particular nature of the material (pratically strengthless in tension).

However, the limitations are not as decisive as they might seem to be at first sight, since the main purpose of this preliminary investigation is to observe the creation of tensile stresses which the material cannot cope with and that lead to cracking. So it seems reasonable to make use of a model of material with linear behaviour provided that, as in this case, the ratio between the maximum compressive stresses and the stress at collapse is not greater that 1/10.

As a consequence, it seems that the maximum horizontal tensile stresses, great enough to induce vertical cracking are confined to the areas of masonry immediately above on below the opening.

In reality, however, this does not happen. This may be due to two factors: a) the width and thickness of the piers are comparable to each other, and this already constitutes a limitation on the plane approach; b) the presence of flat bracing arches imposes, in the area under the opening, boundary conditions that in fact make the problem three dimensional.
Three dimensional solutions in closed form [6] are not suitable for this particular case, for two reasons. First, because they introduce analytical difficulties that are pratically insuperable; second, because the effect of the third dimension only become noticeable for high values of ν (Poisson's ratio), and this does not arise for stone masonry, for which $0.05 < \nu < 0.1$.

For all the reasons put forward earlier, but also to avoid the excessive computations involved in a 3-D analysis that would not be justifiable for present purposes, it seemed suitable, and sufficiently representative of the structural complexity in question, to carry out a numerical analysis by finite elements of a 2-D model able to simulate locally the constraints due to the flat bracing arches.

The numerical model refers to the length of pier that includes the opening, as illustrated in fig. 2. This choice does not imply any thickening of the mesh in individual zones close to the opening, because there was no particular reason in this case to investigate the tensile stress gradients and their consequent maximum values, given the already mentioned characteristics of stone masonry. Each processing took 60" in terms of CPU time.

Only a uniformly distributed vertical load was considered, for the following reasons. Dead loads prevail over live loads in this case. Horizontal forces due to wind and hunting, even though in combination and present up to their maximum level, do not initiate vertical tensile stresses. Beside this, they are relatively small compared to the vertical loads, in percentage, and so do not cause significant variations from the uniform distributions.

In agreement with the De St. Venant postulate, the loaded cross-section is sufficiently far from the top of the pier, and its distance from the top of the opening is greater than the width of the opening itself.

The numerical analysis used the SAP V2 computer code for a plane state of stress. This is only exactly satisfied when the material shows $\nu \to 0$. However, it is reasonable to assume that for $\nu \simeq 0.05$ the approximation is still acceptable. In fact, as shown in [6] for a parallepiped crossed by a circular hole, the maximum stress in the third direction (z) are very modest, and anyway $\sigma_{z\,max}/p \leq 0.1$ for $\nu = 0.05$.

All that is required, and it is quite reasonable in the present case, is to interpret the stone masonry as material with a Poisson ratio $\nu_z \to 0$ in the third dimension.

The flat bracing arches, which provide the constraints against displacements in direction y, is simulated by attributing only to the material in area of the springers, orthotropic behaviour characterised by an ideal elastic modulus E_y much greater than E_x, the latter being the value assumed for the masonry of the pier. In fact the presence of the arch gives a high degree of extensional stiffness to the masonry elements around the springers.

No precise experimental data is available on the value to be attributed the E_y modulus of ideal material with greater stiffness. So a series of calculations were carried out with 10, 20, 40, 100 as the assumed values for the ratio E_y/E_x, in order to have some basis for judging what influence variations in this ratio might have on the stress state, and particularly on the distribution and size of the tensile stresses.

In each case the numerical analysis confirms the presence of horizontal tensile stresses along a good part of the length which is in fact damaged in the pier. Variations in the ratio exert only a modest influence on the exent of the tensile zone and the size of the maximum tensile stresses.

Fig. 3 illustrates two of the $\sigma_y(x)$ stress distributions that lead to damage in the masonry. The amount of this damage, even in the part which in the model is subjected to compressive stresses, should be no surprise. In fact, even a relatively short crack will change the model. So even without any specific analysis it is clear how the crack, once is started, can propagate as in the pier.

Fig. 3 also shows the variation in direction y of stresses σ_y at the height of the springing line. They are pratically independent of the ratio E_y/E_x. As was to be expected, the ratio between the values of the stresses σ_x on the fibres at the two ends is very close to three.

The results overall show that the mathematical model proposed here is sufficiently representative of the real complex structure and is able to show the causes of cracking as soon as the ratio E_y/E_x rises above 10.

It may be of some confort to note that above this limit the solution is little influenced by variations in E_y/E_x. This means that the indeterminacy of this ratio, which can never be eliminate, does not invalidate the model in question.

Any decision about the structural safety of the piers must also be based on a study of their stability. In that sense the catwalks at the top and half way up the piers constitute very stiff longitudinals constraints. Because of the effective slenderness values only transversal displacements are to be feared. The catwalks are in fact rather compliant constraints against transversal displacements, and as such are difficult to define. However, they do offer some modest contribution which, as it is on the side of safety, may be neglected.

The presence of the holes is of no great importance if related to the stability analysis of piers, given their position (figg. 1, 2). When the entire cross-section reacting, the ratio between the span of the opening and their larger side of the cross-section is never greater that 0.2, so that the reduction in inertia is less that 1% .

To sum up, given these conditions the analysis can be performed on a pier model of variable cross-section, with no openings and subject to its dead weight and a load P acting with known eccentricity at the top. The real constitutive law of the material (σ - ε), which in fact is non linear, must be substituted with the ideal law of elastic-brittle material. As Sahlin suggests, the E modulus is given the average value between the tangent at the origin and the secant at failure [7].

As the dead weight must also be taken into account, the indefinite equilibrium equation has to be expressed in terms of shear [8].

The following process was adopted to establish the value of the critical load. For an assigned eccentricity value at the top of the piers, the load P was increased, and for any cross-section length the eccentricity of the vertical force was determined. In this way it is possible to redefine the section which is actually reacting and correct, as necessary, its geometrical characteristics. Iteration of the process can be continued up to the desired degree of approximation.

The stationary point on the load-displacement curve supplies P_{cr}.

For piers of modest slenderness, it may be the condition $\sigma = \sigma_c$ (reached at the point under greatest stress) which signals collapse. This is, in fact, the way that the bridge piers of fig. 1 behave. However, their service load is very confortable on the safe side.

Both the guidelines of this study and the ones of the dynamic behaviour of masonry piers, cracked and uncracked, were presented in [9].

All the results obtained have clearly shown the need to repair the damaged areas of the piers. It is essential to restore the structural continuity endangered by the vertical cracks (fig. 2) by injecting them with cement mortar, which must satisfy a number of requirements [13].

However, it is also reasonable to be expected that the damage may arise again, under normal service conditions. So it is evidently necessary to protect the threatened areas from tensile stresses by applying an equilibrated system of horizontal forces above and below the openings strong enough to replace to tensile stresses with a limited level of compressive ones (fig. 4).

This intervention serves three purposes: (i) it offers precautions against the inevitably approximate results from the numerical analysis which depend on the model used and on the notoriously uncertain mechanical characteristics of masonry; (ii) it gives local improvement to the strength of the material in the presence of compressive stresses in the two main directions of the plane model [11], [12]; (iii) it ensures conformity between the real structure and the model used to study overall stability and dynamic behaviour [9].

These additional horizontal compressive stresses are achieved by a two-by-two system of steel beams tightened with an impact spanner to provide the correct prestressing in the masonry. This too was simulated in the numerical analysis.

Fig. 4 shows the distribution of stress σ_y, which are everywhere compressive, in the previously damaged position. The position and number of the horizontal forces applied depend on the need to spread the effect of precompression and limit the maximum values of σ_y. The numerical analysis also demonstrated that, as was to be expected, the values of σ_x changed hardly at all.

There are also evident computational advantages to be obtained by translating, with suitable expedients, the 3-D reality of the construction into simpler computer codes that make use of 2-D elements. When necessary (if the particular aspect of the problem permits it) the real stress-strain curve (σ-ε) of the old masonry, which is generally so unreliable, can be interpreted as a straight line.

ANALYSIS AND STRENGTHENING OF ARCHES

Brick masonry arches considered herein have a span of 11 m and constant thickness of 65 cm. Their shapes is circular. The span is relatively low with the respect to piers heigth (30 - 45 m). Live loads have been considered as a set of uniformely distributed forces due to the heigth of the embankment existing between the extrados of the arches and the tracks.

It follows that the most severe load conditions to be accounted in the analysis is the one represented in fig. 5, where live loads act on half of the arch. A limit analysis procedure for brittle materials has been implemented. The poor quality of brick masonry prevents the application of Heyman's limit domain, which assumes an indefinite compressive strength.

The N-M domain of fig. 6 has been used, being it able to describe the effective behaviour of a section made by brick masonry with no tension capabilities [15].

The arch collapse mechanism turns out to be determined by four "plastic" hinges, of which two occurs at the springers.

A specific computer program enabling to follow hinges formation throughout the arch was used, according to well known theory of kinematically admissible configurations [15].

Results show that the actual system cannot withstand loads corresponding to a load multiplier greater than 5. This load level agrees with the prescriptions of the Italian code for masonry structures [16].

Pratical strengthening was to be carried out while rail traffic was kept running. Therefore the increase of sections size was performed by concrete shot at the intrados of arches.

A steel mesh was placed as shown in fig. 7 to prevent shrinkage. This figure also shows steel dowels, uniformely distributed on the contact surfaces in order to improve adherence between the existing masonry and concrete. Cautelatively strength properties of the added concrete were assumed equal to the ones of masonry (this was the case for tensile strength too). The characteristic strength in compression was assumed equal to 4 MPa, as shown by experimental surveys.

Loading tests showed a good behaviour of the strengthened arches. Also the aesthetic results were satisfactory, as assessed by the official opinion of the Architectonic Autority for monuments of Lombardy (Servizio dei Beni Ambientali della Regione Lombardia).

ANALYSIS AND STRENGTHENING OF FOUNDATIONS

Strengthening of masonry bridges also requires attention to foundations, both with respect to erosion problems, and to intrinsic load bearing capacity of foundation when soil tests show a poor quality of surface ground. A typical strengthening has been carried out in four phases as is now described. First a set of vertical "micropiles" (foundation piles with small diameter size, i.e. 12-18 cm) have been placed at the edges of the footstall (fig. 8) which turned out to be of poor concrete quality.

Then a r.c. frame was built all around the footstall in order to confine it. In the third phase a set of inclined "micropiles" has been placed in the location shown in fig. 9. Note that they also provide a significant improvement in the internal organization of the existing masonry. Finally a number of horizontal steel bars have been put into action in order to eliminate the thrust originate from the inclined micropiles. These bars have been drilled into the masonry and subsequentely local cement grouting was performed.

A r.c. band was placed around the pier in the area where steel bars were located.

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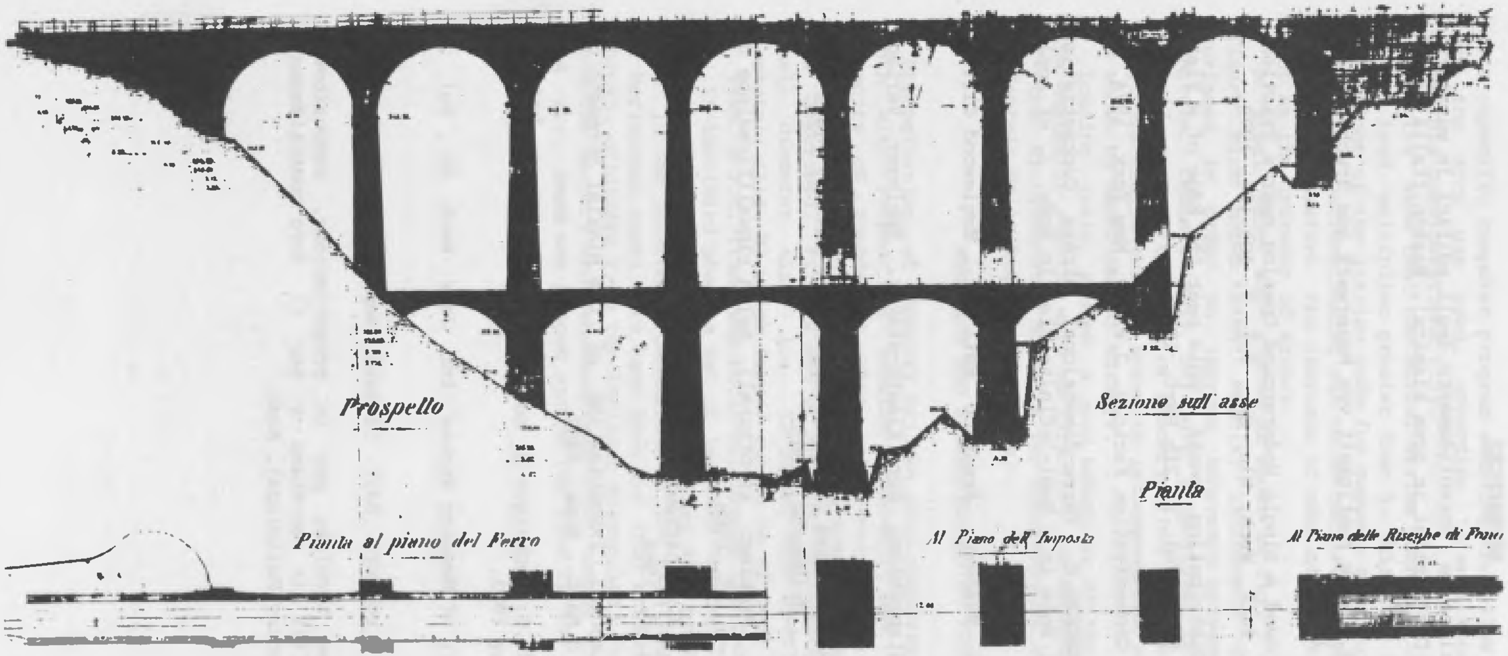

Fig. 1. A typical masonry bridge with catwalks.

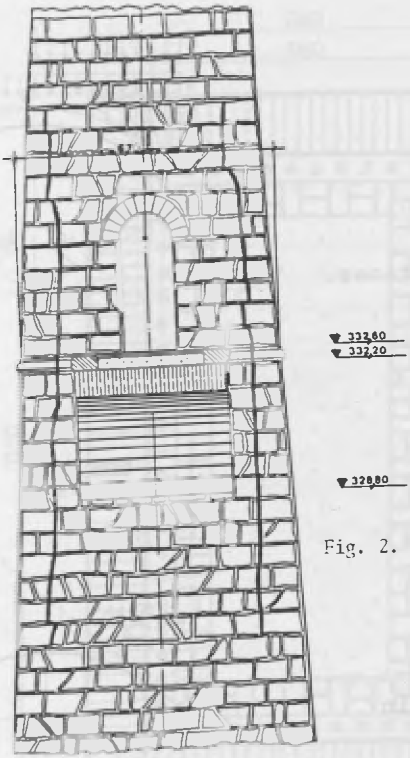

Fig. 2. Cracking along the piers close to the catwalk.

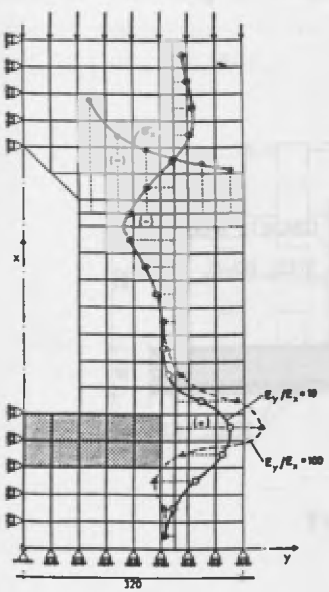

Fig. 3. Stress distribution depending on E_y/E_x ratio.

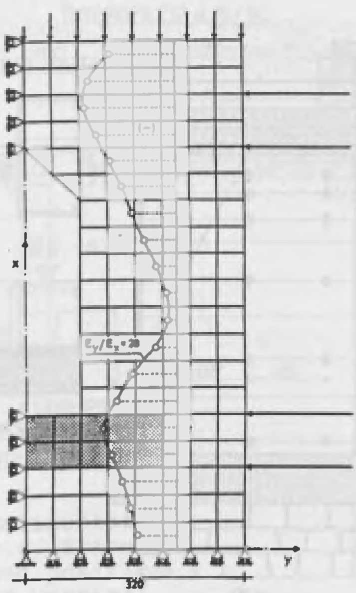

Fig. 4. Distribution of horizontal stresses after prestressing.

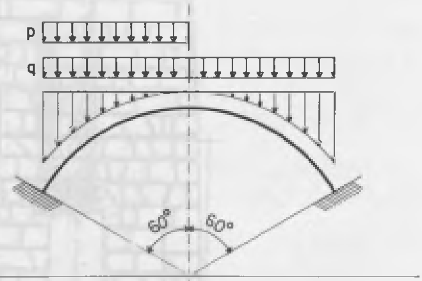

Fig. 5. Load conditions.

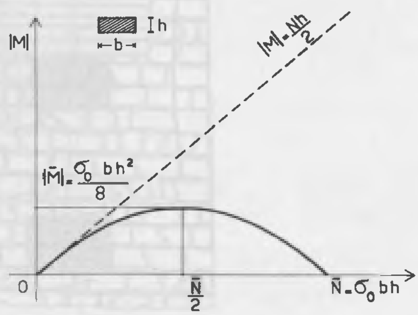

Fig. 6. N-M Domain.

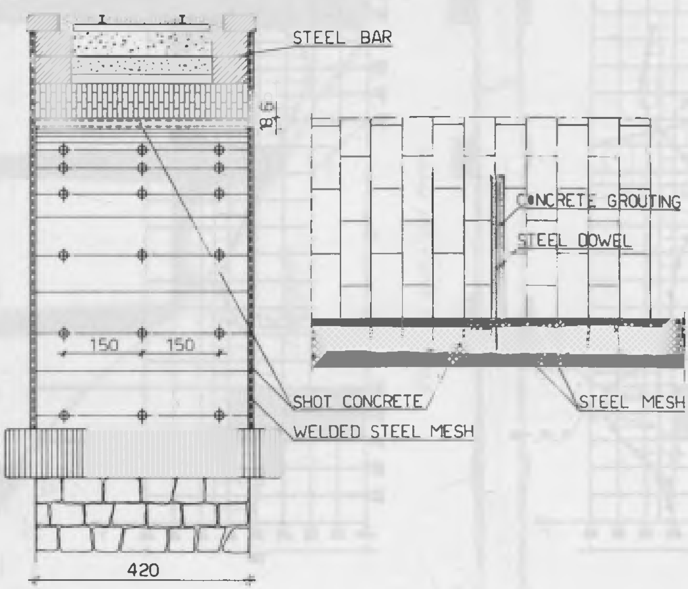

Fig. 7. Strengthening of the arches.

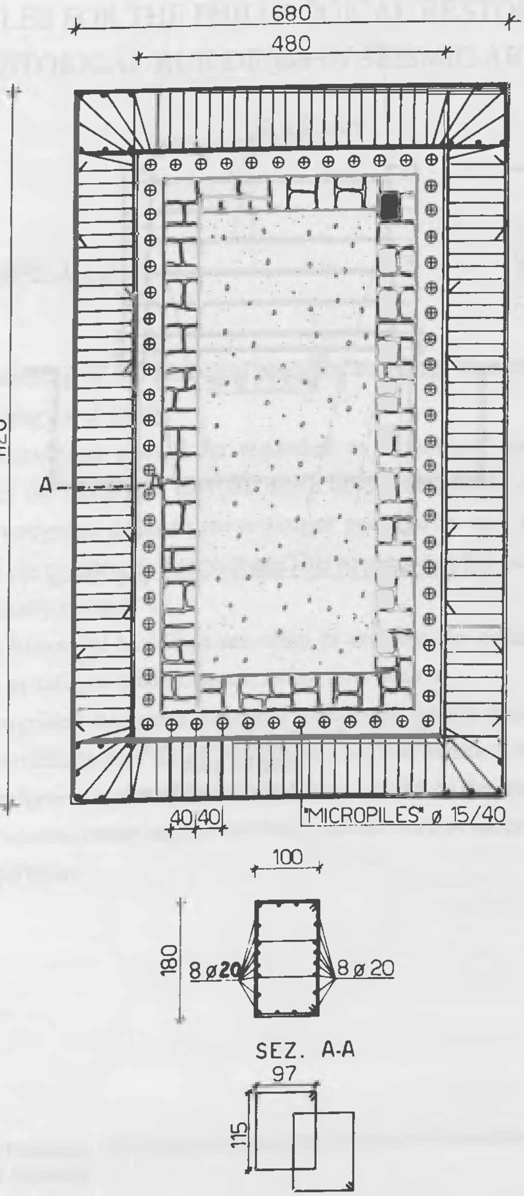

Fig. 8. Horizontal r.c. frame and position of vertical "micropiles".

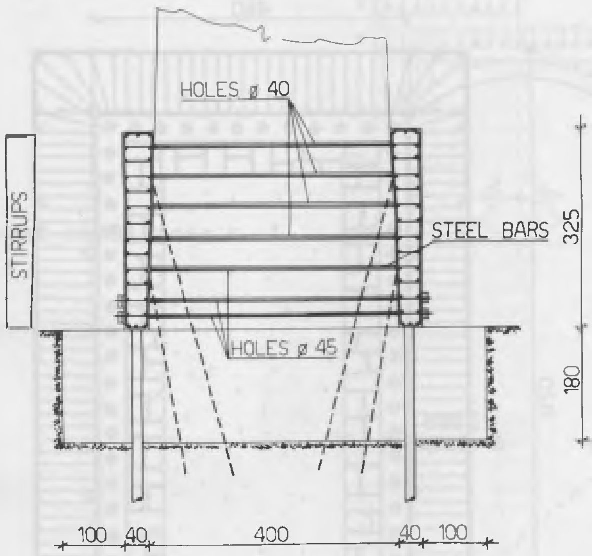

Fig. 9. Position of inclined "micropiles".

PRINCIPLES FOR THE PHILOLOGICAL RESTORATION OF HISTORICAL BUILDINGS IN SEISMIC AREAS

Antonino Giuffrè *

SUMMARY

Restoration or strengthening of the historical masonry buildings in seismic areas involves both problems of philology and safety.

Since the ancient techniques should be regarded as historical documents, like the architectonic feature of the buildings, they are worth being preserved. At this purpose it is firstly necessary to recognize them in their proper peculiarity and in reference to the historical evolution of the constructive tecniques. This approach is the base for the design of interventions philologically correct.

On the other hand the historical buildings are often in use like the modern ones; it follows that they require to be as safe, in seismic areas, as the new houses.

At last it should be recognized that usual teoretical procedures for the structural analysis don't apply to the masonry structures, and safety verification can't be attained in the standard way.

In this paper three foundamental principles are proposed, related to the mechanical behaviour of ancient masonry structures under seismic actions, and the way of ascertaining their safety or designing their restoration.

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INTRODUCTION

Italy, as almost all Europe, is full of historical buildings. The most important part of the main towns retain the features that for centuries have distinguished them. Entire little towns testify nearly a millennium of history and achieve an uncredible continuity of the new way of life with the ancient one.
Quick progress in constructive techniques after the Second World War put a stop to the former culture and attention given to the old buildings. People preferred to leave their family houses for anonymous flats and engineers and architects implemented their skills only within the limits of the modern materials: steel or reinforced concrete frames.
Nowadays the restoration of an ancient building is a very puzzling task. The engineer aims to recognize in it his habitual structural tipology and by force designs his intervention in such a way as to transform the ancient in the new conception.
This is not a successful choice for two reasons: a cultural one and a technical one.
From a cultural point of view the original technologies and the original statics are historical testimonies as important as the architectonic feature, and they must be understood and preserved.
From a technical point of view it should be observed that the seismic strength of the original structure, even though it cannot be demonstrated by the usual computer programs, has been checked by the earthquake and often it is safe enough.
On the contrary, the new materials do not easily achieve a consistent unity of behaviour with the old masonry. Different stiffness and different strength can give rise, during the seismic action, to dangerous separations, and the strengthening can turn out to be weakening.
In order to avoid this double failure it is opportune to settle some fundamental principles and formulate the requirements that must be fulfilled by every intervention on historical buildings, when, of course, their historical nature is considered a positive feature of the architecture, and safety against seismic action is required to be scientifically ascertained as well.
In the following paragraphs such principles and requirements will be proposed.

1. FIRST PRINCIPLE

The foundamental principle related to the technological and structural feature of the monuments is that *their techniques and their statics are important testimony of the culture.* The following requirement ensues:

for interventions on historical buildings their technology and their structural feature must be closely analyzed referring to the cultural context at the time of their construction.

We need to know not only the actual structural consistency, but the historical reasons for it as well.
Sometimes subsequent stages are present, relative to later moments in the evolution of the monument, as in the case of interventions made in the XVII century on medieval buildings. Very different technologies coexist in such cases in different parts of the same building and the stated principle requires that all of them are analyzed and clarified. In addition to the cultural data, essential for the correct preservation of the building, such an analysis gives useful technical information. It is evident, for instance, how important it is for the engineer to know the detailing of the connections carried out by the ancient workmen between the old and the new part.
The historical reasons for the structural choices made by the ancient architects, cannot be inferred just by observation of the analyzed monument. They are the grounds for the constructive rules which were usually applied at the time of the construction, and ensue from the scientific knowledge of that epoch.

In every epoch, in fact, a well defined corpus of rules existed, that from the italian "*regola d'arte*" can be called "rule of art". It is an objective and recognizable way to build which the researchers of the "History of the Constructive Technics" single out, comparing the coeval architectonic production.
The way to build is like a language. Like a language the "rule of art" diachronically evolves and synchronically changes. At a given time regional techniques can be found to be extremely similar over large european areas, with local variants due to local reasons, just like dialects.
A good comprehension of the structure to be analyzed involves the comparison between its actual features and the competent "rule of art" that can be theoretically taken as the "prototype". This is the way to define objectively the structural feature of the building, dividing the analysis into two stages: the study of the prototype and the actual deviation from it.

2. SECOND PRINCIPLE

The second principle is related to the mechanical seismic response of the structure, and is the following:

the earthquake gives rise, within the structural framework of the building, to several kinematic mechanisms, normally involving only parts of the whole building. One or more of them can move as far as causing collapse.

The requirement derived from this statement is:

All the possible collapse mechanism that can be set in motion by the earthquake must be investigated by taking into account the actual connections. They must be pointed out and, if possible, analyzed by numerical procedures.

It should be noted that masonry structures have usually monolateral connections. These invalidates the standard procedures for the dynamic analysis, and the modern code-format based on the ductility-factor loses its effectiveness. The dynamic response to the earthquake cannot be computated through the simplified method of the elastic response spectrum, and a non linear step by step analysis, referred to a realistic accelerogram, and based on effective mechanical models, is the only way to get a significant numerical result.
However, even when this seems a practicable way, the subjectivity of the choice of the input parameters which very seldom can be realistically evaluated, turns the result of a rigorous procedure into a subjective estimate.
Important help can be attained accounting for the alternative way adopted to examine the behaviour of a complex mechanism, when the computational methods fail for lack of realistic input data: i.e. the experimental way.
The dynamic experimentation is usually carried out on the shaking table, moving it according to a given time-history of the ground motion, but in this case the difficulty of working out a physical model is nearly unsuperable.
The results would be as subjective as the analytical ones, or more.
Nevertheless we can often find (unfortunately not always) in literature and in the local archives, precious information regarding the effects of past earthquakes on the examined building. It can be limited and imprecise, but it is derived from an experiment made on the "true" model with a "true" ground motion.
The input data was objective, even if the observer was neither objective nor competent. It is our job to practise a critical interpretation of the chronicles found in the archives, and of the marks left by the earthquake among the stones of the monument.
The aim of this research is to single out the weak parts of the building, the mechanisms brought out by the earthquake and the intensity of the seismic action that has produced the

documented damage. This information often allows the formulation of an effective mechanical model and the evaluation of its parameters. Nevertheless, when the computational way is not practicable due to analytical difficulties, a good documentation and a careful survey of the building give objective data on the structural response to the earthquake, scientifically more reliable than the results of computations based on unjustified or oversimplified models.
It is important to note that, when the structural behaviour has been derived from observation of the historical damage, the earthquake is defined by its intensity scale. It can be useful to keep such a definition as the parameter of the correlation damage-intensity.

3. THIRD PRINCIPLE

The analysis of many historical towns struck by recent strong earthquakes, and the study of many ancient monuments surviving after a lot of them, allow us to outline a third principle involving the problem of seismic safety in the optics of historical preservation.

The ancient constructive technics and structural typologies, if carried out according to the "rule of art", in most cases are seismic resistant.

This statement derives from the common remark, referred to by observers of all times, that extensive damage has been suffered only by very badly constructed buildings.
If that is true or, better to say, when that is true it is evident that a philological way to strengthen an historical building must aim at restoring the original "rule of art" by approaching the prototype as near as possible..
As a matter of fact, the second half of the XVIII century, after the earthquakes of Lisbon in 1755 and Calabria in 1783 which stired up emotion all over Europe, saw important work of strengthening the architectures by buttresses and ties. The alternate action of the earthquake had not been understood and those interventions were not completely effective, but nevertheless from that time through all the XIX century the technique of masonry arose to a very satisfactory level of reliability. So, the third principle can be completed by the following statement:

the XIX century technics can provide seismic resistent detailing, enough to achieve the requested safety.

The corresponding requirement could be so formulated:

Seismic resistence will be achieved by restoring the original "rule of art" and, if necessary, by adding new detailings consistent with the original constructive technique, that can be derived from the technical literature of the XIX century.

In this way the original lexicon is respected, even if completed by the addition of some elements with the aim of controlling the possible kinematisms.

If the original structural framework is seismically ineffective, or the masonry is too far from its "rule of art", seismic safety cannot be achieved without extensive alterations. In this case something must be consciously renounced: either safety, limiting the use of the monument, or preservation, accepting structural modifications. A compromise consists of rebuilding the ineffective portion of the building, using criteria and techniques coherent with the original ones. The choice here involves cultural responsability and will be carefully weighed.

After all that, or, it would be better to say, before all that, as the analysis of the building will be based on the knowledge of its proper techniques, the definition of the "design earthquake" to be accounted for in the verifications must derive from careful local documentation. The

probabilistic definition is not useful when a cost-benefit equation cannot be written, and with reference to the cultural arts all the attempts to transform into parameters the terms of a cultural policy introduce within the optimization process unacceptable arbitrary elements.
The choice of the reference intensity can be more rationally made if a local catalogue is arranged, based on documents.
If information over a long period of time is available, the historical maximum can be accounted for as an expected maximum; the usual procedures for correcting the catalogue, accounting for completeness, source mechanisms, local geology, can be adopted with the aim to improve the local information.

All the philosophy here presented was, point by point, applied for the restoration of the Cathedral of "Sant'Angelo dei Lombardi", damaged by the earthquake on November 23rd, 1980. In a separated paper the study of the restoration has been shortly illustrated pointing out how the above requirements have been fulfilled.

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SEISMIC SAFETY OF HISTORIC BUILDINGS IN MEXICO CITY

by

Roberto Meli[I] and Oscar López Bátiz[I]

ABSTRACT

The performance of old stone and adobe buildings in the 1985 Mexico City earthquake has been in general terms rather satisfactory. Damages were mainly related to previous deterioration due to differential settlements or to weathering and to modifications of the original construction that had weakened the structure.

After a general description of the structural problems in the monumental buildings located in the very soft soil of the central area of Mexico City, the paper describes a research program conducted to ascertain the structural seismic safety of a particular type of historic buildings, those used essentially for housing. The research included the identification of the most common structural schemes, the characterization of the materials by in situ and laboratory tests and the determination of the dynamic properties of typical buildings by measuring their vibration under ambient conditions.

The main problems that can affect the seismic safety of these structures were identified; particulary critical is the stability under seismic forces of walls that are leaning due to differential settlements of the building. Some design criteria are proposed to take into account these effects.

A broader research program is now under way aiming at calibration methods for the linear and non-linear analysis of more complex monumental buildings and at obtaining more information about the mechanical properties of the materials. The main features of this program are outlined.

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1. Structural problems in historic buildings

Historic buildings in Mexico City are mainly heavy masonry constructions from the XVI to the XIX centuries. Their structural safety is governed essentially by two problems: settlements and earthquakes. Both problems are related to the very special characteristics of the subsoil of the part of the Mexico City valley where most of these constructions are sited. The thick deposits of very soft saturated clay that were the bed of an old lake, offer a very small resistance to loads and tend to consolidate under the weight of the buildings and due to the extraction of water from the subsoil, thus producing large settlements of the huge and heavy stone buildings. Furthermore, the same soft clay deposits are very sensitive to the seismic waves transmitted by the underlying hard soil. When earthquakes occur even at distances as large as 400 km, the seismic waves with the lowest frequencies are amplified several times by the clay deposits, thus producing at the surface a quasi-harmonic ground motion of very long duration and with large ground displacements.

The combined effects of the two phenomena have been very harmful to monumental construction in the area. The problem of the settlements has been the most critical. A uniform distribution of pressure on the soil produces settlements that are greater in the center of the area of the construction than in the corners, thus giving rise to a characteristic bowl-shaped profile; the existance of artificial fills and of old buried structures makes the deformability of the soil to be highly erratic and tend to produce large differential settlements of the buildings.

Old masonry buildings have shown to posess a surprising ability to absorb very large differential settlements, if these occur very slowly in time. Nevertheless, large cracks in the walls and vaults are a common feature in old buildings. Cracks tend to open again at the same place, or nearby, after each reparation. The out of plumbing of walls and the distortion of roofs pose stability problems that are more critical than that of the cracking of walls.

The type of ground motion produced by earthquakes in the lake bed area is not particularly harmful to the monumental construction. Ground accelerations are not excessively large (not exceeding 0.2 g) and dominant periods of the ground motion (from 1 to 3 sec) are much larger than the natural periods of these very rigid buildings (from 0.1 to 0.3 sec). Nevertheless, the lateral forces induced on the already distorted buildings could bring about the overturning of out of plumb walls and the collapse of roofs. Furthermore the great number of seismic events that are felt in the lake-bed area, tend to progressively deteriorate the capacity of the buildings.

The repair of the structural damage in the buildings has aimed at avoiding the separation betweeen walls and at providing continuity between walls and roofs. The most accepted criterion has been not to obtain a rigid structure but to allow the structure to follow the deformations of its foundation with a minimum damage and without collapse. Original techniques have been developed to limit, and to correct at least partially, the differential settlements of the buildings. The use of control piles, that are point bearing piles with an adjustable head through which the structure can be jacked up, has been very successful.

The 1985 earthquake has been tremendously destructive for midrise modern construction, but did not produce the collapse of any significant monument. The amount of earthquake damage in these buildings could not be precisely assessed in most cases, due to the lack of information about the part of the damage that existed before the earthquake. In general terms, the opening of cracks in walls and vaults and the increase in the settlements and in the distortion of the buildings was a common feature.

2. Research program on structural safety of historic houses

After 1985 a major program was undertaken to provide new houses to substitute those that collapsed or that were badly damaged by the earthquake. Through 1986-1987 more than 50,000 new housing units were built. The program was associated to the rehabilitation of the downtown area of Mexico City where most of the new houses were built. Within this program 150 buildings qualified as historic monuments were rehabilitated. In association with this job, a research project was sponsored by the agency in charge of the program (Renovación Habitacional) to provide basic information for the verification of the structural safety of the particular buildings of the program and of the historic buildings in general.

The research included the identification of the most typical structural systems of the buildings, the determination of the mechanical properties of the materials and the verification of some simplified models for the seismic design.

In this kind of building, different types of low quality masonry walls are used: adobe, tuff, stone or heterogeneous masonry units joined by lime-sand or mud mortar. Wall thickness varies from 50 to 80 cm. Floors and roofs are commonly supported by parallel timber beams and have thick earth fills. Many of these constructions are badly deteriorated by weathering and by water absorbed from the ground or leaked from the roofs and walls.

The two most typical building plans are shown,in Fig 1 and 2; commonly these are two-story buildings with a story-height of to 5 m.

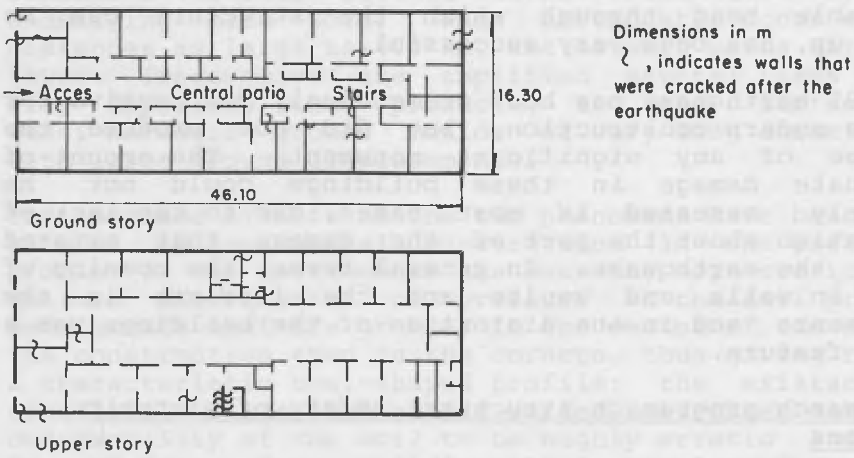

Fig 1. Typical building plan for the "Vecindad" type

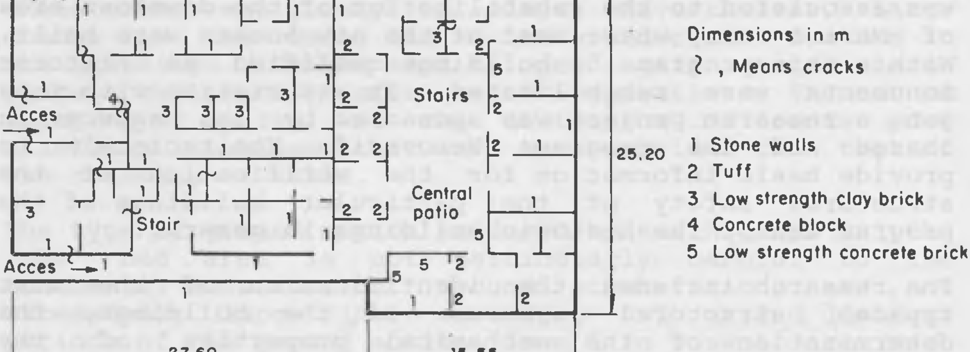

Fig 2. Typical Building plan for the "Bloque" type

The rehabilitation of the buildings consisted mainly of removing appendices and alterations, of redistributing the internal space for the essential needs of modern housing, of providing protection against water and dampness and of replacing deteriorated members. In the structural aspects, the repairing of cracks and the restoration of the connection between walls and between floors or roofs and walls, and specially the stiffening of floor to constitute horizontal diaphragms, were the most common actions.

3. Determination of structural properties

Several laboratory and in situ tests were performed to determine those structural properties that are necessary for the computation of the resistance of the buildings.

Among the non-destructive tests, the most intensive one was the measurement of wave velocity from ultra-sonic pulses, aiming at detecting imperfections in the construction and weakness of the material. Results are not considered to be very positive. Only really major defects could be detected, because the discontinuity and heterogeneity of the material give rise to higly erratic readings.

The attemps for extracting unaltered samples from the walls were successful only for adobe or tuff walls. For stone and other heterogenous materials the samples were

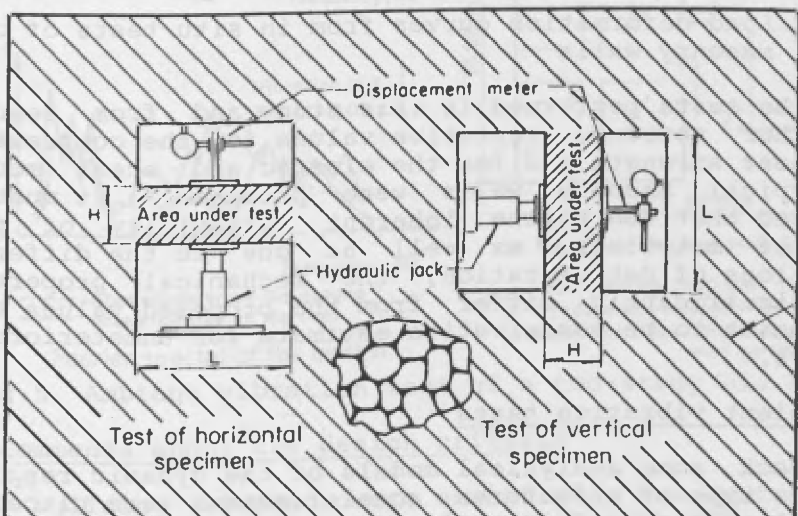

Fig 3. Schematic view of the in situ tests for mechanical properties of old masonry walls

either not representative of the actual composition of without being damaged. The most effective way of determining some basic material properties was by carving the specimen in the walls itself, by opening holes at the in Fig 3. The length-to-depth ratio of the specimens was varied between 1 and 3.5, to change the type of failure from pure shear to diagonal tension and to flexure. Only when the former ratio was of about two, consistent results were obtained; for lower values considerable arching of the load ocurred, whereas for higher ratios the specimen tended to crack due to its self-weight. Load-displacement curves obtained for two walls are shown in Fig 4.

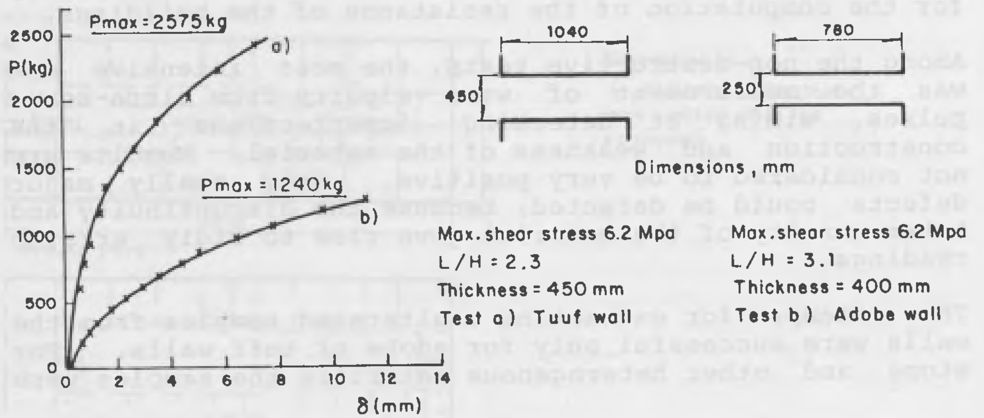

Fig 4. Load-deformation curves from in situ tests of old masonry walls

From the tests performed in this study and from results of other sources, tentative values for the compressive and shear strength and for the elastic and shear moduli of typical masonry walls were proposed. It must be realized that due to the inherent variability of this kind of materials, as well as due to the different conditions of deterioration, the mechanical properties can significantly differ from the proposed values that were meant to be conservative estimate for undeteriorated members.

4. Ambient vibration tests

To check some analytical models of the dynamic response of this type of structures, accelerometers were placed at several points in four typical buildings to measure their vibration under ambient conditions. The records were processed through a spectrum analyzer in order to obtain power spectra and correlation functions between the motions at different points. It was found that the motions of floors and walls were always "in-phase", meaning that no relative sliding ocurred between these elements, at least for the low amplitude vibrations applied in the tests. Fundamental periods of the structure were determined by comparison of the power spectra of the motion at ground level and at the top of the building, to identify the period that appeared only in the building and did not reflect the dominant frequencies of the ground motion, as can be seen from the case illustrated in Fig 5. Fundamental periods of the four buildings ranged from 0.08 to 0.12 seg. They were compared with what computed by simple models considering a story stiffness derived by adding flexural and shear deformations of the walls. Computed periods were consistently larger (by 30% in the average) than those measured. The difference is attributed to an underestimation of the elastic moduli that, for the very low levels of stresses imposed in the ambient vibration tests, are significantly larger than those determined in tests made in laboratory or in situ test, where higher levels of stresses are applied.

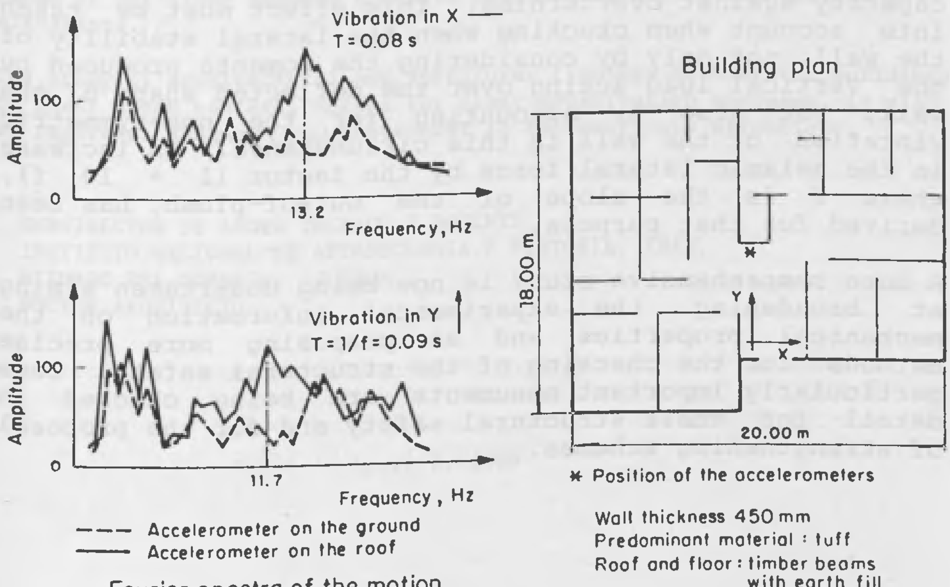

Fig 5. Ambient vibration test of a two-stroy old house

5. Comments about the design criteria

The building studied were essentially box systems that are rather simple structures from the point of view of their behavior under vertical and lateral loads. Most historical buildings have much more complex configurations.

In general, simplified analysis methods based on the equilibrium of forces for realistic failure mechanisms must be preferred over force distributions derived from linear elastic analyses.

For the box systems considered, the main factor differentiating the behavior is the diaphragm effect of the roof and floors. If a proper diaphragm action can be assumed, the shear capacity can be checked for an average shear stress on the walls for each direction of the lateral fources. If that is not the case, the shear capacity of each wall must be checked under the forces generated by its tributary mass. Differential settlements seem not to significantly affect the shear (in-plan) capacity of the walls, even when they have been previously cracked by flexure or diagonal tension. In any case shear capacity is seldom critical in this type of building; its structural safety is commonly governed by out-of-plan bending. Walls tend to separate from the perpendicular ones and to vibrate as cantilevers; failure occurs by overturning. Out-of-plumbing of the wall due to differential settlements is critical to reduce the capacity against overturning. This effect must be taken into account when ckecking when the lateral stability of the wall, not only by considering the moments produced by the vertical load acting over the deflected shape of the wall, but also by accounting for the non-symmetric vibration of the wall in this circunstance. An increase in the seismic lateral force by the factor (1 + 10 f), where f is the slope of the out-of-plumb, has been derived for that purpose.

A more comprehensive study is now being undertaken aiming at broadening the experimental information on the mechanical properties and at proposing more precise methods for the checking of the structural safety. Some particularly important monuments are being checked in detail for their structural safety and for the proposal of strengthening schemes.

CRITERES DES CHOIX POUR DES MESURES D'URGENCE

DIAZ-BERRIO, F. Salvador

SUMMARY

Criteria for selection of emergency measures have to be framed into the general theory and methodology for conservation and restoration of the cultural property.

Considering the difference between emergency situations and emergency measures, and the need of time to establish criteria for the interventions, we have to use pratical methods in order to increase the knwoledge of our stone masonry historic structures, before the emergency situations.

The study of architectural and structural typology of historic buildings in their urban context, useful for other conservation purposes, is also an important and practical instrument in the emergency situations.

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DOCTOR ARQUITECTO

April 7, 1989

INTRODUCTION

En accord avec le programme de cette réunion, notre intention comme conservateurs du patrimoine culturel, est celle d'encadrer les critères de choix, pour les mesures et interventions d'urgence, dans la théorie et la méthodologie génerales de la conservation et la restauration des bâtiments et ensembles historiques.

Nous ferons avant tout deux précisions. D'une part pour parler de critères, choix et méthodes pratiques, il faut avoir d'abord la possibilité d'intervenir. D'autre part, il convient d'éviter la possible confusion entre les mesures et méthodes d'intervention et les situations d'urgence.

Quand une situation d'urgence se manifieste elle est habituellement confiée aux techniciens du génie civil, des travaux publiques, les pompiers, la police et les militaires. Les conservateurs du patrimoine sont souvent exclus, ne peuvent pas ou n'ont pas l'habitude d'intervenir assez vite.

En plus quand les conservateurs peuvent intervenir, ils se trouvent souvent face à un "univers" inconnu ou peu connu, sans disposer des éléments d'information utiles ou suffisants, pour orienter assez vite les critères de choix pour des interventions pratiques, mais appuyées sur les principes généraux de la conservation et de la restauration.

C'est donc question d'établir surtout des bases et méthodes d'ordre pratique, addressées ici particulièrement aux maçonneries en pierre, mais tenant compte nécessairement des élements et phénomènes qui interviennent dans les situations d'urgence.

I. LES SITUATIONS D'URGENCE

Le facteur du temps, qui joue un rôle important dans tout travail de construction ou de réparation, devient pratiquement le facteur décisif dans les situations d'urgence. La vitesse dans l'éxécution du travail semble en principe un élement opposé aux besoins des travaux de conservation et de restauration. La situation d'urgence exige des actions rapides; face aux volumes et besoins de travail imposés par la situation, ce qui manque surtout c'est le temps. (Foramitti - ICCROM, 1970)

Si les heures et les jours passent sans que les critères de conservation soient présents dès les premiers moments, il sera tard pour orienter les actions ensuite. Si le temps échappe vite pendant la situation d'urgence et il est impossible de le récuperer après; la logique nous oblige donc a chercher ce temps avant la situation d'urgence.

Les mesures d'urgence doivent etre établies avant la situation d'urgence, quand on dispose de temps pour définir les criteres et analyser les choix pour etre appliqués au moment nécessaire.

Nous sommes habitués à accepter avec naturalité des critères définis pour une situation d'urgence dans des cas, des espaces et des temps précis. Dans les avions, les navires et aussi dans les trains, de surface ou souterrains (métro) nous connaissons dès l'entrée ces critères et instructions. Ceci arrive même dans certains bâtiments spéciaux ou publiques; musées, biblioteques, archives, laboratoires et lieux de spectacles ou de grandes concentrations: théâtres, cinémas, stades, etc...

En principe, ces normes et critères sont faciles à apprécier dans les usines, grands immeubles de bureaux, écoles et aussi dans certains temples. Par contre, nous ne sommes pas habitués a penser ainsi, dans les espaces ou nous passons la plus grande partie de notre temps: la ville et les logements.

Les musées, les temples et certains bâtiments publiques sont inclus dans la catégorie des immeubles on monuments historiques, mais l'expérience montre que dans les villes historiques, ces bâtiments ne représentent que moins du 10% de l'ensemble des immeubles on monuments historiques; le reste du patrimoine culturel immeuble de nos ensembles urbains est constitué par des bâtiments civils, utilisés en grande partie, de façon exclusive ou mixte pour des logements. (Diaz-Berrio - INAH, 1973)

En plus du manque de temps, d'autres élements manquent aussi, ou ne sont pas disponibles habituellement, dans les situations d'urgence; la main d'oeuvre qualifiée, les spécialistes, les ressources économiques et une série de matériaux et équipements d'appui ou de secours.

C'est évident qu'un ordre de priorités doit etre défini avant aussi que la situation d'urgence se produise. Trois de ces élements doivent etre disponibles immédiatement a) les équipements de secours et d'appui. b) un nombre réduit de spécialistes. c) un mécanisme de participation avec les autorités civiles et militaires. (Foramitti, op. cit.)

Une main d'euvre qualifiée, un plus grand nombre de spécialistes, les ressources économiques, matérielles et humaines ainsi qu'une variété d'éléments pour les relevés des dégâts, consolidations, réparations et l'estimation plus détaillée des interventions, peut se faire après avoir connu et controlé initialement l'ensemble de la situation et ses points critiques. (Pichard - UNESCO, 1984)

C'est justement l'ensemble des constructions historiques et les points critiques qui doivent être le motif de préocupation pour le choix des critères d'interventions urgentes et des interventions qui peuvent attendre, suivant un ordre de priorités. Nous avons souvent sontenu que mieux vaut une bonne consolidation provisoire qu'une intervention précipitée. (Diaz-Berrio - INAH, 1987)

La belle couverture du livre "Between two earthquakes", avec un titre très bien choisi, illustre justement un des cas qui ne représente pas un problème d'urgence, ni de critère de choix. Il y a naturellement un danger sous un arc qui peut s'écrouler, mais c'est une typique question d'intervention soigneuse de consolidation, dans un élément faisant partie d'un ruine non habitée, cas fréquent à Antigua, Guatemala. (Fielden - ICCROM-GCI, 1987)

II. LES CRITERES DE CHOIX

Indépendemment de la valeur culturelle qui leur est attribuée, les "grands monuments" ne sont pas habituellement habités; en cas de séisme par exemple le risque de pertes humaines dans les temples, musées, bureaux, etc... se réduit à 8, 10 ou 12 des 24 heures du jour; le risque dans les logements existe les 24 heures du jour. Maintenant que les critères de risque et de vulnérabilité sont déjà acquis, le besoin d'un changement d'attitude s'impose. (Pichard, op. cit.)

En tenant donc compte que le patrimoine culturel construit est constitué en très grande mejorité par l'architecture historique appelée civile et qui est, en plus, habitée les 24 heures du jour, le choix pour les mesures d'urgence doit s'orienter surtour vers cette grande partie du patrimoine qui est habituellement moins protegée que les grands monuments, reconnus et consacrés.

Les pas habituels, les procédures et méthodes de travail-ou autrement dit, les mesures -face a une situation d'urgence peuvent et doivent etre essentiellement les memes que dans les situations dites "normales".

En premier lieu, toute intervention réalisée sur un bien culturel est qualifiée par l'élément objet de cette intervention utilisant les termes de Cesare Brandi; autrement dit, par la reconnaissance de la valeur culturelle du bien, et non pas par la circonstance qui, dans ce cas, serait la situation d'urgence. (Brandi - ICCROM, 1967)

D'autre part, laissant de côté les interventions régulieres de conservation ou d'entretien et le cas de certaines ruines, il est difficile d'affirmer que les interventions de restauration se réalisent dans des situations "normales"; les travaux ne sont souvent entrepris que quand il y a un danger ou une situation, au moins, relativement urgente.

Le séisme de 1985 à Mexico a montré que les bâtiments historiques en mauvais état, étaient deja en cet état bien avant ce séisme, dans pratiquement tous les cas.

En considérant la "seconde histoire" de ces bâtiments, reprenant les termes de Cesare Brandi, c'était surprenant de trouver encore des étaiements "récents, de 1968" et de faire, dans plusieurs, secteurs, une datation des consolidations "provisoires" ou définitives depuis le début du siecle, déja devenues pratiquement "historiques". (Brandi, op. cit.)

D'autre part le rapport entre l'utilisation des immeubles, leur état de conservation et leur comportement pendant le séisme démontrait la validité d'un nombre d'opinions et d'hypothèses;

Au niveau des bâtiments historiques en pierre ou briques;

1º) Dommages insignifiants dans les cas ou la structure architecturale ou la typologie historique genérale était conservée et dommages qui augmentaient en rapport direct aux modifications réalisées.

2º) Dommages insignifiants ou tres légers dans les immeubles anciens utilisés seulement, ou en grande partie, comme logements.

3º) Dommages plus significatifs dans les cas fréquents ou les rez-de-chaussée avaient subi des modifications pour etre utilisés comme commerces; suppession de murs d'appui remplacés par des poutres et appuis en béton ou fer, introduction de dalles ou planchers en béton, ouvertures excessives en façade pour les devantures, etc.

4º) Dommages plus significatifs dans les cas ou l'utilisation de l'immeuble entraînait la concentration de poids, particulièrement des entrepôts ou machines en rez-de chaussée, ou surcharges sur la couverture.

Au niveau de l'ensemble de constructions dans la zone historique;

1) Dommages insignifiants dans des ilôts homogènes en hauteurs, volumes et bâtis sur plan régulier.

2) Fragilité des batiments d'angle ou des coins des îlots, particu-

lierement dans les cas d'angles aigus, des îlots sur plan triangulaire ou en trapèze.

3) Dommages significatifs par différence de hauteurs, volumes et poids, entre constructions voisines et dans les bâtiments voisins aux terrains vagues dans le tissu urbain.

4) Dommages plus significatifs le long des rues contenant en sous-sol des installations modernes importants d'égoûts, etc..

5) Dommages importants dans les constructions modernes elevées le long de percées récentes (derniers 50 ans) dans le tissu urbain historique.

La matérialisation de l'ensemble de ces problèmes, observés dans d'autres cas et dans les 9 Km^2 de la zone historique centrale de la ville de Mexico, se retrouvaient et pouvaient etre exemplifiés dans de nombreux secteurs réduits de cette zone, appres le séisme de 1985.

III. ELEMENTS DE BASE POUR DE METHODES PRATIQUES D'INTERVENTION

L'etude des dommages individuels mène au rapport avec l'ensemble des bâtiments historiques et oblige donc a considerer les criteres pour les interventions en deux niveaux: a) celui des immeubles (niveau architectural et b) celui de la structure des ensembles (niveau urbain). (Diaz-Berrio, González Pozo - INAH, 1985)

De cette façon le besoin d'un instrument pratique de référence pour orienter le critères face a une situation d'urgence nous conduit assez vite a une étude typologique des structures historiques architecturales et urbaines. Justement cette même étude typologique est nécessaire pour accomplir une lecture objetive des ensembles et des immeubles historiques, tenant compte de leurs structures métriques, constructives, fonctionnelles et économiques plutôt que l'habituelle considération de leurs "valeurs" historiques et esthétiques. (ICCROM-HABITAT, INAH-FONHAPO - INAH, 1986)

La connaissance de ces structures permet non seulement d'appuyer les méthodes de datation, la définition de réglements de conservation et de construction, les critères et paramètres pour les éléments d'intégration, mais devient très utile aussi pour les critères et choix dans des situations d'urgence. (Cesari et al. - RFL, 1976)

Nous retournons ainsi à la considération du temps disponible, avant ou pendant la situation d'urgence; il est évident que les "mesures d'urgence", adoptées sans une connaissance préalable des structures qui forment la typologie des immeubles et ensembles historiques, mènera très problablement a des fauses solutions, constructives, fonctionnelles, etc... et prises souvent de façon générale ou massive pour un ensemble d'élements inégaux ou differents, même s'ils sont tous "anciens"...

Il est vrai que la priorité imposée par le sauvetage des personnes mène à certaines démolitions et mouvements des restes des constructions, mais très souvent un pourcentage important de démolitions ou mouvements, motivé par la bonne volonté de la population, des autorités ou des équipes de secours, est innécessaire.

Il est facile de comprendre que le choix ne seront pas les mêmes pour un îlot ou un batiment qui, en géneral conserve bien; a) ses éléments constructifs (maçonneries en pierre ou briques, poutres, toitures, etc... b) la disposition générale de ses circulations et ouvertures et de ses espaces ouverts et couverts, c) son utilisation originale comme logement ou légèrement modifiée. d) une composition socioéconomique équilibrée de ses habitants.

Par contre un îlot ou un bâtiment qui est en mauvais état de conservation avec; a) ses elements constructifs modifiés b) des ouvertures, circulations et la disposition générale des ses espaces altérée c) une utilisation nouvelle peu compatible avec son usage original (commerces au lieu de logements, par exemple) d) une population excessive ou par contre, l'abandon des habitants..

Entre ces deux cas extremes une série de cas intermédiaires et de variétes peut bien se présenter pour nuancer et orienter les critères, les temps, les frais et les priorités d'intervention. (ICCROM - UNESCO, 1984)

Posons par exemple le problème seulement dans le cas des critères constructifs de réhabilitation, consolidation ou adaptation, plutôt que parler de "reprojetation" quand il s'agit de bâtiments historiques, car ils ne preuvent pas etre "projetés" de nouveau. Si nous reprenons la discussion traditionnelle sur l'introduction d'éléments verticaux et horizontaux en béton ou acier, il semble logiquement impossible de dire oui ou non, de façon générale et définitive pour une variété de constructions en pierre ou en briques et en différents états de conservation qui ont seulement en commun leur caractére d'anciens.

Tenant compte du sous-sol, en plus de la flexibilité des bâtiments, certaines solutions basées sur l'introduction d'éléments structurels modernes, peuvent être convenables dans certains cas, mais dangeureuses dans d'autres cas.

Nous pouvous conclure que ce n'est pas possible d'établir des critères ni des choix à priori et de façon générale pour des situations d'urgence sans disposer préalablement d'une connaissanee typologique des élements sujets à cette situation. Autrement dit, les critères des activités de conservation doivent s'addresser aux causes des altérations plutôt qu'aux effets des déteriorations. (Philippot - INAH, 1973)

Ce serait trés utile et possible d'établir des critères d'intervention d'ordre géneral et même individuel pour des ensembles d'immeubles partant de cette connaissance, pour orienter une variété d'interventions de consolidation, intégration, recupération, réhabilitation et adaptation dans l'activité generale de la conservation et de façon particuliére pour les différents types de maçonnerie, et dans le cas limite des situations d'urgence.C.Cesari op.cit)

Pour illustrer de façon synthétique une méthode pratique d'intervention, un de ces secteurs de la ville de Mexico comprenant 8 îlots, avec 137 bâtiments, 92 desquels classés comme monuments historiques, peut servir d'exemple.

Dans cet ensemble organisé autour de l'ancien couvent de Santo Domingo (Saint Dominique) l'espace ouvert de la place avec 5300 m^2, a déja joué et peut toujours jouer un rôle important pour concentrer provisoirement un pourcentaje important de la population du quartier.

Considérant d'abord les bâtiments publiques, en plus de deux églises, un îlot est presque completement occupé par le Ministère de l'Education Publique, avec un espace de 1245 m^2 en 4 cours. Le bâtiment de l'ancien Tribunal de l'Inquisition,utilisé maintenant par l'Université a aussi 2 cours avec 425 m^2 et le secteur compte en plus avec 2 écoles publiques et quatre terrains vides. Ceci permet de disposer d'un ensemble de 11.570 m^2 d'espace ouvert sans compter les rues elles-mêmes.

En tenant compte du nombre de bâtiments historiques en bon état et des nouvelles édifications bâties après 1985 suivant le plus récent réglement, il est possible de connaître maintenant les élements confiables et ceux qui presentent déjà et peuvent presenter naturellement des problèmes en cas d'urgence.

Maintenant, au lieu de nous trouver face à un "univers" inconnu (137 immeubles) nous savons combien d'immeubles anciens et modernes representent encore un risque réel, quels sont ceux qui peuvent éventuellement présenter des problémes, et quel genre de problème dans leur maçonnerie et leur structure (sept, dix et 21 respectivemente).

Ceci perment donc d'orienter non seulement les interventions dans une situation d'urgence mais celles qui, maintenant dans la situation "normale", sont déja devenues "urgentes". (sept des 38 cas qui ont besoin de différentes réparations, alors que 54 immeubles historiques de l'ensemble de 92 sont en bon état).

En plus de l'utilité de la photogrammétrie terrestre il semble nécessaire d'insister finalement, comme recommendation générale, simple et pourtant presque toujours oubliée, sur l'importance de placer, sur nos maçonneries en pierre, le symbole de la Convention de la Haye, pour indentifier de façon rapide et facile ces biens cultureles que nous voulons sauvegarder avant que ce soit trop tard. (ICCROM, ICOMOS-PNUD, UNESCO, 1983)

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THE CIVIL ENGINEER IN MONUMENT PRESERVATION
CONTINUITY AND CHANGE OF TASKS*

F. Wenzel

SUMMARY

The paper deals with the general approach of the civil engineer to old buildings. The necessity of a methodical proceeding is emphasized. The sequence of four steps similar to the work of a medical physician are presented: anamnesis, diagnosis, therapy and prognosis. Aspects of practical solutions are discussed an criticized in order to find the best solution, which must respect the qualities of the historical building.

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Introduction

Form, function, structure and costs are the criteria generally mentioned in connection with the planning, construction and maintenance of a building. Regarding the construction of new buildings the role of the civil engineer within this range of criteria is basically known. In repair and restoration of old buildings the civil engineer has the additional task to study and assess the historical substance before repair and restoration measures can be considered. This means, for the general renewal of old buildings that the engineer must examine and evaluate the technical suitability of the old structure for its new function. When dealing with historical monuments, however, the monumental value gains importance, especially the historical significance it has. Most civil engineers have not had experience in this field, neither during education nor in practice and are therefore not familiar with it. There were always only a few engaged in monument preservation and even fewer took up research and teaching. In the meantime the range of activity for civil engineers within monument preservation has expanded. Many more buildings have been added to the protection list in recent years. The engineering problems that arise during repair and strengthening of historical buildings and those encountered in general building renewal are similar in many ways. There is no definite boundary between them. In addition to damage caused by aging the atmospheric pollution is beginning to play a major role. Many cases that previously would have been dealt with by craftsmen now require the advice and help of an engineer.

Problems and tasks today

The amount of original building substance in historical monuments is decreasing; this is especially obvious in the oldest among them. It is not possible to reproduce old building substance, therefore interest in preserving it to the greatest extent is growing rapidly. This necessitates methodical proceeding. It can be described at best by the sequence of steps familiar from the work of a medical physician: anamnesis, diagnosis, therapy and prognosis. In practice, these steps are interwoven approximately in the manner shown and explained in Figure 1. This methodical procedure is intended to help find remedies that minimize intervention to the building substance as well as modern technical additions without reducing the efficiency of the engineering aid. Each old building is a case of its own. The type and extent of repair measures must be reconsidered in each case. The civil engineer cannot avoid analysing the characteristics of each case individually. The Special Research Programme "Preservation of historically important buildings" at the University of Karlsruhe will compile recommendations for practice from the results of its work and will publish case examples, though not in the sense of a code. Codes for the repair of old buildings would carry the danger of having more attention paid to the fulfillment of these codes than to the specific conditions of the individual building monmument.

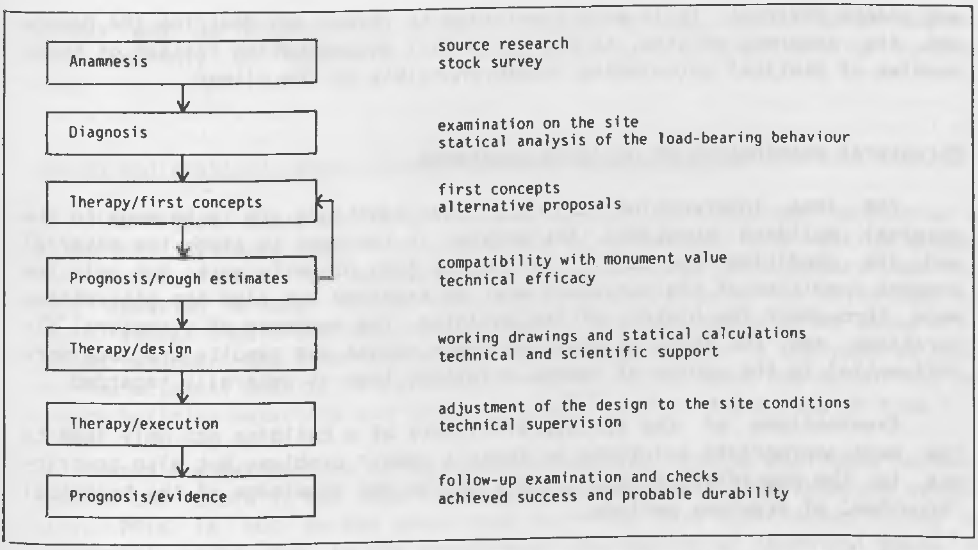

Figure 1

In order to have more preserved than just the façade of the building as is the case when the building's entire core is removed, to preserve as much of the interior structure as possible and to ensure that the new function corresponds with the old structure, it is necessary to consult the civil engineer from the very beginning of the design process. Design and structure, the latter being regarded in the meaning of a conceptional order (and not only as an assembly of bearing elements), are identical in many ways. The concept of design and structure which the building or an alteration followed is of no slighter interest than the material used to realize it. Actually the civil engineer is the person who should know the answers to the questions about structures best and who should contribute this knowledge to the discussion over the future of building monuments. Being one of the descendants of the "master-builder" the civil engineer should regain competence on old building structures in their entirety to be able to participate in monument preservation. This call to grow beyond the limited role of a statical calculator should be understood as an opportunity to broaden the professional scope within the entire field of construction - not only within monument preservation - and thereby to make the profession more interesting and attractive to students.

An old but constantly reoccurring task of the civil engineer is to convince all workers in this field that the structure must always be studied and repaired first before it is painted and polished. This requires a simple, understandable language. High-tuned technical terminology can confuse and create distrust. It is more convincing to reveal and describe the damage and its progress on site, to present a full documentation instead of thick bundles of statical calculating incomprehensible to the client.

Structural examination of building substance

The less intervention and the fewer additions are to be made to the original building substance, the greater is the need to study the material and its condition thoroughly. This means lots of petty work. Not only the present condition of the structure must be examined but also the alterations made throughout the history of the building. The sequence of structural alterations and the facts we learn of their causes and results are much more influential to the choice of remedy solutions than is generally regarded.

Examinations of the structural history of a building not only lead to the most appropriate solutions to today's repair problems but also contribute to the specific building history and to our knowledge of the technical "know-how" of previous periods.

It is surprising how often we find very "modern" structural concepts throughout the past centuries that could not be realized because the necessary means did not exist.

Surveys made available to the civil engineer dealing with a historical monument can be very helpful. They cannot, though, relieve him from his own thorough examinations of the structure and from his own documentation of the material, its condition and structural details. Surveys and basic drawings supplied for example by surveyors may provide general information and measurements. But the civil engineer who intends to deal with the old buildings has special information needs and approaches the old substance from his specific point of view. Therefore he is necessarily interested in conditions and details of no concern and not visible to others.

At this point a word must be said to the deformation-true survey. This is an important tool for the scientific examination of certain important building monuments - that is for building research purposes. This method is expensive and therefore it need not be applied to each conservation project. It is certainly important for a civil engineer to know the deformation patterns of the old structure. This can guide us to the causes of building damages and can indicate which causes have priority in being cured. Often, though, only a few measurements are sufficient to establish an accurate picture of the entire deformed configuration. Very soon an engineer learns to "interprete" these pictures (although they are rather wide-meshed compared with the detailed deformation-true survey), and he can perceive what has happened to the building substance and how aid can be applied. Too many measurements often obstruct the view of the substantial information. This is comparable with thé unreflected use to the computer when the few important values end up with all the unimportant ones in huge cemeteries of numbers and can only be recovered with great efforts to be available for further use.

Design and statical calculation of strengthening measures

The first question to be answered by the civil engineer is whether a historical building must be necessarily strengthened. Intervention always involves some loss of authenticity. If such a step is necessary then the next question arises: which should be the life-time of the buildings? In short terms: should the repair last for 100 or 1000 years? It is being discussed whether the life-time of the repaired structure is the same as that of the original one. It is difficult, though, to estimate the durability of modern building materials and techniques over such a long period of time.

With the perspective of having even less destructive available techniques in the future it may seem reasonable now to do only the absolute necessary. This is not so bad since many buildings have experienced losses in monumental value due to the exaggerated application of technical means. A thoughtless and presumptuous statement still heard today is that the following generations are to be relieved of the burden of the historical substance once and for all. This goal cannot be reached; fortunately, science and technology are not able to achieve it. The care of every generation over its historical monuments is an important link to its history and a basic stimulus to monument conservation in the broadest sense. We should not eliminate this burden entirely. It is our task, though, to ensure the stability and serviceability of the historical buildings and not charge the next generation with an overwhelming burden.

At this point the reversibility of the engineering consolidation measures should be mentioned. The wish for reversibility is often misunderstood as to mean that only those remedies are acceptable which can be removed and replaced by better ones some day. This cannot be the point. Should it be necessary to give technical assistance to a monument then it is reasonable to look for the most appropriate solution, for the minimal intervention and addition. This necessary minimum, however, is to be inserted as a durable addition of our time. Reversibility means rather the ability to repair the inserted elements as well as to replace them in case of failure or deterioration but it does not mean that they should be exchangeable every time a new technical solution is found.

It is still advisable not to intervene to the existing flow of forces even if this pattern is not the original one, but has been developed later on. It may be improved by sealing cracks, by decreasing the eccentricity of forces, by installing anchors or bracing elements. But substantial shifts in load transfer should remain exceptions and should be avoided whenever possible. Why give up consolidation of the building fabric under the present flow of forces and risk the revival of this process elsewhere in the building with new deformations and cracks?

To gain realistic results when proving stability, the building has to be studied thoroughly. It is important to perform reliable investigations of the most critical high-stressed areas and their condition. This can be achieved by making a survey of its condition and damage, e. g. by determining the cracking pattern or by recording signs of overloading such as spalling, splitting, settlements. In addition, the material properties should be determined, in situ, or if possible by taking samples or both. If the crucial areas of the present condition are given the safety factor of 1.0 and if the influence of the consolidation measures - as the result of a reduction of the load or an improvement of its strength - is estimated, a relative increase in safety can be gained. Reduction of load up to 30 % or increase of load-bearing capacity up to 50 % means, e.g., improving the safety factor of the crucial areas to 1.5 compared with the condition before, which was safe even though with only a slight margin. If material properties are not known, if the quality of them varies, if the extraction of explicate samples is a problem then proving such a relative increase in stability is the only reliable way to repair a monument appropriately instead of restoring it "to death".

Repair techniques

When encountering with the objections of the conservationists attention should be drawn to the good experience of past repair operations. Improvement of techniques can also profit from this experience. The objections of the conservationists usually are: masonry can be stiffened through injection and loose its elasticity, corrosion of the added reinforcing bars and prestressed anchors can cause damages. However global these concerns might have been expressed they have provoked some positive reactions among engineers. It is highly welcome that more attention should be paid to structures, to their interior and invisible parts, in the sense of a more comprehensive understanding of monuments than in the previous years. The wish to limit structural interventions and additions has lead to a new way of confrontation with the old structures. Certainly, engineers, who have been strongly engaged in the repair of important historical buildings for a long time, did no more in the past than seemed necessary from their insight and experience. But it is becoming more generally accepted that too much money for repair measures can do more harm to the historicl identity of building monuments than too little money. Unfortunately, there are enough recent examples to illustrate this.

Not every damaged masonrywork should be grouted. This is only necessary when the load-bearing capacity should be improved or when bars and anchors within the masonry should be protected against corrosion. Should grouting be necessary, it is generally and technically not justified to blame it for being harmful to masonry. It has been known for a long time that special care must be taken when injecting masonry containing gypsum. This is to avoid the formation of expansive minerals such as ettringite and thaumasite and to prevent damage that may lead to the irrepairable destruction of the masonry structure.

In recent years numerous systematic engineering investigations of formerly repaired buildings have been conducted within the Special Research Programme "Preservation of historically important buildings". The objective is to determine the strong and weak points of contemporary repair techniques. The results can lead to scientifically-supported proposals for an improvement and development of such methods. Various examination results begin to show that successfull consolidation can generally be achieved using today's experience and the capabilities offered by the grouting and reinforcement techniques for masonry if applied correctly. This means: selection of appropriate grout, choice of sufficient drill hole diameters, use of distance pieces for the reinforcement. At any rate, a preliminary thorough examination of the substance is necessary as well as the frequent presence of an experienced engineer at the construction site. As our investigations also show, carelessness in planning and execution is responsible for most of the defects in repair works - a fact we also know from the new buildings errected in the past decades.

Perspectives

Continuity and change of the tasks of civil engineers in monument preservation not only concern practice and research - as I would like to point out finally - not only methods, procedures and techniques but they also concern the education of engineers, their training towards the handling of historically important buildings. For this purpose knowledge has to be taught and acquired about building history and structural history. Design training is equally necessary including overlapping areas of study with the architects and offering joint design projects at university. Such projects should deal with new buildings as well as with the restoration sector. Specializing in the old is equally wrong at university as it is during practice. The focus on a single field of work to a certain extent can surely be useful to a civil engineer later, but it should not be centered on building restoration problems entirely. The importance of a historic building as a building monument is a value established within each period of time and is not permanent. The work of enginners with building monuments is also exposed to change of opinions and technical opportunities. Whoever works with old and new buildings will find solutions for one side from within his knowledge of the other side and vice versa.

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ON THE STATE OF STRUCTURAL REPAIR OF MASONRY

FRITZ WENZEL *

SUMMARY

The report deals with the techniques of mortar injection, steel reinforcement and prestressing being used since the 1920s to strengthen old masonry. Due to follow-up examinations of buildings repaired in such manner and with recent research results achieved at the University of Karlsruhe, rules for dimensioning and execution can be made available to practice for the structural repair of old masonry.

No universal standards can or should be established for historical buildings, but rules and recommendations can be given for application in practice which can be adapted to the special circumstances of each object.

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1. INTRODUCTION

Grout injection and stitching as well as prestressing of old masonry has been practised for a long time. This technique of restauration helps to save the monumental value of historically important buildings more than taking down and rebuilding them and, as a rule, is distinctively less costly. The Special Research Programme "Preservation of historically important buildings" at the University of Karlsruhe has collected quite a few results regarding the effectiveness as well as the durability of such securing measures (Wenzel, Maus [1], Ullrich [2,3]). These results were achieved on buildings whose restoration and securing can be traced back to the twenties. The gained experience shall be summarized and presented in its practical meaning. The corresponding rules of dimensioning and execution which were developed at the University of Karlsruhe over the last few years (Dahmann [4], Haller [5]) are to be found in the 1987 yearbook of the Special Research Programme. There they are illustrated by sketches and examples of use. The application of these rules in practice presupposes that they are coordinated with the particular conditions of the respective old building.

2. GROUT INJECTION

2.1 The purpose of grouting

Old masonry is grouted to increase its supporting capacity, to close cracks and cavities, to strengthen loose masonry and mortar, to replace missing mortar, to introduce new, larger forces into the masonry at local points, to involve the inner filling of multi leaf walls and pillars in the supporting structure, to link reinforcement bars and prestressed anchor ties to the masonry and protect them against corrosion (Pieper, Hempel [8], Hempel [9]). Where these or similar problems don't occur, grout injection does not need to be performed. (Fig. 1)

2.2 Injection material

All types of cement customary in the trade also with additives of trass are suitable as injection materials. Clay and expanding cement are not suitable. Very important are cements with a high sulfate resistance (HS-cement), which normally help to prevent damage of expansion in old mortar containing gypsum (Dahmann [4], Pieper [7], Pieper, Hempel [8]). The disadvantage of these cements is their dark colour; if they leave the masonry they can easily cause stains on the surface which is why particular caution is advisable. Although super-hydraulic limes may be injected too these do not reach high enough strength in the masonry and tend to expand because of their C_3A-content. For cement injection water-cement factors from 0,8 to 1,0 and pressures up to 6 bar are used.

Synthetic resin as material for injections is expensive, there are unanswered questions concerning the compatibility and aging characteristics, furthermore synthetic resin is an alien material to masonry.

2.3 Experiences from the subsequent examinations

In most of the examined cases the objective of grout injection was achieved. Where this was not the case, deficiencies in planning, execution and supervision were important factors.

The success of grouting injection depends, among other things, on a sufficiently high water-cement factor of the material that is injected, otherwise it will "die of thirst". For this reason pre-moisturizing of the drill-holes is important and should be executed together with cleaning the holes from drill-dust. In addition the pressure must be maintained long enough, otherwise the material will not penetrate the old mortar or the cement paste will not find its way to cavities that are located further back.

2.4 Special Problems

2.4.1 Expanding caused by formation of minerals

There is great danger that mortar containing gypsum is present in old masonry. Even if several specimens of mortar are taken and examined and no gypsum is found, one can't be sure that this result is relevant for the whole building including original parts, repaired parts and added parts. The usage of HS-cement as injection material seems to be obvious. As a rule it helps to avoid expansion in consequence of the formation of ettringite and thaumasit. This applies especially if the masonry is dry or can be dried out. Short moisturizing, e.g. heavy rain, is harmless if the masonry can dry out afterwards. According to todays knowledge (Hempel [9], Pieper, Hempel [8]) even if HS-cement is used expansion can not be excluded with absolute certainty.

Besides cement with a high sulfate resistance as an injection material since a few years, there are also special mortars for use in masonry containing gypsum (Ludwig [10]) which are obviously sulfate resistant but may only be applied over 5° C and have to be treated with moisture subsequently, a task difficult with old masonry. As the strength of these mortars is less than that of ordinary cement mortar and they do not provide sufficient corrosion protection, so that it is necessary to use anchors of rustproof steel, there are also losses in the static-constructive efficiency.

2.4.2 Efflorescences

The moisture that gets into the masonry during the injection may contact salts that are included or sedimented in the stonework and then crystallize by drying. This can lead to efflorescences, destruction of paint layers and plaster surfaces and also to erosion of sandstone (Hilsdorf [11], Althaus [12]). It is therefore advisable to take specimens not only to determine the share of gypsum in old masonry but also to carry out mineralogical and material technological examinations with regard to possible efflorescences. Then at least one knows in advance what to expect after injecting and it is possible to think the concept of rehabilitation over if necessary. To what extent remedy is possible e.g. by harmonization of the injection material with the conditions on site has to be decided according to the special case.

3. STITCHING

3.1 The purpose of stitching

3.1.1 In general

Stitching as subsequent reinforcement happens where tension or thrust occurs which the masonry cannot withstand. Examples can be found with Dahmann [4] and Pieper [7]. Stitching is always connected with grout injection to form the bond between steel and masonry as well as to provide corrosion protection.

3.1.2 In multi leaf masonry

Here the reinforcement bars connect the two outer leaves through the inner filling which was strengthened by injection. As the outer leaves are only one stone thick mostly, special attention must be paid to the anchorage of the bars. (Fig. 2)

3.2 Reinforcement bars

As a rule bars made of ribbed reinforcement steel 420/500 with a diameter of 8 to 20 mm, mainly 12 to 16 mm, with anchorage by bond are used. Also steel with through rolled thread ribs, so called Gewi-steel, has prooved itself. With long anchor bars a sleeve joint may be developed, with short anchor lengths an additional end anchorage with washer and nut or with a special end piece. When the danger of corrosion is regarded as extreme, rustproof steel is sometimes used, for example for strongly moisturized structural elements. Steel with smooth non profiled surface should not be used as the grip is weak.

3.3 Experiences from subsequent examinations

With 20 mm coverage of the reinforcement bar by cement in the drill-hole, corrosion protection is guaranteed. To ensure that the material that is injected covers the anchor uniformly, spacers must be attached to the bar. Sufficient protection of 25 to 30 mm coverage at the ends of the anchor must also be ensured. With shorter cement plugs there exists danger of corrosion, longer ones mostly cause too much loss in bond length in the outer leaves of multi leaf masonry.

3.4 Rules for dimensioning and executing stitching of multi leaf masonry

This refers to the 1987 yearbook of the Special Research Programme. It discusses the following: Appraisal of allowable pressure of the existing, unimproved masonry; Load assumptions for the reinforcement bars; Choosing of the anchor grid; Corrosion protection and bond.

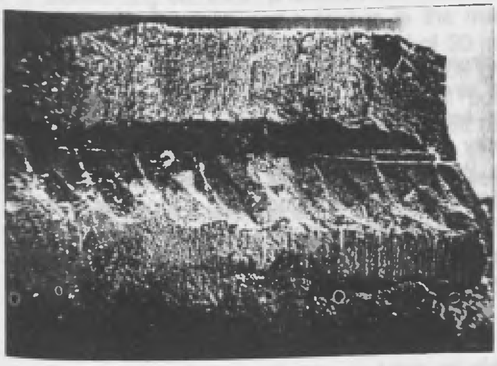

Fig. 1 Grout injection after surrounding and protecting an anchor bar against corrosion

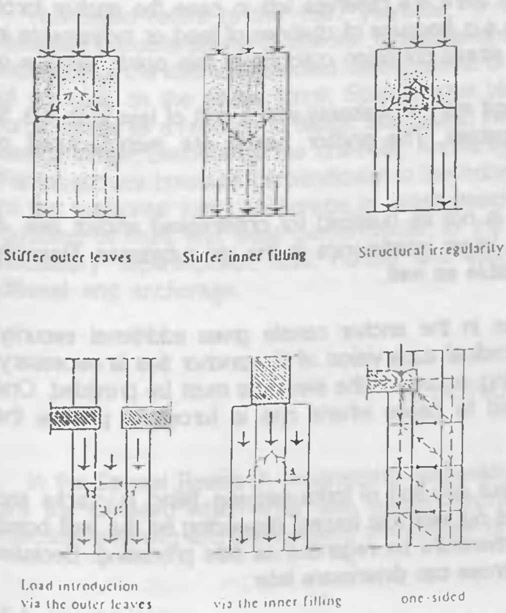

Fig. 2 Stitching in multi leaf masonry adjusting uneven force flow

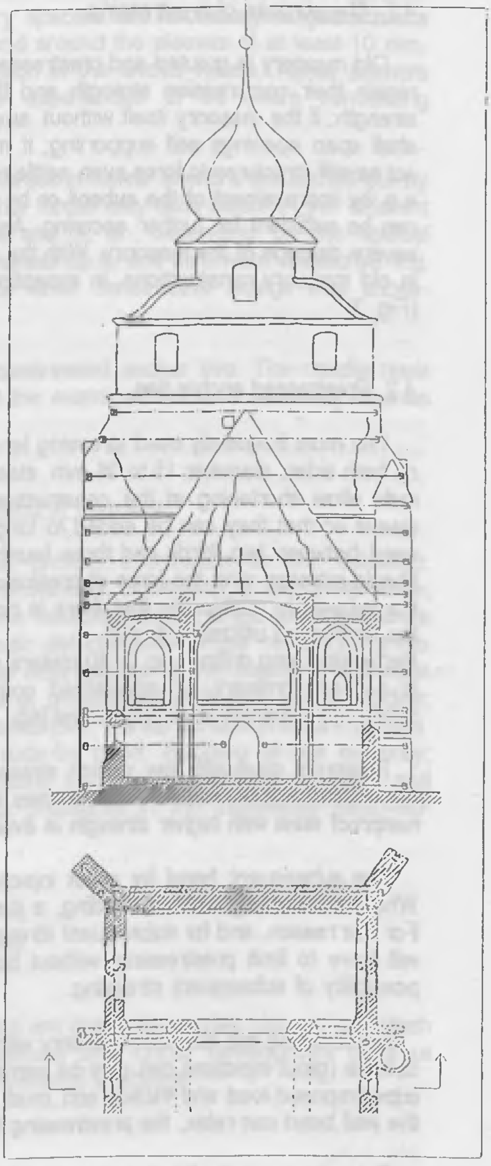

Fig. 3 Prestressing at the Westwerk of the collegiate church in Herrenberg

4. PRESTRESSING

4.1 The purpose of prestressing

Old masonry is grouted and prestressed if strongly torn walls and pillars must be joined to regain their compression strength and thrust strength and in addition to withstand tensile strength; if the masonry itself without auxiliary constructions of steel or reinforced concrete shall span openings self supporting; if masonry buildings because of irregular subsoil shall act as stiff structures to force even settlements. When the causes for the cracks are removed e.g. by improvement of the subsoil or by reinforcement of the foundation, a loose armouring can be sufficient for further securing. As a rule prestressing is only applied in the case of severe damage of the masonry. With the help of prestressing the force flow may be corrected in old masonry constructions, in exceptional cases it may even be changed in its direction. (Fig. 3)

4.2 Prestressed anchor ties

The most frequently used stressing tendons are steel rods with through rolled thread ribs on both sides, diameter 15 to 36 mm, steel quality about 850/1050 to 1100/1350. Such steel rods allow shortening at the construction site with a separator and joining with a thread sleeve so that they can be added to long pretensioning anchors. If the design stress is only used between two thirds and three fourths there are reserves left in case the anchor force should increase over the force of prestress e.g. because of changes of load or movements in the subsoil. In addition to that there is no stress corrosion cracking at this point because of the decreased utilization factor.
Performing long drillings up to 30 meters and more in masonry with a drift of less than 0,3 % is not extraordinary for specialized companies. The anchor heads are manufactured of reinforced concrete or steel (Wenzel [6]).

Rustproof steel with low design stress is not as qualified for prestressed anchor ties. A larger diameter is needed which causes a larger interference in the old substance. Recently rustproof steel with higher strength is available as well.

The subsequent bond by grout injection in the anchor canals gives additional security. When prestressing without bonding, a periodical supervision of the anchor ties is necessary. For this reason, and for subsequent stressing, access to the elements must be provided. One will have to limit prestressing without bond to cases where one is forced to provide the possibility of subsequent stressing.

Prestressing the old, torn masonry without any sort of force carrying filling in cracks and cavities (grout injection) can only be carried out with low forces, depending on the wall bond, superimposed load and friction and must therefore be regarded as little promising. Because the wall bond can relax, the prestressing forces can deteriorate later.

Extensive examinations are presently being carried out in the Special Research Programme, concerning both prestressing without bond and prestressing without or with minimized use of injection material.

4.3 Experiences from subsequent examinations

Concerning corrosion protection the statements on stitching bars also apply to prestressed anchor ties. By guiding the anchor in the masonry spacers are necessary to guarantee a coverage with cement around the steel of 20 mm and around the sleeves of at least 10 mm. Attention must also be paid to the corrosion protection of the anchor heads. About anchors that lie outside the masonry there exist positive experiences of 60 years concerning protection by coating or embeding in cement mortar.

Periodic controlling measurements of the stress forces in newer anchors are carried out by the University of Karlsruhe since 12 years. The anchor forces stay quite stable in the different buildings; the losses of stress move between 3 % and 12 % , depending on the special condition of the building. In special cases losses of stress up to one third were measured e.g. when subsequent underpinning was carried out or when constructive charge was subsequently reloaded on a new pile foundation.

Measurements were also carried out on older prestressed anchor ties. The results must still be evaluated. Up to now it can only be said that the examined anchors obviously now as before fulfill their function entirely.

4.4 Rules for dimensioning and executing prestressing of masonry

This again refers to the 1987 yearbook. It will provide information on the following: Necessary examinations of specific structural circumstances; Dimension of the stress force; Influence of the load independent deformations of the masonry on the stress force; Influence of creeping on the stress force; Spontaneous plastic deformations of the masonry; Stress force losses as a reaction to elastic and plastic initial deformations of the masonry; Allowable design stress; Diameter of the drill-hole; Anchorage of prestressed rods with anchor bodies; Partial surface pressure perpendicular to the horizontal joint; Partial surface pressure parallel to the horizontal joint; Anchorage of prestressed rods by bond; Buckling of the masonry; Buckling of multi leaf walls perpendicular to the plane of the wall; Buckling upwards and necessary superimposed load; Forces and design pressures in the transverse bars; Additional end anchorage.

5. OUTLOOK

In the Special Research Programme examinations are presently carried out among which are the following which shall also support improvement and further development of grout injection, stitching and prestressing of masonry of historical buildings:

5.1 Grout injection

Mineralogical and material technological analysis of the old masonry, especially the historic mortar. Developing of repairing- and replacement mortar for grout injection as well as for jointing with the goal of the best coordination possible of old and new building materials. Differentiated inquiries on the reasons for damage on the church buildings in Lower Saxony which were restored with cement injection and prestressing and are damaged again. Examinations to reduce and to limit the injection material, also for the extended use of the technique of grout injection on masonry with a valuable plaster and stucco or with delicate paintings and panels.

5.2 Stitching

Experiments of pulling out anchor bars

5.3 Prestressing

Prestressing without bonding providing the possibility to stress the anchor subsequently or to remove the anchor. Strain measurements on anchors that were installed longer ago.

5.4 Further examinations

Under the theme "Grout injection, stitching and prestressing" the engineer like inventories at buildings that were restored some time ago including the collecting of data on the building as well as of execution and planning records and the inspection and focused examinations with interventions in the buildings substance to examine the success or failure of the applied securing techniques are going to be continued.

In connection with the theme of this contribution there are also low destructive examinations concerning the determination of strength and deformation values of old masonry, investigations dealing with the values of old bricks or natural stones and surveys related to the behaviour of strength in unimproved multi leaf walls and pillars. A systematic test on low and non destructive examination procedures to determine the inner constitution of masonry of historic buildings has also started.

It was possible to plan many of the above mentioned examinations for a longer period of time - an advantage of the continous promotion of the Special Research Programme by the Deutsche Forschungsgemeinschaft (German Research Community). Results can therefore not be expected at once, but step by step.

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PRACTICAL TECHNIQUES TO IMPROVE THE STRUCTURAL STABILITY OF STONE MASONRY RETAINING WALLS IN SITU

DAVID KORN*

SUMMARY

The British Engineers, Builders of the Nineteenth Century Halifax Citadel deviated from the time proven Vauban's design for fortifications, by decreasing the retaining wall thickness and providing insufficient drainage. No consideration was given to the specific Canadian climate. Part of the fortification was founded on uncompacted backfill. Severe structural distress was the result. Installation of a drainage network, replacement of frost susceptible soils with granular fill or styrofoam and installation of wall monitoring program increased the structural stability of the walls and provided an in situ intervention with long term benefits.

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7 April 1989

PRACTICAL TECHNIQUES TO IMPROVE THE STRUCTURAL STABILITY OF A STONE MASONRY RETAINING WALL IN SITU METHODS USED IN THE RESTORATION OF THE HALIFAX CITADEL NOVA SCOTIA, CANADA

1. Design & construction - historical background

The British Engineers, Builders of the Nineteenth Century Halifax Citadel have based their design of the Halifax Citadel on Vauban's, (Vauban-1704)(4), European time proven model for fortifications. Leduc - 1840 (2). However, they ignored the harsh Canadian winters and were probably unaware of the effects on structures of the freeze and thaw cycle.

In order to get a larger courtyard for the fort, the crest of the 36M drumlin was cut down by 6 metres.

The uncompacted fill was deposited on the south side of the hill on which the large retaining walls were built.

To make things worse, the dimensions of the footing and the thickness of the retaining walls were on average thinner by 0.67M than Vauban's recommendations. (Greenough - 1977.)(1)

Within four years of the beginning of construction the insufficiencies became apparent as several complete walls crumbled. In the next 25 years of construction, however, the quality of the construction improved.

2. Diagnosis of existing problems

a. Site and foundations

The 245M long and 150M wide fort with more than five kilometers of 6-8M high granite and iron stone faced rubble-filled gravity retaining walls is founded on a drumlin, an ice accumulation of sand, silt, clay and gravel. The solid bed rock is 30-35M below the courtyard. The low bearing capacity of the drumlin and the low permeability of the soil make it unsuitable for foundations for massive gravity retaining wall structures. The south part walls of the site was built up from the accumulation of the backfill from the levelled hill, on this uncompacted backfill, the south-escarp, counter escarp and courtyard walls were built. When the major restoration started in 1977 the south part of the fort accounted for the major structural distress ranging from bulges and large cracks to major out of plumb leanings.

b. Climate and drainage

In the harsh Canadian winters, ice forms in the frost susceptible wet soils. The ice and freeze and thaw cycle exert formidable additional forces on the inside, top and under the structure, accelerating the structural deterioration. The drainage originally installed was hopelessly inadequate.

(Razzolini 1979.)(3) The 10MM size clay pipes silted up and became blocked. The low permeable soil added to the building up of water behind the walls and ground water level stayed high.

c. Structure stability

Instead of compensating for the unfavourable harsh weather conditions and the associated additional forces exerted on the structure compared to the mid-European climate on which Vauban's model was designed, the British Builders in the name of economy shaved off 0.67 meters of the recommended time proven walls and foundation dimensions.

Most of the walls failed one of all of the conventional modern structural stability analysis criteria for retaining walls, namely - (N.R.C. 1975)(5)

1. Stability against sliding
2. Stability against bearing failure and overturning
3. Resultant of forces falls in mid third of the base and the appropriate co-efficient of safety

d. Cost of rebuilding

Preserving the authenticity of an historic structure dictates restoration intervention methods that do not require dismantling of the structure. Circumstances where safety of the visiting public is compromised is an exception to the rule. In the case of the Halifax Citadel the cost of rebuilding is $20.000 Can. per linear meter. An exorbitant cost by any standard.

3. <u>Methods to improve retaining wall stability in situ</u>

The challenge for the modern Engineer, therefore, is to increase the structural stability of the retaining walls that show distress symptoms, in situ. In situ preservation intervention has two major advantages over rebuilding. It conforms to departmental and professional restoration philosophy and policies and is more cost effective.

The following methods were used to increase the stability of the Halifax Citadel retaining walls.

a. Improvement of drainage

The site's major drainage problems were solved by installation of 1.5M deep dutch drains composed of gravel wrapped in filter fabric running perpendicular to the under foundations 5-10 meters apart and connecting them to a recently installed perimeter drain in the ditch, it provided the following benefits to the wall stability.

1. The elimination of the hydro static pressure built up behind the wall
2. Prevention of ice formation and associated pressure on the wall
3. Elimination of frost soil heave on foundation.

Installation of an elaborate drainage system in the courtyard in the form of the installation of 2M deep dutch drains covered with filter fabric had the effect of lowering the piezometric and ground water level thus doubling the bearing capacity of the soil.

Installation of the drainage system was the most cost effective method of increasing the structural stability of the fort structures.

b. Soil replacement and cover

The top 1.5M of soil is subject to the freeze and thaw cycle. Replacing the soil with non-frost susceptible material like gravel would eliminate the horizontal forces that significantly contribute to the overturning tendency of the walls.

In cases where the footings are covered with less than 1.5M of soil and authenticity does not allow backfill a horizontal styrofoam layer was placed. Each 25MM of styrofoam is equivalent to 0.3M of missing soil.

c. Surcharge on wall

In cases where authenticity dictated reinstallation of a soil rampart surcharge, replacing the soil with styrofoam slabs instead of soil mass proved to be a successful unorthodox method to fulfill the Historian's wishes without imposing additional streses on the retaining wall below.

d. Repointing

In order to prevent moisture penetration in the walls all walls were repointed and cap stone water proofed. In some cases plastic resins were injected into cracks to harden the rubble wall.

e. Wall movement monitoring

On the most structurally distressed retaining walls a wall monitoring system was employed to determine whether the wall was stable or moving. If it was moving, at what rate?

4. Duration aspect of the interventions

Installation of the elaborate drainage system and wall repointing started in 1977 and was on-going for eight years until 1985. No walls on which intervention was done show distress to date. Drains are working well.

The pleasant surprise is the performance of the styrofoam ramparts. Where the soil filled ramparts need yearly extensive maintenance the styrofoam ramparts are maintenance free. They practically disregard the weather.

5. Summary

The lack of soil mechanics knowledge by the original builders, their wish to economize and their ignorance of the harsh Canadian winters left us with a fort structure with many structural deficiencies. Installation of an elaborate drainage system, replacement of soils with granular soil or styrofoam and wall movement monitoring provided an authentic in situ intervention that has long term benefits on the structural stability of the Halifax Citadel in addition to being cost effective.

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ACKNOWLEDGEMENTS

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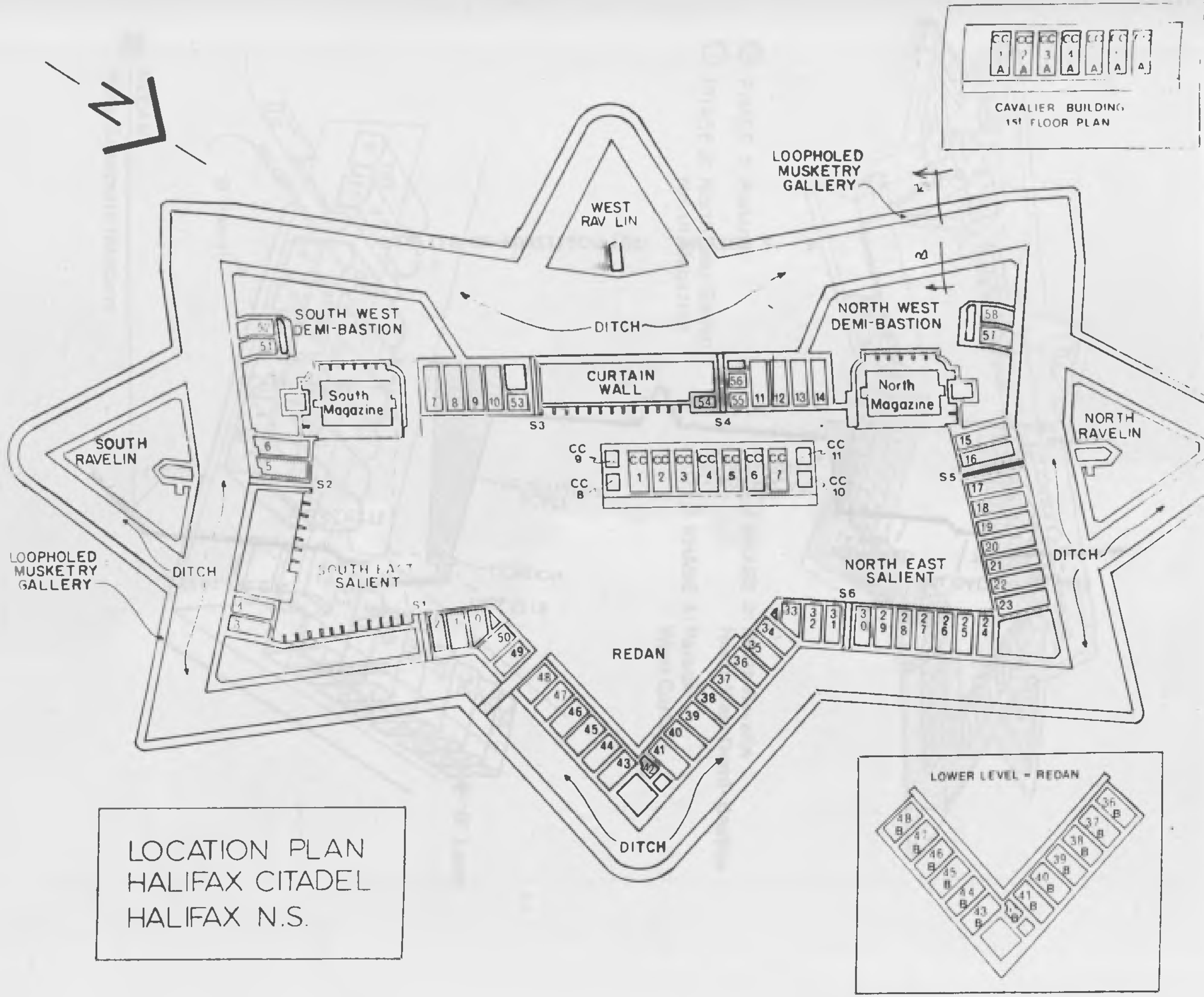
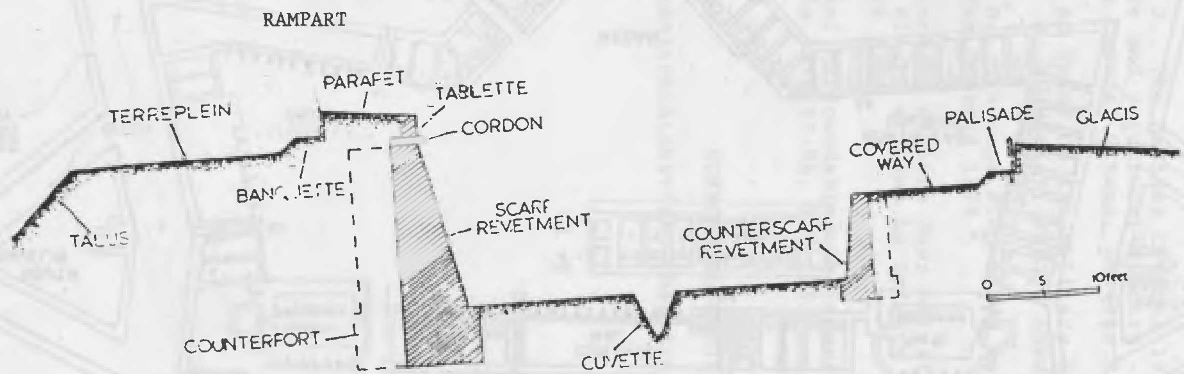
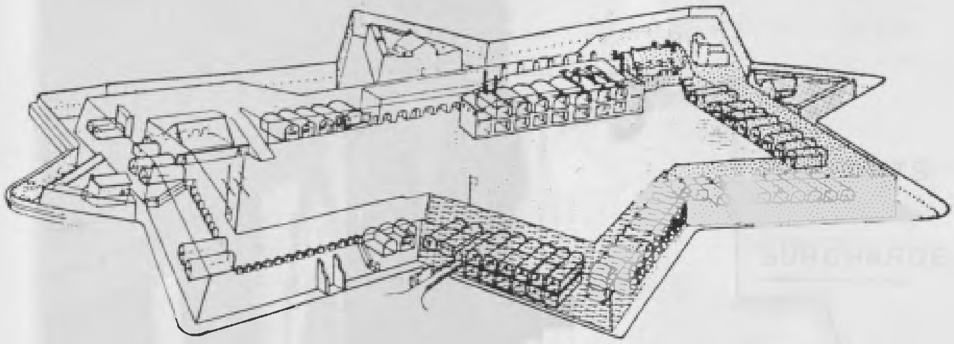


LOCATION PLAN
HALIFAX CITADEL
HALIFAX N.S.



PROFILE OF FORTIFICATION: SECTION B – B

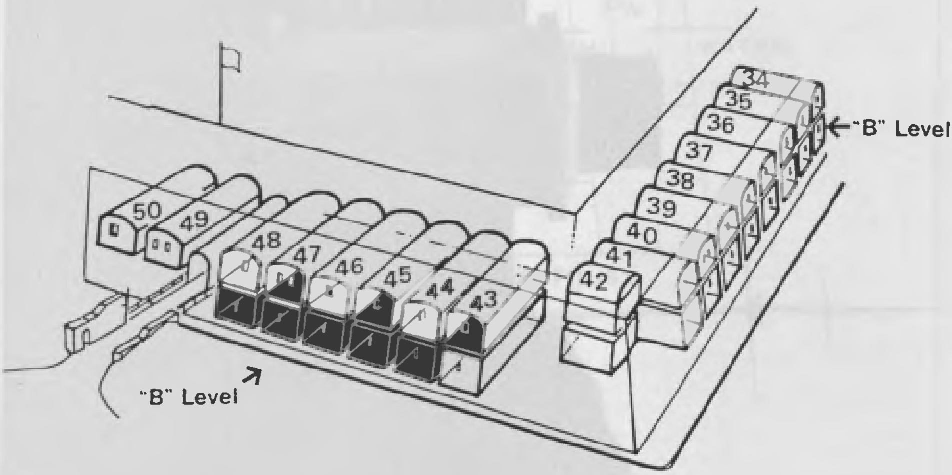


PHASE 1: Redan

PHASE 2: North East Salient
North Magazine

PHASE 3: North Ravalin
North West Demi-Bastion

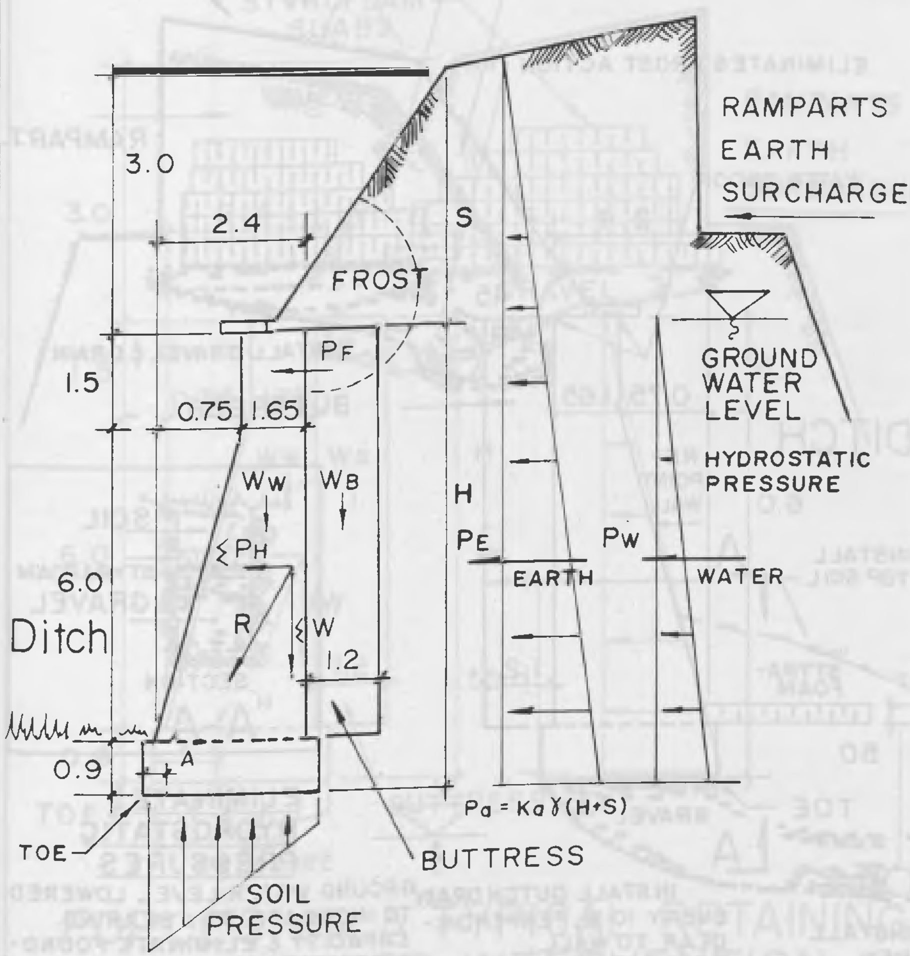
PHASE 4: Parade
West Curtain



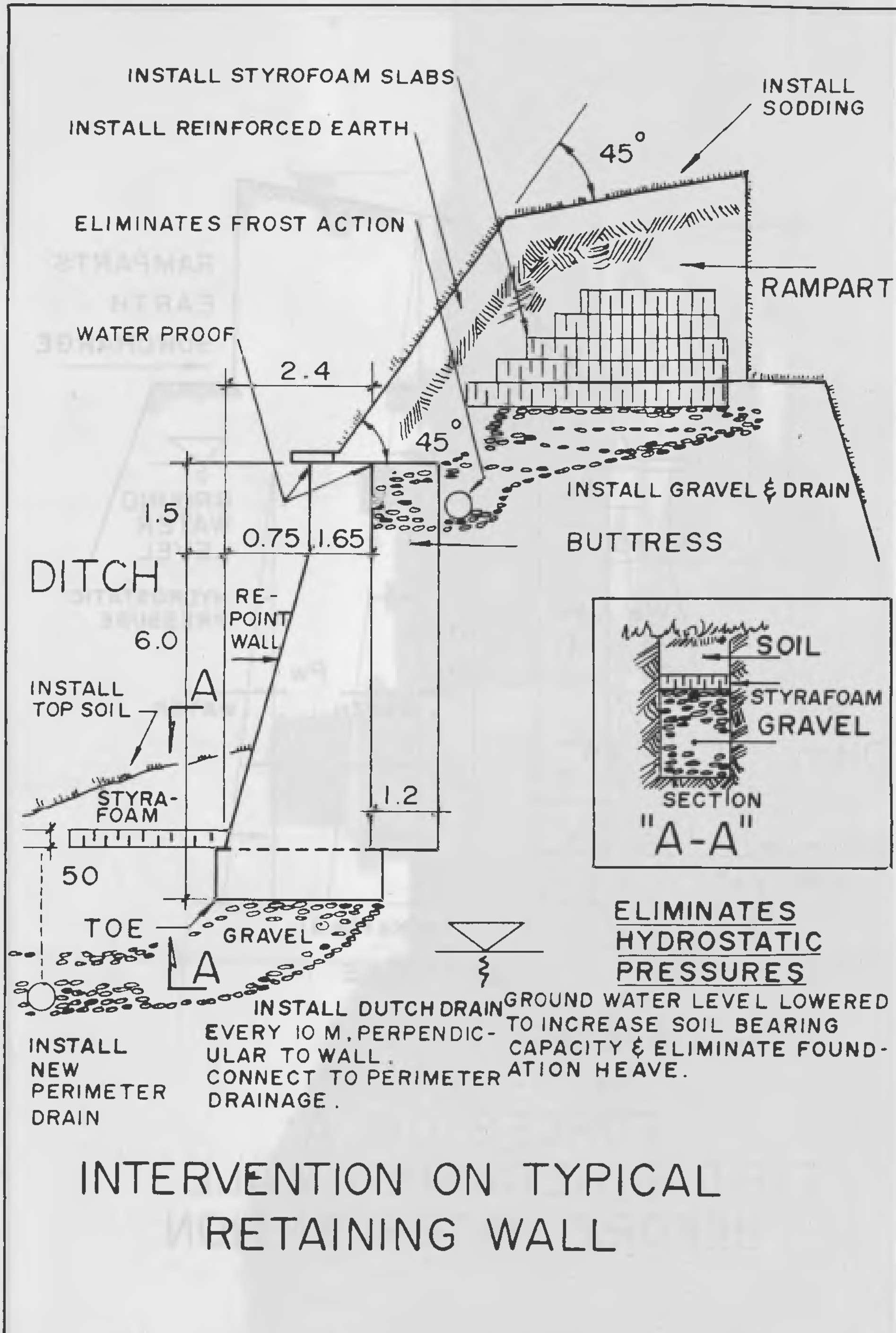
REDAN

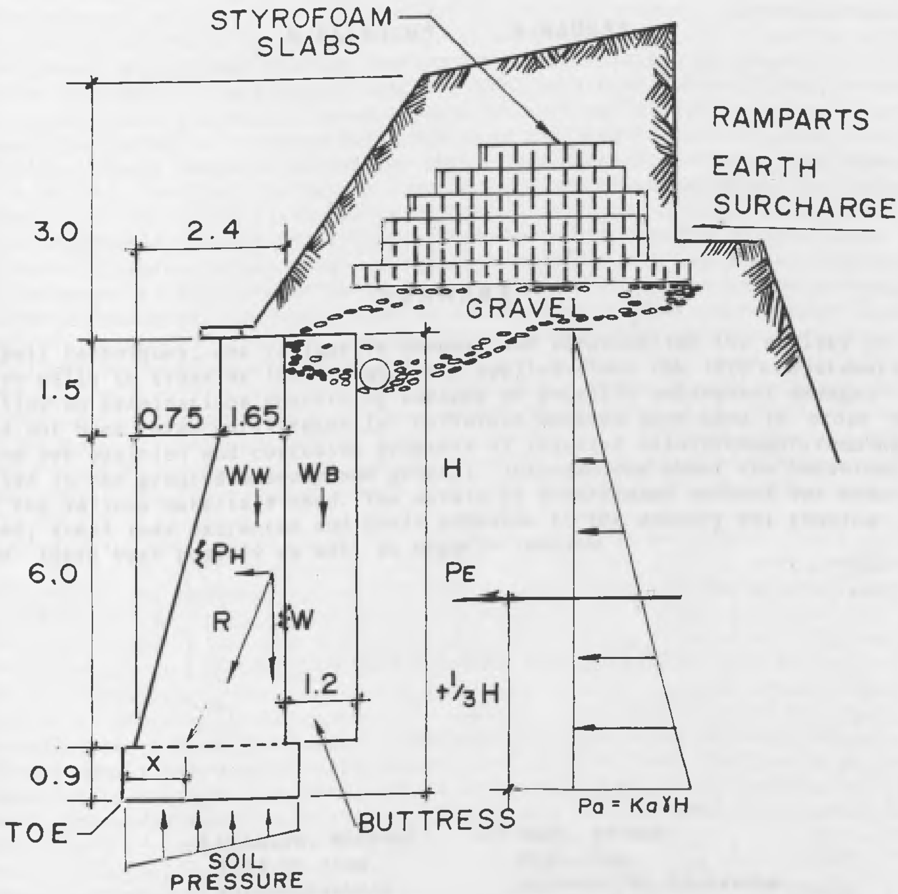
Note Casemate Numbers

A restored rampart with styrofoam
slabs - Halifax Citadel



FORCES ON A
TYPICAL RETAINING WALL
BEFORE INTERVENTION


INSTALL STYROFOAM SLABS
INSTALL REINFORCED EARTH
ELIMINATES FROST ACTION
INSTALL SODDING
45°
RAMPART
WATER PROOF
2.4
45°
INSTALL GRAVEL & DRAIN
1.5
0.75
1.65
BUTTRESS
DITCH
RE-POINT WALL
6.0
INSTALL TOP SOIL
A
STYRA-FOAM
1.2
50
TOE
GRAVEL
A
SOIL
STYRAFOAM
GRAVEL
SECTION "A-A"
ELIMINATES HYDROSTATIC PRESSURES
GROUND WATER LEVEL LOWERED TO INCREASE SOIL BEARING CAPACITY & ELIMINATE FOUNDATION HEAVE.
INSTALL DUTCH DRAIN EVERY 10 M, PERPENDICULAR TO WALL. CONNECT TO PERIMETER DRAINAGE.
INSTALL NEW PERIMETER DRAIN
INTERVENTION ON TYPICAL RETAINING WALL



FORCES ON A TYPICAL RETAINING WALL- AFTER INTERVENTION

W_W = Weight of Wall

W_B = Weight of Buttress

P_E = Total Earth Pressure

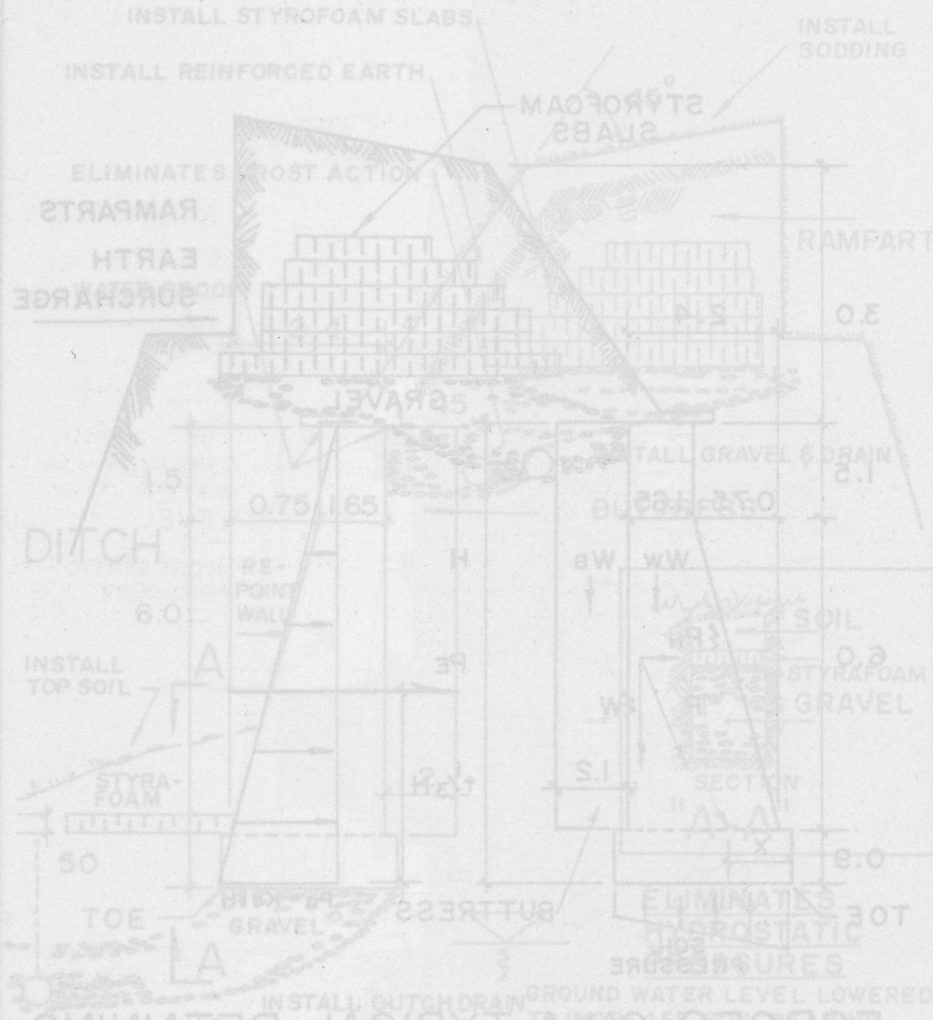
ΣP_H = Sum of Horizontal Forces.

ΣW = Sum of Vertical Forces.

R = Resultant of Forces

P_A = Local Active Earth Pressure.

P_F = Pressure of Frost.



INTERVENTION ON TYPICAL RETAINING WALL

ENGINEERING EXAMINATION OF EXISTING BUILDINGS PREVIOUSLY REPAIRED

M.ULLRICH* H.MAUS**

SUMMARY

Repair techniques, due to improve masonry and re-establish the ability of torn walls to transfer loads, have been applied since the 1920's. Systematic follow-up examinations concerning success or possible subsequent damages had not been taken until recently. Different methods were used in order to find out position and corrosion progress of inserted reinforcement, remained voids in the grouted masonry and general informations about the behaviour of the various materials used. The strain in prestressed anchors was measured, steel rods extracted and their adhesion to the masonry was examinated. There were positiv as well as negativ results.

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PREFACE.

Since ever constructions are exposed natural ageing and wearing. This concerns not only the surface but in many caces the construction parts itself. Further more the interaction of ground and structure resulted sometimes in damages.

In regard to preservation, repair, renovation or higher use loads the techniques of mortar injection, steel reinforcement and prestressing are used since the 1920s to strengthen damaged masonry. Obvious stabilizing effects were achieved by grouting work and reinforcement. Wrecking and reconstruction could be prevented very often and due to this the historical structure was preserved - manly with lower costs.

A deficit in basic research to this structural technique and some regrettable failures in the long term behaviour promoted systematic follow-up examinations of old masonry buildings, that had been structural repaired some decades ago. Within the special research programm "Preservation of historically important Buildings" (research work is done at the University of Karlsruhe) these examinations are carried out by core-boring, endoscoping and exposing reinforcement bars. And in addition to that the flow of injection material, the position of reinforcement inside the drilled channels could be checked in-situ after reopening walls for supplementary repair. This research work is done to find out strong and weak points of these techniques and to define questions for more extensive research and case studies.

First results of these examinations are given hereby for further discussion.

EXEMPLIFYING INSPECTIONS.

The begin of engineering techniques to strengthen masonry of historical buildings was studied by a large number of sacral, profane and technical buildings, which were stabilized in the 1920s and 1930s. The standards of those techniques at that time shall be explained by the following three objects.

The anchorage structure shown in figure 1 was conceived and realized by Prof. PIRLET for the gothic choir-hall at Aachen Cathedral between 1914 and 1920. The construction compensats a gothic ring-anchor, which was destructed by structural modifications in the 18th century and became inefficient. Pirlet discerned the fact, that the lofty, slender pillars and walls needed to be hold at the wall crown, specially against forces due to wind. The construction itself and its tying to the masonry is quite functional up today. Nevertheless the gothic ring-anchors were restored in the 1970s due to newer statical analyses and to better overall stability of the choir - while damages from world war II were repaird.

A water castle close to Münster - the ground plan is shown in figure 2 -

was stabilized by Prof. RÜTH 1928/29. He placed wooden foundation piles, ring- and radial-anchors to stop the detachment of the exterior masonry wall from the interior walls. The anchors out of wrought iron are installed close underneath cellar flooring inside the building and corresponding in the inner court. There are no channels for the iron bars and in consequence there is no grouting around. But it is reported, that the gaps in the walls were grouted. An intensive examination of the wall surfaces, the ceilings in the vaults of the basement and of the anchorblocks on the exterior wall turned out without any earnest damages, i.e. the anchor system seems to be still effective.

At Lübeck Cathedral an examination of the towers made it possible to inspect various masonry repair measures applied to one building over a long-term period. First strengthening efforts were made by Rüth 1934. Due to unsymmetrical settelments the towers got inclined over the time and the walls cracked heavily. To prevent further cracking of the masonry he gave order to place right in front of the interior wall-sides iron anchors in several horizons and to tie them with the masonry - figure 3. (A necessary bracing or underpinning of the foundation, as Rüth wanted to do, was technical not practicable then.) Equipment for drilling channels over the whole length of the tower-walls was not yet available at that time. After the cracks and gaps in the masonry were closed by bricks or grouting with fine-mortar the anchors were set under tension; the anchor-plates were set in pockets beneath the wall surface.- Because the incompatibility of sulfate-bearing material (gypsum mortar) with the tricalciumaluminat phase (C_3A) of the portland cement was known alredy since the 20s Rüth used iron-ore cement; unfortunately this cement is not more produced today.

The north tower of Lübeck Cathedral was damaged 1942 by bombs and fire very heavy. The anchors - placed by Rüth - glowed out und the masonry began to break into parts again. In an ad hoc action Prof. PIEPER arranged the necessary steps to prevent the tower from collapsing. Five reinforced concrete slabs were built in and anchored with needles to the masonry - fig. 4. For the injections Pieper could get then only normal portland cement. Years later - as prophesied and expected - came up new damages caused by expansive minerals (ettringite) and in addition with that frost damages. So Pieper had nearly two decades after his first job to do structural work again. This time he strengthened the masonry walls with needles and tension anchors; the prestressed anchors were placed into drilled channels, which were driven in the middle of the walls over the whole length. For the injection material he used this time highly sulfate-resistant cement and up today new damages became not known. - Certainly an other fact influences this positiv result: all damaged bricks in the wall surfaces were replaced and all mortar joints were repointed to prevent rain penetration.

For all the buildings, which were structural repaired in the last three decades successful an other water castle south of Dortmund may be called in evidence. This castle was under repair between 1974 and 1978. As in many other cases it was possible to compare the shape of the building now a days with the engineering drawings for the structural work - figure 5. An intensive examination of the whole castle and a comparison between the engineering drawings and earlier documentations of the damages turned out with a positiv result; i.e. no symptoms were found, which refered to inefficient needles or prestressed anchors. - The original mortar contains some gypsum and was grouted therefore with trass-lime and trass-cement mortar.

FINDINGS OF IN-SITU EXAMINATIONS.

The reasons of damage, which resulted in structural conservation and repair of structures or parts of them, were quite different. In the main, there were weak or failing foundations (e.g. to small dimensions, rotted wood piles), poor masonry or failures in the constructions right from the beginning. As a matter of these facts redirection of forces, deformations and gaps were caused. Bad or total missing of upkeep, structural conversions or increase of use loads promoted overstress in the masonry, which had original most low mortar quality. Wind, moisture, erosion of cementing material and the natural ageing of materials put the efficiency of mortar down and therewith the bearingcapacity of the masonry.

To all the buildings, which were inspected, the bearing capacity of the walls was increased by grouting with cement or trass-lime suspension; if necessary reinforcement members were placed into multileaf masonry as well as methods of prestressing were applied.

There is no reason not to mention some regrettable failures as a result of applied injection technique. 15 to 20 years after grouting and reinforcing new structural work was indispensable to some buildings; in a few cases wrecking of structures could not be hindered. By a great number of these cases it was possible to examine large sized the distribution of the injection material inside of masonry, the connection of original and injection mortar, the position of needles and anchors inside the drilled channels as well as - in relation to the distance from the injection hole - the covering over the steel members.

It was found out, that inside single- and multileaf masonry only joints or cracks larger than 1 mm were filled up with grouting material respectively only the drilled channels; this is specially due to masonry made with mortar rich in mineral binder. Inside the walls horizontal, vertical, spatial or to stone surfaces following cement disks were found. Penetration or intermixing of mortar suspension into respectively with original mortar - as so far assumed - was not found. If there was used gypsum mortar the separation of "old" and "new" was more obvious then in cases with poor lime mortar. Air bells respectively ungrouted wall parts were found, when air drain was not sufficient or when voids were situated above the injection channels.

In mortar with less binder, that is sandy and sucking, around the injection channel large sized cement stones accumulated. Certainly the original mortar was washed out, while water was used to cool the drill crown and to clean the drill-hole before injection was done. Large porosed pieces of original brick or mortar seem to have evaporated the injection suspension, so that it got cohesive befor all gaps and voids inside the masonry were grouted. A "concrete" out of original mortar, cement paste and stones - as often claimed as a result of injection technique by opponents - could not be found so far in all the examinations.

Even stones, covered with expansive minerals (ettringit, thaumasit) were taken from openings to make further examinations in the laboratory.

The efficiency of needles for holding together multileaf masonry depends on the anchoring in the outer leafs. In many cases the end of the steel rods were not or only unsufficient covered with cement paste and in order to this the anchoring was total or in parts uneffective, in accordance the reinforcement was corroded.

Further more it was found out, that the position of the reinforcement inside the injection channels was not correct. Nearly in all examinated cases the steel rods did lie on the bottom wall of the channels and in cases, where negativ results were claimed, the diameter of the drill-holes was mainly only a few millimeters bigger then that of the steel bar; consequently the cover with cement paste was unsufficient and the bond to the masonry was uneffective as well as the protection against rust. If the cement cover was at least 2 cm thick, there was no corrosion on the surface of the bars, even after more than 20 years.

For pretensioned anchors the same observations and statements were made in principal as for mild steel reinforcement. But damages due to corrosion are of more importance because the normal stress inside the cross-section increases owing to a reduced diameter, quite apart from notch stresses. On the other hand these higher stresses are compensated by natural loss of prestress. - The strain of prestressed anchors was measured periodically since more than 13 years and the values found were compared with available earlier ones, in order to examine the durability of prestressing. These measurements indicate a loss of prestress between 3 and 12 % ; only in cases with structural changes after prestressing turned out with a greater loss.

To proof the bond of injection mortar to the steel rods extraction tests were made with satisfactory results.

RULES AND RECOMMENDATIONS FOR PRACTICAL WORK.

No universal standards can or should be established for historical buildings, but rules and recommendations can be given for application in practice, which can be adapted to the special circumstances of each objekt.

Application of grouting work is convenient

- to strengthen loose mortar and masonry bond
- to close cracks, gaps and voids inside the wall structure
- to increase the load-carrying capacity of original masonry
- to make areas, which are strong enough, to take bigger forces due to changed use loads or supports of new constructions
- to join tension-proof needles and prestress-anchors with masonry as well as to protect steel members against rust.

If there are not these or similar problems it is to advise against application to masonry.

The injection suspension should have a high w/c-ratio and the injection pressure should be maintained as long as possible; so a penetration of new mortar into the old ones will take place, at least close to the injection channel. Simultaneous air and excess water - coming out of the injection mortar - is able to escape from injection hole.

To have sufficient bond bond between reinforcing and injection mortar respectively between new and original mortar the reinforcement and tension anchors must be centered inside the drill-channel with reinforcing rod spacers. Inside multileaf masonry needles must be placed horizontal and be joined tension-proof with the outer leaafs. Injection brings the necessary bond to reinforcement and masonry and simultaniously the interior zone is strengthened.

When there are big cracks in masonry mainly the shear resistence of the walls got lost. To stop moving of detached parts prestressing will help to join these parts durable again. As far as possible the prestress-anchors are placed vertical to the cracks and the anchor-end is joined to the masonry by special anchor blocks or plates. Tensioning can be done before or after cracks and voids are filled up with grouting material.

Before structural work is started an analysis of the original mortar to gypsum content is required as well as to salt content, that could washed out by excess water and effloresce on the surface of the walls. Conservation concept can be changed in case, when the results of analysis give an advice to do so. On the other hand should be used high sulfate-resisting cement or adequate binder in all cases,where gypsum may be (or is) contained in the original mortar,to prevent the growth of expansive minerals. - Further more it is important to dry masonry durable because then formation of expansive minerals is nearly impossible. Short term rain penetration is to neglect as long as rain-water can evaporate again.

Ready mixed mortar specially fabricated for use in gypsum masonry is not without technical problems: placing of these mortars needs temperatures more than 5° C ; setting time is longer as due to normal mortar and constant huminidity content inside the mortar is required for weeks; the strength and bond is less than refered to normal cement mortar as well as the ability to protect against corrosion, i.e. reinforcing must be out of stainless steel.

Success and unsuccess depends highly on the qualification of the egineer as well as on the qualification of the contractor. It is to advise that structural work, due to preserve historical buildings, must be supervised by engineering specialits more frequent as normal - in case daily; further more the workmenship should be done only by contractors which have experience of work like this and have skilled workmen.

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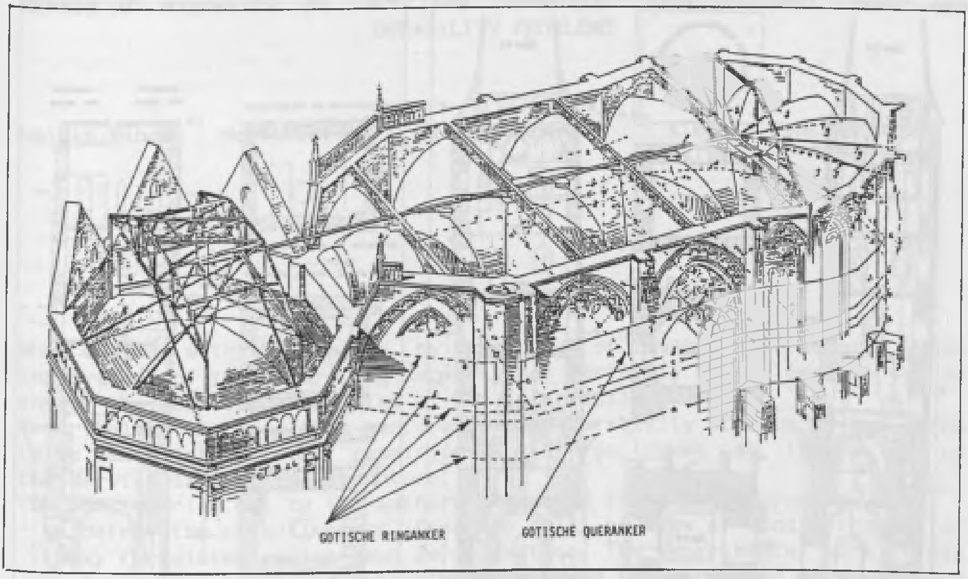


Fig. 1 Aachen Cathedral: anchorage structure above choir-hall (by Pirlet - drawing by Buchkremer)

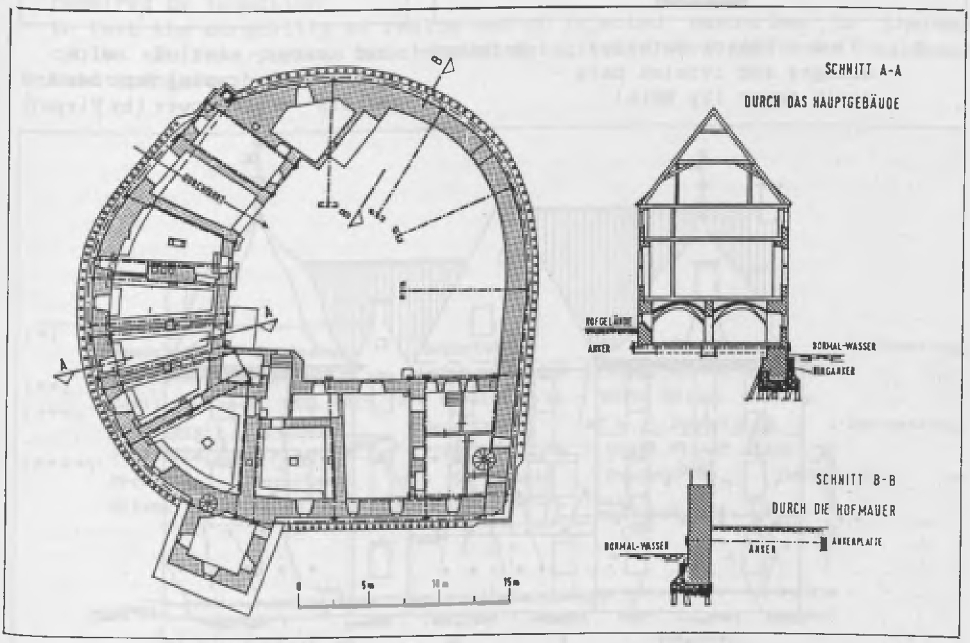
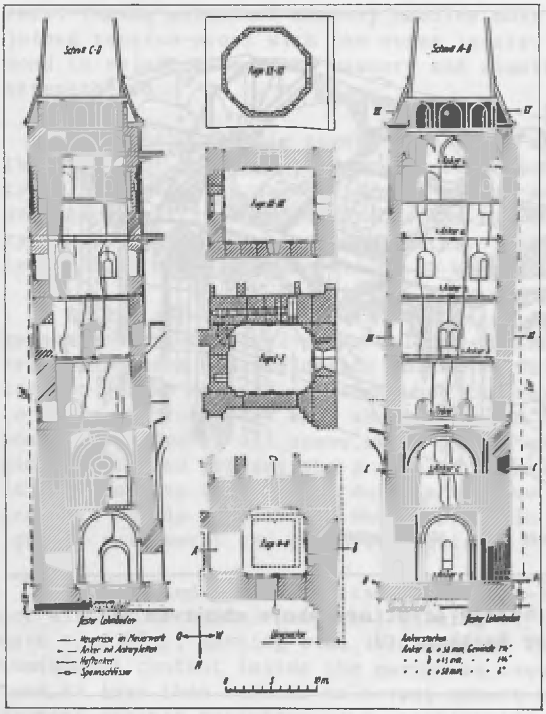


Fig. 2 Ground plan of a water castle with anchor system (by Rüth)



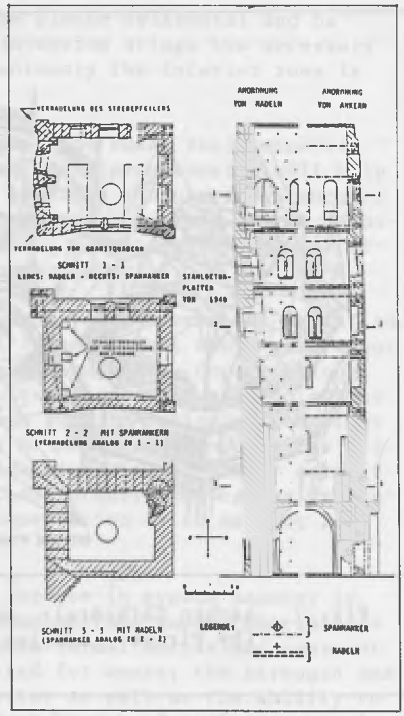


Fig. 3 + 4 Lübeck Cathedral: Vertical- and cross-sections with damages and tension bars - south tower (by Rüth); engineering drawing for rods and anchors - north tower (by Pieper)

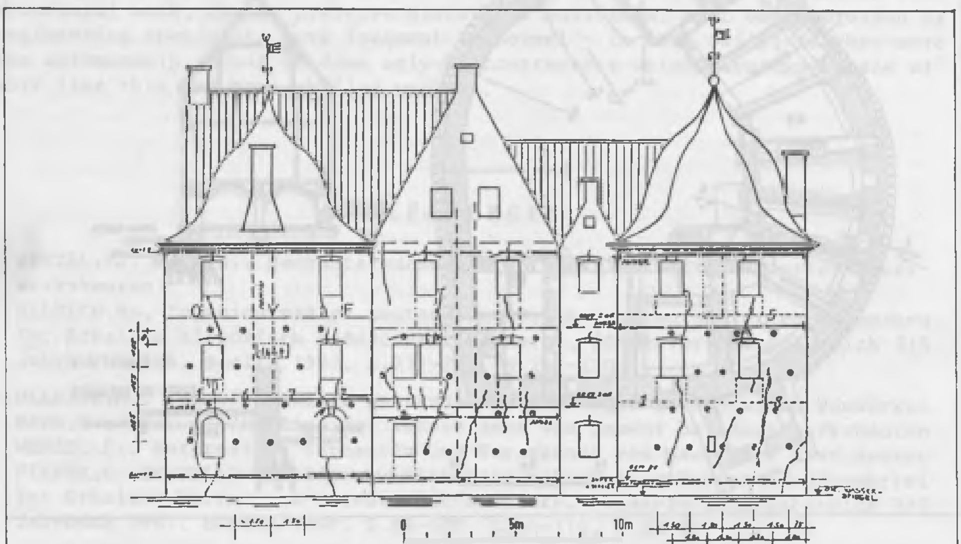


Fig. 5 View to the west-front of a water castle with engineering drawing for rods and anchors (by Schriek)

REPAIR OF MASONRIES BY INJECTION TECHNIQUE: EFFECTIVENESS, BOND AND DURABILITY PROBLEMS

Luigia Binda(*), Mario Berra(**), Giulia Baronio(***), Alberto Fontana(****)

SUMMARY

When dealing with the repair by grouting of masonry structures, the improvement of their carrying capacity has certainly to be considered as one of the most important aims of the intervention.
Nevertheless the long term performance and durability of repairs has also to be taken into account; repairing should also insure an increasing of the service life of the structure.
The work carried out by the authors concerned three main purposes:

- to detect the effectiveness, from the point of view of bond strenght, of epoxy formulated resins used for injection. The experimental work based on mechanical tests and optical observations was performed on shear bond specimens;
- to calculate and evaluate, also with the aid of non destructive tests, the improvement of the mechanical strength of decayed masonry prisms repaired by injection;
- to test the durability of resins and of injected masonries to thermal cycles. To this purpose accelerated aging tests were performed on shear bond specimens.

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INTRODUCTION

Several techniques for repair and retrofitting of masonry structures have been applied to monumental buildings and dwellings of the historical centers, particularly in seismic areas. Grouting by injection of resins, and of polymeric and cementitious grouts is a repair and strengthening technique largely adopted in Europe, particularly in Italy.

The results of a previous experimental research carried out by the authors (Binda - Baronio - Fontana, 1987), (Binda - Baronio, to appear), have shown that the increasing of strength and more generally the effectivess of the injections strongly depend on: (i) the chemico-physical and mechanical compatibility between the grout and the original material, (ii) the penetration and diffusion capacity of the grout, the durability of the grout to frost-defrost actions, thermal cycles, etc. In order to better understand the influence of the above mentioned factor, some damaged brick- masonry prisms were repaired with epoxy formulated resins and their mechanical behaviour was studied in comparison with the behaviour of the same prisms when undamaged. Particular care was taken in applying an adequate technique of injection, such that the injection pressure could be kept constant, the feeding of resin continuous, the loss of injected material minimum. The behaviour of resins when injected in dry or wet masonries was also checked together with the performance of injected specimens to thermal cycles.

Finally the reliability of non destructive tests as a mean to evaluate in situ the degree of mechanical and physical damage of the existing masonries and the efficiency of their stregthening by grouting was widely investigated using both ultrasonic methods and sonic vibrational techniques.

MATERIAL PROPERTIES AND GROUTING PROCEDURE

Eight prisms (250x510x600 mm) were built with solid bricks and a cement-lime mortar named M2 in a previous paper (Binda - Baronio - Fontana, 1987). The bulk density of bricks was 17300N/m^3, the water absorption percentage in cold water for 24h was 22%, the initial rate of absorption 33,2 N/m^2·min, the compressive strenght 22.0 MPa. The M2 mortar had the following composition: 1:3:5 as proportion cement, hydraulic lime and sand respectively and 0.54 as ratio water/binder; its flexural strength was 3.9 MPa and the compressive strength 12.7 MPa. The eigth prisms were previously cracked with a compression test as described in the next Section and subsequently injected. Two types of epoxy formulated resins were used in order to compare the results obtained.

A certain number of specimens used for shear bond tests were prepared in the following way: three bricks were joined together with the chosen epoxy formulated resins in order to simulate cracks or lack of adhesion in a joint (Fig. 1). The operation was done both on wet and dry bricks in order to check the bond strength of the resins in the two cases. The specimens were named respectively ADH-MDi, ADH-SDi if dry and

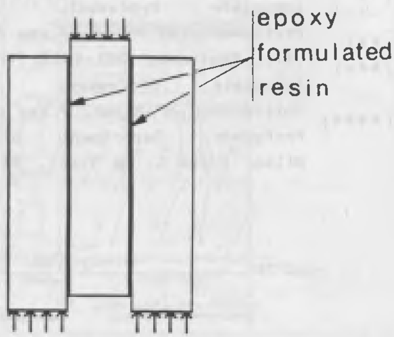


Fig. 1 - Specimen ADH used for shear bond tests.

ADH-SWi, ADH-MWi, if wet (with i = 1, 2, 3); M and S indicate the type of resin which will be subsequently described, D and W indicate the dry or wet condition. From here on, the word dry will have the meaning of room temperature and humidity conditions.

Resin properties

Two epoxy formulated resins which will be named S and M were used for injection. They both were similar to the resin used in a previous research (Binda - Baronio - Fontana, 1987).
Resin S, a commercial resin, was a two component epoxy-amine system curing at room temperature. The resin is principally based on a A-bisphenol epichlorohydrin (having a low molecular weight), modified with 1.4 butandioldiglycidylether in a ratio of 4:1. The hardener was trimethilhexamethylendiamin. The stoichiometryc ratio between the two components was 4:1. The properties of the epoxy formulated resin, as given from the producer, were the following: the glass transition temperature was 57°C, the viscosity 340 ± 50 cps, the pot life (referred to a mass of 10N) about 20 min, the compression strength 80 ÷ 90 MPa, the flexural strength 90 ÷ 100 MPa, the elastic modulus 3500 ÷ 4000 MPa, the linear expansion coefficient $\alpha \cong 38.59 \times 10^{-6}\ {}^{\circ}C^{-1}$. These values refer to a temperature of 20°C and a R.H. of 70%.
The resin called M was an experimental one, particularly designed to improve the mechanical properties at a temperatures below the glass transition temperature. The properties of the resin M are: the viscosity 400 ÷ 450 mpa.s, the pot life (referred to a mass of 6.5N) about 80 min, the compression strength 100 ÷ 105 MPa, the flexural strength 80 ÷ 90 MPa, the elastic modulus 4000 MPa, the linear expansion coefficient $\alpha \cong 30.85 \times 10^{-6}\ {}^{\circ}C^{-1}$, These values refer to a temperature of 20°C and a R.H. of 70%.

Injection equipment and procedure

Before injection, all the external cracks of the prisms were sealed with an epoxy based potty, then the lateral surfaces were covered with a waterproof sinthetic paint which could be easily removed after injection. Subsequently holes were drilled following an adequate direction in order to cross the most important cracks. Water was injected from the bottom, until it reached the top surfaces, into four specimens named SW1, SW2, MW1, MW2 (other four prisms named SD1, SD2, MD1, MD2 were kept dry). Subsequently the epoxy formulated resins S and M were prepared, according to the instructions given by the producers, and injected. The device used for injection is shown in Fig. 2 on the left and consists in a cylindrical steel container, hermetically sealed, equipped with a pressure regulator, a manometer and a thermometer; the material was injected starting from the bottom of the prisms. The pressure of injection was kept between 0.05 ÷ 0.06 MPa and the injection continued until the resin did not appear on the top surface of the prism. The injectors were connected with nylon pipes kept at a level higher than the height of the prisms in order to maintain a constant pressure and a continuous feeding of the resin (Fig. 2). The resin was reinjected, when necessary, a second time, taking into account its pot life. The prisms after curing, until the time for setting was elapsed (10 days), were submitted to the compression tests.

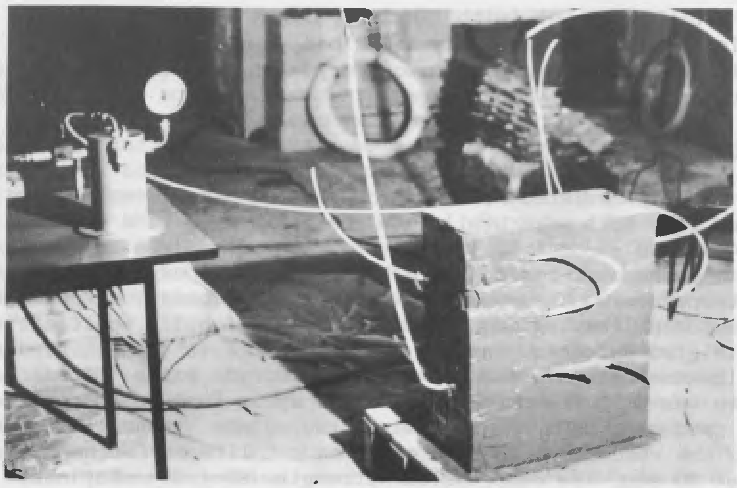


Fig. 2 - Device for injection of prisms.

EXPERIMENTAL DETAILS

The bond strength tests of the ADH specimens were performed using an MTS hydraulic servocontrolled machine at a constant rate of displacements of 0.95×10^{-3} mm/sec. The mechanism of failure was carefully observed.

The compression tests on masonry prisms were also performed with the MTS machine. The prisms were previously capped with a cement mortar and two layers of PTFE of 0.4 mm thickness were positioned between the machine platens and the prism surfaces. Initially a load corresponding to a mean stress of 0.5 MPa was applied, in order to eliminate the settlement effects caused by the defects in mortar joints. Then the loading phase started and continued after the peak stress. When the 80% of the peak stress was reached, total unloading of the specimens (at a rate of 0.75×10^{-3} mm/sec) then reloading were performed. This second loading phase had the aim to show the residual mechanical strength of the specimens (Tassios, 1988). The same loading history was followed after the injection of cracked prisms.

During every phase of the experimental work (on the undamaged prisms, after compression test, after injection and finally after testing up to failure) the prisms were controlled by non destructive measures (ultrasonic tests and sonic vibrational tests). The pulse transmission was monitored along horizontal paths (9 in the transverse direction and 3 in the longitudinal direction) and along inclined paths. Two P-wave transducers with 90 kHz frequency were used for the ultrasonic pulse transmission. They were always applied on the brick surface with a constant load of 10 N. The surface was previously cleaned and smoothed and a couplant (silicone) was employed for a better surface transmission.
The sonic vibrational tests were, instead, carried out with a load transducer and an accelerometer. The applied vibrational pulse was kept always constant. For all the measured pulse transit times were registered through a preamplifier and a waveform analyser coupled with a computer and a floppy disk recorder.

Some of the ADH specimens were subjected to thermal cycles in a servocontrolled climatic chamber. The test was set up following the principles of the Project de Norme Francaise Homologuèe Pr. P. 18-894 and of (Kallel, et al., 1986). The values used for setting the cycles were the following: maximum temperature + 55°C ± 2°C, minimum temperature -20°C ± 2°C; rate of variation of 1.25°C/min, the cycle had a duration of 8h. The maximum and minimum temperatures reached in the interior of the specimens were measured as + 51°C and - 16°C, due to the short duration of the cycles, the thermal inertia of the material and the dimensions of the specimens.

RESULTS AND DISCUSSION

Effects of surface wetness on bond strength

Before carrying out injections on the decayed prisms, the results of the tests on ADH specimens were considered. Soon after injection in most of the resin joints of the wet specimens an expansive phenomenon produced a formation of bubbles with resin release (Fig. 3), and a consequent increasing of the porosity of the resin, hence a change in its physical and mechanical properties. This behaviour was particularly observed in the case of resin S, which is used for injections in deteriorated concrete structures and gives a good bond strength on wet concrete surfaces. The authors intend to study further this phenomenon; the only possible explanation at the moment could be the high water absorption of bricks with a formation of a film of water on the joint surfaces.
Resin M, which is only an experimental product, but was also supposed to give a good bond on wet concrete surfaces, had a similar behaviour with slightly less visible formation of bubbles.
When tested in shear bond the values given by the wet specimens MWi and SWi compared to the dry specimens were generally lower, as shown in Table 1 (left side). The bond strength in the case of ADH-SWi was reduced of the 48.9%. Three different classes of bond were shown by the specimens. (Table 1 - right side)

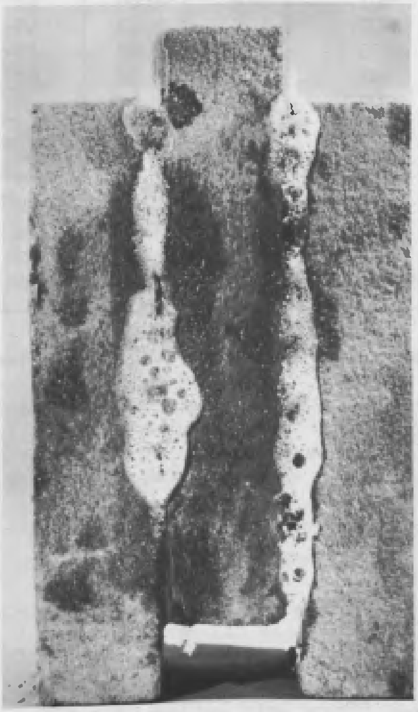


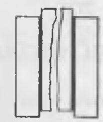
Fig. 3 - Effect of water on resin joints; evident formation of bubbles.

Mechanical strength of prisms

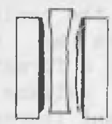
As a consequence of the results reported in the previous paragraph it was decided that some of the prisms had to be injected wet. The results of the compression tests on prisms before and after injection; together with

information on quantity of resin and number of injection points for every prism are presented in Table 2. As for the ADH specimens, S and M indicate the two resins, D and W mean dry and wet respectively.
In Figs. 4 and 5 two typical classes of σ-ε curves obtained for undamaged and subsequently injected prisms are presented.
Table 2 shows that the strength values of the undamaged prisms are rather scattered with a coefficient of variation of 20.0%; the strength values of injected prisms have on the contrary a lower coefficient of variation (8.7%). As a result of the repair intervention a more uniform distribution of mechanical strength values.

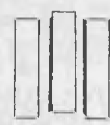
ADH Specimens	Room conditions		After thermal cycles		
	Class	Shear bond MPa	Number cycles	Class	Shear bond MPa
SD1	P	3.06	40	P	2.28
SD2	G	2.93	40	P	2.89
SD3	G	2.83	40	G	0.81
SW1	B	1.26	40	B	0.80
SW2	B	1.34	6	B	/
SW3	B	1.91	6	B	/
MD1	G	3.11	8	B	/
MD2	P	3.02	8	B	/
MD3	G	2.97	8	B	/
MW1	G	3.29	6	B	/
MW2	P	2.96	12	P	3.01
MW3	P	2.95	6	B	/



G = Good bond



P = Poor bond



B = Bad bond

Table 1 - Results of shear bond tests on ADH specimens.

Prisms name	N. of injection $/m^2$	quant. of epoxy N/m^3	σ_{ud} MPa	σ_{da} MPa	σ_{ai} MPa
SD1	18	719	11.06	7.37	16.41
SD2	13	523	10.05	7.62	14.99
MD1	8	189	13.21	8.99	16.63
MD2	8	627	8.94	5.72	16.94
SW1	8	334	8.96	6.19	14.97
SW2	13	366	10.99	6.99	14.20
MW1	14	333	12.69	8.45	13.23
MW2	18	719	6.70	6.10	14.18

where: σ_{ud} = strength of undamaged prisms
σ_{da} = strength of damaged prisms after reloading
σ_{ai} = strength of prisms after injection

Table 2 - Details on injection characteristics and mechanical properties of prisms.

The compressive strengths of SDi and MDi prisms are similar; so happens for SWi and MWi. The results show that there is no substantial difference in the mechanical effectiveness produced by S and M resins. There seems to be instead a tendency of prisms injected wet to have a lower strength than prisms injected dry; in fact a reduction of the 13% can be calculated on the mean strength values.
The results obtained on these prisms can be compared to those previously

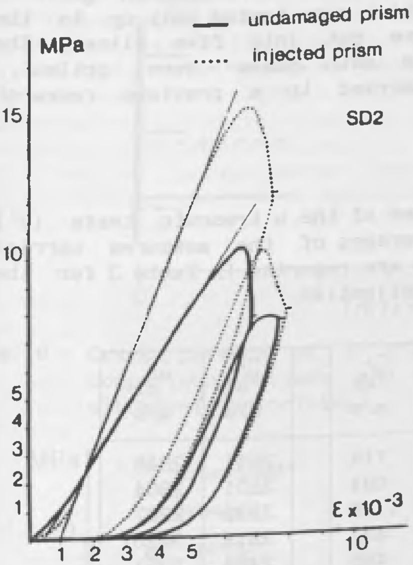


Fig. 4 - Stress-strain curves of SD2 prisms.

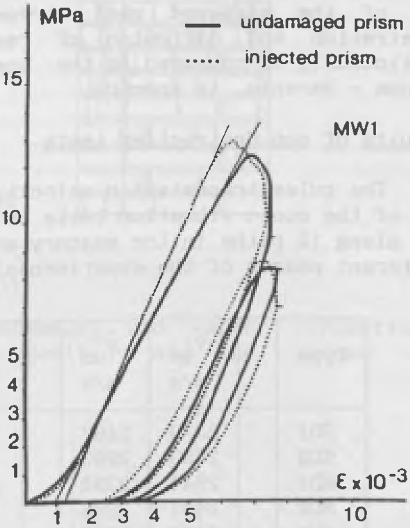


Fig. 5 - Stress-strain curves of MW1 prisms.

obtained on other prisms injected with a similar resin but with a less accurate injection technique (Binda - Baronio - Fontana, 1987), (Binda - Baronio, to appear). In the previous research the strength of injected prisms reached an average of only the 85% of the strength of undamaged prisms. The new results show, after injection, an increasing of 31.2% of the original strength and hence highlight the importance of a proper injection technique on the effectiveness of injection repair.
If the strength reached by the undamaged prisms in reloading is considered as the real residual strength of cracked prisms, then the improvement is even higher (43.4%).
The following results were also obtained from the compression tests:
- the modulus E of elasticity calculated between the 15 and the 60% of the peak stress has values ranging from 1700 MPa to 2470 MPa in the first loading of undamaged prisms.
- after injection the values of E ranges from 2528 MPa to 3350 MPa. There is an increasing of the 28% respect to the modulus in the first loading of undamaged prisms.
- the strain at peak stress varies between 4.73×10^{-3} and 6.5×10^{-3} for undamaged prisms and between 5.26×10^{-3} and 6.33×10^{-3} for prisms after injection.
- the onset of cracking varies between the 41.7% and the 90.0% of the peak stress for undamaged prisms and between the 70.8% and the 85.3% for prisms after injection. These last information show again a minor scattering of data in the case of injected prisms.

If the mechanism of failure of undamaged prisms (this can be considered the situation before injection in a damaged wall) is observed after reloading (Fig. 6, right), the crack pattern is clearly defined as expected under compression test (Hendry et al., 1981). The mechanism of failure of injected prisms shows a tendency of the cracks to be distributed outside of the areas which were cracked during the previous tests. This observation confirms that the injection operation has reached the goal. Two prisms SW1 and MD2, which after injection were tested only up to the 80% of the supposed peak stress, were cut into five slices. The penetration and diffusion of resin in both cases were optimal, particularly if compared to the ones observed in a previous research (Binda - Baronio, to appear).

Results of non destructive tests

The pulse transmission velocity values of the ultrasonic tests ($\bar{v}'$) and of the sonic vibration tests ($\bar{v}''$) (averages of the measures carried out along 12 paths in the masonry prisms) are reported in Table 3 for the different phases of the experimental investigation.

Type		$\bar{v}'_{ud}$ m/s	$\bar{v}''_{ud}$ m/s		$\bar{v}'_{da}$ m/s	$\bar{v}''_{da}$ m/s		$\bar{v}'_{ai}$ m/s	$\bar{v}''_{ai}$ m/s
SD1		2379	2467		697	714		2641	2548
SD2		2453	2263		885	684		2521	2804
MD1		2544	2398		1555	1168		2339	2267
MD2		2401	2206		589	432		2618	3420
SW1		2432	2292		383	485		2464	2391
SW2		2445	2339		784	615		2285	3429
MW1		2533	2319		947	844		2371	3502
MW2		2289	2186		519	629		2207	2228

where: $\bar{v}'_{ud}$, $\bar{v}''_{ud}$, $\bar{v}'_{da}$, $\bar{v}''_{da}$, $\bar{v}'_{ai}$, $\bar{v}''_{ai}$ are the mean velocities on undamaged, and damaged and prisms, after injection, as obtained from ultrasonic (') and sonic vibrational (") on tests.

Table 3 - Results of non destructive tests.

The ultrasonic velocity values are also correlated with the corresponding strength data in Fig. 7. A clear distinction between prisms in good conditions (undamaged and injected) and prisms damaged after the compression tests can be observed.
In Table 4 two comparisons are shown: the first, between the prisms in

Specimen condition		$\bar{\sigma}_{ud}/\bar{\sigma}_{da}$	$\bar{v}'_{ud}/\bar{v}'_{da}$	$\bar{v}''_{ud}/\bar{v}''_{da}$		$\bar{\sigma}_{ai}/\bar{\sigma}_{da}$	$\bar{v}'_{ai}/\bar{v}'_{da}$	$\bar{v}''_{ai}/\bar{v}''_{da}$
Dry		1.45	2.62	3.11		2.18	2.72	3.68
Wet		1.42	3.68	3.55		2.04	3.53	4.50

Table 4 - Ratios between mechanical strength and ultrasonic and sonic velocity values of the prisms.

undamaged and damaged conditions and the second between the prisms after injection and those damaged. The mean ratios between strengths and between velocities are calculated for both dry and wet prisms. These ratios could

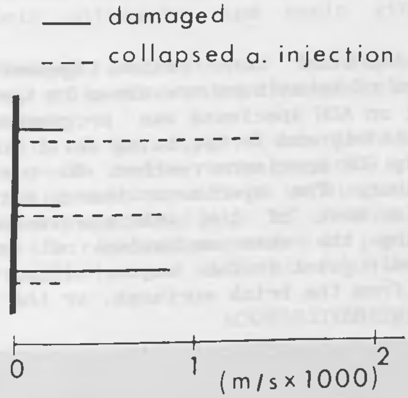


Fig. 6 - Crack pattern of a prism undamaged and after injection; comparison between the crack position and the values of ultrasonic velocities.

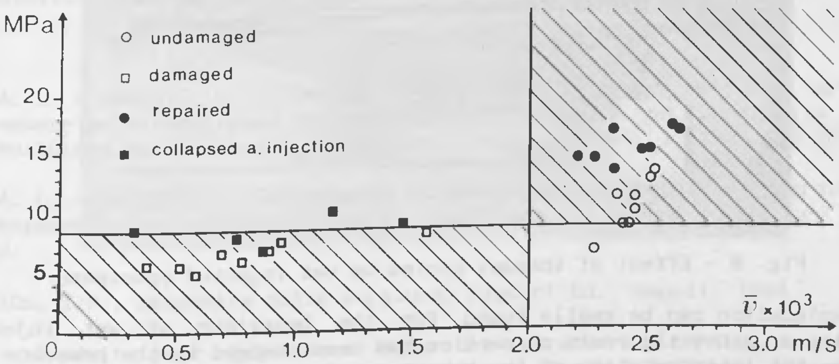


Fig. 7 - Correlation between strengths and velocities.

be assumed as "damage coefficient" in the first case (undamaged/damaged conditions) and as "amelioration coefficient" in the second case (after injection/damaged conditions).
For wet prisms a lower value of the amelioration coefficients concerning strength ($\bar{\sigma}_{ai}/\bar{\sigma}_{da}$) can be noted, probably because of the harmful presence of water in the masonry during the injection. On the other side the water caused an increase of the coefficients concerning velocities.
As general observation both ultrasonic and sonic vibrational velocities (v' and v") appear to be consistent and the results confirm the reliability of the pulse velocity values to define the global changement of material characteristics (Berra et al., 1988).
Furthermore the existing damaged situations and the effectiveness of the repair by injection are well characterized also at the local level. Fig. 6 (left) shows, in fact, a typical example of the very good correspondence

between the crack pattern of the masonry prisms before and after injection and the relative ultrasonic pulse velocities. The same type of result is given by the sonic vibrational tests.

Infuence of thermal cycles

The results obtained from the shear bond tests together with the number of cycles and the classes of behaviours are shown in the right hand side of Table 1. The aging test on ADH specimens was programmed to last for a number of cycles between thirty and forty, being still unknown the behaviour of the specimens. Only SDi specimens reached 40 cycles, still displaying an acceptable behaviour. The specimens named MDi were all collapsed after 8 cycles, while most of the wet specimens collapsed between 6 and 8 cycles following the same mechanism of failure. The failure took phase along the resin joint in two ways: either the resin joint was completely detatched from the brick surfaces, or the resin had detatched part of the brick (Fig. 8).

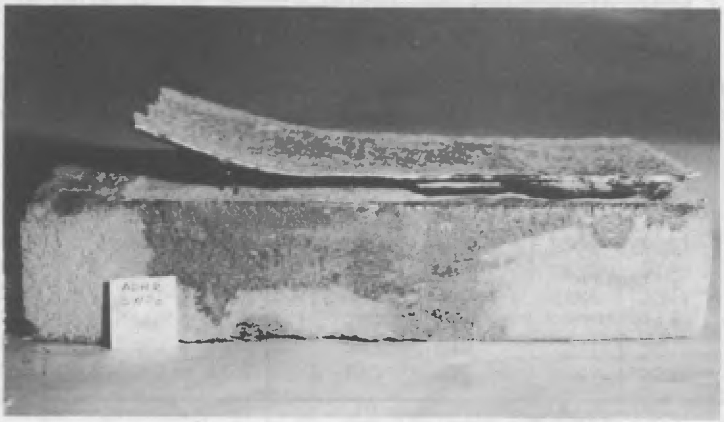


Fig. 8 - Effect of thermal cycles on wet injected specimens.

An explanation can be easily found for the behaviour of wet injected specimens, since the resin properties had been changed by the presence of water; the interpretation of the behaviour of specimens MDi is certainly more difficult. At the moment the only possible comments to the results can be as follow: (i) the behaviour of the injected material depends on the mechanical and physical properties of the components, particularly on the thermal expansion coefficients, strength, deformability and their variation according to the different temperatures, (ii) the presence of water on the surface of cracks reduces the bond strength of the resins, and also affects their behaviour under thermal cycles.

CONCLUSIONS

The following conclusions can be drawn:
- The effectiveness of injection resins in strengthening of decayed masonry structures is strictly related to the technique procedure. Care has to be taken, during the injection to maintain a constant pressure and a continuous feeding of the material.
- The injection pressure should be kept no higher then .06 MPa in order to obtain better diffusion and penetration of the material.

- The reliability of non destructive tests as a mean to evaluate in situ the degree of damage of existing masonry and the effectiveness of the repair by injection has been proved through a comparison between velocity values and mechanical strength values. For the masonries studied both ultrasonic and sonic vibrational measures gave good results.
- When the damaged masonries to be injected are in wet conditions, problems may arise concerning the bond strength and the durability under thermal cycles.
- The durability of the repaired material under thermal cycles even in dry condition depends on the resin physical and mechanical properties.

In the future the authors will be working at the solution of two main problems: (1) the compatibility of resins with wet support; (2) the durability of the repaired masonries in particular temperature conditions, (3) the interpretation and elaboration of data from n.d. tests.

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APPLICATION OF CEMENT GROUTING TO HISTORIC BUILDINGS

R. Zepnik and W. Schönbrodt-Rühl

This article deals with the application of cement grouting to four historic buildings in the district of Dresden. The stabilization of masonry at the Dresden palace "Palais im Großen Garten" will be reported on as our first example. The prevention of further settlements at the castle "Albrechtsburg" in Meißen and at the castle "Ortenburg" in Bautzen by subsurface grouting are the next examples to be dealt with. Finally we tell about the renovation of the fountain "Mosaikbrunnen" in Dresden. The foundation and the fountain basin were reconstructed by cement grouting.

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02.06.1989

STABILIZATION OF MASONRY AT THE "Palais im Großen Garten"

At the end of World War II the palace was severly damaged by bombs. The situation at the north side was especially dangerous. The thrust of a wide-spanned vault caused the wall to rotate outwards. The masonry was bowed some 25 cm. Two pillars made of quartzitic sandstone are situated in the centre of the wall (see fig. 1). Each of them is loaded with about 1,800 kN from the dead weight of the building. The original cross -

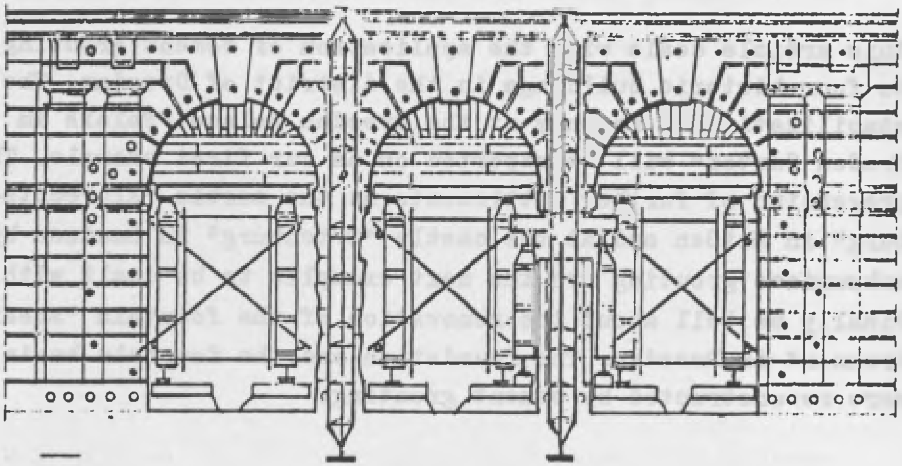

Figure 1: Dresden palace "Palais im Großen Garten", north front

sectional area of one pillar amounted to 1.37 m^2. It had been reduced to 50 per cent due to fire. In the early fifties the arches beside the pillars were filled with masonry to prevent the building from collapsing. The pillars were to be replaced by new ones but the essential problem was to assure the stability of the wall as a whole. It was decided to groute the voids by cement-clay-water-mixture before changing the pillars. The joints were filled by cement-lime-mortar to a depth of 80 mm. 154 holes were drilled by a wet type rock drill. In the ground floor reinforcing rods of 14 mm diametre were inserted into the holes as anchorage to connect the wall and the vault. About 3.4 tons Portland cement (PZ 1/475) were used. The works were carried out in 1984. Until today there have not been any clues yet that the cement stone and the old masonry would not be compatible. As there is no heating in the building and the palace is to be used only in summertime we do not expect cristallisation of sulphate at the sandstone facing.

SUBSURFACE GROUTING AT THE CASTLE "Albrechtsburg" IN MEISSEN

The castle "Albrechtsburg" in Meißen near Dresden is an unique historic monument. It was built at the ent of the 15th century. The retaining pillar at the south-west gable-front (see figure 2) was erected in 1864 because the stability of this wall had been endangered even at this time. But this method is generally not suited for this purpose because settlements occur under the new pillar foundation and the weight of the pillar additionally loads the wall. The formation of cracks went on slowly. Since 1984 an considerable increase of the crack widths between the end-wall and the ceilings has been observed. At the beginning of the analysis we supposed that thermal stresses have caused the displacements. In the architectonic description the depth of this foundation was said to be about 6 m. An exploration of the pillar support was carried out. The digging point (see figure 3) showed that the foundation was absolutly insufficent. A leaking water-pipe was the real cause for the deterioration during the last years. The fine grain of the soil had been washed off.

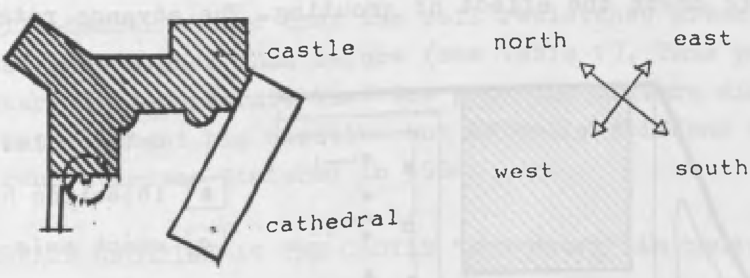

Figure 2: Plan Albrechtsburg

The foundation was reconstructed by cement grouting. Investigations were carried out before. The concentration of sulphate in the masonry amounted to 0.33 % and the pH-value was 8.4.

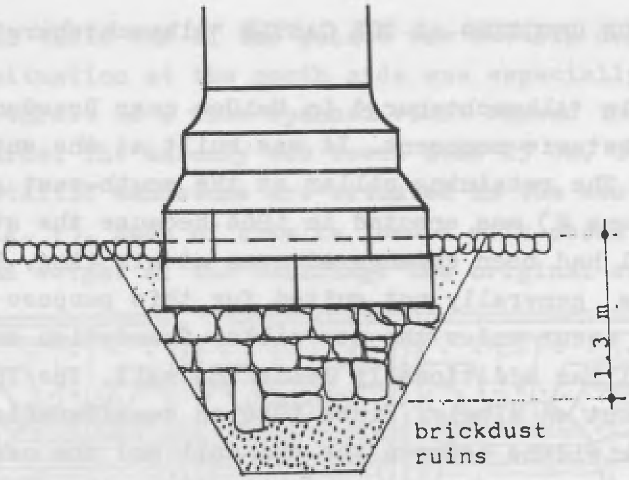

Figure 3: Pillar foundation

The mixture consisted of cement, fine sand, clay and water. Four drill holes of 55 mm diametre at an angle of 30 degrees to the vertical were made until the maximum depth of 4 m. Fourteen grouting lances of 33.5 mm diametre were driven in by pile-hammer until the depth of 5 m (see fig. 4). 16.4 t cement and 5.2 t fine sand were used. After hardening of the cement two further holes were drilled and two lances were rammed to check the effect of grouting. The advance rates of

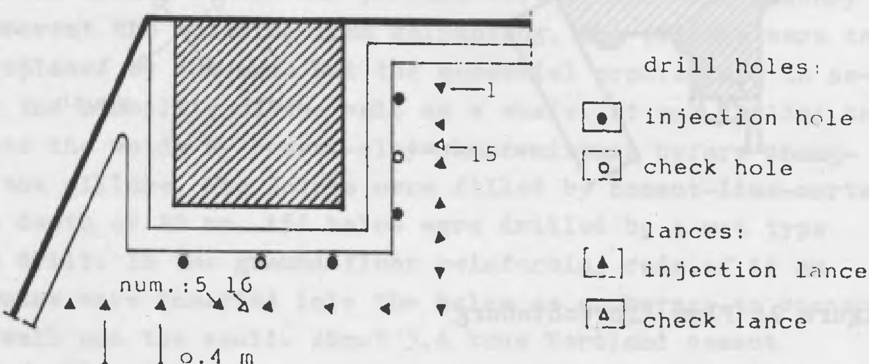

Figure 4: Plan of the pillar foundation,
Location of drill holes and injection lances

Table 1: Advance rates of the pile-hammer before and after grouting

DEPTH FROM - TO (metre)	TIME (second) before grouting lance 1	before grouting number 5	after grouting lance 15	after grouting number 16
0 - 1.0	7	5	7	7
1.0 - 2.0	11	10	18	16
2.0 - 3.0	12	14	90	95
3.0 - 4.0	42	48	-	-
4.0 -	61	122	-	-
MAXIMUM DEPTH (metre) no further advance	4.7	5.0	2.2	2.9

the pile-hammer showed that the soil resistance after grouting was much higher than before (see Table 1). This proof was necessary to demonstrate that the grouting mixture did not pour into distant big cavities but actuelly stiffend the soil. This renovation was finished in 1988.

SUBSURFACE GROUTING AT THE CASTLE "Ortenburg" in bautzen

The north-eastern part of the castle has displaced as a consequence of water influence. The walls were damaged in such a degree that in this part of the building the security of visitors could not be guaranteed (see figure 5). In preparation of the cement injection the geological situation under the foundation was explored by experts. Tests were

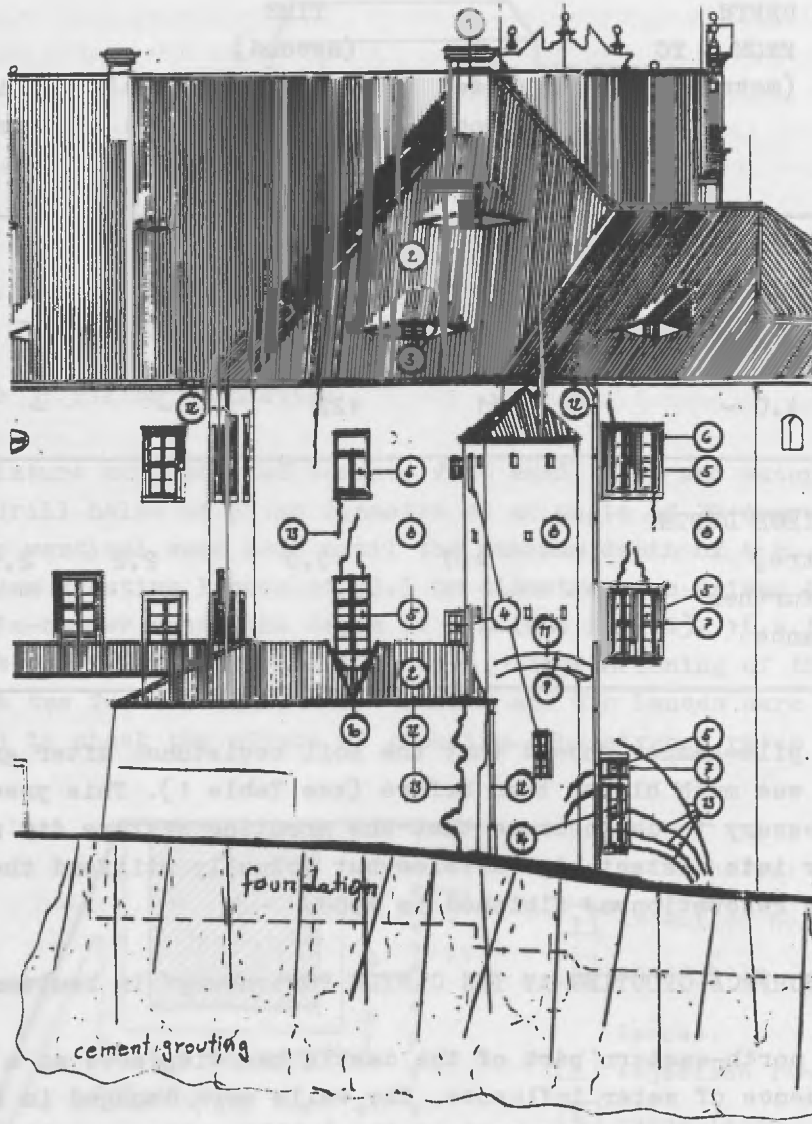

Figure 5: Ortenburg, elevation of the north-eastern part

carried out to investigate the concentration of sulphate in the ground and the pH-value of the soil. The subsurface grouting was carried out in 1987 to prevent further settlements. 83 holes of 45 mm diametre were made by pile-hammer (total length 410 m) and rock drill (total length 35 m). The greatest depth amounted to 7 m. The composition of mixture was as follows (volume parts): 10 parts cement, 1 part fine sand (o ... 0,8 mm), 1 part clay and 8 parts water. The total volume of grouting mixture was 198 m^3. The works proofed to be succesful. No further settlements were observed.

CEMENT GROUTING AT THE FOUNTAIN "Mosaikbrunnen im Großen Garten" in Dresden

On occasion of the great horticultural show in Dresden in 1926 several fountains were built. They were temporary constructions and should be used for some months only, However, one of them remained working until 1980. The fountain basin is covered with mosaics of coloured glass. This architectonic gem is to be preserved although the structure is dilapidated. The fountain basin is standing on eleven single footings. There were many cracks in the basin consisting of a 5 to 8 cm thick reinforced concrete shell and a 2 cm thick mortar layer in which the mosaics were pasted. Many reinforcing rods are rusty because they are not covered by concrete. While erecting the fountain the coarse basin shape was made of loamy sand and the concrete directly placed on it. Settlements occured resulting in a 2 cm high cavity under the whole shell which now became a load-bearing element. The renovation of the foundation and of the basin itself was carried out in 1988. The soil under the foundation level and around the single footings was stiffened by grouting of cement suspension. Eleven injection lances of 33.5 mm diametre were rammed until the depth of 2.3 m. The cement consumption amounted to 1.4 tons. The cavities under the concrete shell were also filled with cement suspension. 43 pipe nozzles were put into drill holes of 25 mm and were fixed by cement mortar (see figure 6 and 7). The impermeability to water and the load-bearing capacity are restored. However, the remaining service life is limited because it was not possible to insert supplementary reinforcing rods.

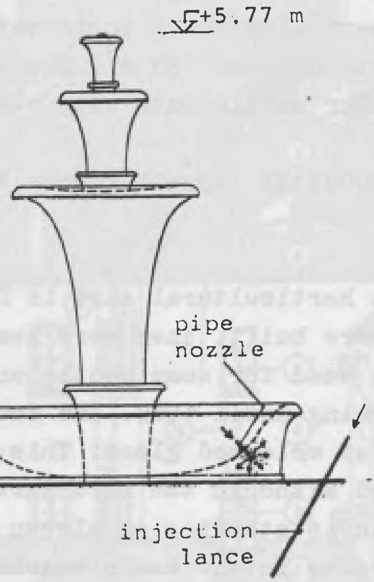

Fig. 6 Fountain "Mosaikbrunnen"

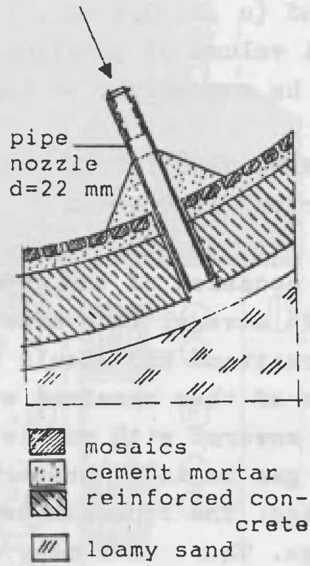

Fig. 7 Reinforced concrete shell of the basin

SUMMARY

The four examples showed to which kind of tasks we are able to apply the method of cement grouting. As any other method were more expensive or even there was not any other alternative of injection technique thismethod is very important for our work.

ARTA BRIDGE: INVESTIGATIONS AND STUDY OF STRUCTURAL BEHAVIOUR

Mr P. Plaines *

SUMMARY

In August 1983 the old stone bridge at Arta (Epirus-Greece) was in risk of serious damages due to underscouring, which was the result of irregular flows of the river Arachthos, over which the bridge is built.

To protect the bridge, remedial measures were decided immediately and first stage protection works were executed. Permanent protection works for the river channel and bridge pillars followed and completed the first stage works [1].

Parallelly with the execution of the protection works, a limited investigation programme was carried out. As part of this programme, Historical-Archaelogical data were collected, site and soil investigation works were carried out and structural analyses were performed to provide the basis for the assessment of the structural damages (which occurred prior to scouring) and lead to proposals for repair of the bridge [2].

The results of the first stage of this study proved the need for an extended investigation programme in the area of the west abutment, the findings of which provided additional data for the foundation of the bridge.

This report aims to present the main features of the structural analyses and their correlation with the actual damages of the bridge, as well as with the results of the in-situ investigations.

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HISTORICAL DATA - DESCRIPTION OF THE BRIDGE

The Arta bridge over the river Arachthos is located at the west part of the city of Arta.

Its construction is placed in the 13th century although it is probable that the bridge is founded on structures belonging to the classic or hellenistic period.

According to some information, part of the bridge collapsed and was rebuilt in the beginning of the 17th century [3], [4].

The present form of the bridge has been maintained since at least the beginning of the 19th century (as displayed on a copperplate print of 1813 and a lithography of 1855).

Photographic documentation of the bridge starts from 1897. From 1962 on, the bridge is used for pedestrian crossing only.

The bridge of Arta (see fig. 1, South elevation), has four main arches of 23.95, 15.83, 15.43 and 16.16 m spans respectively. At each pillar there are arched relief spillways through which water flows during floods. Below the spillways the pillars are widened to 11.5 m whilst the superstructure overall width is 3.70 m and the net deck width is 3.0 m approx.

The foundation of bridge is made of limestone block stones 0.5 to 1.5 m long, while stones of 0.20 m average thickness were used at the arches. On all visible surfaces hydraulic mortar was used. Stones of irregular shape are seen on the drum walls. The interior of the drums is filled by limestone rubble of various sizes without mortar (fig. 2). The bridge is founded at varying levels (fig. 3).

BRIEF DESCRIPTION OF STRUCTURAL DAMAGES

The historical-archaelogical documentation, the detailed geometrical survey of the bridge, the mapping of the damages and the monitoring of their development with the time together with the at-site investigations yielded new data as

- Remainings of an old masonry were located in the channel between the piers (fig. 3).
- Rotation of and discontinuity at the west springer of the longest arch were noticed (fig. 4).
  The above discontinuity (given also that the west approach appears to have been non-uniformly built) is likely to be related with the serious damages of that region
- Some of the major superstructure cracks, near the west embankment were noticed on some 1955-60 photographs. By comparison it was found that the crack width increased from 10 to 16 mm in the 30 years elapsed (fig. 5).
- In some other photographs of the same period the second relief arch is seen to be filled with rubble stones (as it

is today). This operation, apparently, was made necessary due to rupture (fig. 6) which however happened at a time unknown today. In the lithography of 1855, mentioned earlier, the said relief arch is displayed open. The masonry which fills the arch is cracked in a way of projection of the arch cracks. This fact indicates that the cause of damage of the arch remained, even after the filling operation.

- At the keystone of the long (west) arch there is a compressive rupture at the underside and a 30 mm wide crack at the upperside (fig. 8). This crack does not extent to the drums, it appears, however, also in the barrier (which was built later). This fact which suggests that the cause of the damage existed after the construction of the barrier.

INVESTIGATION OF STRUCTURAL PERFORMANCE

Introduction

The aim of this phase of the study was to reproduce numerically the response of the structure under actions imposed to the bridge in the past, or likely to be imposed in the future.

The structural response can be expressed in terms of stresses developed, safety factors against various failure modes or via possible damages which could have occurred as a result of the imposed actions etc. The interpretation of the damages actually occurred is one of the major targets of the present investigation.

The available data for the Arta bridge were rather limited (incomplete record of the construction phases and the corresponding loading conditions) and, futhermore, the urgency under which the study was carried out did not permit thorough investigation of the strength characteristics of both materials and masonry. Thus, in several cases the results of the analyses should be considered as quantitative.

Two groups of analyses were performed

a. Transverse stability analyses and analyses of single arches by applying conventional static methods
b. Investigation of the longitudinal response by means of finite elements techniques.

Stability Analyses

The main conclusions drawn can be summarized as follows:

a. The average bearing pressures is 4.8 MPa (pillar M1). The bearing pressures can vary, due to geometrical eccentricities by ±1,0 MPa approx.

b. Stability analyses in the transverse direction assuming critical flood conditions (from uncontrolable overflow of the dam without water discharge through the flooded approaches) resulted in safety factors 3.1/2.8/2.0 at pillars M1, M2 and M3 respectively. At the arch springers, the corresponding safety factors become 8.0/7.1/3.8 respectively.

c. The safety factor against overturning at the floor level of the relief arch assumes the limit value of 1.0 under earthquake loading $F_h = \varepsilon Q_v$ when the earthquake coefficient is $\varepsilon = 0.51/0.46/0.42$ for the pillars M1, M2, M3 respectively.

Approximate analyses of single arches

In this part of the study, the main arches were considered isolated: No connection was assumed between arches and drums under the permanent vertical loads.

The analysed arches were discretized into segments, at the ends of which, the magnitude, eccentricity and the slope of the transmitted forces were calculated (fig. 9).

Several admissible thrust lines through the springer and the key stone were attempted.

The extreme thrust line position is the one which led to the maximum allowable compressive stresses (30 MPa appr.).

The basic conclusions are briefly presented herebelow:

- Under normal loading conditions, the line of thrust passes within the kernel of the section and hence the stability of the arch is secured.
- Stability is maintained even under the extreme deviation at the keystone.
- The shear stresses developed at the joints are small compared to the friction forces which can be mobilized.
- Even under extreme loading condition there is a possibility for geometric readjustment of the arches without violation of the external equilibrium.

Finite Element analysis of the Bridge

Linear elastic finite elements were employed to analyse the structure. It was assumed that the bridge is symmetric w.r.t. the axis of the central pillar and, thus, only half the bridge was analysed.

The simulation of the structure was based on the assumption that the drums do not carry vertical loads, although they are acted upon by imposed deformations. The above assumption was adopted on the basis of the actual failure modes and found to be in good agreement with the results of the relevant calculations.

The different parts of the structure (according to their construction pattern) and the foundation soil around the pillars were introduced in the model with their appropriate engineering properties.

The initial model was based on foundation data which were available at that time. From the evaluation of the first results however it was indicated that the damages in the region of the west abutment could be explained on the basis of the stress resultants under the combined action of permanent loads and a differential settlement at the west approach.

Further to this conclusion, test trenches were dug and showed that the approach was founded at a shallow depth on a deformable clayey layer.

The same test trench revealed clear signs of differential settlement in the area of the filled arch (K2 in fig. 9).

In the light of the new evidence, the structural model was improved to its final (fig. 10) form and analysed under "normal conditions" i.e. permanent vertical loads and uniform foundation conditions, with imposed differential settlements at the pillars (parametric analysis) together with horizontal seismic loads.

Although the construction sequence of the bridge and its loading history are not known, the appropriate combination of some probable cases identified the vulnerable points of the structure which coincided with the points where serious damages actually appeared. At these points, and in particular above the arches K1, K2, K3, combinations of loads and settlements result in significant tensile stresses normal to the existing cracks (fig. 11).

In summary the analyses shown the following

- Above the "broken" arches K2, K3, tensile stresses develop even under vertical loads only
- Pillar settlements greater than those of the abutments should be excluded as in such a case compressive stresses would develop at places where wide cracks were observed.

The representative deformation mode which reflects all the major damages is settlement of the west approach and rotation of the west abutment to the left.

CORRELATION OF THE STRUCTURAL ANALYSES WITH THE OBSERVATIONS AND INVESTIGATIONS

The results of the site investigation, the laboratory tests and the structural analyses yielded the likely causes of damage, (which can be eliminated by appropriate interventions) and located vulnerable areas which can be repaired or strengthened.

Furthermore, another important contribution of these investigations became apparent, which is to assist the Historical and Archeological investigations, concerning the Arta bridge.

In fact, as described above, the first results of the study revealed some building features which were previously unknown.

The entire results of the structural analyses can provide the background for assumptions related to the original shape of the bridge, assumptions which, of course can be later confirmed or rejected by Archeological investigation.

Indeed,

- The sudden narrowing of the channel at the location of the bridge to the west
- The lack of symmetry of the main arches (unusual in stone bridges)
- The form of the "west abutment" which is the same with that of pillars M1, M2 and totally different from that of the East abutment.
- The illogical formation of the three arches of the west approach
- The unskilled building of the west approach at which tensile stresses develop even under vertical loads.
- The foundation of part of the west approach on clay, whereas all the other piers are safely founded on strong soils.
- The outlook, building pattern and the general condition of the drum wall could suggest that the main arch was the central arch (ref. par. 3) and the reported partial collapse and reconstruction in the early part of the 17th century would actually be collapse of main arches which were unsuitably replaced by the small arches and the walls of the west approach as it is today.

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The Investigations, studies, underpinning and protection works were co-ordinated by the joint committee of representatives of the Ministry of Public Works (A. Iakovidis, D. Koroneos, S. Christoulas) and the Ministry of Culture (T.P. Tassios, E. Delinicola, N. Miltiadou).

The corresponding works were carried out by the Consulting firm "Group of Technical Studies". Detailed data are included in the relevant design documents.

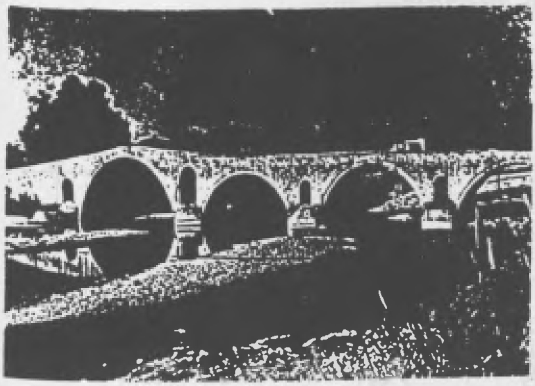


Fig. 1

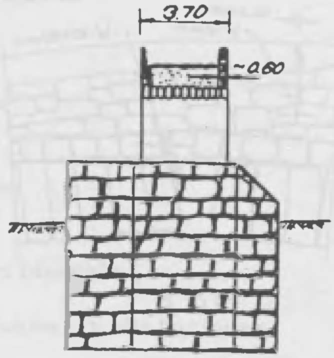


Fig. 2

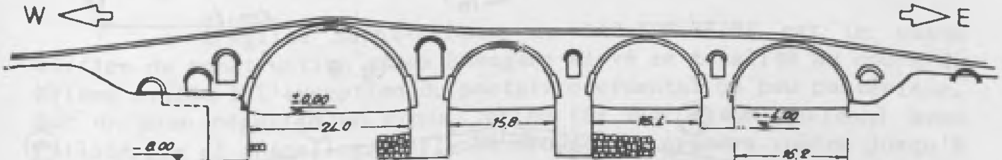


Fig. 3

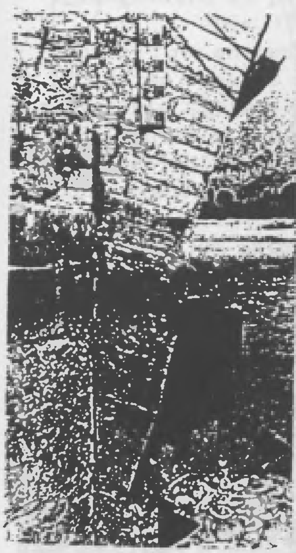


Fig. 4

Fig. 5

Fig. 6

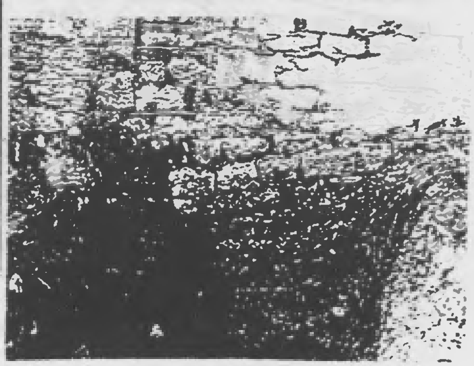


Fig. 7

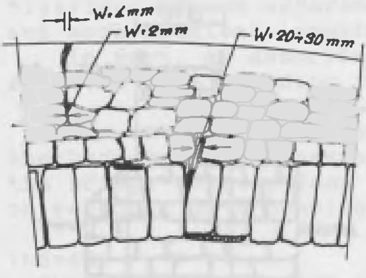


Fig. 8

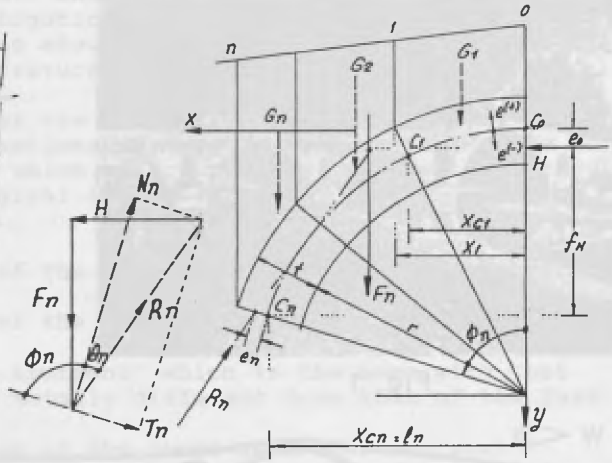


Fig. 9

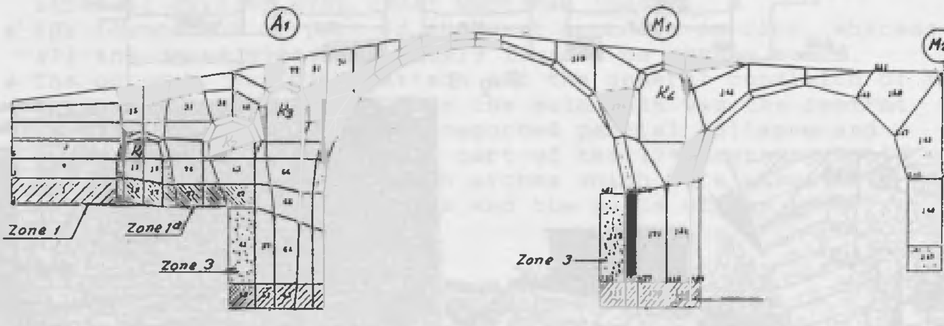


Fig. 10

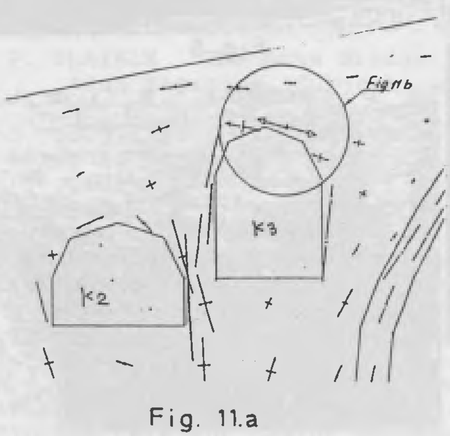


Fig. 11.a

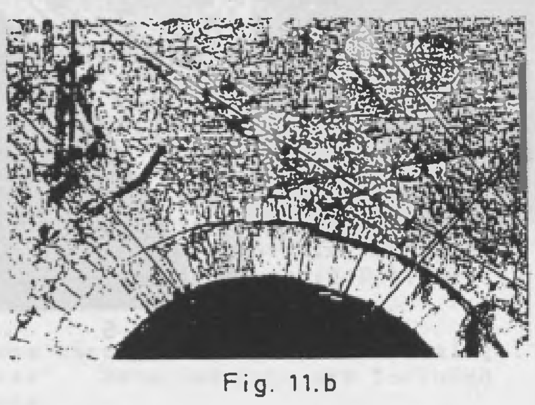


Fig. 11.b

AUBE
BAR-SUR-SEINE
Eglise Saint-Etienne

Restauration de la rose occidentale

J.M.MUSSO - Architecte en Chef des Monuments Historiques

L'église Saint-Etienne de BAR-SUR-SEINE est un vaste édifice de construction assez homogène élevé en totalité au cours du XVIème siècle à l'exception du portail occidental un peu postérieur. Sur un plan régulier en croix latine (de 60m x 40m environ) avec collatéraux et chapelles, elle développe ses grandes voûtes jusqu'à 20m de hauteur. La façade occidentale présente un grand mur nu encadré de deux contreforts puissants, surmonté d'un pignon triangulaire et percé d'une grande rose ajourée.

Dès 1984, la restauration des parties hautes de cette façade fut entreprise en prévision de la restauration de l'orgue situé au revers ; celle-ci présentait en effet d'importants désordres (grand coups de sabre sur toute la hauteur, tassements différentiels, désorganisation à peu près complète de la rose) dûs vraisemblablement à certaines faiblesses de fondations n'évoluant plus depuis quelques décennies, mais plus encore à la vétusté générale accentuée par les rigueurs d'un climat humide et froid à l'encontre de matériaux fragiles employés dans cette région.

L'état de dégradation de la rose en particulier nécessitait une intervention complexe : les remplages, constitués d'un faisceau de seize colonnettes cannelées avec base et chapiteau, s'appuyant au centre sur un quadrilobe ajouré et prolongées par des arcatures en plein-cintre entrelacées, apparaissaient dans leur presque totalité fissurés, éclatés, minés par le gel et par la rouille ; en effet, de grandes pièces de ferraille entrecroisées renforçaient la rose en sus des goujons, chaînages et barlotières d'origine, l'ensemble dans un état de corrosion et de pulvérulence accentué, provoquant l'éclatement de très nombreux éléments de pierre. Par ailleurs, d'importantes déformations avaient affecté la structure au point de réduire le diamètre vertical de la rose de 30cm par rapport à son diamètre horizontal (5,50m environ), par affaissement des parties supérieures et accentuation du coup de sabre de la façade.

LA RESTAURATION

Celle-ci se décompose en deux parties :

I - Le mur de façade.

II - La rose.

I - Le mur de façade

La restauration des maçonneries du mur occidental fut réalisée selon les techniques habituellement appliquées sur les monuments classés : remplacement des parements dégradés en pierre neuve de constitution similaire sinon identique à la pierre d'origine, rejointoiement au mortier de chaux, injection de coulis de chaux pour consolidation interne des maçonneries, remaillage des fissures et coups de sabre avec renforcements par épingles en acier inoxydable, etc.

En vue d'annuler les poussées latérales provoquées par la masse du pignon s'écrasant sur la rose, un fort chaînage en béton armé, calculé et constitué pour travailler en arc de décharge, fut coulé à la base de celui-ci en arrière du parement de pierre de taille.

II - La rose :

La rose nécessitait un type d'intervention plus particulier du fait de sa structure très ajourée et des efforts dûs aux effets du vent.

Si d'une manière générale on cherche à conserver le maximum d'éléments d'origine, surtout lorsqu'ils sont sculptés ou décorés, ici s'imposait la purge et le remplacement systématique de toutes les parties malsaines ou seulement douteuses pour ne conserver que les seuls éléments parfaitement sains, ceci afin de garantir la plus grande homogénéité de l'ensemble du réseau de pierre, et éviter tout risque de rupture d'élément dans l'avenir.

Par ailleurs, afin de parer à tout phénomène de corrosion importante, les goujons, chaînages et barlotières ont été réalisés en cuivre, en ayant soin de doubler les sections du fait de la moindre résistance mécanique de ce matériau, toutes les pièces de renforcement parasites étant éliminées.

Restait une question importante : cette structure, épurée de tous renforts externes, pourrait-elle résister à la poussée du vent, alors même qu'elle s'insère dans une façade occidentale très exposée ? Certaines pièces de renforcement en ferraille, aujourd'hui supprimées, n'auraient-elles pas été mises en place sinon dès l'origine du moins très tôt, après les premiers signes de faiblesse ?

Il fallait éviter la repose de toute structure métallique visible (en fer les problèmes de corrosion seraient inévitables surtout à cette hauteur, en cuivre les sections seraient énormes), tout en assurant une résistance accrue des remplages de la rose. C'est ainsi que se développa l'idée de traiter celle-ci comme une roue de charrette, par cerclage périphérique.

Une étude particulière fut entreprise en étroite collaboration avec Messieurs BANCON, Ingénieur Conseil Expert et XERCAVINS, Ingénieur spécialiste en précontrainte, ainsi qu'avec le C.E.B.T.P. (Messieurs BOUINEAU et CHAGNEAU), portant sur l'analyse de la rose, ses caractéristiques et sa stabilité, puis sur la définition d'un projet de mise en précontrainte.

L'analyse :

Celle-ci a montré que la stabilité sous poids propre dans le plan vertical ne pose pas de problèmes particuliers (efforts de l'ordre de 3 bars). La stabilité aux effets du vent est, par contre, beaucoup plus préoccupante : la flexion des réseaux engendre des contraintes de l'ordre de 14 bars, chiffres acceptables en compression mais pas en traction surtout au niveau des joints. Le dessin des remplages conduit de façon très naturelle les efforts du centre vers la couronne, mais un point de faiblesse apparaît à l'intersection des arcatures au point qu'il fut décidé de renforcer la section des barlotières correspondantes. Par ailleurs, le noyau central en forme de quadrilobe, du fait de ses sections très variables, fut renforcé lui aussi par un anneau en cuivre destiné à répartir la pression des seize colonnettes. Enfin, l'analyse des effets de la poussée du vent a débouché sur deux hypothèses de fonctionnement de la structure étudiée :

a - décomposition des efforts horizontaux en bielle oblique qui engendre des poussées radiales importantes dans le plan du pignon (de l'ordre de 700 kilos par colonnette)

b - flexion de la structure de la rose qui engendre des efforts de flexion dans les membrures et une simple réaction annulaire tangentielle en feuillure de berceau.

La première solution nécessite une cohésion massive importante de la part des maçonneries enserrant la rose, ne tolérant aucune défaillance. C'est pourquoi c'est la deuxième solution qui a été retenue pour l'élaboration du projet, consistant à conférer à la rose une résistance à la flexion suffisante en elle-même.

Des tests effectués en laboratoire ont permis de mieux connaître la résistance des pierres employées, neuves et anciennes, celle des mortiers et celle des joints avec ou sans goujons ; en particulier le calcul théorique de transmission de la flexion par des goujons fut confirmé.

Le projet :

La mise en compression de la rose s'effectue par deux systèmes associés :

1 - Cerclage périphérique.

2 - Expansion du noyau.

Des mesures de contrainte et de déplacement latéral seront effectuées avant précontrainte, puis en cours et en fin de mise en compression, la rose étant soumise à des efforts transversaux simulant les effets du vent, afin d'analyser le comportement de celle-ci à vide et à différents niveaux de pression et de compression.

1 - Cerclage périphérique :

Celui-ci est constitué de deux cerces placées de part et d'autre du plan de symétrie vertical de la rose, c'est-à-dire du plan du vitrage ; il s'agit de deux câbles d'acier monotorons T 15 graissés et protégés par une gaine en polyéthylène, l'ensemble étant logé dans deux saignées engravées dans la feuillure de berceau recevant la rose. La mise en place des cerces et le bourrage des tranchées autour des gaines sont exécutés avant le remontage de la rose. Les câbles sont ancrés dans un massif de butée de type "ancrage passif" logé en partie basse dans un évidement ménagé pour ce faire dans le parement intérieur du mur de façade.

2 - Expansion du noyau :

Afin d'assurer une meilleure répartition des efforts de précontrainte dans l'ensemble des remplages, il est apparu nécessaire de dilater l'anneau central. Celui-ci est formé de quatre éléments de pierre en quart de cercle. Des vérins plats, de type Freyssinet, sont logés par paire dans chacun des quatre joints ; ils sont réalisés en cuivre; diamètre 70mm, épaisseur à vide 9 à 10mm ; des plaques de plomb de part et d'autre des vérins répartissent les pressions au contact de la pierre.

L'exécution :

Après mise en place des cerces inertes, la rose fut remontée en totalité, hourdée au mortier de chaux hydraulique ; afin de laisser libres les quatre joints du noyau destinés à recevoir les vérins, le quadrilobe central fut provisoirement étrésilloné par deux parois de brique mince. Pour procéder aux essais de simulation du vent, un dispositif fut mis en place en avant et en arrière de la rose : une double structure primaire de poutre HEB solidaire du mur de façade constituant le point d'appui des vérins destinés à simuler la poussée du vent d'une part, une double structure secondaire en forme d'octogone constitué de poutres IPN et s'appuyant sur les remplages par l'intermédiaire de patins en plâtre d'autre part.

Quatre comparateurs mesuraient les déplacements de la rose et de nombreuses jauges à billes les déformations et les contraintes des éléments de pierre en traction comme en compression selon les efforts appliqués.

Les essais à vide, c'est-à-dire câbles détendus et vérins plats, effectués par paliers progressifs jusqu'à atteindre 1.200 Kg de poussée sur la rose, valeur retenue au calcul initial pour la simulation du vent, ont provoqué un déplacement de l'ordre de 1mm. Si ce résultat peut laisser croire en la stabilité de la rose sans appoint, il ne faudrait pas sous-estimer le rôle important joué par les parois de brique du quadrilobe et par la double structure octogonale d'IPN partiellement liée aux remplages qui, ensemble, renforcaient très sensiblement ceux-ci. La preuve en est qu'après mise en précontrainte définitive, les mesures effectuées après suppression des parois de brique ont donné un déplacement presque double de celui relevé en présence de celles-ci (0,34mm avec les parois pour 0,52mm sans parois).

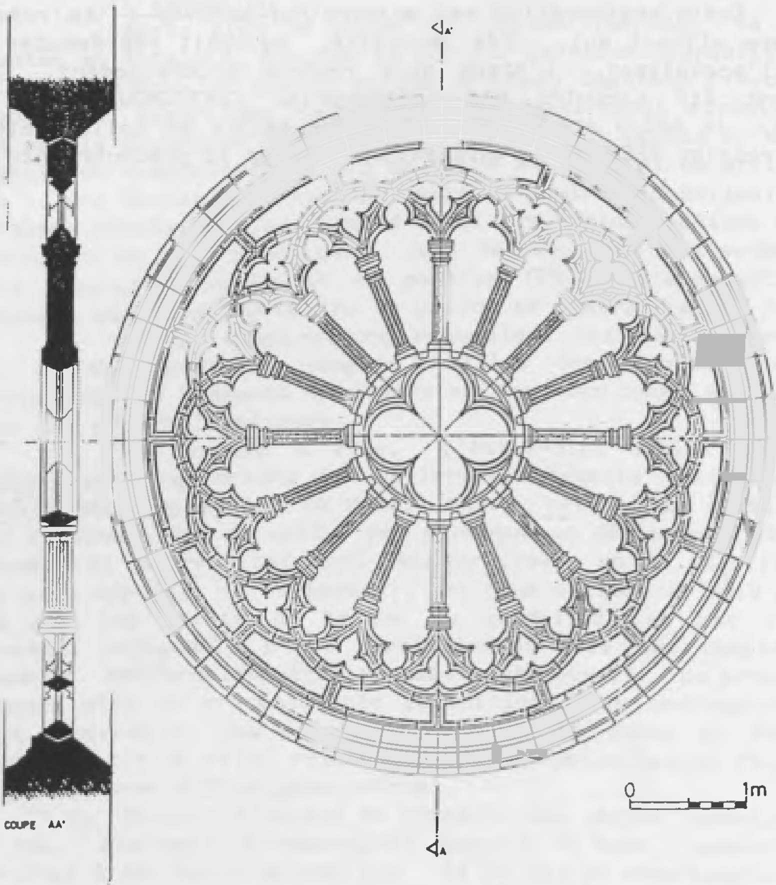
Plusieurs essais de pression des vérins furent effectués à l'eau, par paliers successifs jusqu'à 70 bars, avant blocage définitif à la résine époxydique. Il en est de même pour la mise en tension des câbles qui fut portée pour chacun à 2,5tonnes. On doit signaler l'extrême sensibilité de cette paroi ajourée en pierre qui se déplaçait de quelques dixième de milimètres d'avant en arrière au fur et à mesure que l'on tendait alternativement les deux câbles.

L'ensemble des mesures effectuées en cours d'opération a justifié l'intérêt de la double intervention au centre et en périphérie de la rose : les mesures ont montré la répartition progressive des contraintes à travers tous les éléments de la rose, lorsque les deux systèmes ont été en place, ce qui n'était pas le cas lors de la seule mise en tension des câbles ou de la seule mise en pression des vérins.

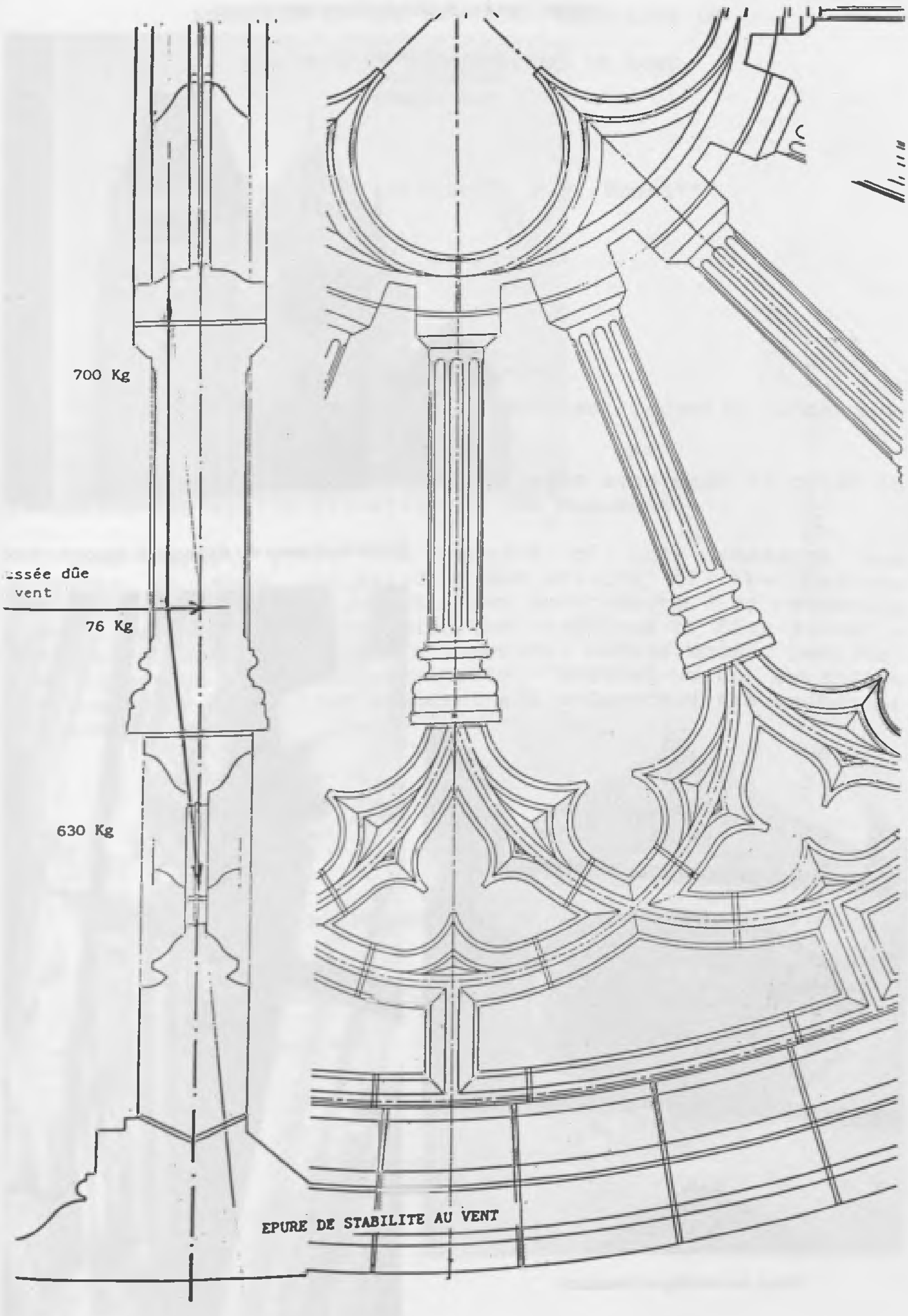
Pour ménager l'avenir un logement a été réservé entre les deux vérins dans chaque joint du quadrilobe afin de permettre la mise en place éventuelle d'un autre dispositif, ou même la décompression complète de la rose. Quant aux câbles, ils furent recépés assez loin des ancrages pour pouvoir à nouveau y exercer une traction ou les détendre. Enfin, une plaque gravée, placée prochainement au bas de la rose explicitera l'ensemble du dispositif afin de prévenir toute fausse manoeuvre dans l'avenir.

Cette restauration est aujourd'hui achevée ; la rose a retrouvé son vitrail qui, très lacunaire, pourrait représenter le thème de l'apocalypse. L'orgue sera remonté prochainement. Les travaux ont été exécutés par l'entreprise CHATIGNOUX pour la maçonnerie, le béton armé et la pierre de taille en collaboration avec l'entreprise FREYSSINET INTERNATIONAL pour la précontrainte.

AUBE - BAR-SUR-SEINE - Eglise Saint-Etienne



Relevé : entreprise CHATIGNOUX et J.M.MUSSO, Architecte en Chef des Monuments Historiques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AUBE - BAR-SUR-SEINE - Eglise Saint-Etienne

Relevé de l'Agence des Bâtiments de France de l'Aube

Détail des remplages (désordres)

ANALYSIS OF THE STATICAL BEHAVIOUR OF THE ARCH OF CONSTANTINE IN ROME

C. Blasi*, Conforto**, P.P. Rossi***

SUMMARY

The Arch of Constantine in Rome has recently undergone complete restoration.

During restoration work surveys were also made in order to establish the static situation of the monument.

This report gives the results of this research and attempts to show the validity and utility of the results obtained both through co-ordinated experiments - by relieving tension at certain points with the technique of flat jacks - and theoretically by accurate numerical models whose results, when compared with the experiments, enabled us to establish tensions over the entire monument and understand the causes of the present damage.

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*** Ing. ISMES - Bergamo

Florence, April 1989

1. INTRODUCTION

The restoration of a monument involves a process of both knowledge and trasformation. Knowledge is acquired from a direct analysis of the materials, which may reveal new relationships and facts; transformation is the result of the interventions made to counteract the degradation wrought by time.

One might tend to assume that the state of an ancient monument has remained essentially the same from the time it was built until its present day restoration. However, the actual events of the classical era prove the contrary: the structure and surface of each monument often preserve signs, however corroded by time, of those alterations - both ancient and more recent restorations, changes of use and re-surfacing - to which the monument owes its very survival.

From times of antiquity to the present day, Roman monuments constitute a repertoire of technical interventions and restorations made in different periods that have gradually changed the appearance of the buildings and their individual relationship to their urban surroundings.

The present from of a monument is thus the product of the different cultures that have influenced the planning and realisation of the original work, and subsequent restoration works. Thus knowledge of an individual monument does not attempt only to assess its state of conservation, but also to interpret the palimpsest resulting from ancient and modern transformations of both its architechtonic and its structural aspect, while giving due emphasis also to the way previous restorations have been carried out and to the less obvious maintenance practices, as well as restoring the original structural concept wherever possible.

Until recent restoration work the architectural drawings on which knowledge of the Arch of Costantine was based were those made by A. Desgodez in 1682 before the monument underwent restoration by Clement XII. The new drawings made it possible for considerations that had hitherto been limited to the art historical study of the sculptures to be applied to the construction and stability of the monument.

Even in antiquity, at the time of its origin, this monument dedicated to Constantine's victory over Maxantius in 312 A.D. was already a palimpsest of materials assembled with different techniques. The construction is massive and extremely austere. The lower part, formed by four columns and three arches, is built entirely of large blocks of marble, while the upper part is built of blocks of marble on the exterior and brick walls in the interior; on these rests a vault in tufa conglomerate. Core boring testes carried out recently have showns the continuity and homogeneity of the construction in marble up to the crown of the vaults and in the piers (fig. 1).

The rich and varied iconography has always been a source for the study of the monument. The present of reliefs coming from different monuments makes the Arch a compendium of Roman sculpture.

The reliefs are partly sculptured on the blocks already in position and partly on huge slabs of marble using different techiniques and figurative concepts.

Dismembered panels of a continuous frieze originally more than twenty-eight metres long, from the Trajan Forum, are mounted in the central vault and on the smaller sides of the attic. This frieze celebrated the campaigns against the Dacians, which are also commemmorated by the fully sculptured statues on top of the columns; the facades are decorated with eight medallions representing hunting and sacrificial scenes that were originally part of a monument built by Hadrian; on the attic there are eight large rectangular panels, probably from an arch erected near the columns of Marcus Aurelius, narrating episodes from his life. The reliefs celebrating Constantine's victories and other episodes during his reigh are all sculpted on blocks that were already in position, with the exception of the two lateral reliefs.

Recent architectural drawings, drawn up on a scale of 1:25, have made it possible to develop a new line of research into the construction and structural behaviour of the monument, with the aim not only of conserving the building, but also of understanding it as a work of architecture.

2. DESCRIPTION OF THE DAMAGE

The monument is particularly robust and well constructed from a static point of view. The pillars are made entirely from squared blocks of marble, as are all the lateral surfaces. In the central part above the arches, containing an internal niche, shown in Figure 1, the walls and vaults are built in masonry.

The parts in conglomerate, as shown in Figure 1, are few and limited to areas that have no static function.

However, at some time in the past, the columns and the statues decorating the facade of this monument had to be consolidated.

Moreover, there are further cracks whose regularity reveals the presence of excessive strain in the material.

Four different types of cracks may be identified (see fig. 2):

a) cracks in the abutments of the large central arch, near the corners, following a course that runs parallel to the curve of the arch;
b) cracks in the masonry columns in the internal cella;
c) cracks in the corbels above the columns;
d) extensive cracks in the columns.

The cracks described in a) and b) reveal the presence of excessive compression tensions, while those described in c) appear to indicate the presence of traction tensions due to phenomena of flection.

The cracks found in the columns may probably be attributed to the poor quality - from a mechanical point of view - of the marble.

3. NON DESTRUCTIVE IN SITU TESTS FOR THE MECHANICAL CHARACTERIZATION OF MASONRY STRUCTURES.

The determination of the parameters necessary to evaluate the static conditions of the Arch was carried out by means of the well-known flat-jack testing technique developed some years ago by ISMES. This non-destructive testing technique, which is based on the use of thin flat-jacks inserted into the masonry has been set up in order to give reliable information concerning the following parameters.

- Measure of the state of stress.
- Determination of deformability and strength characteristics.

The reliability of the flat-jack technique has been deeply investigated by means of a wide range of calibration tests carried out on large masonry samples.

On the stone masonry structures of the Arch the measure of the state of stress was carrie out by means of flat-jacks having the shape of a circular segment (length 32 cm, height 12 cm and thickness 4 mm), inserted into cuts made by means of a circular plate with diamond tools.

The values of the state of stress, measured in several significant points of the structures, are indicated in the scheme of Fig.

The four tests carried out at the base of the monument show a stress concentration in point 12 (SOUTH-WEST corner) with a value of 2.0 MPa which is much higher than the average value (0.7 MPa) measured at the other three points (11, 3, 14).

Stress concentration was also determined on the South face of the main arch especially in the point 8 where a value of 3.2 MPa was measured.

The tests carried out over the columns show, at the South face, slightly higher stress values (0.5 - 0.6 MPa) than those measured at the North face (0.4 MPa).

The state of stress was also measured on the brick masonry walls of the chamber in the upper part of the monument (fig. 3) A uniform stress distribution was observed.

The deformability characteristics of the brick masonry were also determined by using two parallel flat-jacks. The masonry shows a linear elastic behaviour up to a stress value of about 1.5 MPa.

4. THEORETICAL SURVEY OF THE TENSIONAL STATE

The tensional state was examined by means of a popular Finite Element numerical procedure. Conventional modules of elasticity, using a different one for each material, were set up in order to describe mechanical behaviour.

While bearing in mind that the actual behaviour of the structures under examination is more complex than the theoretical linear one, the absence of relevant tensional states due to traction suggests that the results obtained are fairly significant.

Figure 4 gives the tensional state in a transversal section, and Figure 5 the tensional state in a longitudinal section.

A comparison between the theoretical values of the tensional states and those obtained experimentally reveals a high rate of correlation; especially worthy of note is the correspondence between the areas of the model revealing greatest tension and areas of damage in the monument.

It thus seems reasonable to conclude that the information given by the numerical model provides a valid indication of the present tensional state.

It should however be noted that the tensions revealed both by experiments and on the model, even in the areas under most strain and where cracks have been revealed, although worthy of remark, are not such as to explain the fractures completely.

Further increases in tension should therefore probably be attributed to local factors.

In particular, with regard to the corners at the abutments of the main arch, it is reasonable to assume that the more careful working of the exposed surfaces of the marble blocks has resulted in points of increased adherence and hence of greater concentration of tension in these areas.

Finally, the traumatic events the monument certainly underwent (earthquakes, demolitions, etc.) should not be forgotten: these could have increased the tensions beyond the limits of resistance at particular points, such as the corbels, the columns or the masonry ribs in the internal cella.

ACKNOWLEDGEMENTS

We should like to thank Prof. Chiarugi who has been the responsible for organization of this research.

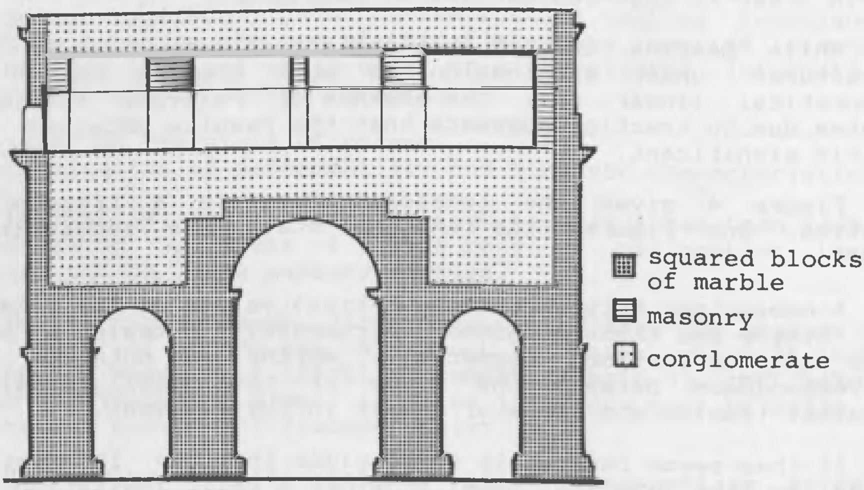

Fig. 1: Section of the arch with the different materials

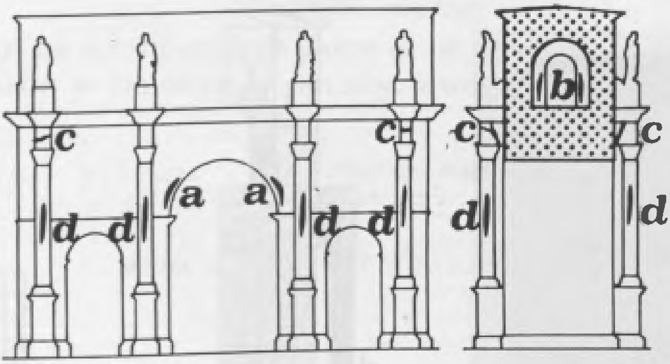

Fig. 2: Cracks on the monument

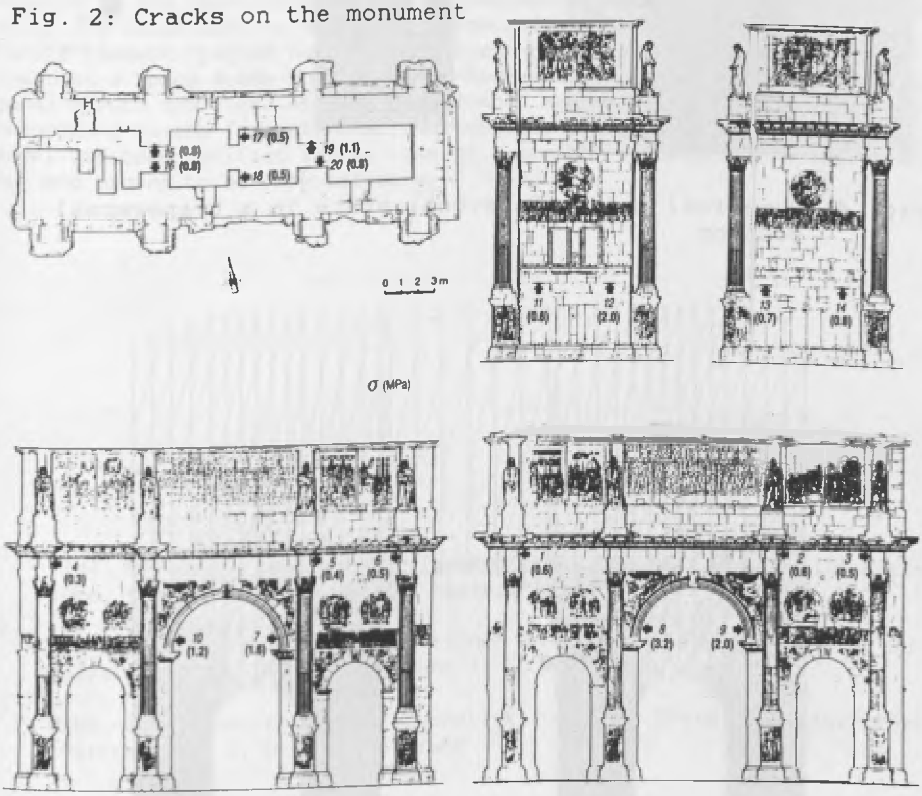

Fig. 3: The state of stress (experimental tests)

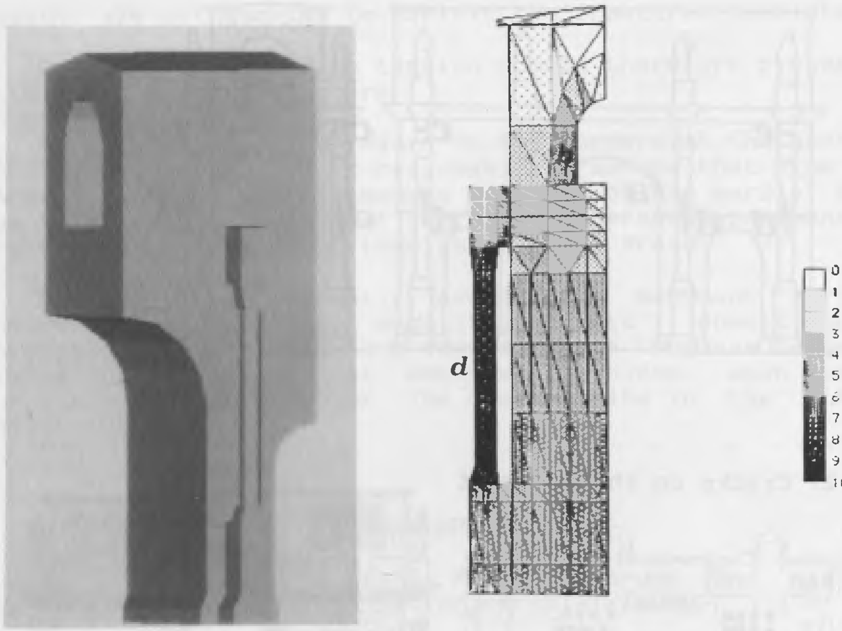

Fig. 4: Numerical model: Tensional state in a transversal section

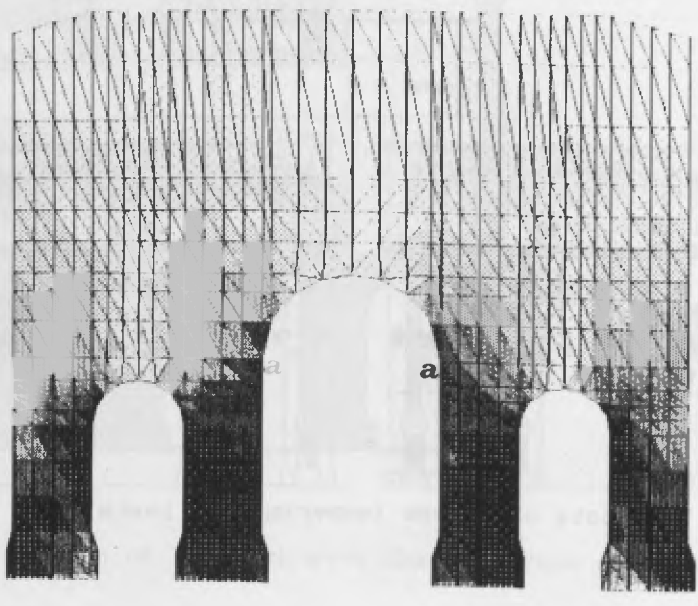

Fig. 5: Numerical model: tensional state in a longitudinal section

STRUCTURAL ANALYSIS

OF THE AUGUSTE-VICTORIA CHURCH OF THE ASCENSION IN JERUSALEM. ASSESSMENT OF ITS SAFETY BY EMPLOYING RESPONSE SPECTRUM MODAL ANALYSIS

F. WENZEL ([1]), B. FRESE ([2]), V. VRATSANOU ([3])

SUMMARY

Response spectrum modal analysis of the Auguste-Victoria Church of the Ascension on the Mount of Olives in Jerusalem has been conducted in order to study its capability to withstand seismic loads safely. For this purpose a Finite Element Program has been employed and the structure has been analysed as a space frame consisting of horizontal and vertical beams and diagonal struts and ties. At each node lumped masses have been specified corresponding to the free translational degress of freedom. This simulation model has been verified in the case of a simpler structure, the church belfry and proved to be very reliable.

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INTRODUCTION

Description of the Church

The Auguste-Victoria Church of the Ascension was built at the beginning of this century. It is situated at the S. part of a Hospital Complex on the Mount of Olives in Jerusalem (Fig. 1).

At the NW. corner of the Church the belfry rises; its walls are 45m high and 2m thick. Perpendicularly to the E. outer wall of the nave there is a 16m high wall, which surrounds the kitchen of the Hospital (Fig. 1). At the W. side, near the entrance, a forebuilding was added at a later time as a protection against rain and wind; no structural connection exists between the nave and this subsequent projection.

The nave of the Church has a rectangular section of 27m (length) by 20m (width) and a height of 16m (Fig. 2, 3, 4, 5). At the S. side there is the apsis with a semicircular section of ca 5m radius. The exterior walls have a thickness of approximately 1.20m.

In the ceiling there are several steel joists, which were supposed to support the roof slab; due to their improper dimensioning and bad quality of construction they do not contribute to load-bearing capacity of the structure. The roof truss is a light steel construction.

At the S. side of the Church above the triumphal arch there is a masonry gable of about 9m height.

The walls of the Church are made of ashlar stone masonry with rubble infill. The supporting pillars in the nave are made of stone. The upper (roof) and gallery floors belong to the so-called "Ackermann" type of construction; they consist of perforated clay blocks reinforced with steel mesh between their joints.

The soil formation of the Mount of Olives consists of alternate layers of hornfels and chalk marl. Hornfels is a hard, compact, metamorphic stone, whereas the properties of chalk are similar to those of clay (in presence of water the strength of the chalk decreases considerably).

Proposed strengthening measures

Because of the large and wide cracks in the upper part of the Church-especially in the S. and SE. side -, which were probably caused by a strong earthquake in 1927, it has been decided to repair the masonrywork by means of grout injections, anchor bars and dowels. In order to improve the interconnections between the structural members and to ensure the "tying" of the peripheral walls the upperstructure will be strengthened by the construction of a shotcrete belt and the addition of steel ties at the eaves level.

Apart from these measures, the construction of two new r.c. gables at the W. and E. side has been planned; the existing roof truss will be extended and modified accordingly.

F.E. ANALYSIS

Calculation method

In order to calculate the maximum values of the earthquake response of the structure the Response Spectrum Modal Analysis has been employed. The Finite Element Program A.D.I.N.A. of the University of Karlsruhe has been used for the computation.

The calculation has been performed separately for the two principal directions: the longitudinal (cases L1 and L2) and the transversal (cases T1 and T2). Due to the relatively good soil conditions and the absence of horizontal cantilever members the vertical component of the seismic action has been disregarded.

The normalized acceleration response spectrum provided by a proposal of 1988 for a new Isr. Standard (Fig. 6) has been given as input to the program (seismic intensity of the Jerusalem region: I=VIII-IX of the MM Scala). A behaviour factor K=1.30 has been assigned to the structure.

Simulation model

The Church of the Ascension has been modelled as a space frame consisting of vertical and horizontal beams (153 BEAM elements) and diagonal struts and ties (117 TRUSS elements). An isometric view of the simulation model is given in Fig. 7.

Soil-structure interaction: As no settlements appear in the structure, it has not been considered necessary to simulate the foundation soil using SPRING elements. Nevertheless, an unfavourable soil factor has been chosen as input parameter for the response spectrum in order to be on the conservative side.

Adjacent structures: The existing cracks indicate that the seismic response of the Church is mainly influenced by the belfry at the NW. and the surrounding wall of the Hospital at the E. side. A preliminary quasi-statical analysis according to the Isr. Standard SI 413 has shown that:

- the surrounding wall is very stiff (fundamental period T=0.10s) in its longitudinal direction, which means that this wall remains practically unmoveable during an earthquake excitation in this direction, and
- the displacements of the belfry at height H=17m (height of the Church) are rather small, although the structure is in itself relatively flexible (fundamental period T=1.50s).

Therefore, the nodes at the contact surfaces (Church-belfry and Church-surrounding wall) have been regarded as fixed. In this way:

- the torsional effect, i.e. the rotation of the Church around the belfry, which is evident through the large and wide cracks of the SE. corner, has been taken into account, and
- the inaccuracies of a calculation, when considering the Church, the belfry and the surrounding wall as a monolithic structure, have been avoided (in asymmetric structures the higher eigenmodes can play an important role and, therefore, cannot be disregarded).

Apsis: Being stiffer than the rest of the Church, it tends not to follow the central part of the structure in its movement (the crack pattern around the apsis provides sufficient evidence of this fact). Very probably a future strong earthquake will once more cause - independently of the strengthening measures - an interruption of the continuity between nave and apsis. So, practically, the apsis does not contribute to the seismic response of the main structure, and, consequently, it has been decided to exclude it from the simulation model.

Shear walls: In order to simulate the shear and flexural behaviour of the masonry walls in a simple way, a frame model with one diagonal strut - capable of carrying compression - has been adopted (Fig. 8). By comparing the model "A" with the model "B" (Fig. 8) the unknown parameter "w" (width of the diagonal strut) can be defined:

$$\text{Model "A"}: \quad \text{Stiffness} \quad K = \frac{Et}{4r^3+3r} \tag{1}$$

$$\text{Model "B"}: \quad \text{Stiffness} \quad K = Et\left[\left(\frac{c}{h}\right)^3 \cdot \frac{(6r+1)^2}{18r^2+15r+2} + \frac{x}{1+r^2}\right] \tag{2}$$

$$\text{wherein}: \quad J_1=J_2 \,, \quad r=h/l \,, \quad x=(l^2+h^2)^{1/2} \,, \quad K_Q=1.20 \,, \quad G=0.40\,E$$

$$(1)=(2) \qquad x = \left[\frac{1}{4r^3+3r} - \left(\frac{c}{h}\right)^3 \cdot \frac{(6r+1)^2}{18r^2+15r+2}\right](1+r^2) \tag{3}$$

Equation (3) can be shown in the diagram of Fig. 8. According to this diagram the cross-sections of the diagonal struts of all vertical frames have been evaluated.

Gallery floors: The stiffness of the gallery floors has been simulated by means of two diagonal TRUSS elements with a very small cross-section. Thus the softness of these horizontal diaphragms has been taken into account.

Roof slab: According to the proposed intervention scheme the roof slab will be strengthened by a number of diagonal prestressed steel ties. For this reason the roof diaphragm has been simulated by diagonal steel ties (Fig. 9), whose cross-sections are equal to the proposed ones:

Ties at the wings: $F=50\text{cm}^2$

Ties at the center: $F=10\text{cm}^2$ (cases L1, T1) or 50cm^2 (cases L2, T2).

Masses: At each node lumped masses have been evaluated corresponding to the free translational degrees of freedom. The weight of the proposed new r.c. gables has also been considered.

VERIFICATION OF THE SIMULATION MODEL

The main assumption of the simulation model, i.e. the replacement of the shear masonry walls through beams and diagonal struts - capable of carrying only compression -, has been verified in the case of a much simpler structure, the belfry. A response spectrum analysis according to SI 413 has been performed and the following simulation models have been employed:

- Model "A" : Simple oscillator with five lumped masses
- Model "B" : Space structure consisting of plane frames with one diagonal strut
- Model "C" : Space structure consisting of plane frames with two diagonal elements (whose stiffness is half as great as the stiffness of the struts of Model "B").

In Fig. 10 and 11 the diagrams of the horizontal displacements and lateral forces vs. height of the structure are given. The comparison of the results shows that Model "B" is reliable.

The deformed shape of the belfry can be seen in Fig. 12.

RESULTS

In order to evaluate the earthquake response of the structure the lowest five modes of vibration have been considered. The natural periods of these modes are listed below for each case:

		T (sec)				
	Cases \ Modes	1	2	3	4	5
Excitation in the longit. direction	L1	0.727	0.601	0.577	0.557	0.435
	L2	0.713	0.596	0.577	0.546	0.386
Excitation in the trans. direction	T1	0.727	0.637	0.633	0.545	0.418
	T2	0.684	0.637	0.617	0.440	0.417

The contribution of the higher eigenmodes, although negligible, has been taken into account by calculating the residual terms using the SRSS (square root of the sum of the squares) method. This contribution is static and is governed by the ZPA (zero period acceleration) value, thus not dynamically amplified.

From the comparison of the cases L1, T1 with L2, T2 it can be clearly seen that the strengthening measures (installation of steel ties at the eaves level) tend to make the structure stiffer. Some of the results (horizontal displacements and shear forces) can be seen in Fig. 13 and 14.

According to the output of the numerical analysis it has been decided to intervene only in the upper part of the structure by means of anchors and dowels. In the lower part, where the displacements are very small, the masonrywork is fully capable of carrying the shear forces and, therefore, no reinforcement is required.

CONCLUSIONS

The Church of the Ascension in Jerusalem is a structure which is characterized by an irregular shape, very flexible horizontal diaphragms and very stiff vertical bracing members, and so presents a very complex load distribution. In order to evaluate its earthquake response, the response spectrum modal analysis has been employed and a simulation model, which consists of plane frames with diagonal struts and ties accounting for the stiffness/flexibility of the walls and floors, has been developed. This simulation model has been verified numerically in the case of a simpler structure, the belfry of the Church, and proved to be a reliable tool for the prediction of the ultimate loads and displacements. Also it enables us to avoid a much more complicated F.E. Analysis (e.g. using SHELL or SOLID elements), to considerably reduce the number of the unknowns and, consequently, to decrease the computer expense.

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Fig. 1: Site plan of the Hospital Complex

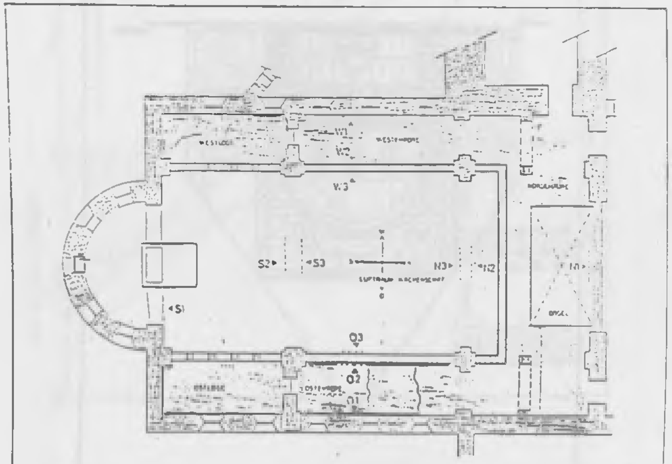


Fig. 2 : Ground plan of the Church

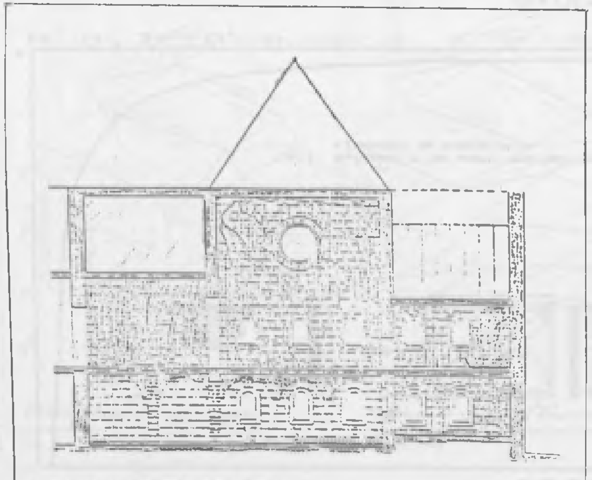


Fig. 3 : Cross-section Q1 of the Church

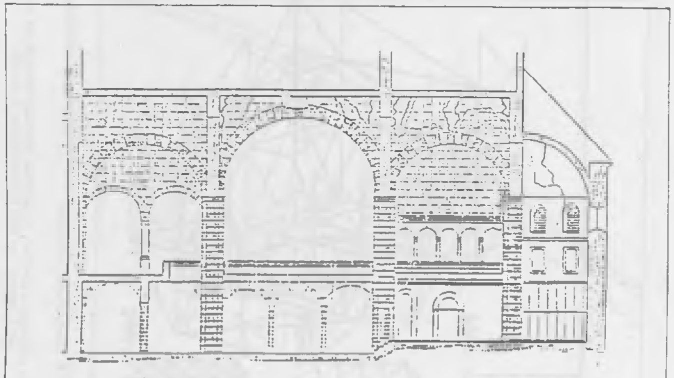


Fig. 4 : Cross-section Q3 of the Church

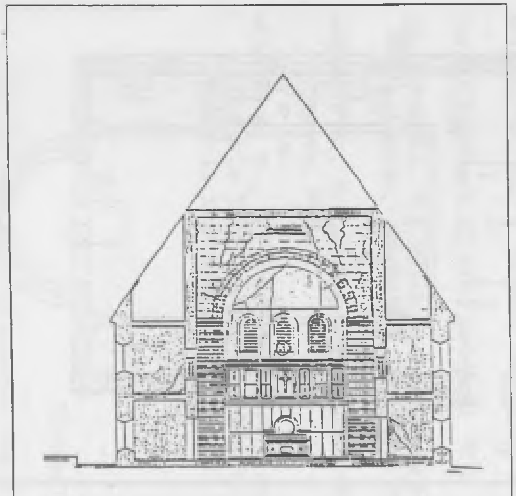


Fig. 5 : Cross-section S1 of the Church

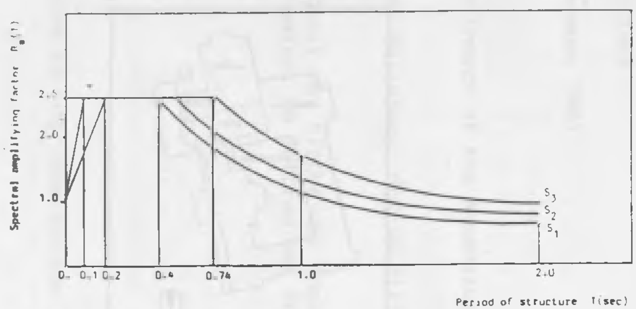


Fig. 6 : Normalized design spectrum according to the proposal of 1988 for a new Isr. Standard

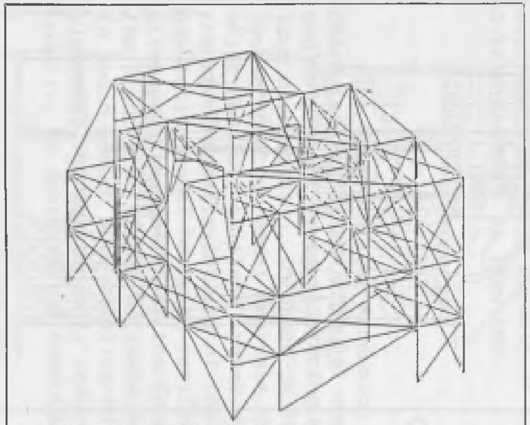


Fig. 7 : Isometric view of the simulation model

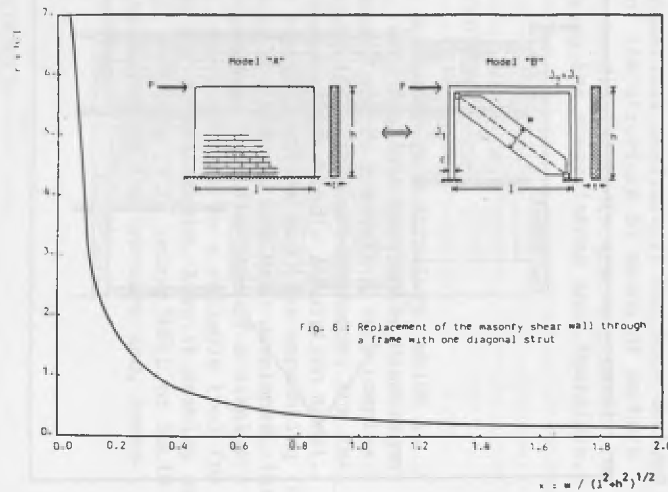


Fig. 8 : Replacement of the masonry shear wall through a frame with one diagonal strut

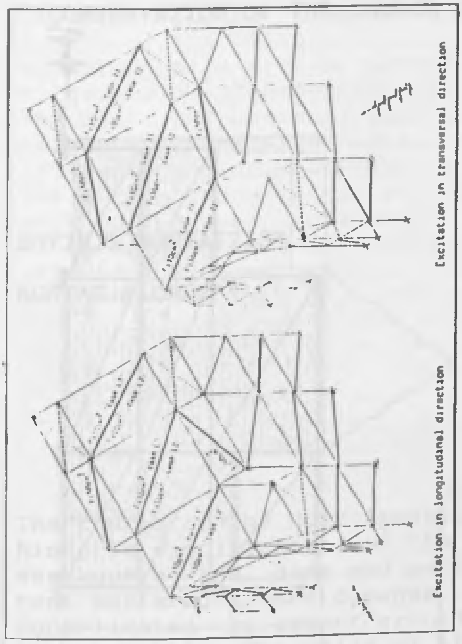


Fig. 9 : Simulation of the roof slab through diagonal (steel) ties

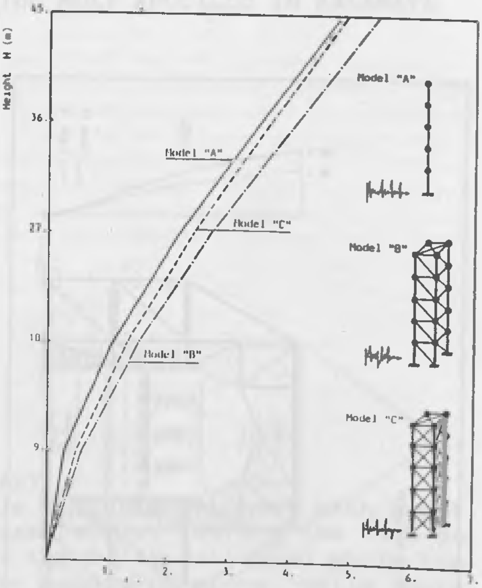


Fig.10: Response spectrum analysis of the belfry according to SI 413
Horizontal displacement δ (cm) vs. height H (m)

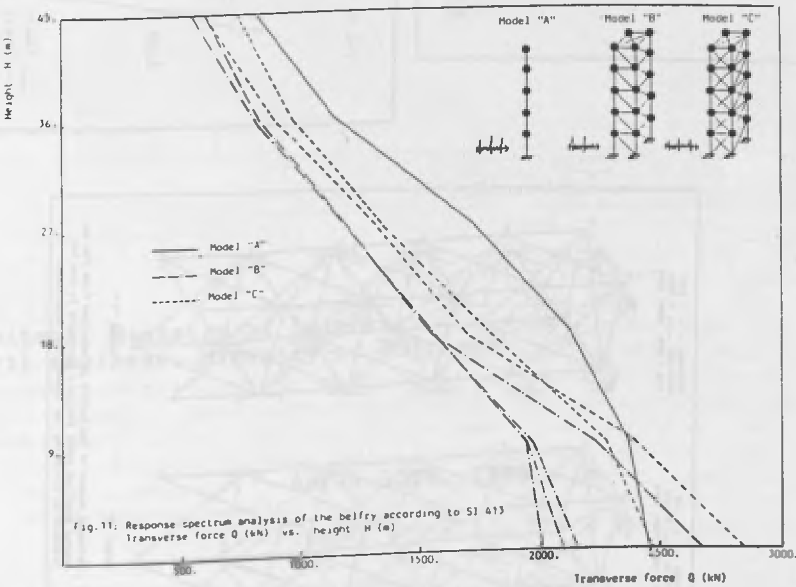


Fig.11: Response spectrum analysis of the belfry according to SI 413
Transverse force Q (kN) vs. height H (m)

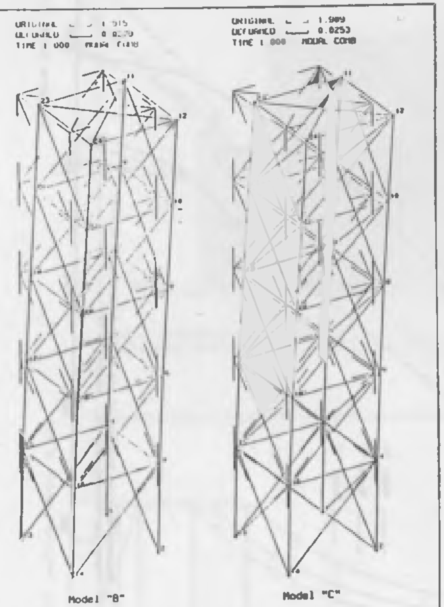


Fig. 12 : Deformed shapes of the belfry as resulted from the response spectrum modal analysis according to the Isr. Standard SI 413 using the lowest five eigenmodes

Fig. 14 : Horizontal displacements of section S1

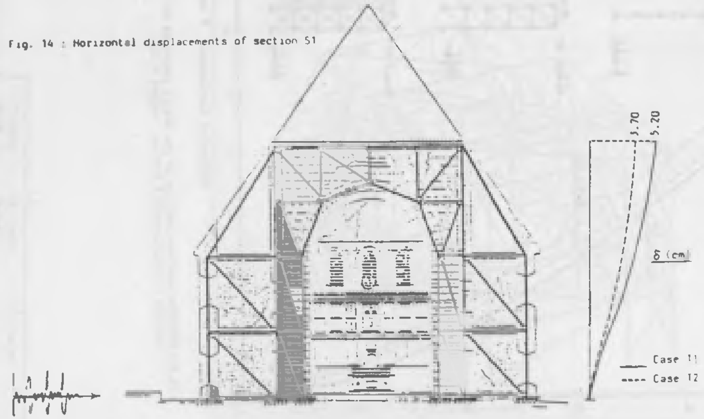
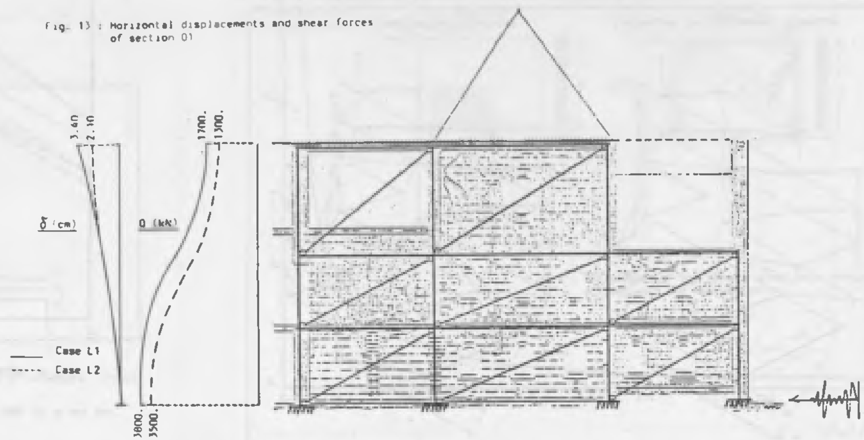


Fig. 13 : Horizontal displacements and shear forces of section 01



CONSERVATION OF THE CHURCH OF THE HOLY APOSTLES IN KALAMATA

SOTIRIS VOGIATZIS*

KOSTAS FLOROS**

SUMMARY

The church of the Holy Apostles is a double monument with great historic significance for the Greek People. During the 13-6-86 earthquake the dome and part of the walls collapsed while the rest suffered severe damages. The remaining stone walls were consolidated by cement grouting and the vaults, dome and drum were rebuilt. The walls of the Holy Bema, built in the 12th century, were repaired by cement grouting up to the springing of the arches and epoxy resin grouting where the existence of frescoes prevented the use of cement mixtures. The two buildings were then structurally separated.

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April 15th, 1989

DESCRIPTION - HISTORY

The Church of the Holy Apostles is a historic building with great significance for the town of Kalamata - located in southern Peloponnese - since it was here that the 1821 Liberation Revolution got started. It is found in the centre of the historical district of the town. It has a composite structure, being literally two buildings built with a lapse of about 400 years. The eastern one is older, byzantine, built probably during the 12th century, with frescoes dating in the 14th (Καλοκύρης, 1973, Bon, 1951). It is a small, single-domed, greek-cross type church. Its western wing was demolished (or collapsed) during the Venetian Era (1685-1715), while at the same time a newer part was added as an extension on the west side (Fig 1). The latter is a larger, domed, single-spaced, quincux-shaped structure used now as a Main Naos, while the older part is used as a Holy Bema. In the place of the demolished west wing an arch was built to take up the loads.

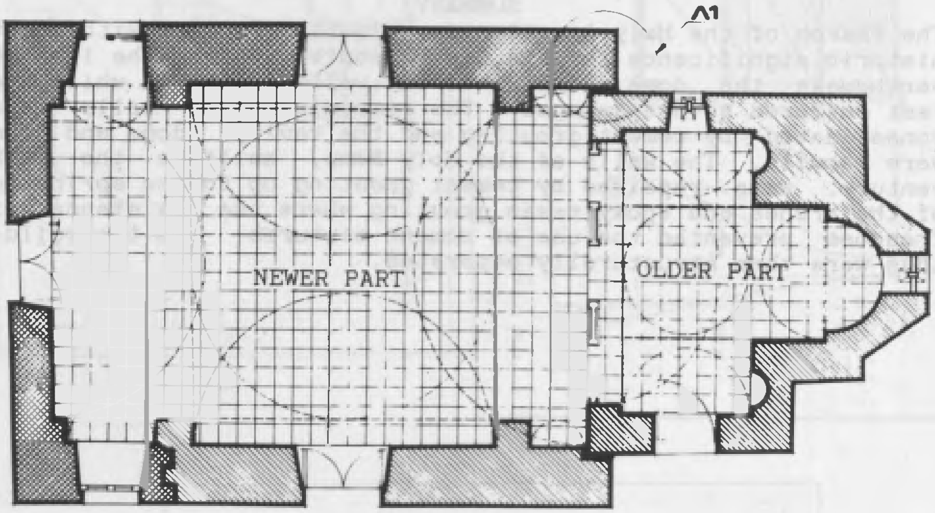

Fig 1. Plan of the monument

The byzantine Bema is built by relatively good quality porous lime-stone, "cloisonne" masonry characteristic of its era. At its north western corner a heavy belltower was added probably at the same period with the main Naos. The latter is built by sand stone of the worst quality, that comprises the outer layer of the walls, while the rest of the wall width is built by large pebbles and rough stones. The dome, rebuilt by reinforced concrete during the 50's, without proper dome base support, replaced the older wooden, makeshift dome.

During the severe earthquakes of September the 13, 1986, the reinforced concrete dome collapsed intact inside the temple, bringing down the whole roof of the main naos as well as part of its walls (Fig 2). The remaining walls were also severely damaged. Older cracks were reactivated on the northern wall, while a new diagonal crack was formed running from the peak up to two meters from the bottom of the wall, which acquired an outward lean of almost 20cm. Also the outer layer of the wall was disconnected from the sea pebbles core. The western wall almost totally collapsed, while only part of the southern wall remained intact, having however a severe horizontal displacement crack at its bottom due to the thrusts of the collapsing dome (Fig 3).

Fig 2. After the earthquake.

The byzantine part, although it did not collapse, suffered considerable damages. The old temple displayed severe cracking at the base of the dome, accross the barrel-vaults and disintegration of its structural material. Also several voussoirs were detached due to the movement and mortar failure and were ready to collapse. The frescoes were in bad condition too, due to cracks at the substructure, but also to previous water penetration. The belfry tower was in better condition structurally, however bell vibrations had over time cracked its upper part.

WORKS UNDERTAKEN

Two days after the earthquake the remainings of the monument were shored up in order to prevent further damage. It was measured, plotted by photogrammetric methods and restituted making use of photographs existing before the earthquake destruction by the Department of Topography (Fig 4). Then a conservation project was undertaken by the Department of Byzntine and Post-byzantine Monuments of the Ministry of Culture. The operation was undertaken by a specialized contractor and the collaboration of the 5th Ephory of Byzantine Antiquitties, that also undertook the work of the final appearance of the monument, under the supervision of the above Dpt. The cost amounted to 30.000.000 dr, 10% of which was sponsored by the Commission of the European Communities, while the whole operation lasted from July to November 1988.

Fig 3. South Facade.

Investigations

A number of investigations were performed in order to decide the best course of action:
a. Laboratory analyses of mortar specimens in order to establish its physical, mechanical and chemical characteristics. The most important result was the fact that salt water was used in the construction of the newer part.

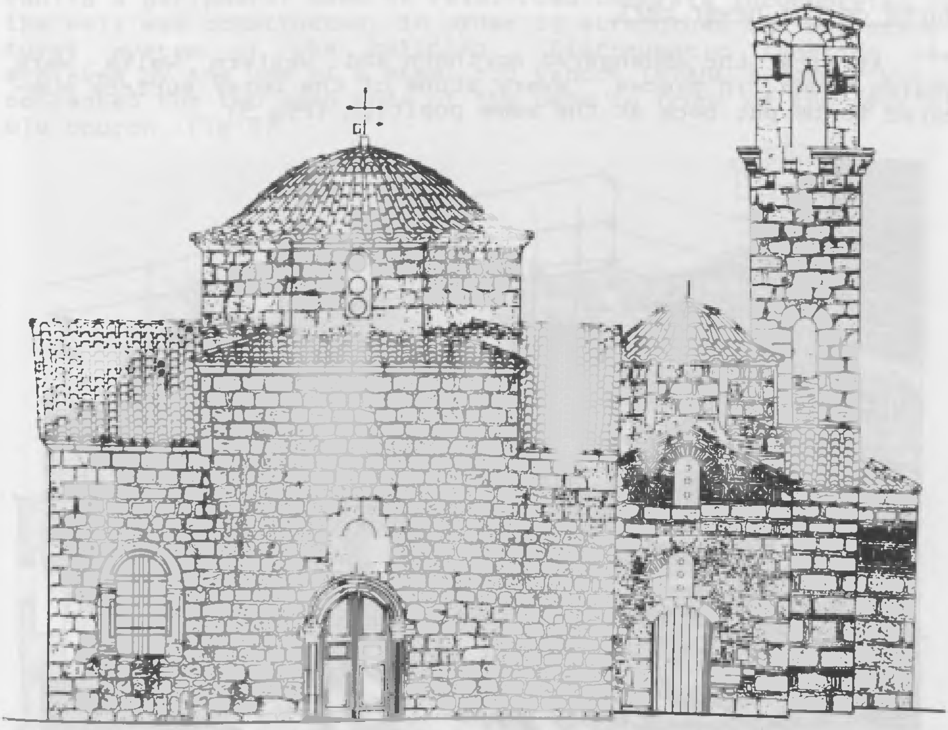

Fig 4. Restitution of the south Facade.

b. Ground excavations were performed inside and outside the temple, by which it was established that none of the damages was due to foundation trouble, since the church was founded on compacted clay soil to a depth of 1,00-1,50m.
c. Borescoping was performed on the walls of the byzantine part to measure the amount of voids, the results being the establishment that although the outer faces of the walls were in generally sound condition, age had resulted in a general decay of the mortar, having as a result a 30% amount of voids.

Finally the following design lines were followed:
a. Reconstruction of the building as it was before the earthquakes due to its high symbolic significance.
b. Structural separation of the two different parts of the church, in order to avoid the thrusts of the newer part to the older during earthquake action in the future.
c. Use of traditional materials (stone, bricks) for the new constructions avoiding as much as possible the use of reinforced concrete so as to achieve a nearly homogenous structure.
d. Repair of the existing walls by the use of different types of grouting.
Works on the monument started on June 1988 and were finished by the end of November on the following order:

Works on the newer part

At first the endangered northern and western walls were hauled down in pieces, every stone of the outer surface numbered to be put back at the same position (Fig 5).

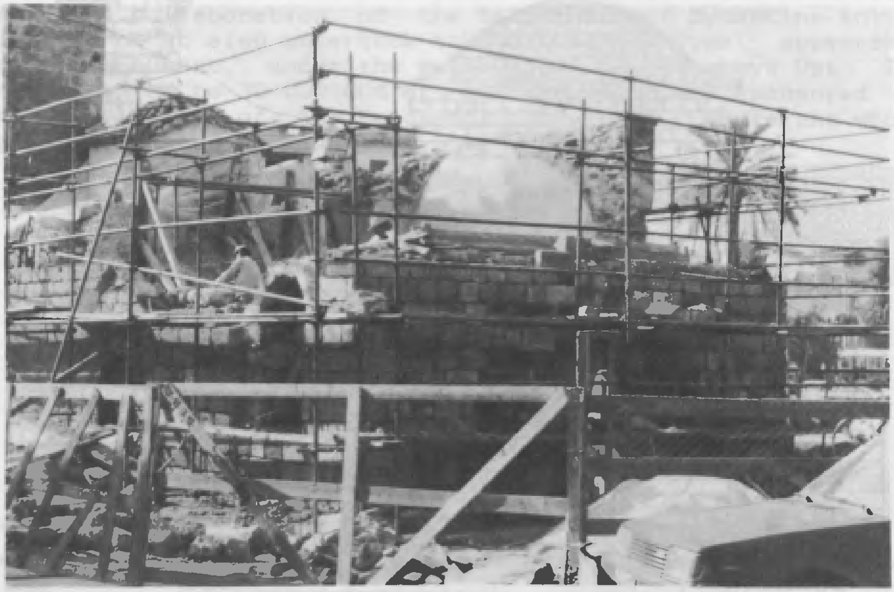

Fig 5. View from the north-west.

The remainings of the walls were consolidated by extensive cement grouting, in order to reinstate their initial mechanical characteristics. The method used was low pressure (1atm) grouting of a Portland cement-fine grain sand-water mix having a ratio 1-1-2 1/2 with the admixture of a plasticizer in order to establish the permeability and stability of the grout. The use of Portland cement was compatible since chemical analysis of the old mortar showed that its SO_4 content was 41.57mg/100gr well below the level of 100mg/100gr of the Standards. Grouting was performed through cracks and holes bored in the mortar joints on a grid every 1m in height with a hand operated grout pump fed through a paddle mixer. No need for extensive sealing arised since the old pointing was in good condition. Records were kept for the quantity of the mixture injected.

The horizontal crack of the still standing southern wall was additionally stiched by the use of four vertical "stiches". Polymer Concrete was used for its high tensile strength and very low linear shrinkage. It was applied on "spaces" formed by taking out stones from both sides of the crack.

Rebuilding of the walls followed, using the old outer stone layer. When their height reached the springing of the vaults a peripheral beam of reinforced concrete incorporated in the wall was constructed, in order to strengthen the new structural system of the building. Diafragmatic function was achieved by the use of a steel rod tendon (STAHL I,D40mm) which connected the two open ends of the beam in front of the arch of old church (Fig 6).

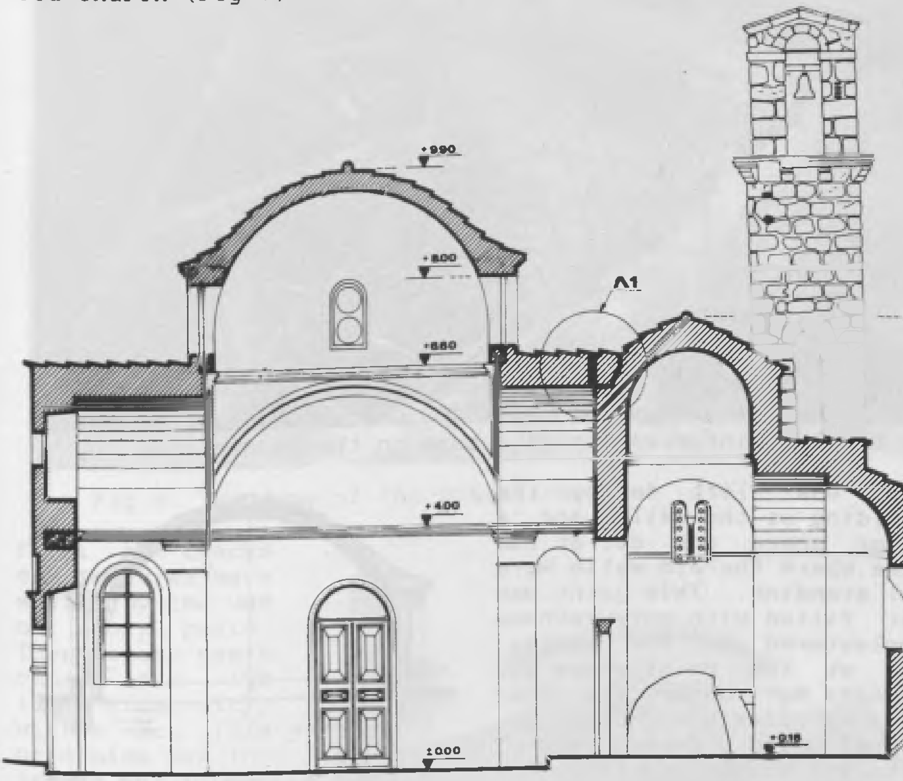

Fig 6. Proposed section.

The four vaults were then rebuilt using rough cut stones and a typical cement mortar. In order to bind the scuthern vault to the old wall additional cement grouting was used. After the four pendentives were completed another beam of reinforced concrete was constructed in order to bear the horizontal thrusts of the dome (Fig 7). Old hand made bricks were used for the building of the dome which was hemisperical to the inside, while a shallow octagonal stone drum was erected to the outside (Fig 8). The whole construction was roofed by traditional hand made clay tiles. Finally the walls were repointed externally with a white cement/lime mixture with stone dust as an aggregate, while they were plastered in the interior. The floor was covered by paving stones following the pattern of the initial floor, found by the excavations at a lower level.

Great care was taken so that the two parts of the building would become structurally independent. To achieve that a 5cm

Fig 7. Reinforced concrete beam on the pendentives.

joint was left during the rebuilding of the walls and a similar space was cut at the places where the old walls were still standing. This joint was later filled with polyurethane and plastered on the inside, while at the point where the two roofs met under the roof tiles, a flexible joint was designed using U formed lead sheets and bituminous felt.

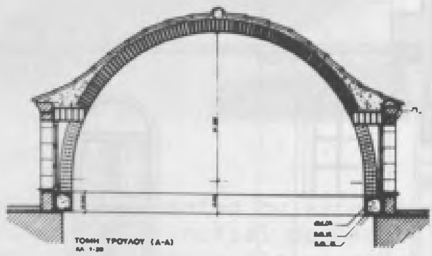

Fig 8. Section of the dome.

<u>Works on the older byzantine Part</u>

Works on the byzantine part run parallel to the ones of the 18th century part as follows:

First the old roof tiles of the vaults were removed in order to get access to their structure. All the walls from the level of the ground up to the springing of the vaults were consolidated by cement grouting with a composition similar to the one used for the newer part of the church, through holes bored on the mortar on a 60X60cm grid approximately.

It was decided not to use cement grouting for the vaults, the drum and the dome since the concertration of salts on the surface would eventually ruin the frescoes. Instead an appropriate epoxy injection was to be used to fill the voids and repair the structure of the vaults (Fig 9).

Fig 9. The dome of the byzantine part

Fig 11. Final appearance.

First the cracks and fissures were sealed by the use of epoxy paste. Then epoxy resin chosen for its low viscosity, with a pot life of 45mins was injected by hand operated pump. The resin was mixed with a ratio 1:3 with fine aggregate (marble dust), in order to lower extensive setting temperature. since the voids to be filled in the stone wall were almost over 30%. However after a few trials this method was abandoned because through the very fine cracks underneath the frescoes and through the pores of the exterior sandstones the resin tended to infiltrate the outer layers of the masonry and overflow, staining the external surfaces. So a change of material was decided on the spot and after some trials a thixotropic combination of medium viscosity was finally chosen, to be pumped manually into the mass of the wall without any further trouble.

One of the fine points of the repair was the stabilization of the load bearing arch that was built underneath the remains of the cut off western vault (Fig 10). It was at first proposed by the contractor to connect the broken arch, to the mass of the wall above by the means of four steel braces through holes bored on the arch consolidated by grouting. However when the ramshackle templon was demolished a second arch above the first was discovered bearing interesting frescoes not seen before. So the steel bars were abolished and an ahesive epoxy resin was used to bond the single line of stones that remained of the lower arch. Finally hand made ceramic tiles were used to cover all the roofs (Fig 11).

Fig 10. Interior view of the western arch.

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Preliminary comments on the collapse of a medieval tower

G. Michele CALVI * M. J. Nigel PRIESTLEY †

SUMMARY

At 9 a.m. on March 17, 1989, a 900 years old rectangular masonry tower collapsed without warning in Pavia, killing four persons, damaging the adjacent 15th century Duomo (cathedral) and destroying adjacent shops and dwellings. The paper describes preliminary attempt to understand the possible causes of the collapse. Initial results of analysis indicate that the form of masonry construction used in the tower, and the influence of a stairway that spiralled up around the tower within the thickness of the walls may have been significant.

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INTRODUCTION

In the medieval age it was common for Italian families of wealth and influence to build tall rectangular towers (fig. 1), certainly as status symbols, but perhaps also with defense purpose, either against outside enemies and in internal wars for power.

Such towers were commonly built at the corner of blocks (in the case of Pavia almost perfectly square, on the Roman perimeters) and often connected through arches.

More than one hundred towers are documented to have existed in Pavia, but most of them are now cut at the level of the surrounding buildings. Both failures and demolitions are documented. Two major towers are documented to have failed in Pavia in 1584 and in 1712, but the best known collapse happened in Venice (1902) when the bell tower of the Basilica di S. Marco failed with only a few minutes warning.

DESCRIPTION OF THE TOWER

The Civic Tower of Pavia (fig. 2 and 3) was started in the old middle age as the bell tower of the medieval cathedral, but the known history of the tower starts in the 11th century, when the lower part of the destroyed tower was built. The upper part was conceived and constructed in the last part of the 16th century.

This tower was therefore different from all the others, both because of its history and of its geometry: it was much wider and consequently apparently squatter, and a stairway was spiralling up around the tower within the thickness of the walls, which were of uniform thickness, an unusual detail. A consequence of the discontinuity of the construction was also the dishomogeneity of the materials used, with roman and medieval bricks, stones, river gravel, and mortar of different quality.

The Civic Tower collapsed without warning on March 17th, 1989, at 9 a.m., killing four people and damaging surrounding buildings and the adjacent 15th century cathedral (Duomo).

A recent survey (after a severe wind storm, september 1988) performed by town officials had not given any concern; on the contrary the tower was considered in excellent shape, and a project for its opening to the public was going to be funded.

In figures 4 and 5 views of the about 6000 cubic meters of materials that had constituted the tower are shown; it is particularly significant to notice the presence of large pieces of walls perfectly conserved together with a lot of material reduced to powder.

The collapse had aroused concern about the safety not only of the remaining towers, but also of the Duomo, because of different reasons. New cracks opened in the Duomo, which is a very composite structure, constructed over five hundreds years, with different materials and techniques.

Since the day after the event the Duomo has been temporarly monitored by electronic Whittmore bases, displacement gauges and displacement transducers.

POSSIBLE CAUSES OF FAILURE

Since there wás no observed evidence of distress prior to the collapse of the tower, and since the collapse will almost certainly have distroyed any remaining evidence, it will be difficult to determine the precise cause or causes of failure. However, at the time of writing this paper, the lower section of the tower debris remains to be excavated. This will be carried out with considerable care in case clues to the causes are found.

Figure 1: Medieval Towers in the historical centre of Pavia

Figure 2: The Civic Tower and the Cathedral before collapse

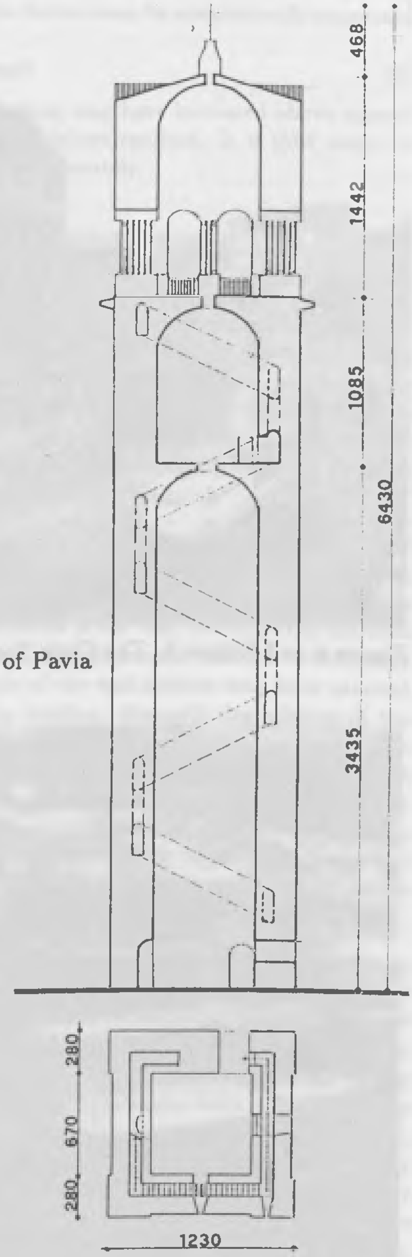


Figure 3: Plan and section of the Civic Tower

Figure 4 and Figure 5: The Civic Tower and the Cathedral after collapse

Figure 6: Corner section of wall showing brickwork skin

Figure 7: Conglomerate inner section of wall

In the meantime, it is possible to draw some conclusions on the various aspects which may have some significance to the collapse. The criterion for failure may be simplistically expressed as

$$\text{Loading} > \text{Strength} \tag{1}$$

The implication of Equation 1 is that for failure, loading may have increased above normal levels, or strength may have decreased to the level where failure resulted. It is thus useful to consider aspects of material strength and applied loading separately.

Material Strength

(a) **Foundation failure.** The possibility of a foundation failure was an early suspect. The streets of the historical centre of Pavia have many old brick sewers dating from Roman times, and the collapse of one of these could have destabilised the tower. Records indicate intersecting sewers some 5m outside two walls, but not underneath the tower. It is possible that leakage from these sewers, or underground natural waterways could have leached the predominantly alluvial materials under the tower, thus reducing foundation strength, but it is felt that this is unlikely, as such an occurrance should have been preceeded by some visible tower displacement. Current excavations will clarify this point.

(b) **Tower compression strength.** The tower walls are of a construction known in Italy as 'sacco', where outer layers of stretcher-laid brickwork confine a central region of an artificial conglomerate formed by broken pieces of brick and river gravel embedded in a mortar matrix. Figures 6 and 7 show photos of sections of the wall material that survived the collapse. The quality and strength of this central conglomerate material is extremely variable. It is possible that compression strength of the wall section may have reduced to the extent that failure occurred under gravity loading. Strength degradation of the bricks is less likely than that of the lime mortar, wich is more susceptible to attack from environmental pollutants. Howewer, the great thickness of the walls would indicate that such a degradation would be concentrated in the comparatively well constructed outer skins. In this respect it should be noted that conventional models of masonry compression strength indicate that the strength is rather insensitive to mortar strength. It is also felt that gradual reduction in compression strength would have been accompanied by visible warning signs long before collapse occurred.

(c) **Tower tensile strength.** The tensile strength of the tower masonry, particularly in the outer layers, would clearly be more dependent on the possible degradation in mortar condition, and a failure mechanism can be proposed which involves degradation of tensile strength. Considering the 'sacco' construction of figure 8, it is clear that the competent outer layers will carry a disproportionate amount of axial load, compared with the weaker conglomerate layer. It is possible that in some regions of the tower walls, the stress state was such that the central conglomerate layer was close to its strength. As a consequence, it would exhibit high transverse tensile strains which would be restrained by the stiffer and more competent outer layers. The conglomerate would thus be confined laterally to some extent, which would increase its compression strength and enable it to carry its share of the axial load. The outer layers would be placed in equilibrating tensile strength, as shown in figure 9. If the transverse tensile strength of the outer layers degraded to the current level of tensile stress, vertical cracking would result, relieving the confining pressure on the conglomerate, and hence reducing its strength. The outer layers would have to carry an increased axial stress in conjunction with a condition of vertical cracking, and failure could result. Note that such a failure mechanism could be very brittle, since a step-decrease in compression strength of the wall as a whole would result from the onset of vertical cracking, and it is feasible that there would be no warning of failure.

Applied loading

At the time of failure there was no obvious unusual loading that could explain sudden failure. Wind conditions were calm, and the tower was certainly not subjected to seismic loading. Traffic vibrations in the locality had been reduced some years earlier by a ban of heavy traffic in the vicinity of the Duomo. In a severe wind storm in Sept. 1988, extensive damage occurred in Pavia, but the tower was apparently undamaged. In fact, estimates of the bending stress induced by the wind storm indicate a maximum of about 0.15 MPa, or less than 10% of the peak gravity stress. Early in the morning, when failure occurred, the tower could have been subjected to thermal stress as a result of incident solar radiation on the East-facing side of the tower, but this would induce small lateral and vertical tensile stresses in the outer layer of brickwork on that face, and insignificant stresses elsewhere. There is also no reason to believe that thermal conditions were unusual. The collapse occurred at the end of an unusually dry winter, and it is possible that drying of the outer skin of brickwork relative to the interior may have induced lateral and vertical tensile stresses in the outer layers, contributing to the tension failure mechanism proposed above.

It is concluded that there was no significant unusual stress-inducing action that could have contributed to failure. It is, howewer, of interest to investigate the levels of stress induced by gravity load.

(a) **Effect of section eccentricity at the tower base.** As will be seen from Fig. 2b, the section at the base of the tower is unsymmetrical as a result of positioning of the openings for ingress, and a mayor window opening, in conjunction with the cavity resulting from the initial rise of the internal stairway. Approximate calculations, based on a material unit weight of $20KN/m^3$, indicate that the average compression stress at the base of the tower was about 1.2 MPa, and the effect of the section eccentricity was to increase the compression stress at the S.W. corner to approximately 2.0 MPa. Smaller variations in stress across horizontal sections at higher levels occur would result from the influence of the stairway, and smaller transverse openings for light.

(b) **Local effect of internal stairway.** Of particular interest is the possible influence of the internal stairway in inducing undesirable local stresses. In order to qualitatively assess this, a simple linear 2-D finite element analysis was carried out representing one wall of the tower penetrated by an inclined stairway inclusion. As shown by the mesh of figure 10, this was represented, somewhat simplistically, by a series of elements of reduced thickness. Under uniform axial load applied at sufficient distance from the stairway to ensure that local end effects did not influence stresses in the region of interest, the compression and tension stress contours of figures 11 and 12 resulted. These contours are expressed as percent of the average compression stress. As expected, the compression stress contours indicate increased stress in the region of reduced wall thickness, but it is the tension stress contours of figure 12 that merits special attention. Areas of significant tension stress (up to 9% of the average compression stress) occur adjacent to the ends of the stairway inclusion. These principal tensions are inclined at a small angle to the horizontal axis. To ensure that these tensile stresses were not a spurious result of the unsymmetrical mesh, the analysis was re-run with uniform thickness elements throughout. Tension stresses resulting from this analysis were insignificant. It will be noted that because of the similar influence of

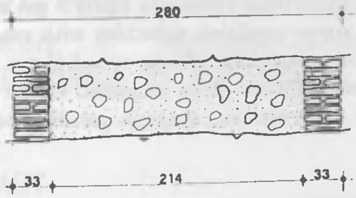


Figure 8: Section through wall thickness showing 'sacco' construction

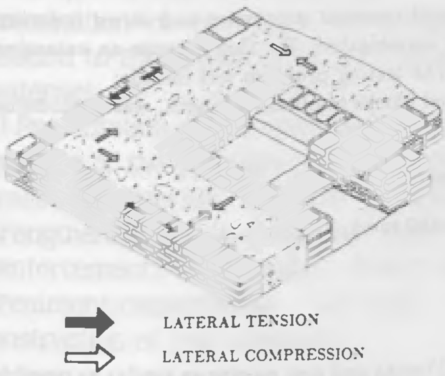


Figure 9: Lateral equilibrium of forces in wall under uniform compression

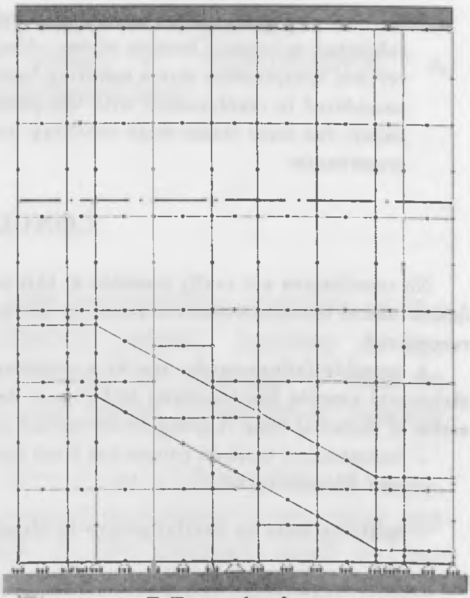


Figure 10: F.E. mesh of a portion of the Civic Tower

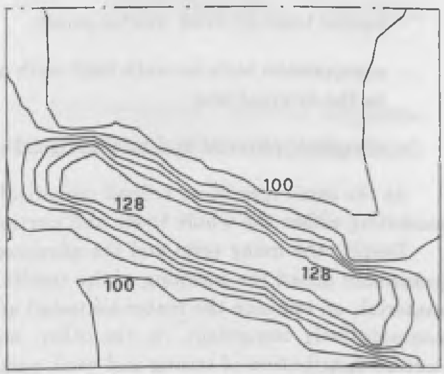


Figure 11: Contours of principal compression stress (as percentage of average compression stress in uniform wall)

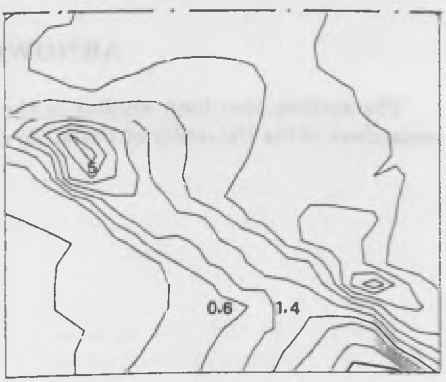


Figure 12: Contours of principal tension stress (as percentage of average compression stress in uniform wall)

the stairway in the intersecting transverse wall, the critical regions in figure 9 are actually subjected to biaxial tension stress. Also, this stress condition coincides with maximum vertical compression stress resulting from section eccentricity effects, noted above. When considered in combination with the possible tensile failure mode described earlier in this paper, the local stress state resulting from the finite element analysis assumes increased importance.

CONCLUSIONS

No conclusions are really possible at this early stage of research, but the existence of significant lateral tension stresses induced by the geometrical characteristics of the tower should be recognized.

A possible failure mode due to a combination of material properties and stress inducing structural aspects has therefore to be more deeply investigated. For this purpose an extensive series of material tests is going to be carried out. The testing program will include:

- compression tests on prisms cut from remained blocks to obtain strength, elastic modulus and Poisson ratio;
- splitting tests on similar prisms to obtain tensile strength;
- compression tests on masonry cylinders (D=150 mm);
- compression and splitting tests on bricks;
- tensile tests on brick mortar joints;
- compression tests on walls built with original bricks and new mortar as similar as possible to the original one;
- chemical, physical and mineralogical tests on bricks and mortar.

At the same time more refined numerical analyses using also 3D elements will be performed, modelling either the whole tower and portions of particular concern.

Despite the many tests and the advanced numerical analyses that will be possible a note of pessimism about the meaning of the results is drawn in recognition of the heterogeneity of the material: on one side the materials tested are those that survived the collapse and are therefore comparatively competent, on the other, no numerical analysis will be able to reproduce the correct distribution of strong and weak parts within the structure of the tower.

AKNOWLEDGEMENTS

The authors have been working in the three months after the collapse within a group of researchers of the University of Pavia. To all of them they wish to express their appreciation.

MEDIEVAL TOWN OF RHODES. RESTORATION AND ITS PROBLEMS

N. Zarifis, K. Manoussou-Della, G. Dellas, P. Papatheodorou, A. Paraskevopoulou, Th. Anapolitanos, D. Lazaridou-Koliadi

SUMMARY

The paper has two parts:

a) Analysis of construction techniques with load-bearing masonry of the buildings in the medieval town at different periods, focusing on the examination of structural performance through the passage of time in relation to the seismicity of the region and the special properties of building materials.

b) Presentation of the operations of the Office from October 1985 till today. Analysis of the principles directing interventions in relation to structure and available materials. Detailed presentation of the methods used in the strengthening of load-bearing walls and stone vaulting (surface repair, reinforcement) and timber floors and roofs carried on timber beams (treatment-conservation, securing of their diaphragmatic function, construction of roof covering).

Office for the Restoration and Conservation of the Medieval Town of Rhodes
Contract between the Ministry of Culture, the Archaeological Receipts Fund and the Municipality of Rhodes
architects - civil engineers

Rhodes, 15th May 1989

ON THE BUILDINGS OF THE MEDIEVAL TOWN

Introduction

The buildings of the medieval town of Rhodes are the result of the building activity mainly of the Hospitaller period, when Rhodes was ruled by the Order of the Knights Hospitallers of St. John of Jerusalem (1309-1522) and the Turkish period (1522-1911). The Hellenistic and Byzantine cities had previously stood on the same site. Significant restoration work was carried out during the Italian occupation of the island (1911-1945).

A feature typical of buildings in the town is the use of earlier structures in the construction, either by re-use of building materials (stone etc.) or by incorporating entire sections of older buildings into the new ones. Intensive building activity usually followed a major destruction such as a siege or earthquake. A date significant in the evolution of Hospitaller architecture was 1480-1481: it marks the first -unsuccessful- siege of the town by the Ottoman Turks and the devastating earthquake which followed scarcely a year later. Most important Hospitaller buildings retain some pre-1481 elements but underwent radical alterations after that date, under the rule of the grand-masters Pierre d' Aubusson, Emery d' Amboise and Fabrizio del Carretto, a period marked by intensive building activity. Then follow the building phases of the Turkish period after the widespread destruction caused by the successful siege of 1522 and the huge explosion that destroyed the area around the conventual church of St. John in 1856; and recently of the Italian occupation.

It is thus understood that the examination of edifices aiming to distinguish the various building phases is a very important first stage of any study and offers valuable information for the diagnosis of damages and the interpretation of the reasons which contributed to the historical evolution of the building.

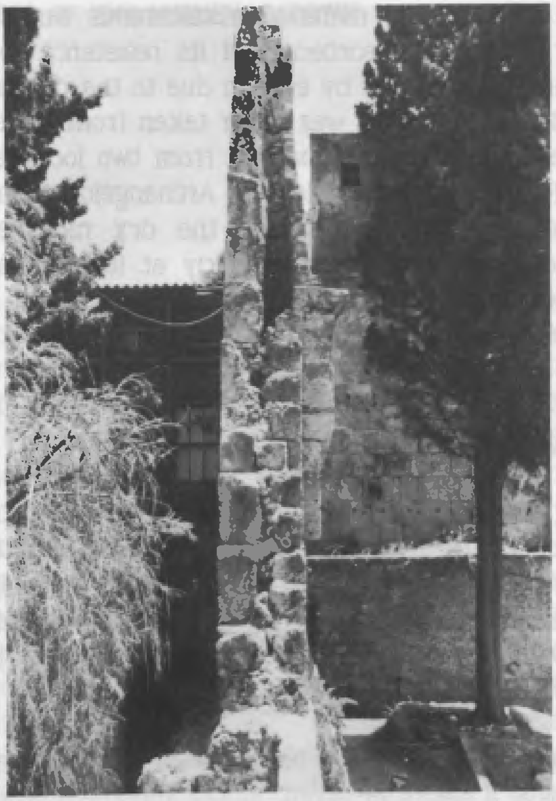
The load-bearing structure of the buildings in the medieval town includes stone masonry on the ground and first floors by the use of vaults as load-bearing structures for the upper floor and wooden beams for the flat roofs. The local building material, a calcareous sandstone of yellow-brownish colour, is highly absorbent and its resistance to compression is low. It is also seriously affected by erosion due to the deposition of salts by atmospheric and rising damp. It was either taken from ancient or Byzantine buildings and re-used or freshly quarried from two localities on the island, Sfoungaria and Psartos near the village of Archangelos. Stone has also been quarried in the process of deepening the dry moat surrounding the landward defences of the town, particularly at its SW part. Stones from different quarries show distinctive properties, which are currently being studied in the laboratories of the Athens Polytechnic.

Load-bearing stonemasonry walls

Load-bearing masonry of the Hospitaller period typically has few openings, are bonded by a singularly strong mortar and the technique of construction varies with time. In the early years Hospitaller walls were constructed with hewn stones of varying sizes, the height of a course being usually 45 cm with potsherds occasionally inserted in the joints, a method carried over from the preceding Byzantine period.

In Hospitaller buildings dating before 1480 the courses of the hewn stone masonry have a height of about 22 cm and the stones are small (20-30 cm wide with joints of 0,5-1 cm). In buildings of the second period (1481-1522) the stones are larger, 35-50 cm long, meticulously laid, with narrow joints (0,2-0,5 cm) in courses 21-22 cm high. The mortar employed in Hospitaller construction was lime-rich or a mixture of lime and pozzuolana. The latter was quarried in the neighbouring islands of Nissyros and Cos, where it is still abundant. This was an extremely strong mortar and was used as a rule in the construction of vaults and unplastered stone faces.

The ground-floor walls of Hospitaller buildings were 50-80 cm thick and consisted of two independent faces of hewn masonry without bond stones, the space between them filled with a mixture of rubble and mortar. First floor walls were usually of small thickness, particularly during the second Hospitaller period.



Even the thickness of the outer walls of the buildings consisted of a single block of stone hewn in the round with a pozzuolana mortar. These walls have suffered seriously, particularly in cases where they incorporated several openings or were of considerable height (up to 5 metres - e.g. the arcaded porch of the Hospice of St. Catherine).

In the construction of these walls, stones of various origins have been used not only in the same building but also in the same wall. As a result the walls have a polychrome appearance which is not incidental: it appears that the Knights knew the properties of the different varieties of stone at their disposal[1]. Thus they were using resilient stone for parts of the wall exposed to atmospheric damp. Stones of greater strength were used deliberately to frame openings and in critical positions like the courses forming the springing and crown of vaults. The presence of reinforced belts in the form of arches incorporated in the masonry is attested in places where partition walls were carried in the floors above, and is evidence of planning in the construction of a building.

[1]. These observations were made during the restoration of the Hospice of St. Catherine.

In constructions of the Turkish period the skilled tradition of the Hospitallers was continued, particularly on load-bearing walls of the first floor, while lightweight timber-framed walls are encountered very occasionally. The 18th century upper-class house with its repetition of openings in two parallel levels has proved extremely vulnerable structurally: no reinforcing vertical or horizontal elements were employed in walls 20-25 cm thick. This, in conjunction with the use of low quality stone even on outer walls (e.g. House of Hassan-Bey and Villaragut) creates serious difficulties for the restorer who aims at the structural consolidation of the buildings.The construction of thin walls here differs from corresponding Hospitaller structures. The outer "better" face is carefully laid with courses about 20 cm high (this is not always the rule, though) with very thin joints (0,2-0,5 cm); on the inner face, however, the joints were much wider (2-3 cm), with thin potsherds inserted in them with the lime mortar: this meant that in section the stones presented a narrowing on the inner side of the wall, being slightly wedge-shaped

During Italian restoration, which at times was simply the erection of new buildings in the Hospitaller style, the technique of the second phase of Hospitaller masonry construction was followed. This was often combined, however, with modern methods of structural consolidation like reinforced concrete elements (slabs, beams, lintels), strong cement mortars to strengthen the surface of stone masonry walls and even metal tension rods employed internally in arcades etc. Now, fifty years later, we are able to observe that the buildings show serious deterioration caused by the decay of the metal due to the great permeability of the sandstone; also the erosion of the latter through contact with the cement due to retention of moisture and the consequent deposition of sulphuric salts.

Upper storey floor structure

For the construction of the upper storey floor structure the use of vaulting, particularly barrel-vaults[2], became universal during the second Hospitaller phase; vaults were added even on earlier structures, an arrangement that improved the seismic resistance of the edifice.

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[2]. Examples of similar constructions can be seen in the Palace of the Knights, the house of Villaragut, the Hospice of St. Catherine, the Inn of Spain, etc.

Roofs

Hospitaller roofs used thick Anatolian cedar beams as load-bearing elements, slightly inclined to improve the drainage of rainwater. The typical construction of the Knights, with stone and timber corbels, and purlins overlaid by planking, was also used in Turkish buildings. Important Hospitaller edifices like the "Kastellania" and the Hospice of St. Catherine still preserve their painted ceilings. For the insulation of the flat roof a layer of lime mixed with small stones corrected the final drainage angle and this was covered with a thin, polished waterproof layer composed of powdered clay potsherds, mixed with lime and sand ("kourasani"). In buildings of the Turkish period this construction was replaced by a layer of clay ("patelia") whose of moisture-retaining properties necessitated its replacement at regular intervals.

The timber roof is the most vulnerable part of the buildings in the old town; the failure to ensure complete insulation from moisture without constant maintainance has caused serious problems from dampness which rots the timber load-bearing structure. The great height of upper storeys in combination with the slenderness of the walls, the often considerable surface erosion of the stone and the unreliable lateral cohesion of the roof area are structural problems which have to be effectively met by the restorer, given the increased earthquake risk of the area.

RESTORATION WORK BY THE OFFICE FOR THE CONSERVATION & RESTORATION OF THE MEDIEVAL TOWN

General Principles

A fundamental restoration principle is the treatment of a building as a contained unit rather than as a sum of separate elements. Efforts are made to preserve the original structural disposition with suitable repair - consolidation of load-bearing elements, use of the walls to carry at least part of the load and the avoidance of incorporating new structural elements which usually prove incompatible with the original structure. The aim is to ensure the reliability of the load-bearing structure as a contribution to the lifespan of the building; to be achieved through the linkage of vertical and horizontal carrying elements and correct setting/anchoring of the floors and roof in order to better exploit their connective capabilities.

Conservation of Buildings

The aim of conservation is to preserve the original structure and building materials as much as possible. Depending on the condition of the foundations repairs include surface consolidation, masonry wall reinforcements, or additional concrete foundations.

Masonry repairs in load-bearing walls include joint cleaning and pointing. In the case of extensive stone decay stone replacement is advisable. Depending on the condition of a wall reinforced plastering, grouting and other special techniques may be used. In the case of severe damage the rebuilding of the wall or its consolidation with an interior concrete layer is inevitable.

During the restoration of the Hospitaller house in 49 Pythagora street its walls, which have been seriously damaged due to the explosion of a World War II bomb nearby, a wall has been rebuilt using as much of the original building material as possible. The same method will be applied to the SE wall in the Hospice of St. Catherine.

Research on the composition of mortars continues in order to achieve durability. Experiments are being carried out with the use of materials such as white cement, pozzuolanic additives or volcanic ash in mortars.

In repairs of stone ornamentation the original parts are being preserved as much as posible. New parts are simplified versions of the originals in order to be easily distinguishable.

Special care is being given to the binding of horizontal and vertical load-bearing elements. Roof construction follows traditional principles with the use of modern materials. During restoration the original structural forms are preserved and attempts are being made to reduce nonessential loads.

Repairs in the medieval fortifications

In places where the fortifications are urgently in need of attention, lest they deteriorate further, we can distinguish:

a) Repairs on the face of sections of wall, towers and the counterscarp of the moat, in areas where weathering is advanced, with the methods already mentioned above. Such work seeks to "heal" discontinuities in the masonry by replacing material no longer able to perform its structural function; the new building material used here is as close to the original as possible

b) Repairs of damage to the foundations of the masonry where the gradual deepening of the dry moat and the fragmentation of weak sections of the bedrock undermine the masonry and may eventually lead to its collapse. In such cases new masonry is constructed, in slight recess to distinguish the intervention from earlier work, with a foundation of reinforced concrete constructed in blocks about 5 metres wide

Grouting will provide the means of carrying loads from the overlying old masonry onto the new section and will fill empty spaces in the cracked bedrock at the same time.

ARC DE TRIOMPHE DE L'ETOILE - PARIS

Michel MAROT *

Michel BANCON *

SUMMARY

In 1983, a stone weighing 200 grams fell from the vault.

In 1985, 800 kilograms of stones in danger of falling were removed. This operation revealed the presence of continuous cracks, which indicate the movement of the monument.

In 1987, a survey was made to determine the measures to be taken in restoring the monument.

In 1988-89, two programs of restoration work were carried out :

A) Façades : Cleaning, in addition to :

a) Replacement of damaged stone blocks and sculkptures;
b) Mineralization of blocks which suffered surface damage oly;
c) Waterproofing the whole monument.

B) Masonry :a) Injection consolidants and mineralisants into the joints of the foundations;
b) Placing three networks of tension cables above the vaults;
c) Waterproofing the ground-level esplanade.

* Architecte des Bâtiments Civils et Palais Nationaux.
* Ingénieur Conseil.

15 juin 1989

ARC DE TRIOMPHE DE L'ETOILE
RESTORATION WORK 1988-1989

DRAPING AND FENCING

The historical monuments of France receive constant attention from the Direction du Patrimoine. These monuments are often hidden behind scaffolding or fences. This creates the opportunity for artists to decorate these coverings, important surfaces where artists may display their talents on a monumental scale. The ephemeral nature of these decorations gives the artists more freedom.

A safety net donated by the Lancelin-Locapose company provided for the workers' security and comfort. Catherine Feff painted the monumental covering in the french national colors (blue, white , and red), while Mlles Bouchon and Le Breton designed a large tricolor fresco for the fencing. This monumental urban decoration was sponsored by the following corporation : Stic B, groupe Pelège, CCF, Fédération Parisienne du Bâtiment, Compresseurs Envé, Layher, SPR, Tollens, and SEERI.

CLEANING

The method used in 1965 was repeated: it involves spraying the monument with water for several hours through piping that produces a pulverizing action, in order to remove sulfates impregnated in the stone by natural means. The blackened traces of recalcitrant concretions were removed by sandblasting. The traces of pigeon-resistant glue were removed with steam jets and an application of chlorure methylene gel. This work was conducted by SPR Company, using scaffolding provided by Layher Corporation.

TREATING MASONRY AND SCULPTURES

Detailed drawings for the replacement of stone blocks and sculptures were prepared as the cleaning work progressed. Rather than replacing stone blocks and sculptures which have begun to deteriorate, products and techniques developed in the 25 years since the last cleaning of the monument were used to reinforce these blocks.
At the same time, the Centre Expérimental des Bâtiments et Travaux Publics and the Laboratoire des Monuments Historiques at Champs-sur-Marne supervised in-situ tests on new products developed by Rhône-Poulenc and Dupont de WNemours. These tests, performed on identical materials subjected to identical environmental conditions, will provide useful information for dealing with a wide range of preservation problems encountered by the Monuments Historiques.
These products, called consolidants or mineralisants, are able to

regenerate damaged stonework. The one employed here was PR 70, a Rhône-Poulenc product. Of course, only damaged blocks were treated. The masonry work was executed by the Rontaix and Quelin companies.

SCHEDULE OF WORK

Scaffolding	May - june 1988
Cleaning	July - August 1988
Masonry and sculpture work	August - November 1988
Mineralization	October - November 1988
Waterproofing	October - December 1988

CONSOLIDATION

After a small stone fell from the monument, the structure was thoroughly inspected in 1985. This inspection revealed the presence of cracks caused by the movement of the structure; a safety net was placed under the vaults. Specialized laboratories and companies, including the CEBTP, Simecsol, Rontaix, and Soletanche, conducted the following studies :

a) Measuring vibrations at the ground level and in the upper parts of the monument.

b) Placing measuring instruments (with a precision of 1 micron) on all cracks and recording movement-inducing temperatures.

c) Placing 16 leveling measurement devices (with a precision of 1/100 mm).

d) Measuring the rotation and verticality of the piers.

e) Verifying the horizontality of the cornices.

These studies revealed settling in the foundations and a spiral movement in the structure centered on the notrhwest pier.

Rainwater from the ground-level esplanade, the facades, and the terrace is directed into collectors, which are undoubtedly leaky. These collectors originally emptied into a pit under the esplanade on the Avenue Carnot side of the monument; around 1900, this rainwater was redirected into the sewer of the quartier Carnot, and after 1965, into the quartier Iena sewer. This flox of rainwater is responsible for washing the joints of the foundations, which produced a significant change in the composition of the mortar (atmospheric salts); this produced the differential settlement we observed.

The terrace has taken on a distorted shape : it has a slight concavity on the long facades, and a slight convexity on the short facades. A relaxation method analysis indicated that the loads are unequally distributed between the piers (from 0 to 50 bars).

In order to stop the deterioration of the foundation joints, the following sequence of work was planned out and undertaken:

a) January-March 1988 - Testing the injection procedure under 1/8 of each pier, and under 1/4 of the northwest pier.

b) September 1988 - Performing injections in the cracks above the vaults.

c) September-December 1988 - Installing tension cables in the superstructures.

d) January - April 1989 - Performing injections in the remainder of the foundations.

e) April-June 1989 - Waterproofing the surface paving of the esplanade.

Injections in the foundations

At 3-meter intervals, holes were drilled at angles ranging from 30° to 70°, thus allowing acces to all points where injections were to be made. The material removed from these drill holes was analyzed in order to find ways to avoid disturbing the foundations or getting the joins too wet during the injection process.

Two liquid compounds were chosen for the injections : Microsol, a cement-base product with a low viscosity, was used to fill cavities ands pores in the portar; it was injected at a pressure of 4 bars. Silacsol, a chemical product which reinforces disintegrating portions of the mortar by inducing a remineralization of the mortar, was injected at a pressure of 2 bars.

The injection process was carried out in three phases :

1) The primary drilling was used to inject Microsol.
2) The secondary drilling was used to examine the results of the primary injection, and to inject additional quantities of Microsol.
3) The tertiary drilling was used to examine the results of the first two injections, and to inject Silacsol.

Injections in the superstructure

As in the case of the injections performed in the foundations, a product was carefully chosen to be inject into the cracks of the superstructure before the tension cables were tigntened. This product was selected to match the superstructure's stone in hardness and imperviousness to water. The outer revetments of the walls, which are 2m50 thick, are of a very high quality stone. The infill, however, consists of inferior materials. This situation was the cause of severe difficulties.

Installation of tension cables

On three levels - above the small vaults, above the main vault, and in the lower flooring of the great hall - 50 pairs of tension cables were installed. These three networks are composed of intersecting cables running underneath the floor paving; these cables were routed around obstacles. The installation of these cables was made more difficult because we wished to avoid restricting public access to the monument while the work was underway.

The anchors set in the outer walls were installed from the inside, thus leaving them invisible from the exterior. The force in each cable is regulated at its center; the networks actually consist of 200 half-cables.

This work was carried out by the Frayssinet, Soletanche, and Rontaix Companies.

Public accomodations

In order to improve public accomodations in the monument, it was decided to install the following facilities at the level of the pedestrian access tunnel :

1) Restroom facilities
2) Ticket booths
3) A baggage checkroom.

This work was carried out between January and May 1989 by the Rontaix Company.

Illumination

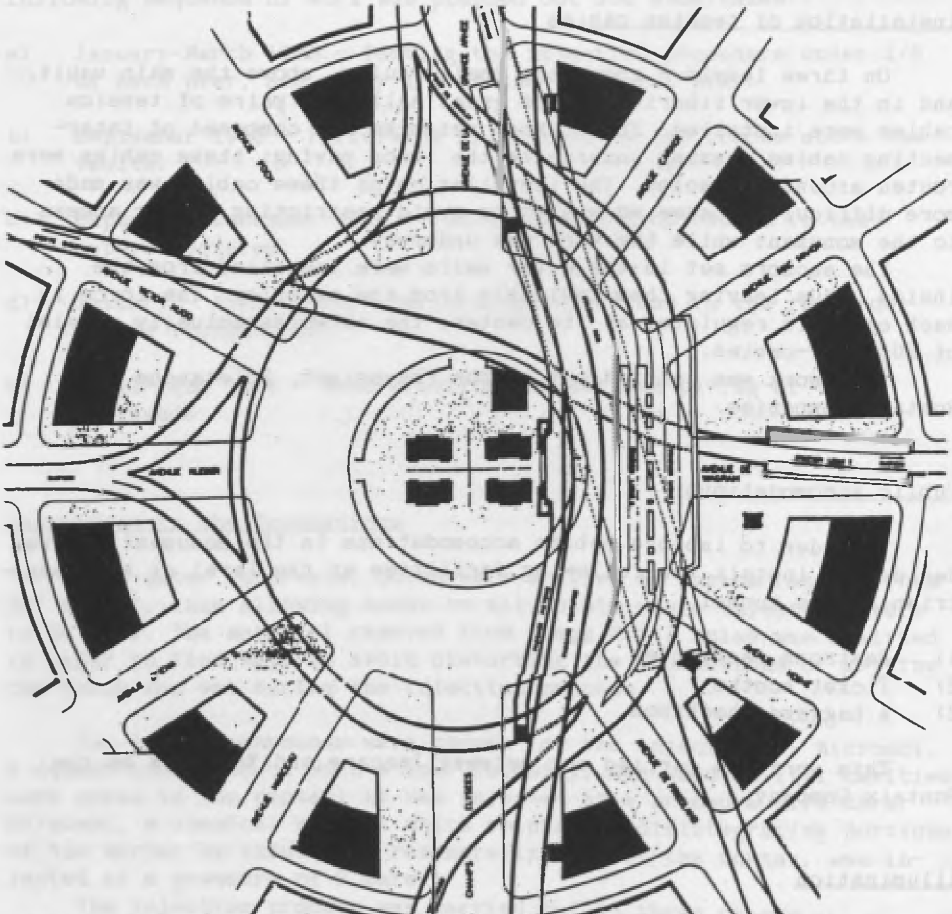
The waterproofing and paving of the ground-level esplanade also permitted us to design and install a new lighting system. Much progress has been made since the first lighting system was installed 30 years ago, so it was possible to reduce the size of the lighting wells. In addition, the new glass panels used to cover the lighting wells have eliminated the daily openings and closings of the trap doors formerly in use. Sodium-vapor lights are used in the new system.

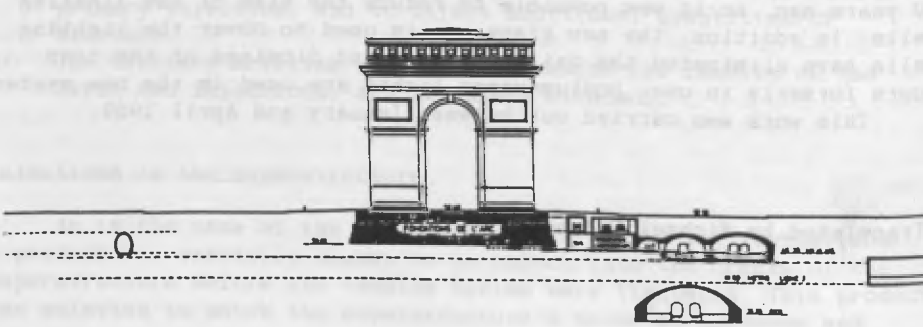
This work was carried out between January and April 1989.

(Translated by Michael Rabens)

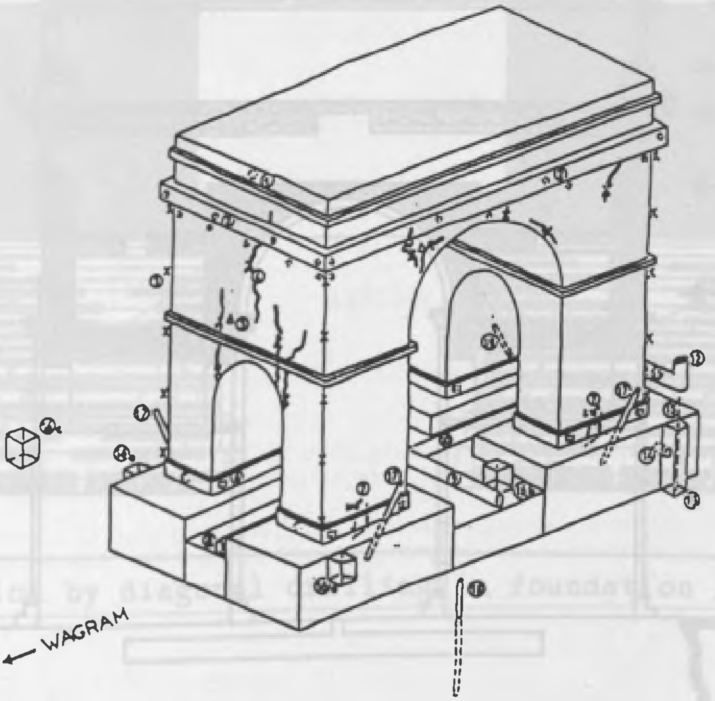
Place de l'Etoile

Underground Railways and Roadways





Locations of measurements and interventions

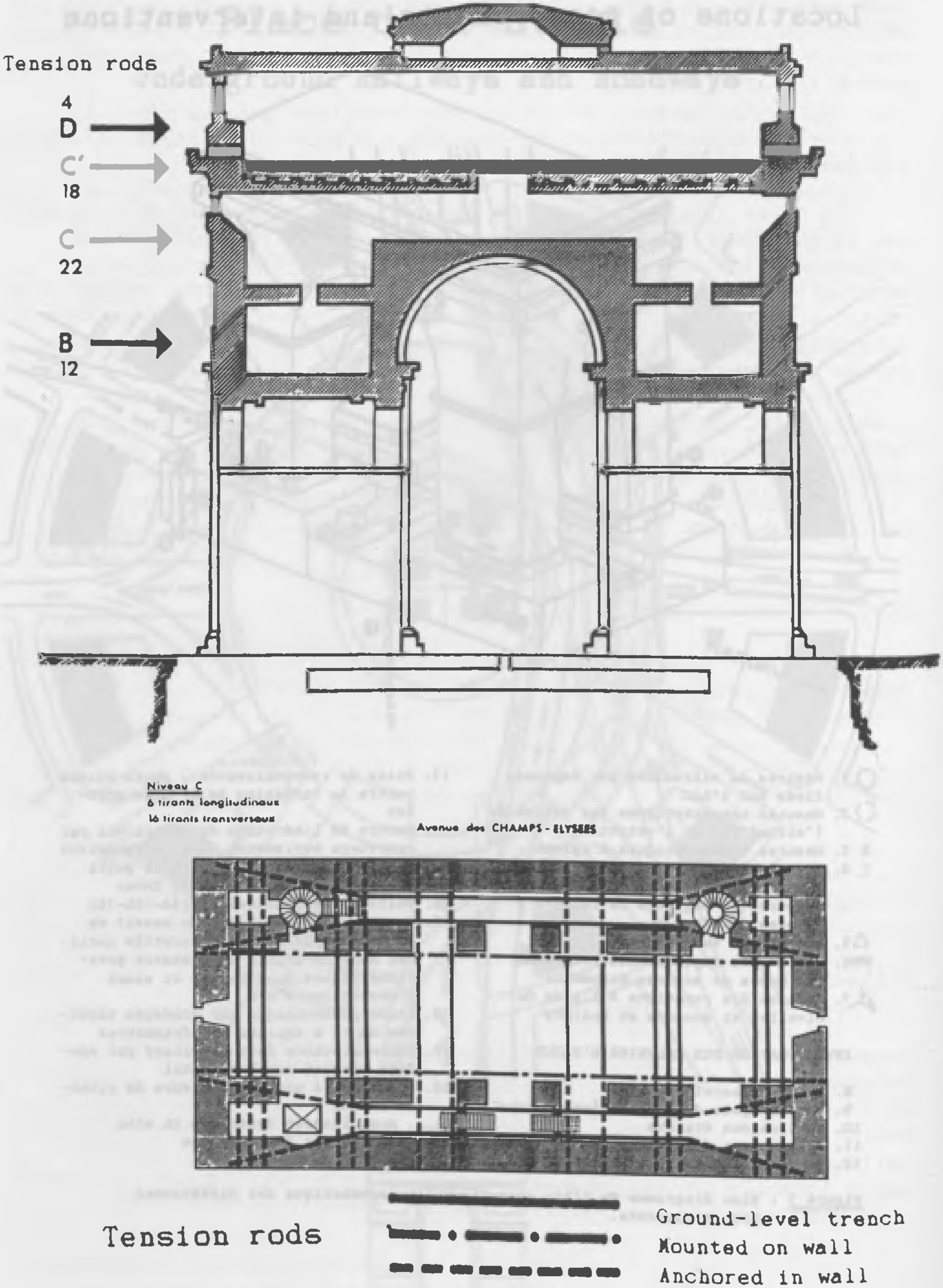


1. Mesures de vibrations par capteurs fixés sur l'Arc
2. Mesures topographiques par relevé de l'altimétrie de l'entablement
3. Mesures topographiques d'aplomb
4. Mesures d'évolution de fissures par fissuromètres électriques sur les fissures et boîtiers de lecture à la base de l'Arc
5. Mesures de température
6. Mesure des déplacements verticaux par bases de mesures Nivomatic
7. Mesures des rotations à l'aide de nivelles et données en radians

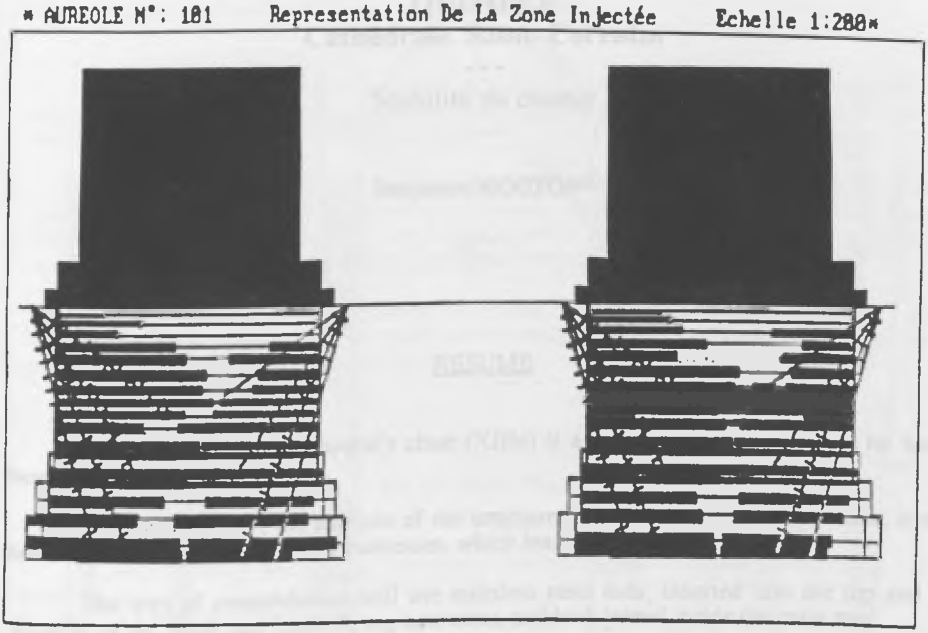
INVESTIGATION DES GALERIES D'EGOUT

8. Egoût en service
9. Ancien égoût désaffecté non étanche
10. Portion non étanche
11. Branchement d'égoût
12. Puits de visite
13. Puits de reconnaissance : puits blindé contre la fondation de la pile Nord-Est
14. Mesure de libération de contrainte par carottage horizontal dans la fondation
15. Mesure de déplacement dans le puits par convergencemètre à fil Invar
16. Puits de reconnaissance 16a-16b-16c
17. Reconnaissance interne du massif de fondation par sondages carottés inclinés dans le massif avec essais pressiométriques dans le sol et essai d'absorption d'eau
18. Etude géotechnique par sondages carottés de 15 m équipés en piézomètres
19. Reconnaissance dans un pilier par sondage carotté sub-horizontal
20. Etude de la pierre et mesure de vitesse du son
    - Auscultation dynamique in situ
    - Essais sur échantillons
    - Essais de ravalement

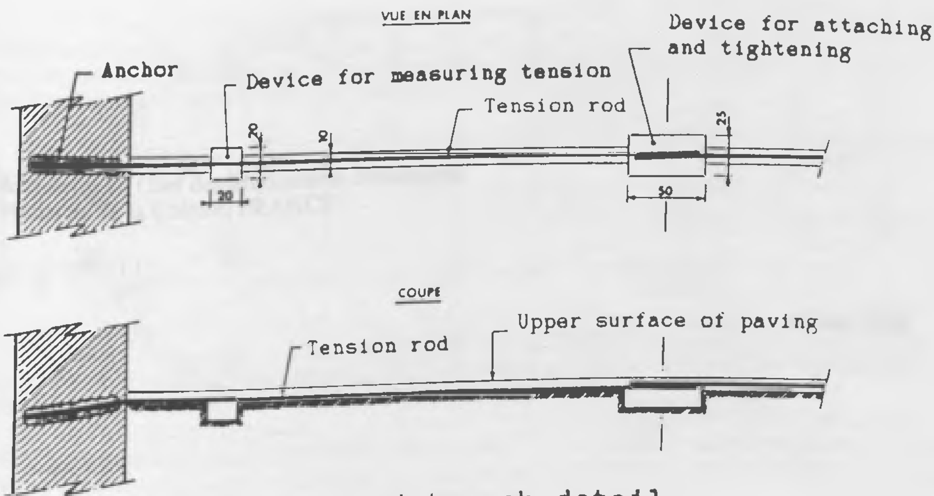
Figure 3 : Bloc diagramme de l'Arc avec figuration schématique des différentes reconnaissances.



Tension rods
4
D
C′
18
C
22
B
12
Niveau C
6 tirants longitudinaux
16 tirants transversaux
Avenue des CHAMPS - ELYSEES
Tension rods
Ground-level trench
Mounted on wall
Anchored in wall



Injection by diagonal drilling in foundation joints



Tension rod trench detail

QUIMPER
Cathédrale Saint Corentin

Stabilité du choeur

Benjamin MOUTON*

RESUME

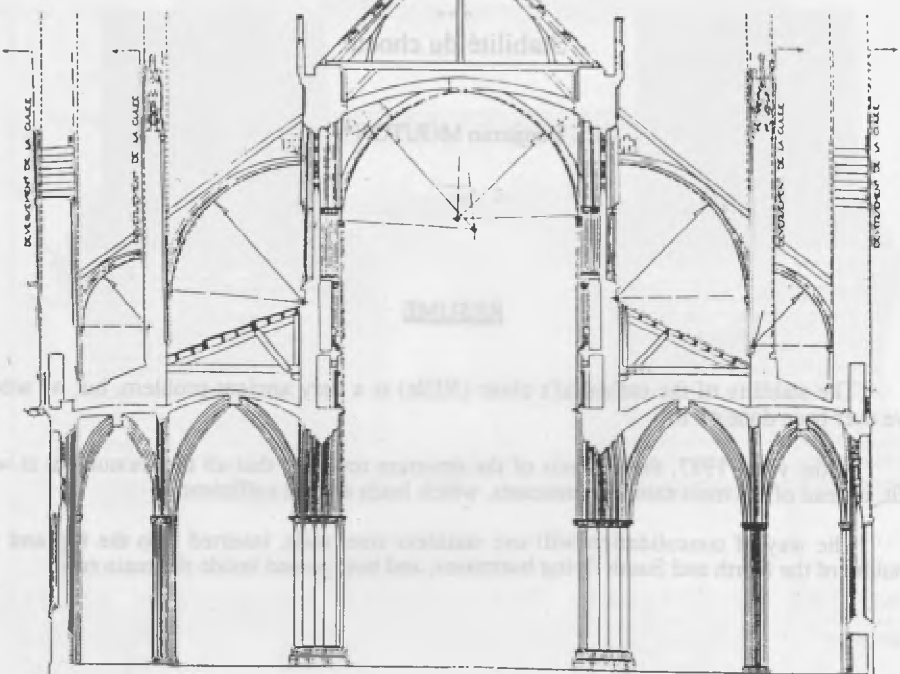
The stability of the cathedral's choir (XIIIe) is a very ancient problem, but no works have ever been done on it.

In the years 1987, the analysis of the structure revealed that all the monument is well built, instead of the most exterior buttresses, which loads are not sufficient.

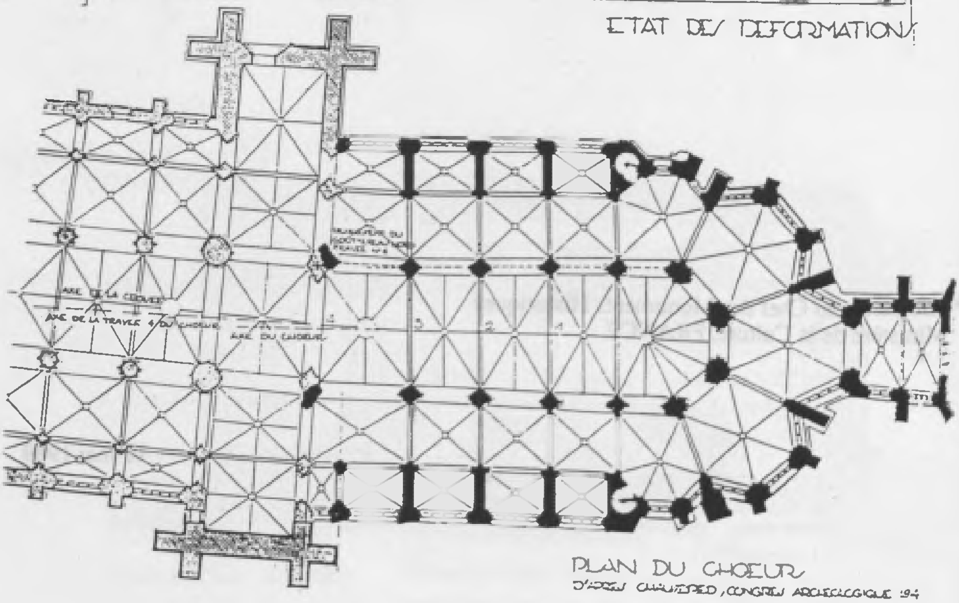
The way of consolidation will use stainless steel rods, inserted into the top and the shoulder of the North and South flying buttresses, and both joined inside the main roof.

* Architecte en Chef des Monuments Historiques
Ministère de la Culture, FRANCE

18 mai 1989



ETAT DES DEFORMATIONS



PLAN DU CHOEUR

D'APRES CHAUSSEPIED, CONGRES ARCHEOLOGIQUE

ETAT ACTUEL

Le choeur de la cathédrale Saint Corentin est composé de quatre travées barlongues, avec déambulatoire et chapelles latérales, terminé d'une abside polygonale à trois côtés, cinq absidioles, et une chapelle mariale saillante.

L'élévation intérieure est à trois niveaux (grandes arcades, triforium et baies hautes), couverte de croisées d'ogives quadripartites à liernes, contrebutées de batteries d'arcs-boutants simples à deux volées successives. Les culées externes sont à l'aplomb des murs gouttereaux, lesquels sont dépourvus de contreforts, sauf dans la partie tournante.

L'ensemble est bâti en maçonnerie de granit, dressée en grand appareil dans les parties vues intérieures et extérieures. Les voûtes sont en maçonnerie de moellons enduits, nervures de granit, avec clés armoriées et polychromes.

ETAT SANITAIRE

Les défauts de stabilité de l'ensemble voûtes/arcs-boutants du choeur sont connus depuis longtemps.

1 - Les travaux de charpente, qui ont eu lieu en 1777, ont redressé la pente des fermes anciennes à l'aide de fourrures : doit-on y voir un effet du déversement des gouttereaux, ou un relèvement des chéneaux ?

On peut néanmoins supposer que c'est à cette date qu'ont été mis en place les premiers tirants métalliques reliant les gouttereaux aux entraits de charpente, car en 1802, ils sont décrits "en mauvais état".

Entre 1862 et 1867, l'ancrage des tirants sur les entraits de charpente a été reculé. Par contre, aucun refichage des fissures ne semble avoir été exécuté.

2 - C'est très probablement en raison de ces désordres que les baies de circulation traversant les culées externes au niveau des chéneaux ont été murées au Nord. On peut supposer que ces travaux sont apparentés à ceux du XIXe siècle, sans oublier que c'est surtout en 1856 que l'on évoque, de façon claire, les problèmes de stabilité des voûtes.

3 - L'analyse des déformations, effectuée à l'aide du relevé stéréophotogrammétrique, révèle :

. une parfaite stabilité des gouttereaux du haut choeur et des bas-côtés ;
. un important déversement des culées d'arcs-boutants, de l'ordre de 10 cm pour 10 m au Sud, jusqu'à 25 cm, soit 2,5 % au Nord.
. d'importantes déformations des voûtes d'ogives de la nef et des arcs-boutants.

4 - L'analyse des sols a fait l'objet, en octobre 1987, d'une campagne du CEBTP.

Des carottages de 8 m de profondeur ont été exécutés au Nord et au Sud ; ils ont révélé une succession de couches de remblais, argiles et schistes altérés, avec pendage vers la rivière au Sud, de l'ordre de 30 %.

Les calculs de charge ont conclu à des compressions situées entre 2,5 et 3 bars, ce qui, pour des fondations reconnues jusqu'à 1,50 et avec les caractéristiques des sols obtenues par les essais pressiométriques, entraînerait des tassements approchés de l'ordre de 0,014 à 0,011 mètres, ce qui est considéré comme satisfaisant.

C'est donc au niveau de la stabilité des voûtes et des arcs-boutants qu'il faut rechercher la cause des désordres.

ANALYSES DE STABILITE

Exposé de la méthode

A - VERSANT NORD

1. Gouttereau haut

Somme des poussées = - 1,1 T
Somme des poids = 160,6 T

La trajectoire des pressions est très correctement centrée dans la masse des maçonneries. L'interruption par la coursière et le triforium peut avoir comme effet de reporter vers l'extérieur une partie des pressions, expliquant ainsi les fissures qu'on y rencontre.
L'arc-boutant et la voûte sont correctement implantés.

2. Culée médiane d'arc-boutant

Somme des poussées = 3,1 T
Somme des poids = 19,8 T

La trajectoire des pressions est excentrée, et paraît sur le point de sortir de l'aplomb externe de la culée au moment où celle-ci rencontre le mur diaphragme situé dans le comble sous la toiture. La stabilité est donc limite.

3. Culée externe d'arc-boutant

Somme des poussées = 1,4 T
Somme des poids = 22,6 T

La trajectoire est excentrée, et la résultante sort de l'aplomb externe de la culée, au-dessous de la corniche. L'effet de surplomb (35 cm) de celle-ci est de nature à aggraver de façon déterminante cette situation. On doit donc conclure à un état de déséquilibre de la culée externe.

B - VERSANT SUD

1. Gouttereau haut

Résultats identiques à ceux du versant Nord.

2. Culée médiane d'arc-boutant

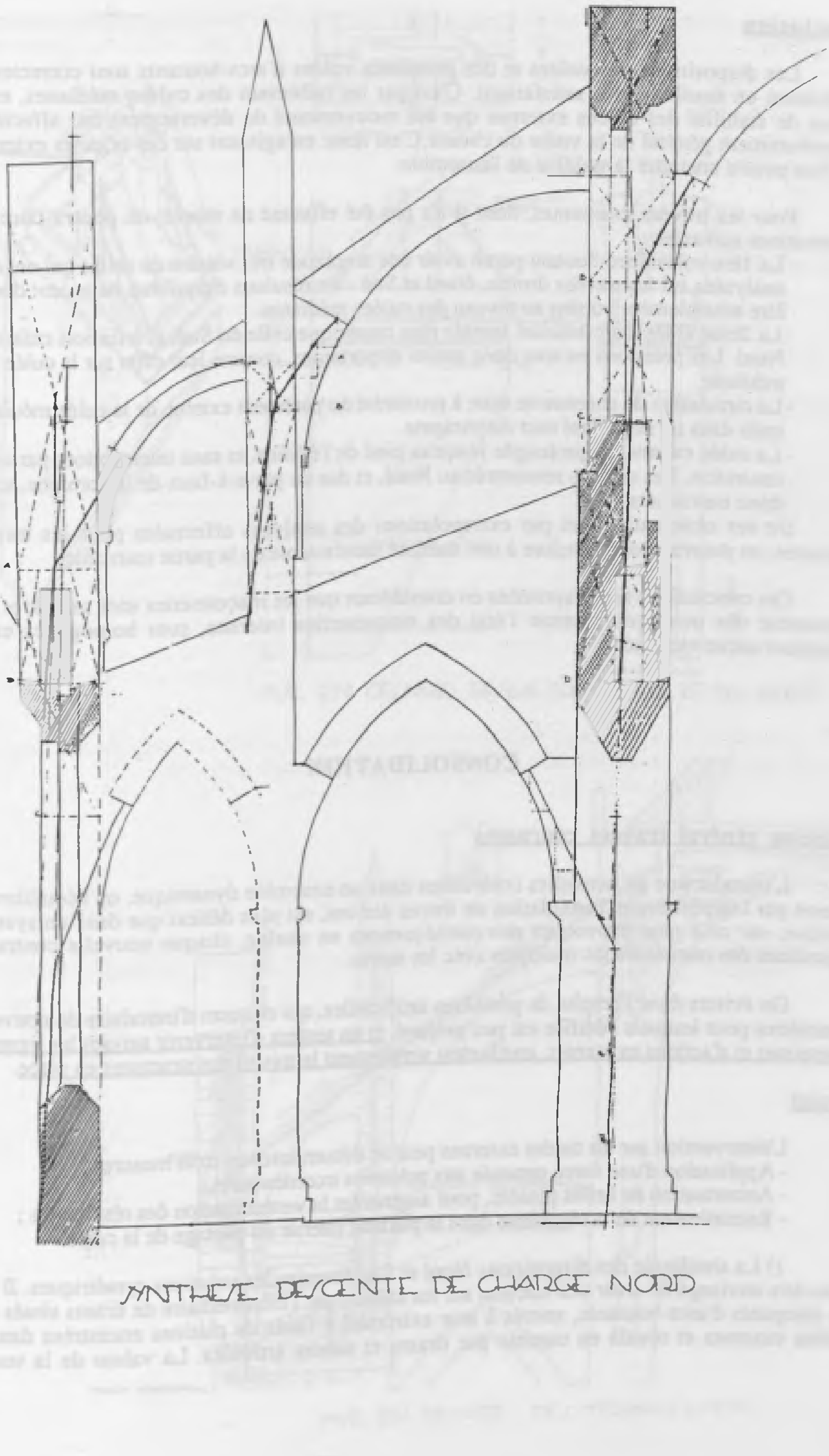
Somme des poussées = 3,3 T
Somme des poids = 24,2 T

La trajectoire est excentrée, et comme celle du Nord, sort de l'aplomb externe de la culée, à l'approche du comble. La stabilité est donc, là aussi, limite.

3. Culée externe d'arc-boutant

Amplitude plus courte que celui du Nord (50 cm sur moins de 2 m), et d'une inclinaison supérieure, de l'ordre de 4,5 °.
L'équilibre peut être considéré comme presque satisfaisant du fait de deux facteurs :

- Inclinaison plus forte de l'arc-boutant : la résultante des pressions est plus inclinée, et l'action de l'effet pinacle de la culée s'opère plus bas.
- L'aplomb externe de la culée n'est pas en porte-à-faux sur le gouttereau, mais c'est au contraire celui-ci qui marque une légère saillie.



SYNTHESE DESCENTE DE CHARGE NORD

Conclusion

Les dispositions des voûtes et des premières volées d'arcs-boutants sont correctes et produisent un équilibre très satisfaisant. C'est par les faiblesses des culées médianes, et le défaut de stabilité des culées externes que les mouvements de déversement ont affecté le contrebutement général de la voûte du choeur.C'est donc en agissant sur ces organes externes que l'on pourra atteindre la stabilité de l'ensemble.

Pour les travées tournantes, dont il n'a pas été effectué de relevé, on pourra faire les observations suivantes :

- La 1ère volée d'arc-boutant paraît avoir une amplitude très voisine de celles qui ont été analysées sur les travées droites, Nord et Sud : les résultats d'équilibre devraient donc être sensiblement voisins au niveau des culées médianes.
- La 2ème volée d'arc-boutant semble plus courte que celle du Sud, et à fortiori celle du Nord. Les pressions en sont donc moins importantes, comme leur effet sur la culée médiane.
- La circulation du chéneau se situe à proximité du parement externe de la culée médiane, mais dans un important mur diaphragme.
- La culée externe est prolongée jusqu'au pied de l'édifice, et sans interruption, par un contrefort. Les défauts rencontrés au Nord, et dus au porte-à-faux de la corniche, sont donc inexistants.

De ces observations, et par extrapolations des analyses effectuées pour les travées courantes, on pourrait donc conclure à une stabilité satisfaisante de la partie tournante.

Ces conclusions sont exprimées en considérant que les maçonneries sont saines, et que l'ensemble des ouvrages, comme l'état des maçonneries internes, sont homogènes et ne présentent aucun vice caché.

CONSOLIDATION

Principe général travées courantes

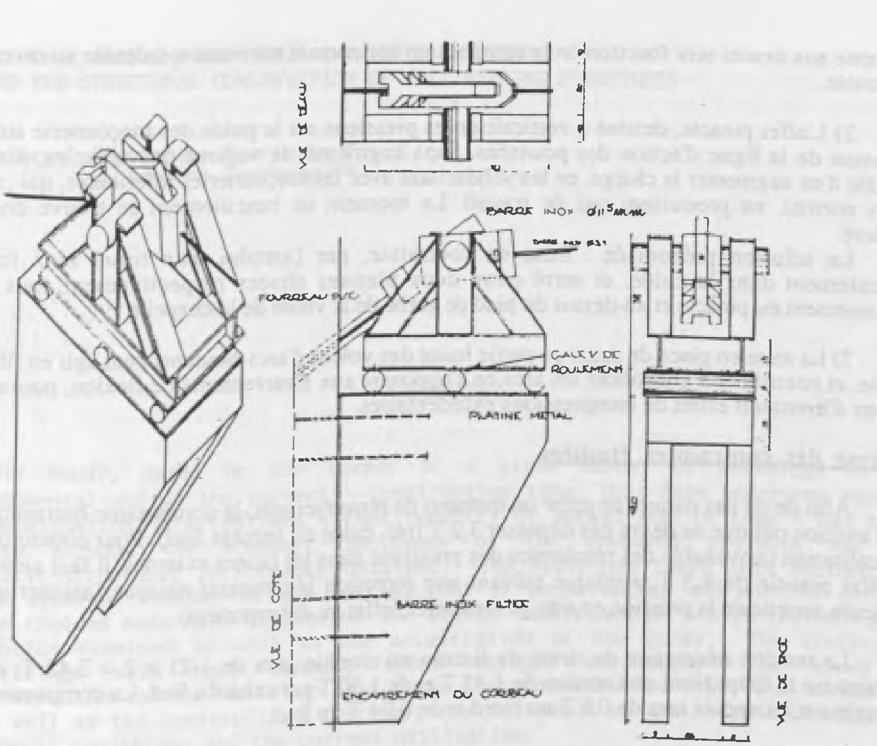
L'introduction de nouvelles contraintes dans un ensemble dynamique, où l'équilibre est obtenu par l'opposition et l'annulation de forces actives, est plus délicat que dans un système statique, car celà peut provoquer des conséquences en chaîne, chaque nouvelle contrainte engendrant des combinaisons multiples avec les autres.

On évitera donc l'emploi de prothèses artificielles, qui risquent d'introduire de nouveaux paramètres pour lesquels l'édifice est peu préparé, et on tentera d'intervenir suivant les lignes de contraintes et d'actions existantes, améliorant simplement le travail des structures en place.

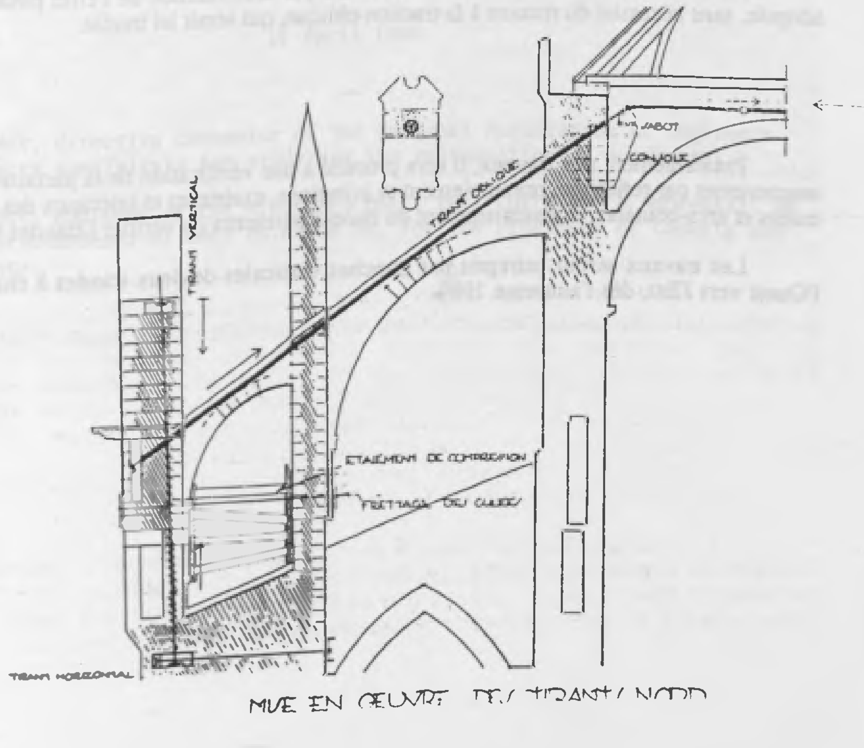
Projet

L'intervention sur les culées externes peut se décomposer en trois mesures :
- Application d'une force opposée aux poussées excédentaires ;
- Accentuation de l'effet pinacle, pour augmenter la verticalisation des résultantes ;
- Recentrement de la résultante dans le piédroit interne du passage de la culée.

1) La similitude des dispositions Nord et Sud favorise les solutions symétriques. Il peut donc être envisagé de créer une traction sur les culées, par l'intermédiaire de tirants situés sous les rampants d'arcs-boutants, ancrés à leur extrémité à l'aide de platines encastrées dans les culées externes et réunis en comble par tirants et sabots articulés. La valeur de la tension



MISE EN OEUVRE DE LA CONSOLE ET DU JABOT



MISE EN OEUVRE DES TIRANTS NORD

imprimée aux tirants sera fonction de la composante horizontale nécessaire, calculée au niveau de la culée.

2) L'effet pinacle, destiné à verticaliser les pressions est le poids des maçonnerie situé au-dessus de la ligne d'action des poussées. Sans augmenter le volume des pinacles, il est possible d'en augmenter la charge, en les solidarisant avec les maçonneries inférieures, qui , en temps normal, ne produisent pas de travail. Le moment de basculement se trouve donc amélioré.

La solution préconisée : mise en contrainte, par l'emploi d'un tirant inox foré verticalement dans la culée, et serré entre deux platines situées respectivement sous le couronnement du pinacle et au-dessus du pied de gerbe de la voûte de la chapelle.

3) La mise en place de tirant en partie haute des volées d'arcs-boutants peut agir en fibre tendue, et contribuer à consolider les arcs en s'opposant aux mouvements de flexion, pouvant résulter d'éventuels effets de compressions excédentaires.

<u>Analyse des contraintes étudiées</u>

Afin de ne pas risquer de créer un moment de renversement, la composante horizontale de la traction oblique ne devra pas dépasser 1,2 T (réf. culée du versant Sud). Pour obtenir une verticalisation convenable des résultantes des pressions dans les culées externes, il faut ajouter un effet pinacle de 4,5 T, appliqué suivant une direction légèrement oblique (assurer une meilleure assiette de la pression en tête, et contrarier l'effet au déversement).

La tension nécessaire du tirant de liaison en comble sera de 1,21 x 2 = 2,42 T, qui produira sur le tirant Nord une tension de 1,45 T et de 1,50 T sur celui du Sud. La composante verticale sur les appuis sera de 0,8 T au Nord et de 0,94 T au Sud.

<u>Conséquence sur les travées tournantes</u>

Par sécurité, la technique du renforcement par accentuation de l'effet pinacle peut être adoptée, sans nécessité du recours à la traction oblique, qui serait ici inutile.

Préalablement aux travaux, il sera procédé à une vérification de la parfaite hygiène des maçonneries par refichage, rejointoiement et injections, extérieurs et intérieurs des gouttereaux, culées et arcs-boutants. L'assainissement du choeur permettra de vérifier l'état des fondations.

Les travaux seront entrepris par tranches verticales de deux travées à chaque fois, de l'Ouest vers l'Est, dès l'automne 1989.

ECONOMIC PROBLEMS AND THEIR IMPACT ON THE DESIGN
AND THE STRUCTURAL CONSERVATION OF WALL BEARING STRUCTURES

Autors: Giuseppina Ferriello *
Giuseppina Torriero **

SUMMARY

This study, based on the survey of a given number of buildings - both monumental and of the current - construction type, that have undergone partial consolidation interventions either recently or spread out in time - has taken into account the economic incidence of the executed projects and the static-structural performance obtained. The study has taken into account how the apparent economy of the initial cost is unconvincing and most of all how the choices made have determined the static conditions of a high percentage of fabrics examined as well as the acceleration of the decay. The listing of buildings also has, amongst other things, evidenced: the epoch of construction of the buildings and their transformation in time, the morphology as well as the constructive typology, the incidence of the seismic damage, the general conditions and the current utilization.

11 April 1989

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CURRENT BUILDING AND MONUMENTAL PATRIMONY

Implementation methods of the consolidation projects and the incidence of economics on the final results of the works.

This study, beginning with the critical analysis of some consolidation interventions performed on wall bearing structures - both monumental and of the current construction-type, aims to contribute to formulate a first case-by-case study of problems connects with the implementation methodology of consolidation projects and to verify the percentage of incidence of such problems on the outcome of the construction works as well as the performance in time. The study intends to demonstrate how, regardless of any budget issue, there are a whole series of elements that concur to make the final result: the methodological approach extended to the entire "ancient built patrimony", the discontinuity in the availability/distribution of funds (this to be considered the main cause of th discontinuity in the execution of the rehabilitation interventions and consolidation of buildings), the lack of quality craftsmanship employed on the building sites, the partial modifications of the static-structural model of the buildings, etc.

The results of the consolidation interventions, on wall bearing buildings particularly, does not derive only from the quality of design (1); often the economic variants and the discontinuity of the execution, influence to a large degree the construction work. The complexity deriving from the coexistence and the overlap of the aforementioned variants is often burdened by the overlap of the competences that produce time lapses between the various burocratic approvals necessary for a restauration project, before the various burocratic approvals necessary for a restauration project, before its implementation (2).

In light of research made on a vast number of sample buildings located in seismic areas that have undergone consolidation work (3), it has been possible to notice the incidence of economic factors on the project and on the structural conseravation of the buildings. The projects of static consolidation in the majority of the cases assume also the nature of Public Work since often we are witnesses to extensive interventions of the monumental and currently built patrimony in consequence to Special Decrees emanated after natural calamities, the exceptions of course are the isolated examples of scientific restauration executed on monumental buildings of high historic-artistic interest forever consacrated to fame (4); very few, however, are the monuments that may benefit of studies that approach with harmony the technical, critical-historical aspects and that may utilize fundamental preliminary studies to the making of the projects given to a group of the experts from various technical and historic disciplines closely connected to static and structural interventions of restauration.

In reference to particular research performed on the monumental patrimony unde scrutiny, two cases are considered: the cases whereby the variation to the original project in execution was determined by the findings of works intially hidden to view, from those cases, instead, in which the variation was determined by the static-structural worsening and by the decay due to pauses during the construction work.

Further, the increase of the expenses for the necessary provisional structures (such as saffolding), since the amounts of the single project phase financing provided had not allowed/consented to conclude at once all structural and finish work even in the reduced dimensions of areas of interventions. Such last possibility has been the most recurring one, with rare exceptions, and is always recurring constantly in the various samples analyzed by the research (5).

A further distinction should regard even the Public Bodies that provide financing. In fact, for example, it is not possible, for the interventions of the Ministery of Public Works (save for very rare cases) to budget in the projects presented for approval provisional works whose priority objective is to reduce to a maximum the possibility of decay of buildings during the long period it takes for the execution of the construction work. The research has pointed out how the economic cost of such structures would more than repay the increase in the costs that a designer or field inspector - in order to repay recurring damage produced by decay caused by atmospheric agents - is forced to foresee afterwards, during the execution of the works; these actions by thej designer or field inspector vary once more the design with the most direct consequence being the work stoppage in order to obtain the typical survey by the competent authorities. In any case, if for the restaurant of the monumental patrimony there is no theorical limit of expenditures, in reference to the possible financing of rehabilitation and consolidation of current buildings, the limited amount allowed by the available financing determines often only partial interventions on some part of the structures; here is then thhe ten fragmentation of the interventions that during a medium or long term demonstrate the risk for the lifes of buildings and even more often produced as direct consequence the acceleration of the decay (6).

The in-depth analysis of the research done for civil construction - in two areas of the historic center under scrutiny and the observations of the static worsening of a large part of the buildings consolidated immediately after the earthquake of the 23 November 1980, have brought to a list of more detailed data:

- the nature of the interventionsj executed - utilizing the survey on all the authorizations consented by the competent authorities before the initial work start date;
- the cost of the interventions;
- the presence of other work executed of foreseen for the immediate future;
- the total and final cost of the projects (7).

The results have been synthesizes in an all-encompassing note in which it is possible to read the interaction between: the materials and the construction techniques, the entity of the damage/quake, the type of works executed, the static conditions surveyed after the repairs (8).

HYPOTHESIS FOR A UNIFICATION OF THE RESEARCH

Once the various problems emerged have been identified, it should be useful to quantify the incidence of each problemj on the result of the intervention. This operation is however made difficult by the coexistence and by the interaction of the single variants that all concur to determine the final state of the intervention object and its resistence in time. This, of course, should be verified both in normal conditions and in presence of traumatic variants such as, for example, the natural calamity and first of all the seismic events which are the most recurring. With this objective it is indisensible to have available a vast series of data which consents the comparison with cases considered. The survey of the vast data gathering on the built patrimony and on the interventions on the same, allows the verification of the performance in time considering also the problems emerged in the construction pahse.

In light of recent definitions of "Bene Culturale", the groups of the buildings included in the historic centers and the ancient nuclei, which should be considered as such. The recurring uniformity of the methods and of the constructive techniques of the vast patrimony of the ancient buildings allows the possibility of using a typical index-card similar for buildings that constitute minor construction and monumental buildings since the methodological approach - in both cases we are dealing with a "Bene Culturale" - should be similar even taking into account, naturally, the indispensible distinctions that must characterize the single project and the single interventions.

A first index-card model useful for the survey which may be utilized widely with the objective of verifying the results of the initial research may be, with small modifications and integrations, the one already expermented in a positive way regarding the survey of data for minor buildings.

This index-card, conceived for a propedeutical survey for the design of the "Rehabilitatin Plans" of the historic center, may be considered valid for the ancient wall bearing buildings. It should take into account predominantly the constructive history of the building, the typology, the morphology and the constructive typologies, of the "vestustas" as well as the utilization of the building in time. Elements of particular interest are the surveys of the conservation state of the building at the time the index-card is filled as well as the entity and the location of the damages in consequence of calamities, repaired and to be repaired. Furthermore, in presence of repairs and consolidations already performed, notice should be taken also for the works executed and of all the problems that have emerged during the construction phase. In this way, it is possible to constantly update the index-card and it is, further possible to compare the conditions of the buildings in a vast time frame.

Following the sample survey made, it has been possible to verify the aforementioned and expecially notice a close existing relationship between the economic problems and the results of the interventions executed (9).

Naturally, it should be clarified, that a listing of the interventions either as a preventive measure and for works already executed, should not be considered a novelty, since the Charter of Venice tens of years ago, suggested the keeping of an updated restauration journal (interpreting also precedent suggestions). In any case, even though this is not the forum for writing on the scarse utilization made of the journal, still today, it is possible to evaluate the positive aspects of a tool much more simple to compile, such as a simplified index-card; such card may become computerized, useful also to those professionals who are not mainly concerned with restauration but are definitely the majority of the "operators" on the ancient-centers. Finally, thej institutional obligation of updating the index-card, may allow, at the level of Public Bodies, proposed for the protection (the State Administration) the formation of a data bank which in a country rich of monuments such as Italy, may guarantee in the future better results for the rehabilitation and the safeguarding of the vast patrimony considered "Bene culturale". It is superfluous to underline the utility of such a survey for those who are interested in the experimentation of the techniques and of the consolidation systems. The possibility of referring to a vast number of exampls of interventions executed and the analysis of the results also in relationship to morphology and to the typology in particular would represent a noticeable saving of time and a common reference for those who are concerned with the study of building and particular of that wall bearing which characterizes not only the monuments but generally all the current buildings which are commmon and always considered minor construction (9).

NOTES

1) There is a substantial difference between a consolidation and/or a restauration project and a building to be built ex novo: the restauration and consolidation project may be considered finished only when the construction site is closed due to its frequent possibility of finding initially hidden to sight structural and works of art, whose presence correctly requires the revision of the project from a structural and a design viewpoint. Therefore, also the necessity to have available on the construction site qualified craftsmanship in particular execution of works on the ancient constructions.
2) The financing for the execution of the works may come from various ministries and the consignee may be either a private citizen or a Public Body.
3) The initial research to which reference has been made to by this study, is the same performed by the designers Arch. M. Elena Alfano and Giuseppina Ferriello in the field, while preparing two "rehabilitation" plans for the historic center located in the town of Arienzo (Caserta) in the years 1984-1985, as well as the site surveys performed on the same locality after the seismical events of 23-11-1980 by technical staff so entrusted by the local town. The historical center considered is listed as a seismic area equal to nine (9) and has been greatly damaged by the quake.
4) In light of the modern definition of "Bene culturale" as such may also be considered the historic center with its construction, for a long time retained "minor"; it should be useful to revise the norms that regulate and interventions of works foreseen in the historic centers.
5) In reference to the monumental patrimony expecially if it is not famous, the very limited financing which is available from time to time has evidenced the following and most frequent partial interventions: partial consolidation of the foundation, demolition of the roof with consequential absence of temporary structures and atmospheric agents infiltration which provoke a recurrent decay of the structure and art-work.
6) The research performed on the minor building patrimony has demonstrated how the static conditions have worsened in about 40% of the cases, after the execution of particular consolidation works. The works performed were generally: partial substitution of vault and floors, roof demolition, cement mix injection generalized also to the part of the complex with the structural elements not interested. All works were financed by the "ordinanza commissariale N. 80/1981" which allowed 10 million of lire for each housing unit in order to repair seismic damage.
7) The law 219 of 14.51981 gave the owners of buildings already repaired the opportunity to present again another project in order to complete works in course or to forsee new works as necessary. About 90% of the owners responded to this new legislation; the total cost of the works foreseen in both projects has always gone beyond the cost of performing only one intervention. The major part of the cost included were due to the

provisions for interventions or areas already interested by recent precedent work. With reference to the economic survey the cost for the project and site supervision have not been taken into account.

8) Due to space problems it is not possible to attach to this document the images that offer the comparison between the static condition surveyed soon after the execution of the works of partial consolidation.
Hereafter, due to lack of space, follows only a synthesis of the principal values surveyed in reference to the structural morphology of the sample buildings observed; the vertical structures are indicated with the following abbreviated nomenclature: tufo stone wall (1); stone wall (2); mixed masony wall (3); brick wall (4); vertical structures in reinforced concrete (5) and as a result the structures observed in the historic center have been:

	first area	second area
housing units surveyed	355	115
buildings	48	46
vertical structures and their combination within same building:		
1-2-3-4-5	2	0
1-2-3-4	2	0
1-2-3	1	0
1-2	20	1
1	1	38
1-3	4	2
1-5	12	2
1-2-4	1	0
3	1	0
2	0	1
3-5	0	1
1-2-5	3	0
1-3-4-5	0	1
1-4-5	0	1
1-4-5	0	1

N.B.: the term "housing units" indicates also utilizations such as shops, deposits, offices and other uses.

9) The elaboration of the research on the built patrimony of the historic centers and the first chapter have been particularly investigated by Arch. G. FERRIELLO; the research on the monumental patrimony and the elaboration of the second chapter have particularly investigated by Arch. G. TORRIERO; the translation of the text into English has been done by Arch. F. POSSEMATO.

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6) The ... building patrimony has demonstrated ... in about 40% of the cases, after ... addition, ... part of the complex with the ... for ... the ... works in ... as necessary. About 90% ... tion; the total cost of the works foreseen in ... gone beyond the cost of performing only one ... part of the cost included were due to the

L'ENTRETIEN ORDINAIRE SUR LES SITES ARCHEOLOGIQUES: PROPOSITIONS ET CHOIX POUR LA CONSERVATION DES MACONNERIES A HERCULANUM

Valerio PAPACCIO* Antonio SALVATORI**

SUMMARY

The possibility of proceeding to longlasting conservation of the tuff stonework of the monuments of Herculanum must be the fruit nowaday's of global plans and projects, which see restoration operations as normal practice, avoiding the previous approach of seeing such interventions as an extraordinary matter. It is important that these interventions also take into account prevention systems against the damage that time, wear and atmospheric agents cause every day on the structures after they have been restored. In the past, in Herculanum, these problems were dealt with efficiently by small maintenance operations carried out by well trained workers. It is thus logical to propose that such previous types of intervention be included in a global strategy, aimed at a careful and thorough analysis of the real motives of degradation of the stonework.

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15 avril 1989

La restauration en tant qu'activité constante d'entretien ordinaire des monuments - en particulier de ceux archéologiques - constitue l'un des objectifs jamais atteints dans le cadre du binome problématique conservation-jouissance.

Cela est probablement dû à la discordance évidente entre gestion et programmation des opérations de restauration (toujours plus importantes et extraordinaires) et des interventions d'entretien (limitées et déclassées).

Une telle discordance a entraîné la dénaturation du sens même des termes, conférant ainsi à la restauration une importance notable de vulgarisation, au détriment de l'entretien ordinaire.

Heureusement, depuis quelque temps déjà, lorsque l'on aborde le problème de la survivance des parcs archéologiques (1), l'accent est mis sur le danger lié à l'absence d'un entretien constant et diffus, dans le cadre d'interventions de restauration multiples. Il arrive toutefois que les interventions se concentrent en opérations isolées de restauration - extrêmement coûteuses - qui ne sont que des "cathédrales dans un désert", et finissent par altérer les prévisions modestes de valorisation à peine ébauchées.

Par ailleurs, il est nécessaire de signaler l'apparition, relativement récente, d'un nouvel élément, "de principe", pourrait-on dire. L'exigence de la restauration documentaire (2) des ouvrages archéologiques étudiés sous leur aspect de ruines a entraîné la nécessité de les traiter comme des "objets architectoniques", avec le même soin que l'on accorde habituellement aux biens mobiliers de valeur historico-artistique.

En effet, il a été précisé qu'une intervention de restauration archéologique ne doit négliger aucune trace, aucun détail ou signe présent en courtine ou sur la structure.

Et ce pour plusieurs raisons.

En premier lieu, car - comme c'est le cas bien souvent - les ouvrages archéologiques se présentent à nous transformés par des restaurations datant de cinquante ans (parfois même cent ans).

En second lieu, car la restauration - en tant que nouvelle intervention - tend à remplacer et à régénérer la matière antique à travers des opérations symbiotiques qui provoquent souvent confusion et doutes.

En troisième lieu, car bien souvent la restauration est sélective et place au premier plan le respect de l'original antique, négligeant ainsi, voire atténuant, les aspets essentiels.

Enfin car, bien souvent, l'histoire du monument est peu connue (manque de temps?) et on a tendance à l'assimiler à celle d'autres monuments, en élaborant une casuistique structurelle et figurative de cas, basée sur des schémas, des typologies et des classifications erronés.

Par ailleurs, la baisse de l'attention que la restauration à grande échelle semble susciter, en raison des aspects précités, a bien souvent provoqué des conflits - irrésolus - entre techniciens et amateurs d'archéologie. Culturellement parlant, il est impensable d'accepter de semblables discordances entre catégories, alors que l'institution elle-même impose la présence conjointe de spécialistes.

Aux fins d'une réorganisation fonctionnelle future, il est indispensable de bien comprendre quels sont les secteurs et les référents d'une correcte conservation des monuments archéologiques, réalisée grâce à un recours accru à l'entretien ordinaire.

En ce qui concerne les fouilles d'Herculanum, la documentation photographique - qui date d'environ cinquante ans - montre, sans aucun doute possible, que la zone fouillée entre 1828 et 1875 (quartier ins.II et VI) bénéficiait il y a trente ans encore d'un entretien soigné, prouvé par la bonne tenue des jardins et la propreté de l'environnement. Après l'extension des fouilles (1927-1955), le problème de l'entretien s'est rapidement aggravé en 1965-1970, jusqu'à devenir insoutenable durant la dernière décennie.

Les effondrements répétés des toitures vétustes et lourdes d'Herculanum - qui se détériorent par manque d'imperméabilisants -

témoignent de la gravité du phénomène.

Comme ailleurs, les fouilles d'Herculanum souffrent tout particulièrement de la diffusion incontrôlée d'eaux météoriques, stagnantes et provenant des égoûts (3).

Le problème était déjà perçu au début du siècle. Même si la zone de fouilles était sensiblement plus réduite, elle subissait tout autant l'incidence négative des ravinements et des eaux stagnantes (situées à cinq mètres au-dessous du niveau du sol).

Les notes de surintendance (4) de l'architecte Luigi Fulvio, technicien de l'époque, mettent en évidence des préoccupations quant: "...aux pluies torrentielles qui remplissent le puits absorbant.." et encore: "...il est urgent de prendre les mesures nécessaires afin d'éviter que les eaux de pluie remontant du puits de dépot s'écoulent librement dans les fouilles...".

Par ailleurs, les rapports de l'architecte Fulvio (5) mettent en évidence la nécessité de réaliser les interventions de restauration grace à la mise en place de structures de renforcement, à des substitutions, à la réparation des faites des maçonneries. Toutes ces interventions sont classées sous la rubrique "petit entretien".

Entre 1920 et 1927, conformément à la pratique courante (6), les techniciens réalisaient conjointement les travaux plus spécifiquement structurels, à savoir la mise en place de tuyaux d'écoulement ou la fourniture de matériaux d'utilisation rapide aux ouvriers.

Tant et si bien que l'exploration constante des structures par les équipes d'ouvriers permit de réaliser une action d'entretien appropriée, évitant ainsi de graves dommages.

Les principaux programmes de fouille qui suivirent, bien qu'ayant obtenu des résultats positifs, soulevèrent le grave problème de l'entretien à plus grande échelle: d'où la crise gestionnaire et l'incontrolabilité des phénomènes de dégradation.

La principale cause de dégradation est l'eau, dans la mesure où l'exhumation d'environ six hectares de ville antique a entraîné la concentration d'importantes masses d'eau dans le bassin de vallée de la ville, situé à environ quatre mètres au-dessous du niveau de la mer. En outre, la présence de puits de source et de nappes d'eau a nécessité l'installation de puissantes pompes élévatoires refluant les eaux dans une canalisation débouchant en mer.

Mais, à Herculanum, le problème de l'écoulement des eaux requiert de gros effort afin d'éviter que celui-ci s'attaque directement à la conservation des maçonneries tufacées. En effet, le contrôle de ces dernières dépend essentiellement de la stabilité de ses composants par rapport à l'action de ravinement, filtrante et stagnante des eaux météoriques.

Les interventions les plus urgentes prises en considération concernent la remise en fonction des anciens réservoirs d'eau: à l'origine, chaque habitation disposait d'un système d'écoulement des eaux de pluie dans des citernes (ainsi de nombreux péristyles étaient dotés de caniveaux spéciaux).

Restaurer de telles structures signifie limiter en grande partie les dommages causés aux structures antiques.

Ces dernières, en courtine, sont extrêmement endommagées dans la mesure où elles sont généralement constituées de matériau tufacé (gris s'il est antique, jaune en cas de restauration) extrêmement poreux et qui ressent fortement tant les amplitudes thermiques que les effets directs et indirects de l'eau (pluie, remontée capillaire, humidité, etc.). Le phénomène le plus grave se manifeste par l'érosion et la pulvérisation accrues du matériau superficiel, d'où une perte d'épaisseur de tous les éléments tufacés jusqu'à 2-3cm par rapport au mortier de jointoiement; ce phénomène d'érosion concerne rarement le mortier lui-même. Sur tous les matériaux (tufs, mortiers, enduits, fresques), on trouve une présence majeure d'efflorescences salines blanches et tendres, surtout dans les zones inférieures des structures, moins exposées au soleil (7).

Un autre phénomène d'évolution plutôt rapide s'observe sur les faites des maçonneries.

Dans le passé (8), le problème était affronté par la recomposition des faites émoussés avec du cailloutis réutilisé et un abondant "capping" de mortier et de mortier de tuileaux. Plus récemment, on a préféré laisser aux pierres leur profil émoussé de "sacrifice" (9), mais il est bien évident que l'intervention doit etre parfaite afin d'éviter que ne s'ouvrent des voies d'accès aux agents déstabilisants (10).

La durabilité des matériaux composant les maçonneries d'Herculanum est aussi liée aux conditions particulières dans lesquelles les fouilles ont été réalisées.

En effet, on sait que la masse boueuse alluviale qui bouleversa et ensevelit Herculanum sous une couverture de plus de dix mètres, défonça, du fait de sa force inouie, de nombreuses maçonneries avant de se figer et d'englober les structures.

Au moment de la réalisation des fouilles, cette situation a nécessité de délicates interventions de recomposition et, plus souvent, de "sous-fondation", avant de poursuivre les fouilles.

A l'aide de travaux provisionnels, on tenta de rassembler des morceaux situés à des hauteurs différentes à travers des reconstructions à texture différenciée (appareil incertain polygonal en tuf jaune) en relief.

Dans certains cas plus intéressants, on a conservé les bombements et les gites des murs, presque comme pour témoigner de l'état de pre-collapsus des ouvrages (11). Par conséquent, il existe des conditions structurelles que les interventions de conservation et les choix y relatifs ne peuvent ignorer.

Enfin, il est nécessaire de prendre en considération la présence importante d'enduits couverts de peinture à fresque, dont l'état de conservation est rendu difficile par l'obscurcissement, l'altération, la fragmentation.

Bien souvent, la restauration des peintures échoue par manque de consolidation et de restauration des maçonneries qui jouent le rôle de support.

Cela peut non seulement rendre les interventions inefficaces, mais aussi aggraver la situation et empêcher d'atteindre les objectifs préfixés.

Il s'agit là de l'exemple le plus simple des cas où le travail du conservateur, pour atteindre des résultats optimaux, doit pouvoir être coordonné dans le cadre d'un programme complet de restauration où trouvent leur place tant les travaux de consolidation proprement dits que des mesures de prévention facilement réalisables.

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(5) Archivio Soprintendenza Archeologica di Pompéi, fasc.41, notes du 4 mars 1910.

(6) Archivio Soprintendenza Archeologica di Pompéi, fasc.42, expertise de frais de mai 1926.

(7) Des études sont en cours de réalisation aux fins d'individualiser les causes de tels phénomènes de dégradation, et d'y apporter les remèdes appropriés.

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(10) MELUCCO VACCARO, A., Archeologia e Restauro, Rome, 1988, pp.234-273.

(11) L'un des exemples les plus frappants est donné par le mur d'enceinte de la maison du "Rilievo di Telefo" (ins. orientalis, I, n.2) et par le stabulum de la même habitation.

(1) [illegible], M.C. - TIBALLI, M., Il parco archeologico: analisi di [illegible] la conservazione sullo scavo [illegible] pp.89-107.

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(6) Archivio Soprintendenza Archeologica di Pompéi, fasc.42, expertise [illegible]

(7) [illegible] aux fins d'individuali[illegible] de dégradation, et d'y apporter les [illegible]

ADDITIONAL STRESS ON MASONRY DURING REPAIRING AND STRENGTHENING WORKS, AND APPLICATION PROBLEMS OF RESTORATION TECHNIQUES ON MASONRY WITH HEAVY DAMAGES

A. VOYRDERIS – G. TZANETOS – A. FROUSSOS

SUMMARY

In this paper an examination and analysis is made for additional actions on masonry produced during the application of the most common repairing and strengthening techniques (injections, "cast in" concrete jackets, sprayed Concrete, pretensioning etc).The theoretical presentation and analysis is compared to existing data from already completed projects (in which necessary estimations have been made before application). Reference is been made also to high risk cases. Final conclusions are reached.

ACADEMIC TITLES : CIVIL ENGINEERS N.T.U.

DATE : 15 / 4 / 89

INTRODUCTION

Applications of repairing and strengthening techniques on masonry in general,produce additional - temporary in most cases - stresses.These stresses are generated by forces or constrains on masonry, prodused during works,and are characteristic of the restoration technique used. We can generally classify them in two categories.

The first category includes actions developing during the technique application, and disappear with the end of works.Such actions are the internal pressure applied on the mass of masonry during all kinds of injections,the external pressure applied on the surface of masonry during "cast in" or sprayed concrete jacket constructions,the additional load actions (new supports etc.)in the case of lifting and moving masonry,as well as pre-tension loads at temporary stone pretensioning applications, collision loads during drilling or degradating etc.

The second category includes actions applied on the structure while working, and continue acting after the end of works. These are usually pretension cases on stone and are examined in relation to time.

The control of these additional stresses interests in all kind of works on masonry, but becomes critical in cases of works on masonry with heavy damages, where it is directly connected to the safety of parts - or even -the whole construction during the works.

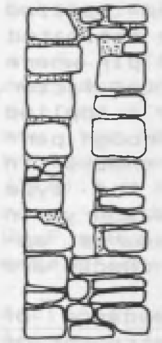
ACTIONS DEVELOPING DURING INTERVENTIONS AND DISAPPEAR WITH THE END OF WORKS.

<u>Injection applications</u>

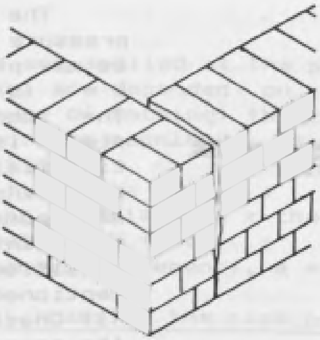
The determination of pressure distribution as well as the analysis of stresses applied to the mass of masonry during injection are - in general - difficult to be solved theoretically and simultaneously. The difficulty stands on the inability of geometrical presentation of the voids system,the peculiarity of the system of voids itself,as a flow duct,the great variability of flow conditions,and further more on the number of parameters involved (Kind and composition of grout, kind and geometry of masonry etc.).

A schematic approach of the problem is made in the following. The analysis concerns mainly inorganic grouts. We give a schematic presentation for three characteristic cases of voids. (Dr. 1,2,3).

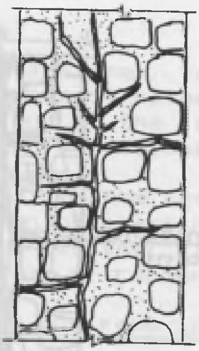
In the fist case, actions developing on the void "walls" initially correspond to hydrostatic pressure of a liquid mortar, and therefore can be estimated as usual (see dr. 4) while at the end of the void filling pressures reach the inlet pressure (pressure at the pump).



Dr. 1 Case of masonry with large voids.

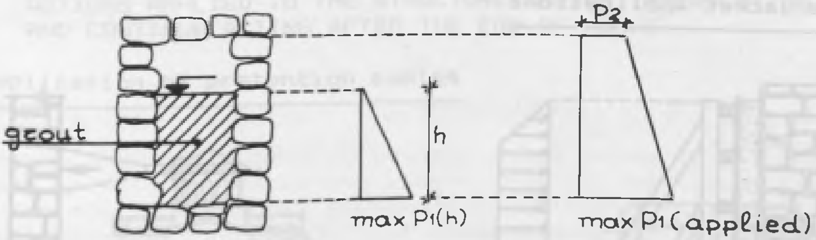


Dr. 2 Case of single crack in masonry with low percentage of voids ("porosity" < 2.5%)



Dr. 3 Case of many connected voids with low widths (w ≤ 5mm)

In the second case the magnitude of the pressure depends on the width of the crack. The narrower the crack is (e.g. the section of "duct") the higher the required pressures are. For a significant crack width (w > 10 mm) the flow conditions are similar to those of the first case while for limited crack width they are similar to the third case's ones.

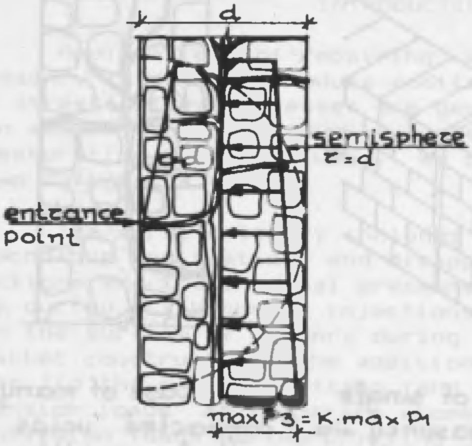


Dr. 4 Diagrams of pressures applied to the void "wall".

In the third case of injection, the flow conditions are the most uncertain. For the area of maximum pressure (area of grout inlet) the simplified schematic presentation of Dr. 5 can be given from which with the use of proper coefficients the practical control of pressures is possible. Using the vertical and horizontal advance of the grout Rv, Rh correspondingly in Ft per hour, observed at the "control points", and the grout flow rate Q in Ft3/h, the mean "porocity" of masonry in the specific point can be estimated: $n = Q.100/Rv.Rh.d$ or

$$n = 100.Q/Rf.d.f_1.(1+f_2)$$

where d: the width of masonry, Rf: advance of grout in masonry in Ft^2 per hour, f_1, f_2: percentages of voids filling and grout loss.



Dr.5 Area of maximum pressures in the case of Dr.3

The maximum applied pressure can be estimated as max p3 = k max p1: where k = f (n)· reduction factor. The control of applied stress is made through partial limit state checks in accordance with the type and geometry of masonry in the point of pressures application. Three checks are mentioned :

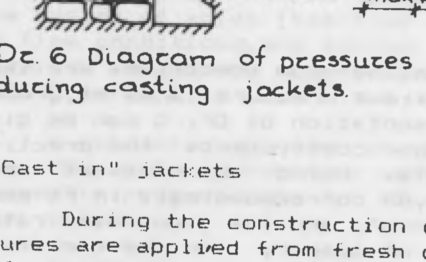
(i) Check of exceedance of the compressive strenght of existing mortar.

(ii) Check of blowing stones out of the masonry face.

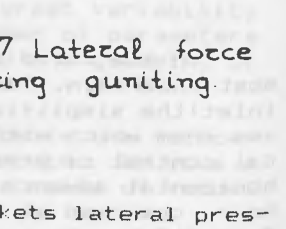
(iii) Check of horizontal swelling of masonry.

In cases of injections into masonry with heavy damages, low pressures are recommended, - of about 1 atm (data coming from the author's files - M. Lizzi 1982). Examples of applications with the maximum applied pressures and safety measures taken are given in Dr. 12,13,14,15.

Concrete jacket applications



Dr.6 Diagram of pressures occuring during casting jackets.



Dr.7 Lateral force during guniting.

"Cast in" jackets

During the construction of "cast in" jackets lateral pressures are applied from fresh concrete to masonry. The estimation of pressures can be made by the use of formulae of the type max p = $C_1 + C_2 . R/T$, where C_1, C_2 constants, R the vertical advance of casting and T the temperature. Thus the concrete height in each zone of casting can be determined (and reduced).

Gunite jackets

Lateral forces are applied at the point of blasting. Forces (due to guniting) are depended on the flow rate of material and air (e.g. type of maching) the composition of the dry mix, the rebound etc. The magnitude of forces can be estimated by formulae of the type:

$(\Sigma Fx).t = m\ (Vx\ \text{initial} - Vx\ \text{final})$: where m is the total mass, of the blasting material and x the direction of blasting. Or $F = Q.d\ (V\ \text{initial} - a.V\ \text{final})$: where Q is the flow rate of dry mix (aggregate, cement), a = percentage of rebound.

Action of additional loads in the case of lifting and removing masonry

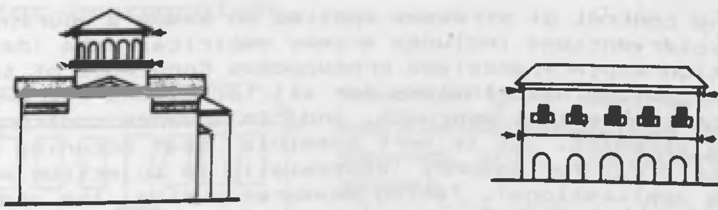
The static model of the stucture changes (change of supportings) New static calculation of the structure is necessary under the existing loads at the time of intervention and the temporary supports.

In the case of additional removing, calculations include :

1. Estimation of maximum load actions in the different lifting stages.
2. Estimation under inertial loads during the removal.
3. Strain estimation of masonry because, generally, the temporary supporting systems have less stifness.

ACTIONS APPLIED TO THE STRUCTURE DURING INTERVENTION AND CONTINUE ACTING AFTER THE END OF WORKS

Application of pretention cables

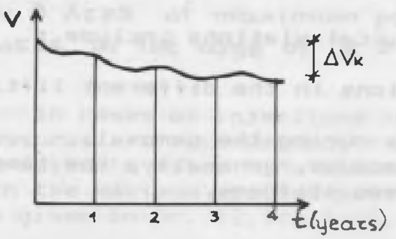


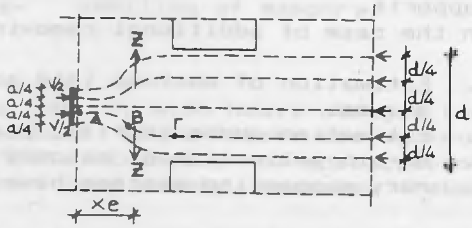
Dr. 8-9 Prestressing cable applications in zones with tensile stresses

By the use of pretention cables horizontal forces are introduced in masonry. Cables are placed so as to introduce vertical loads in sections under bending moments, see dr. 8-9. The method used for checking is the permissible stresses one. The applied mean compressive stress on masonry due to pretention is $\sigma_{m,v} = V/A_{m,v}$: where V is the prestressing load, (constand along the cable in the usual case of unbonded tendons application) and $A_{m,v}$ the mean cross section of the mainly prestressed zone.

The type of the(V,t)curve is shown in Dr. 10.The variation of the prestressing load whithin each year is due to different thermal expansion of steel and masonry correspondingly. From the existing experimental data it comes that the expected loss in prestressing loads due to creeping is of the order of 10-15% According to [Hage, 1972] a percentage of 80-85% of the creep deformations is expected to happen whithin the first 2 1/2 to 3 1/2 years.

The expected stress distribution in the anchoring regions can be seen in Dr. 11. A rough estimation of the tensile force Z can be done with the assumption of a mean path of principnle compressive stresses ABΓ consisting of arces, thus: $Z \simeq V (1 - a/d) / 3,2$ at a distance $Xe \simeq 0,3de\ (1+a/d)$. Resulting Z is compared to the existing vertical compressive load Q to the masonry (t/m).





Dr. 10 Typicaldiagram v-t   Dr. 11 Splitting tensile forces.

CONCLUSIONS

The control of stresses applied on masonry during restoration interventions includes anyway empirical data (data from application expierieance)and presupposes Knowledge of the available technology.Nevertheless for all techniques there is knowledge for theoretical approach, and in advance confronting of expected stresses. It is very possible that occuring stresses can be critical for masonry (espessially in injection and prestresing applications). Taking measures during the application is usually necessary (determination of maximum allowable quantities, work in stages, side supportings etc.).

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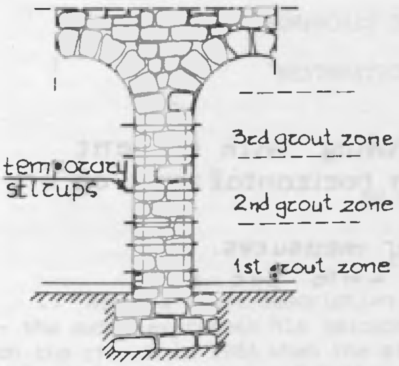
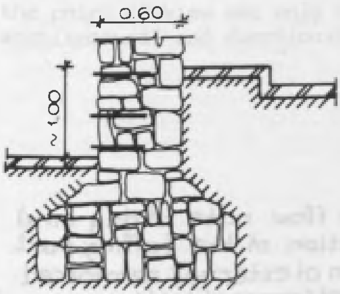


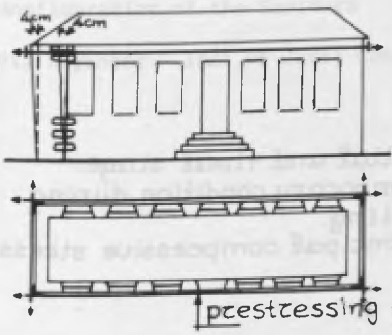
section of columns: 70×70 cm^2
void percentage: 20%
maximum grout pressure: ≃1 atm
safety measures: temporary tightening with stirups.

Dr. 12 Cement grout injections in columns with significant voids.



void percentage ≃ 2.5%
maximum injection pressure ≃ 10 atm
side supportings of masonry: through existing floors.

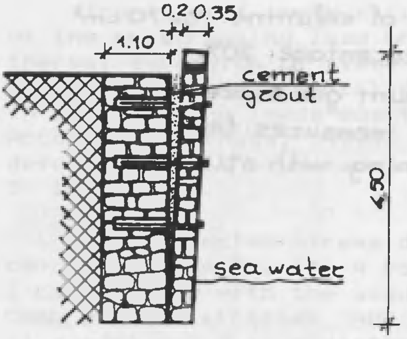
Dr. 13 Resin injection for void filling and masonry mortar impregnation.



prestressing cables (Φ 1/2" unbonded)

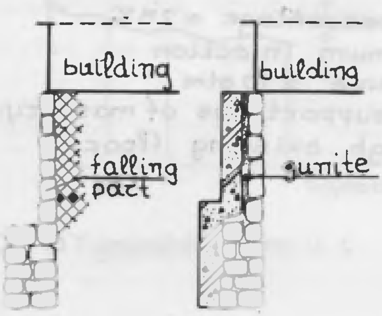
maximum grout pressure ≃1 atm
additional measures: "crack sewing" with additional new stones.
placement of 3 pretention cables (V = 2.00 tn per cable)

Dr. 14 Cement grout injection in a case of severe crack and wall inclination.



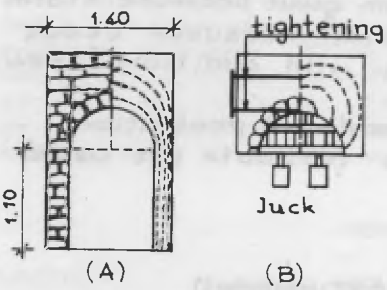
1. Void filling with cement grout in horizontal zones of 0.5 m
2. Safety measures:
Bolts 4M16 per sq.m.

Dr. 15 Stabilization of inclined stone retaining wall on stabile existing.



1. Gunite (low flow rate of dry mix) for stabilization of the falling part.
2. Construction of external reinforced concrete jucket (zones of casting per meter)

Dr. 16 Repairing of stone retaining wall with partial collapse



A) Initial and final stage.
B) Temporary condition during lifting.
----Principal compressive stresses.

Dr. 17 Raising and moving of vault (bell tower)-temporary stress redestribution.

I. METAMORPHOSIS SOTIROS CHURCH* at VILLIA ATTIKA

RESTORATION STUDY AND ITS APPLIANCE

BY NIKOS ALEVRAS**

After a short description of the church - architectural and structural - the author expresses his opinion about the causes of the damages evident on the church in 1984 when the study was carried out by : Anna Skaltsioti (Arch.) El.S.Menagias and N.Alevras(Civ.Eng.) Following that he presents the principles followed in the restoration of the building, the assumptions made, and the structural model used on the computer and describes in short the works carried out. He separates the various phases in order to underline the need for the careful study and restoration of old buildings from the point of view out only of their structural adequacy but also of their architectural and functional aspects.

* Transfiguration of the Saviours

**Civil Engineer - Inst.of Civil Construction,Bucarest.

I. METAMORPHOSIS SOTIROS CHURCH at VILLIA ATTIKA

RESTORATION STUDY AND ITS APPLIANCE

One of the little known works of E.Ziller in Attika is the Metamorphosis Sotiros Church, built in 1893. The Church symmetrical, cross-shaped with vault, was constructed from reddish porous stone with strong lime mortar. Its four (4), big arches, from brickwork, have a radius of about 4m. The tympani have openings over the ladies´storey which is supported by steel beams, bridged by little vaults of solid brick. The vault loods uniformly the 12 columns which, through the arches, lean on the four central columns. Over the entrance there is a steeple (from the same porous stone) which joins the ladies´ storey to the court-yard by an impressive double staircase, forming an autonomous (statically too) well balanced whole. Let us note here that one of Villia´s Mayors added a cement canopy on the steeple in order to place there a clock* With only its interior plastered, the Church has beautiful frescoes as well as naive decorations. In 1981, the surface earthquake of the Alkyonides Islands (6,6R), which are very close to Villia shook almost vertically the Church, as is easily proved by the location of the cracks which are greater in number at the joints (grinding of mortal) than at the carved stone. The disintegration and loosening of the binding mortar is, luckily not very extensive. Besides the earthquake, the moisture has created lasting decay such as salt deposits, fungi etc., thus forouring the disintegration of the mortal.

Having in mind that the Church is one of the "newer" monuments at Greece, it is clear that for its restoration we have to follow the principles of the "Charter of Venice" of 1964.

For this reason we propose its restoration without the clock canopy, but preserving its "cement" columns in remembrance! Rescue restoration. The solutions to be proposed for the restoration of the structural adequacy of the Church aim at the attainment of its anti-seismic function, taking also in consideration the resistance (reserves) of the building to the dynamic loading Their methodiical application will result from the investigation of the structural behaviour of the building as well as from the fitness of the restoration materials to those already existing. We shall not insist here on the details of calculations. We shall note that together with the colleaque civil engineer Mr.El.Menagias, we have chosen structural models which have helped us to a better approximation of reality. We considered that the vault sits on the four arches, that the arches operate in three directions, that the whole of the vault weight is apportioned among the four columns, that the seismic load "vault-vault base" is assumed by the near by walls and that the mansory is not loaded with tensional stresses. Also, from the form of the supporting structure it seems (also proved by the computer) that the construction is not subjected to twisting and that we therefore can study it by examiningonly one section at the ladies´storey level and at the ground floor. Before we close the "few words" for the study, we state that we did not think it necessary to examine the steeple because, in spite of the ill treatment to which it was subjected, structurally as well as ... aesthetically it has given tokens of a good anti-seismic behavior.

As we have stated at the begining, one of the main themes of the restoration and reinforcement studies of monuments is the fitness of the repair materials to the original ones, especially in their present condition, under the various effects of humidity or/and other corrosive phenomena. It is just due to this reason that the use of synthetic materials should be restricted. The grouting also would have to have their machanical resistance

*Foto 1.

lowered($\leq$70Kg/cm² breaking in 28 days) and the specific gravity of the hardened injection must be about 1,5 kg/lit. As it is known these values are usually found in the mortars of old buildings as in those which are constructed in the traditional ways. Theoretically we can divide the repair works applied on the Church in two categories : "restoration" and "reinforcement" of the walls, this meaning in no case that distinction is clear. Works were therefore done for the restoration of the structural adequacy, the architectural appearance and function as well as works for the reinforcement of the walls, the columns and the arches. Because, in general, the classical methods of repair are already well known methods for the restoration of the structural adequacy, we shall not insist here on their description, noting only that the materials are also known : sand, pozzuolana, cement etc. The only exception : the synthetic ameliorative in minimun quantities (e.g. 1 Lit special mix in about 180 Lit water) in order to achieve a better fluidity of the cement injection or for a PH-necessary for the protection of the steel reinforcement and the epoxy resins in combination with very fine quartzy sand (about 0,1 - 0,3 mm) in order to diminish the mechanical resistances of the unaltered material-fitness with the hard porous stone of the Church. An interesting example of the architectural and function restoration is the steeple. With the replacement of the vault its original place (as a pre-cast element) we achieved the restoration of the original appearance of the Church. At the same time this way allows its probable removal : REVERSIBILITY. Before the restoration of the vault, its three bells were suspended in their original place : FUNCTIONALITY. By not removing the old concrete plate on which the "canopy" was placed we preserved a disc: REINFORCEMENT. And speaking of reinforcement let us see how and where it was applied on the Church. For the walls we used the method of "postreinforcement". Let us call it therefore short reinforcement rods. The method has been used by Italian Colleaques on many restoration works (F. LIZZI-RESTAURO DEI MONUMENTI-GENOVA 1981). There is also the relative description of the Mechanics of Wallbuilding of Professor Mr.Tassios-N.T.U of Athens 1984 where the "short reinforcement rods" are mentioned. (Fig. 2)

After the anti-seismic examination of the walls we found out that on the most structural elements of the Church where there are cracks of the ... "structural model" which we chose, the computer developed tensional tendencies*. It was therefore natural to choose the "postreinforcement" as the most appropriate method of restoration and reinforcement. The sequence of works at this technology is the following : after the cleaning of the crevices for joints from mortar, dust and rotten materials, which are always there, we make holes with a boring drill with the needed dimensions. The opening of the holes has to done by a skilled workman and successive changes of the dimension of the drills, especially at cement little root-piles over 16 mm diameter of and/or longer than one meter. In the case of this Church the cement little root-piles reach a length over 2.50 m and with a 20 mm reinforcement diameter, particularly for the reinforcement of the four central columns of the Church. During the works frequent blowing of the hole is done, cleaning with air under pressure up to 6 atm. not only from its mouth but also in depth with the help of a special tool. The position and direction of the openings depends on what are want to achieve : resistance to tension and/or in section. (Foto 3 a-b). After the (very careful) final airing, the sealing of the joints (or crevices) is done and possibly the little pipes for the injections are also placed, according to the needs. After the placing of the reinforcement in the hole (ST I for cement little root-piles or bronze pipe for epoxy little root-piles) and before the anchoring injection, airing will again be done. The way in which the conveyance of the injection through the pipes will be done depends on its quantity and kind as well as on the

*** Fig. 1a-b.**

direction of the small reinforcement rods. It can be done as an injection or with the help of gravity. In both cases we must see that no air is confined and generally the hole is filled up with the injected material so that we have a good packing of the reinforcement. In order to achieve this it is better for the injection to be poured in at least two phases with the in between time less than the "pot life" with the indirect result to diminish the shrinking of each injection as well. (Foto 4 a-b).

At the columns too, we used the post reinforcing method, this time with the help of epoxy little root-piles. Essentially there is no difference between the methods applied here from the ones described for the cement little root-piles of the wall. The injection and reinforcement are the only ones to differ, as well as the dimensions (the epoxy little root-piles have no more than 18 mm hole diameter and 40 cm depth). (Foto 5 a-b).

At the begining in the study, we had proposed in order to cope with the tensional and breaking tendencies at the key area of the arches as well as of the tension which rises from the earthquake and the vertical load, the well known solution with epoxy little root-piles and tension bars corresponding by.

In the end though, we came to suggest the application of resin injection and of the "small reinforcement rods". But as usual ... the Church had the last word. The disclosures of the cracks at the key of the arches showed that the appliance of the small reinforcement rods was not mecessary. The width for the cracks was of the class of 1 to 2 mm at the most. The resin injection was enough for the key of the arches. The same happened also with the knocks as we have called the symmetrically placed at the area cracks of the four central columns.

In our opinion these cracks appeared because of the knocking of the great arches of the Church at the begining of the earthquake ($G=\psi\cdot G_o$ - where $\psi \geq 2$), which resulted in momentary increases of the tensions

Once more it is proved that the "reserves", the resistance margins of good construction at dynamic loadings are always greater than the ones exspected. We also found out the existence of tension bars at the parapet of the ladies´storey (well hidden behind the mortar) made from steel plates 50 X 7 mm as well as of external metal collars at the four central columns of the Church. So we did only resin injections at the knock cracks in depth and improved the anchoring of the existing tension bars. The horizontal forces will be transfered (as we have mentioned) to the already existing tension bars and the well preiserved metal beams of the set ladies´storey (floor) and the walls now reinforced by cement little root-pile. The mechanical characteristics of the epoxy mortars will assure the reception of the tensional and breaking tendencies as far as their fitness with the original materials of the Church allow it.

These in brief were the reinforcement works done at the Metamorphosis Sotiros Church at Villia Attikis. It is quite obvious that at the same time many of them are works for the restoration of the structural adequacy too.

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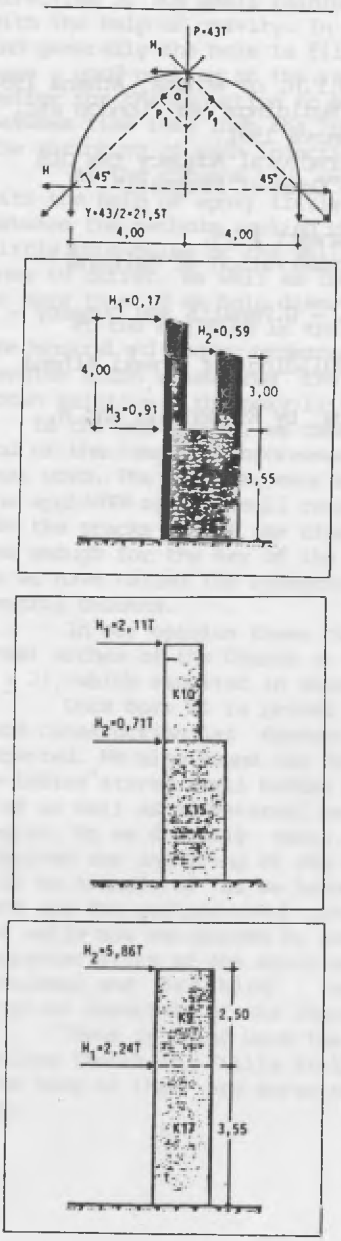


SUMBOLISMOI EKTYPOSEOS

1 St : Tasis Toixopoias Aneu Seismou
2 CmX : Megisti thlipsi Toixopoias kata XX
3 TrX : Efelkismos kata XX
4 CmY : Megisti thlipsi Toixopoias kata YY
5 TrY : Efelkismos kata YY
6 Td : Tasis diatmiseos
7 Ert : Metro Elastikotitos Toixou

PARATHRHSEIS
Ta fortia dinontai se tonous - Oi diastaseis se metra
Oi Entaseis katagrafontai se kg/cm2
To metro Elastikotitos se t/m

ANTISEISMIKOS ELEGXOS TOIXOPIAS - OROFOS V-IS

Seismikos syntelestis = 0.08

ARA	ax	ay	X	Y	Nx	Ny	P	Ox	Oy	Mx	My
1	1.00	1.00	13.0	4.8	3.0	3.0	50.0	0.74	0.73	2.22	2.19
2	1.00	1.00	13.0	13.0	3.0	3.0	50.0	0.74	0.73	2.22	2.19
3	1.00	1.00	4.8	12.9	3.0	3.0	50.0	0.74	0.73	2.22	2.19
4	1.00	1.00	4.7	4.8	3.0	3.0	50.0	0.74	0.73	2.22	2.19
5	0.70	3.50	5.0	1.8	3.5	3.5	55.0	0.59	8.25	2.07	28.89
6	3.50	0.70	6.4	0.4	3.5	3.5	45.0	8.35	0.58	29.23	2.05
7	3.50	0.70	11.4	0.4	3.5	3.5	45.0	8.35	0.58	29.23	2.05
8	0.70	3.50	12.8	1.7	3.5	3.5	55.0	0.59	8.25	2.07	28.89
9	3.00	0.70	13.9	3.0	3.5	3.5	30.0	5.98	0.50	20.93	1.75
10	0.70	3.00	14.8	4.0	3.5	3.5	20.0	0.50	5.91	1.77	20.69
11	0.70	3.50	17.5	6.5	3.5	3.5	45.0	0.59	8.25	2.07	28.89
12	0.70	3.50	17.5	11.4	3.5	3.5	45.0	0.59	8.25	2.07	28.89
13	3.50	0.70	16.1	12.7	3.5	3.5	55.0	8.35	0.58	29.23	2.05
14	0.70	3.00	14.6	13.8	3.5	3.5	20.0	0.50	5.91	1.77	20.69
15	3.00	0.70	13.6	14.0	3.5	3.5	30.0	5.98	0.50	20.93	1.75
16	0.70	3.50	12.8	16.0	3.5	3.5	55.0	0.59	8.25	2.07	28.89
17	3.50	0.70	11.3	17.3	3.5	3.5	45.0	8.35	0.58	29.23	2.05
18	3.50	0.70	6.4	17.3	3.5	3.5	45.0	8.35	0.58	29.23	2.05
19	0.70	3.50	5.0	16.2	3.5	3.5	55.0	0.59	8.25	2.07	28.89
20	3.00	0.70	3.9	14.9	3.5	3.5	30.0	5.98	0.50	20.93	1.75
21	0.70	3.00	2.9	13.8	3.5	3.5	15.0	0.50	5.91	1.77	20.69
22	0.90	1.70	0.4	10.5	5.0	5.0	10.0	0.21	0.69	1.05	3.48
23	0.90	1.70	0.4	7.4	5.0	5.0	10.0	0.21	0.69	1.05	3.48
24	0.90	1.30	0.4	5.2	5.0	5.0	15.0	0.16	0.32	0.80	1.61
25	0.70	3.00	2.9	4.0	3.5	3.5	18.0	0.50	5.91	1.77	20.69
26	3.00	0.70	3.9	3.0	3.5	3.5	30.0	5.98	0.50	20.93	1.75
27	1.30	0.70	0.6	13.8	5.0	5.0	15.0	0.25	0.07	1.27	0.37
28	3.50	0.70	16.1	5.1	3.5	3.5	55.0	8.35	0.58	29.23	2.05

ΣP= 1043 t

ENTATIKH KATASTASIS PESON

ARA	St	CmX	TrX	CmY	TrY	Td	Ert
1	5.0	6.3	3.6	6.3	3.6	0.0	6000
2	5.0	6.3	3.6	6.3	3.6	0.0	6000
3	5.0	6.3	3.6	6.3	3.6	0.0	6000
4	5.0	6.3	3.6	6.3	3.6	0.0	6000
5	2.2	2.9	1.5	4.2	0.2	0.3	6000
6	1.8	3.8	-0.2	2.5	1.1	0.3	6000
7	1.8	3.8	-0.2	2.5	1.1	0.3	6000
8	2.2	2.9	1.5	4.2	0.2	0.3	6000
9	1.4	3.4	-0.5	2.1	0.7	0.3	6000
10	0.9	1.6	0.2	2.9	-1.0	0.3	6000
11	1.8	2.5	1.1	3.8	-0.1	0.3	6000
12	1.8	2.5	1.1	3.8	-0.1	0.3	6000
13	2.2	4.2	0.1	2.9	1.5	0.3	6000
14	0.9	1.6	0.2	2.9	-1.0	0.3	6000
15	1.4	3.4	-0.5	2.1	0.7	0.3	6000
16	2.2	2.9	1.5	4.2	0.2	0.3	6000
17	1.8	3.8	-0.2	2.5	1.1	0.3	6000
18	1.8	3.8	-0.2	2.5	1.1	0.3	6000
19	2.2	2.9	1.5	4.2	0.2	0.3	6000
20	1.4	3.4	-0.5	2.1	0.7	0.3	6000
21	0.7	1.4	-0.9	2.6	-1.2	0.3	6000
22	0.6	1.1	0.1	1.4	-0.1	0.0	6000
23	0.6	1.1	0.1	1.4	-0.1	0.0	6000
24	1.2	1.7	0.8	1.9	0.6	0.0	6000
25	0.8	1.5	0.1	2.8	-1.1	0.3	6000
26	1.4	3.4	-0.5	2.1	0.7	0.3	6000
27	1.6	2.2	1.0	2.0	1.2	0.0	6000
28	2.2	4.2	0.1	2.9	1.5	0.3	6000

KENTRON BAROUS KENTRON ELASTIKIS STROFIS

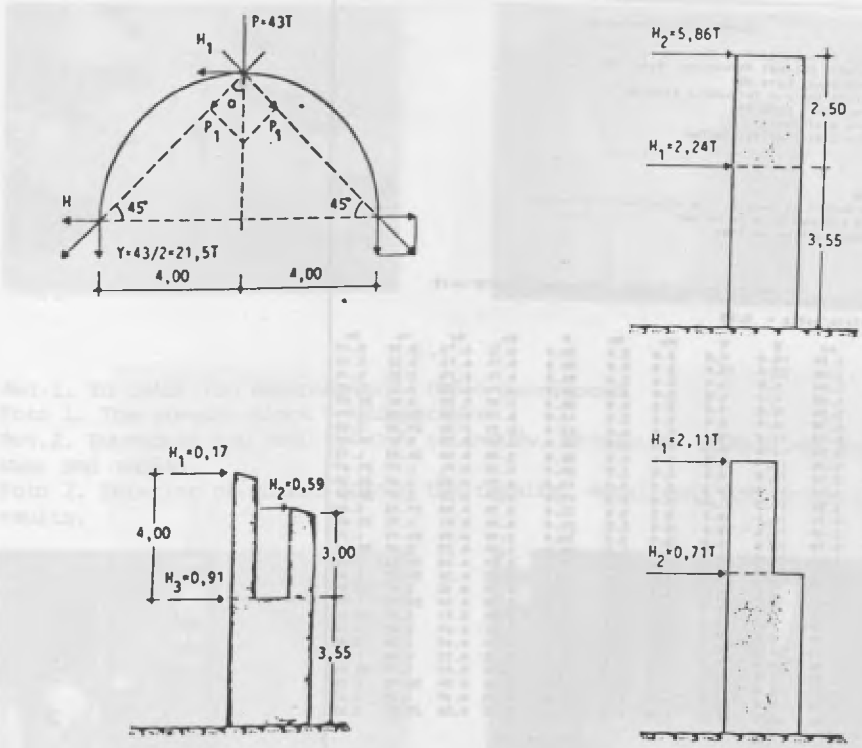
Xo= 10.01m Yo= 8.85m Xo= 10.09m Yo= 8.83m

EKKENTROTITES X-Y DIATMISIA X-Y

D(Xo-Xo)=-0.47m D(Yo-Yo)= 0.01m ΣDx= 6755.65t/m ΣDy= 6431.12t/m

Εικ.1. Στατικά μοντέλα και πίνακας υπολογισμών με τον Η/Υ που χρησιμοποιήθηκαν στην στατική μελέτη της αποκατάστασης του Ναού.

Fig.1. Static models and table of calculation with computer,used in the study of restoration of the Chapel.



Εικ.1α. Στατικά μοντέλα τα οποια χρησιμοποιήθηκαν στην μελέτη αποκατάστασης του Ναου.

Fig.1a. Static models used in the study of restoration of the Chapel.

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5 TrY : Efelkismos kata YY
6 Td : Tasis diatmiseos
7 Eyl : Metro Elastikotitos Toi/as

PARATHRHSEIS
Ta fortia didontai se tonous - Oi diastaseis se metra
Oi Entaseis katapomiseis se kg/cm2
To metro Elastikotitos se t/dm

ANTISEISMIKOS ELEXOS TOIXOPIAS - OROFOS V-IS

Seismikos sintelestis = 0.08

A/A	ax	ay	X	Y	Mx	My	P	Ox	Oy	Mx	My
1	1.00	1.00	13.0	4.0	3.0	3.0	50.0	0.74	0.73	2.22	2.19
2	1.00	1.00	13.0	13.0	3.0	3.0	50.0	0.74	0.73	2.22	2.19
3	1.00	1.00	4.8	12.9	3.0	3.0	50.0	0.74	0.73	2.22	2.19
4	1.00	1.00	4.7	4.0	3.0	3.0	50.0	0.74	0.73	2.22	2.19
5	0.70	3.50	5.0	1.0	3.5	3.5	55.0	0.59	0.25	2.07	28.89
6	3.50	0.70	6.4	0.4	3.5	3.5	45.0	0.35	0.58	29.23	2.05
7	3.50	0.70	11.4	0.4	3.5	3.5	45.0	0.35	0.58	29.23	2.05
8	0.70	3.50	12.8	1.7	3.5	3.5	55.0	0.59	0.25	2.07	28.89
9	3.00	0.70	13.9	3.0	3.5	3.5	30.0	5.98	0.50	20.93	1.75
10	0.70	3.00	14.8	4.0	3.5	3.5	20.0	0.50	5.91	1.77	20.69
11	0.70	3.50	17.5	6.5	3.5	3.5	45.0	0.59	0.25	2.07	28.89
12	0.70	3.50	17.5	11.4	3.5	3.5	45.0	0.59	0.25	2.07	28.89
13	3.50	0.70	16.1	12.7	3.5	3.5	55.0	0.35	0.58	29.23	2.05
14	0.70	3.00	14.6	13.0	3.5	3.5	20.0	0.50	5.91	1.77	20.69
15	3.00	0.70	13.6	14.0	3.5	3.5	30.0	5.98	0.50	20.93	1.75
16	0.70	3.50	12.8	16.0	3.5	3.5	55.0	0.59	0.25	2.07	28.89
17	3.50	0.70	11.3	17.3	3.5	3.5	45.0	0.35	0.58	29.23	2.05
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20	3.00	0.70	3.9	14.9	3.5	3.5	30.0	5.98	0.50	20.93	1.75
21	0.70	3.00	2.9	13.8	3.5	3.5	15.0	0.50	5.91	1.77	20.69
22	0.90	1.70	0.4	10.5	5.0	5.0	10.0	0.21	0.69	1.05	3.48
23	0.90	1.70	0.4	7.4	5.0	5.0	10.0	0.21	0.69	1.05	3.48
24	0.90	1.30	0.4	5.2	5.0	5.0	15.0	0.16	0.32	0.80	1.61
25	0.70	3.00	2.9	4.0	3.5	3.5	18.0	0.50	5.91	1.77	20.69
26	3.00	0.70	3.9	3.0	3.5	3.5	30.0	5.98	0.50	20.93	1.75
27	1.30	0.70	0.6	12.8	5.0	5.0	15.0	0.25	0.07	1.27	0.37
28	3.50	0.70	16.1	5.1	3.5	3.5	55.0	0.35	0.58	29.23	2.05

SP= 1043 t

ENTATIKH KATASTASIS FESON

A/A	St	CmX	TrX	CmY	TrY	Td	Eyl
1	5.0	6.3	3.6	6.3	3.6	0.0	6000
2	5.0	6.3	3.6	6.3	3.6	0.0	6000
3	5.0	6.3	3.6	6.3	3.6	0.0	6000
4	5.0	6.3	3.6	6.3	3.6	0.0	6000
5	2.2	2.9	1.5	4.2	0.2	0.3	6000
6	1.2	3.3	-0.2	2.5	1.1	0.3	6000
7	1.8	3.8	-0.2	2.5	1.1	0.3	6000
8	2.2	2.9	1.5	4.2	0.2	0.3	6000
9	1.4	3.4	-0.5	2.1	0.7	0.3	6000
10	0.9	1.6	0.2	2.9	-1.0	0.3	6000
11	1.8	2.5	1.1	3.8	-0.1	0.3	6000
12	1.8	2.5	1.1	3.8	-0.1	0.3	6000
13	2.2	4.2	0.1	2.9	1.5	0.3	6000
14	0.9	1.6	0.2	2.9	-1.0	0.3	6000
15	1.4	3.4	-0.5	2.1	0.7	0.3	6000
16	2.2	2.9	1.5	4.2	0.2	0.3	6000
17	1.8	3.8	-0.2	2.5	1.1	0.3	6000
18	1.8	3.8	-0.2	2.5	1.1	0.3	6000
19	2.2	2.9	4.5	4.2	0.2	0.3	6000
20	1.4	3.4	-0.5	2.1	0.7	0.3	6000
21	0.7	1.4	-0.0	2.6	-1.2	0.3	6000
22	0.6	1.1	0.1	1.4	-0.1	0.0	6000
23	0.6	1.1	0.1	1.4	-0.1	0.0	6000
24	1.2	1.7	0.8	1.9	0.6	0.0	6000
25	0.8	1.5	0.1	2.8	-1.1	0.3	6000
26	1.4	3.4	-0.5	2.1	0.7	0.3	6000
27	1.6	2.2	1.0	2.0	1.2	0.0	6000
28	2.2	4.2	0.1	2.9	1.5	0.3	6000

KENTPON BAPOUS   KENTPON EL/KIS STPOFIS

Xo= 10.01m   Yo= 8.85m   Xo= 10.49m   Yo= 8.83m

EKKENTPOTITES X-Y   AKAMPSIA X-Y

D(Xe-Xo)=-0.47m   D(Ye-Yo)= 0.01m   SOx= 6336.65t/dm   SOy= 6431.12t/dm

Εικ.1β. Πίνακας που χρησιμοποιήθηκε στην στατική μελέτη της αποκατάστασης του Ναού.

Fig.1b. Table of calculation with computer used in the study of restoration of the Chapel.

1. 2.

Φωτ.1. Το ρολοΐ του Καμπαναριού - Παλιά φωτογραφία
Foto 1. The steeple clock - Old picture
Φωτ.2. Εσωτερικό του Ναού προ των επισκευών. Μεταλλικοί ράβδοι και θολίσκοι από τούβλα
Foto 2. Interior of chapel before the repairs. Metal rods and brick little vaults.

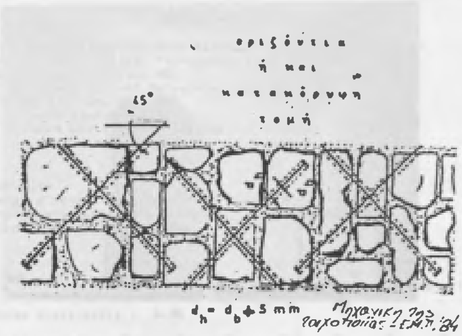
3α

Φωτ.3α. Οι εργασίες άρχισαν. Ανοίγονται οι οπές.
Foto 3a. Repair work has started. The holes are drilled.
Φωτ.3β. Το φύσημα γίνεται επιμελημένα
Foto 3b. The blowing is done carefully.

4α

4β.

Φωτ.4α. Οι τσιμεντενέσεις θα αποκαταστίσουν την τοιχοποιΐα. The cement injection will restore the walls.
Φωτ.4β. Συμπλήρωση με ένεμα των τσιμεντοβλίτρων. Filling in of the cement little root-piles with grout.



Εικ.2. Στο Ε.Μ.Π. τους λένε κοντούς ράβδους οπλισμού.
Fig.2. In the Natiotional Techical University they call them "short root piles".

Φωτ.5α,5β. Τα ρητινόβλιτρα.„Μετ΄οπλισμός" των πεσσών.
Foto 5a,5b. Epoxy "short root-piles". Post reinforcement of columns.

Φωτ.6. Επανατοποθέτιση του τρούλου
Foto 6. Replacement of the dome.

Φωτ.7. Σημερινή εμφάνιση του Ι.Ν.Μεταμόρφωσης του Σωτήρα
Foto 7. The church as it appears today.

PHILOLOGICAL RESTORATION OF HISTORICAL MONUMENTS
the Cathedral of "Sant'Angelo dei Lombardi" in Irpinia

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SUMMARY
The aim of this lecture is to demonstrate that in order to perform a mechanical analysis of historical monuments we need to have a thorough knowledge of their structural types and the original constructive techniques used, compared to the codified "rule of art". This enables us to pinpoint the possible weak mechanisms, to formulate correct mechanical models, and leads us towards philologically proper and effectively safe interventions. The philosophy of philologic restoration, presented in an other paper, is here applied in the restoration of an italian monument damaged by the earthquake.

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INTRODUCTION

The peculiar characteristic of ancient masonry structures is their historical nature. The restoration of such buildings has to account for that feature for two reasons: the aim of conservation of the cultural heritage, and the effective compatibility of the intervention.
The original techniques often don't accept the introduction of different materials and structural sistems; but historical structural detailings, if well made, are able to make safe the construction.
The Cathedral of Sant'Angelo dei Lombardi, damaged by the strong earthquake of Irpinia in 1980, was restored following these principles. The studies carried out were principally directed to understand the the seismicity of the soil and the original structure; The detailings of the restoration strightly followed from the first analyses.

LOCAL SEISMICITY

The Cathedral in question owes its present facies to important work of reconstruction carried out after damage caused by earthquakes which struck the area in 1688 and 1694.
One can observe that destructive earthquakes are not rare in Irpinia, and the town of "Sant'Angelo dei Lombardi" has been many times a victim.
Here is a brief list of the earthquakes, with regard to the Cathedral, with macrosismic intensities higher than the V degree:

Year	1694	1732	1805	1857	1910	1930	1962	1980
Intens. (MM)	X	IX	VI	VII	VIII	VIII	VII	X

It is useful to observe that the two X degree earthquakes, which occurred at a distance of a little less than three centuries, indicate through the form of the isoseismal maps [1] the same focal mechanism and the same propagation on the surface. One can conclude that the recent event (1980) can be considered representative of an "expected earthquake" of high intensity. The frequency contents of the expected motion can be assimilated to those registered on this last occasion.
The fact that the Cathedral has been struck twice by an earthquake of the X degree, and that the inhabitants of Sant'Angelo intend to religiously preserve this monument that unities them with Italian and European culture, leaves no doubt about the choice of the intensity of the verification, independently from the probabilistic value that a statistic elaboration of the data could give at the X degree earthquake.
Therefore, the aim of the restoration must be to enable the Cathedral suitable to withstand an earthquake of the X degree. Serius damage can be accepted but the possibility of collapse, even partial, must be prevented.

ARCHITECTONIC AND STRUCTURAL CHARACTERISTICS

The architectonic aspect of the Cathedral was closely studied: from its mediaeval foundation (XI century) to the Rinascimental enlargement of the XVI century, and to the reconstruction in the style of the Counter- Reformation at the end of the XVII century. This is seen in the basilical form with three naves that, also with regard to dimension, one finds in most of the coeval churches in Irpinia.
To satisfy the first requirement, a thorough recognition of the structural organism and

constructive details are briefly indicated below.
Something could be said about the foundations, but the analysis of the structure offers the most interesting features.

STRUCTURES

Observation of the outside surface of the walls, as seen in archaelogical surveys, is not enough to ascertain its mechanical characteristics. However, one soon discovers that every masonry typology has its own way to appear to the outside, and with the aid of some checks it can be recognized from the outside.
The walls of the Cathedral enumerate six different masonry aspects, but they are only variations of different qualities of the same typology. The two most significant ones are the following:

- a set of squared stone, laid without mortar, appears on the outside of the XVI century parts of the construction;
- pebblestones, broken in half and arranged with the cut surface on the outside, laid with pieces of tiles and bricks (called "tegolozzo" in the old italian language of the last century), and three layers of bricks recurring about every 80 centimetres.

Making some tests one sees that the thickness of the wall is made up everywhere of large pebblestones, well bonded one to the other, and arranged in horizontal layers made regular by the "tegolozzo", filling the spaces with smaller pebbles and lime mortar (one recalls that brick powder lends hydraulicity to the lime mortar, making it more effective on the inside of the wall).
This bulding technique, which is very popular in Italy where there are streams in abundance, is cited by the XIX century authors of treatises as a possibility, even though not the best, to construct a masonry [2] [3]. The criteria for the arrangement of the stones is described very clearly and it is not possible to make a mistake in judging whether or not a wall of this type is executed according to the "rule of art". For example, the high walls of the central nave, on which the wooden trusses rest, show the big pebblestones distant one from the other with the interposition of a lot of mortar. The smaller pebbles are not used just to fill the spaces between the bigger ones, but they themselves also form a very poor masonry horizontally irregular and with too little "tegolozzo".
This wall does not conform to the rules. Walls of pebblestones only, surrounded by large quantities of mortar, have been found after the earthquake to have disintegrated, and reduced to a pile of rubble. On the contrary well constructed walls of pebblestones, even if overturned, have remained whole. The difference in the mechanical behavior is manifest, and directly correlated to the quality of workmanship. It can be verified more objectively than the compression strength measured with a press on a portion of the wall extracted at random. workmanship
This brief presentation on the walls of the Cathedral immediately brings to the fore their most peculiar characteristics: an ancient masonry wall is not a "material" for which constitutive laws can be stated, but a "structure" built of a well balanced assortment of stones and mortar. laws can't be stated
The same stones that well cut and well arranged, constitute the miracle of equilibrium of a Gothic Cathedral, used with different but rigorous skill constitute the masonry. Hence the masonry "weaving" must be questioned to ascertain the mechanical characteristics of the wall, abandoning the sterile checks of an improbable "Cauchy" stress state.
Before concluding this paragraph on the masonry, it should be added that the pillars of the central nave, the arches and the pilasters of the Cathedral, were constructed with excellent brickwork, like the vaults. It is interesting to note that in the more ancient Irpinia tradition, one finds the use of stone masonry for the vaults, like in the vaults of the Cathedral of Castelvetere, a town not far from Sant'Angelo but always spared by the earthquakes.
From the XVIII century onwards, stone has disappeared at Sant'Angelo dei Lombardi, in favor of brick in the building of vaults. This is how the "rule of art" learnt the earthquake lesson and turned towards a goal of safety.

ORIGINAL CONNECTIONS

It is useful to observe the detailings of the connections between the various structural elements in their original form. In some cases one sees that the solution adopted by builders was not the most suitable to resist the seismic action. The "rule of art" that such connections imply does not yet contain earthquake experience, or derives from an incomplete understanding of its nature.

antiseismic intervention

The most serious "defect" which we will see has been the cause of the damage, concerns the construction of the lateral buttresses. These were added at a later date, perhaps after the earthquake of 1732. This emphasizes the conscious use, in the XVIII century, of antiseismic interventions.

The buttresses were built by joining their vertical edge to the wall of the central nave, without any connection.

This solution is coherent with the intention to impede a one-direction thrust, the reaction of the vaults, or to restrain the eventual out of plumb of the wall, observed after the earthquakes; but the seisma's way of acting, its dynamics, were not understood. Nevertheless, one observes that in the original structural typology, in the "rule of art" of a three naves basilica, the wall of the central nave was standing between the vaults and the trusses that contrasted it from the inside and the buttresses from the outside. In this situation the connection defect is clearly less serious.

Another constructive detail of extreme interest, on the subject of connections, regards the crossing of the orthogonal walls.

Except for the front wall, which was badly connected to the longitudinal masonry, all the other corners are made according to the "rule of art": squared stones of remarkable dimensions correclty bonded. How far this solution was effective with regard to the earthquake, we will discuss in the next paragraph. Here we will just observe that in the corners of the transept, and in many other places, numerous metallic bolts, made of short tie-rods (about 1.5 m) were put into the wall and locked from the outside with the usual bars: device used in the constructive phase to improve the bond. In the Irpinia buildings of the XVIII century, such tie-rods are frequently to be found. The architectural treatises of the XIX century describe and recommend them. The "rule of art" passes from the practice to the literature.

rule of art passes from the practice to the literature

KINEMATISMS AND MECHANICAL MODELS

If we had examined the Cathedral before the earthquake we would have had to ascertain the kinematisms that the unilateral connections allow, both as regards its actual disconnections as well as those that the earthquake would have been able to produce. We would have then tried to discover which of these kinematisms would have been set in motion by the seismic action and how this motion would have progressed in a X degree earthquake. Relatively easy the first analysis, decidedly unrealistic the second.

Since November 23rd 1980, the Irpinia subsoil has naturally produced the feared (and still to be feared) X degree earthquakr, the kinematisms of the Cathedral have been ascertained and moved. We should only observe them.

off plumb

The surveyors discover an off plumb of three or four centimetres between the top and the bottom of the facade, but do not know if this slight anomaly existed before the earthquake. However, the chipped stones at the base of the XVI century portal indicate a concentration of stress on the outside edge. The facade has certainly swung towards the outside. The rocking mechanism is easy to foresee but it is difficult to quantify the effect of a X degrees earthquake. If we evaluate the intensity of the horizontal force H that produces the overturning of a wall 100 cm thick and 16 m high, we obtain an eighth of the weight: H = 0.125 P, but the maximum peak of acceleration has been certainly nearer 0.35 g than 0.125 g, as shown by the available records. In any case, the effect of a static force does not have

any relation to the dynamic action and the comparison has no sense.
If, however, we study the wall as a rigid body rocking on its base, set in motion by accelerograms similar to those registered on 23/11/1980, we will find that there is no realistic value in the peak of acceleration able to overturn it, as, in fact, has always happened to obelisks and minarets which survive undamaged any earthquake. But also this result is not useful because the wall, restrained from the back by the longitudinal walls, cannot oscillate freely as an obelisk.
Taking into account in the dynamic model that movement towards the inside is contrasted, we get extremely scattered results: different accelerograms, even though of equal intensity, produce very different displacements, from slight ones to the unacceptable. Collapse is not certain, but it is very probable.
These analyses do not add very much to the ascertainment that kinematism is possible and can be set into motion. It would be desirable to know what seismic intensity puts it in danger, and how much chaining is necessary, but today we only know that the X degree of November 23rd 1980 did not cause collapse, and we cannot affirm that this is because of some intrinsic resistance or just by chance.
Beyond the examination of the kinematisms put into motion by the seismic action, it is very useful to observe those foreseeable but not activated. For all the walls the rotation kinematism towards the outside is foreseeable, as has happened to the facade, except the effect of restrainment offered by the bond at the corners. In fact the facade was very badly bonded at its corners.
For all the other walls this mechanism has not been activated. The walls of the transept, as high as the facade, profited by reinforced bond. Their perfect condition shows that this system has been sufficient to avoid the rocking experienced by the facade, or perhaps worse. Until we have collected sufficient experimental data, we can't accept umbonded walls, but can give credit to the reinforced joints.

Finally, we come to the analysis of the serious damage suffered by the right wall, inclined towards the outside, and by the first pillar on the right, overturned towards the inside. The key to this inexplicable behaviour has been provided by a document found in the State Archives in Rome. It is a letter that was sent on September 10th 1910 to the Naples Superintendant reporting on the effects of the earthquake on the Cathedral on June 7th of the same year (VIII degree). In substance, the report says that nothing happened, except for some longitudinal cracks in the arches and in the vaults of the minor right-hand nave, and the separation of the buttresses from the corresponding wall of the central nave.
For such slight damage the Superintendant did not grant any aid for repairs.
Perhaps the situation worsened after the earthquakes in 1930 and in 1962. No document mentions it, as though, accustomed to living with that cracks, the parish priests of the Cathedral did not complain. However, 70 years later, the earthquake demonstrated that in the first chapel, the pillar with the upper wall on one side, and the lateral wall with the buttresses on the other, had become separate elements, subject to rocking in an asynchronous way and to collapse under the effect of the hammering.
It is interesting to observe in the photograph of the broken pillar, the net breakage at the base that underlines the behaviour of a rigid body. Also the outside wall, as can be seen in the photograph, bends towards outside turning rigidly on its base at the same level as the base of the pillar.
These observations allow us to formulate some assertions that we have seen systematically confirmed by innumeral cases and that we have adopted in order to establish the principles outlined at the beginning.

- The structural organism of a masonry building does not behave as a whole. The seismic action can be amplified in one portion of it and not in another. Therefore, the structure must be examined in every part.
- The collapse mechanism of masonry structures is often well indicated by the kinematism of rigid bodies, as assumed in the XVIII century studies on the resistance of the arches [4], and as proposed today by the Napoletan experts, M. Como and A. Grimaldi [5].

- A masonry constructed according to the "rule of art" is able to behave as a rigid body, locating the sections of rotation and allowing the development of kinematic mechanisms, in some way controllable with numerical analyses.

One can assert that this is the most significant mechanical characteristic, more useful to modelling, more objectively verifiable than the tensional resistance which is not measurable and can be used only with unrealistic continuous models.
We will see in the following paragraph that these observations allow us to formulate a useful model for a quantitative verification. It is, however, opportune to state that if the damage had been repaired in 1910, eliminating the dangerous separation that had formed and restoring wholeness to the pillar-wall, as still existed in the next three chapels, the hammering would have been avoided, and collapse excluded which, in fact, elsewhere has not been verified.
The earthquake has demonstrated that without this particular serious defect, the Cathedral would have suffered damage but not collapse in the X degree earthquake, like a modern well-designed antiseismic building. It is evident that that defect must be eliminated, connecting the outside wall to the pillar. And it is furthermore demonstrated experimentally that with this intervention the right side of the Cathedral will acquire, with regard to the foreseen X degree earthquake, the required safety.

NUMERICAL ANALYSES

Although in the previous non-numerical analysis everything has said as regards safety and strengthening, a computational work can be carried out making use of the mechanical information already acquired. Since both the damaged pillar and wall were constructed with good masonry (and will be reconstructed according to the "rule of art"), therefore we can assert that they are able to implement a kinematic mechanism through the partialization of the horizontal sections at the base and at the top of the vertical elements. A system of three rigid bodies able to oscillate is schematically derived from. The value of the horizontal force that causes the overturning of the system is soon calculated, making use of the principle of the virtual works (with some hypotheses on the position of the rotation centres). It is not possible to establish a realistic force-displacement relationship for the system. A destructive test would be necessary, but it would be valid only for the experimented element. One can make various hypotheses and obtain qualitative indications. In order to utilize the algorisms available in the sector of non-linear dynamic analyses, it is also necessary to attribute a value to the initial elasticity, contradicting in this way the definition of "system of rigid bodies". Nevertheless that does not contradict the physical reality of the problem, which naturally presents almost a linear initial phase, but introduces the necessity to evaluate a further difficult quantity. A non-destructive experience can't provide a value significant the actual behaviour up to the limit of resistence.
In order to study the dynamical behaviour of the lateral wall various possibilities can be explored. Three different F-δ functions have been adopted, more or less dissipative, and aninterval of the initial elasticity has been explored such as to vary the actual period from 0.15 to 0.55 seconds. The results obtained by numerical integration, using ten accelerograms generated from the power spectrum of that registered on November 23rd 1980 and scaled to a peak of acceleration equal to 0.35 g, have been averaged.
The maximum displacement produced by the seismic action at the highest part of the wall is what seems most significant. It slowly increases with the actual period of the system. It also depend on the dissipative characteristics of the F-δ functions, and increases at the decrease of these. In the heaviest conditions: model not dissipative, period between 0.4 and 0.5 seconds, the maximum displacement is worth about 6 cm. In the most favourable conditions, however, the result is 2 cm.
These values have an order of magnitude coherent with the experimental considerations: that the X degree earthquake would cause damage but not collapse. In fact, a displacement of 6 cm would cause damage at the base and at the top of the wall, but would be perfectly supportable. Such agreement allows to state that the model interprets the physical

Examination of the experimental data and direct observation

phenomenon, but here one should note how many subjective hypotheses it has been necessary to introduce, and that a result contained between 2 and 6, that is between a value and three time as much, gives only a qualitative information. It seems that the analysis of the seismic behaviour of the building carried out through examination of the experimental data and direct observation, has coverd a more univocal path, that it has left less space to subjective choices, and it is more reliable to be followed.
It is opportune to conclude this paragraph of analyses by putting forward a last consideration.
The two procedures substantially lead to the same result:

- as regards the facade wall, the mathematic model warns of the danger of collapse, and the experimental observation does not give enough elements to reassure us.
- as regards the right side of the Cathedral (accounting for an effective connection between pillar and wall), the mathematic model furnishes results not very well defined, but certainly below the limit of resistance, and the observation leads us to exclude collapse.

If the experience is correctly interpreted and the mathematic models well chosen, it is natural that the same results are obtained; in fact, the mechanical model describes the physical interpretation that the analyst had already expressed in the preceding phase. Often the mathematic modelling does not add information since, as has been seen, the results that it provides can be interpreted only with qualitative optics. The consequence of the last observation is that often it is not possible to obtain a deterministic correlation between seismic intensity and safety, neither a quantitative indication of the necessary intervention regarding a given value of the seismic accelleration.
It seems that the only possible way to follow is to ascertain the collapse mechanisms and prevent their formation. We will speak rapidly about them in the following paragraph where it will be shown how the third requirement has been satisfied.

TECHNIQUES OF RESTORATION

Our task was to *restore the original "rule of art", and if necessary, add new details consistent with the original structural lexicon.* The fallen pillar and the wall pushed out of plomb, which was later demolished, must obviously be reconstructed. The pillar was in brick masonry and it is easy to rebuild it. The outside wall was, as has been said, in pebblestone masonry and even though that technique was satisfactory in the past, today we are not able to remake it. The art of the mason that puts the right stone in the right place, choosing it from the heap with a glance, after he has memorized all the available forms and for each of them has decided its place, that art which has held up the Cathedral for about three centuries, and for the first two was handed down from father to son, today is lost. And our bookish competence, useful for judging, is worthless for accomplishing. We shall be forced to construct also the wall in bricks.

walls pushed out of plumb

The art of the mason that put right stone in the right place

The same applies to the upper walls of the central nave, in part collapsed together with the pillar that supported it. The part remaining denounces a slackening in the quality of the work of the ancient workmen and all of it has to be demolished and reconstructed in bricks as a "work of art".
The problem of the connections remains.
The masonry walls which will be reconstructed will have regular joints, also between the buttresses and the corresponding wall of the central nave, thus filling in the lack of knowledge of the engineers of the XVIII century. But we have seen that we cannot rely on the simple bond. It is necessary to obtain greater guarantees.
The XVIII century custom to introduce metallic tie-rods within the masonry, to improve the connections, is documented, as we have observed, also in the Cathedral of "Sant'Angelo dei Lombardi". At the end of the age of Enlightenment, the Rondelet, in the first modern treatise on architecture [6], wrote: *"It is not enough to construct the walls of a building in the desired dimensions and with all the proper attention; since they will be weighed down by the weight of the floors and the roofs that naturally tend to push them out into space, an effect that is increased still further by the continuous shaking caused by the movement of vehicles in the*

big cities (that appened in Paris; in Irpinia what is not caused by the vehicles is left up to the earthquakes). *Certain precautions are taken in regard to this, floor by floor, in the construction of the walls to prevent any separation, putting in the center of the walls or in their thickness, horizontal chains of flat or square iron, firmly applied and solidly soldered at the ends with anchors, which hold the walls together in such a way to prevent one moving without the other and, therefore, helping each other. These chains are placed in the walls at the time of construction. Meanwhile, only in buildings of a certain importance chains shall be put along the whole length of the walls".*

This indication was actually applied in the XX century, and examples can be found everywhere. The prescription contained in many seismic regulations, to insert reinforced-concrete tie-beams in the building masonry, is just an unconscious remembrance of a technique already used before reinforced concrete was invented. In actual fact, the indications that Rondelet had matured through experience of a century of building constructions (the Galileian culture still bore its fruit), is much better than the present prescription. The beam element of reinforced concrete, adapted to support bending stress, naturally requires that the reinforcement is arranged on the outside edges of the section. This is also its weakness because the inevitable crack of the concrete exposes the steel to corrosion and the structure to a rapid degradation. But the tie-rods that must "*prevent any separation*" of the walls, does not suffer bending, and it is not, therefore, of any use arranging the iron on the outside edges as is usual in the tie-beams of reinforced concrete. To place the rod of iron at the center of the wall, avoiding the interruption of the masonry structure with concrete, is evidently more protective.

The technique of restoration is all here, as formulated by Rondelet. In the building of habitations, where the walls are not very thick, the tie rods can run along the floors and also make use of their girders. In the Cathedral, it is better to arrange large bars of steel type DIWIDAG at the center of the wall. In the reconstructions these will be arranged in special slots; in the original walls longitudinal holes will be made, after having improved the route of the rotary rig with a little grouting. In any case the highest extremity of the walls will be reconstructed using brickwork for a strip of 40 cm, to house the tie-rods and offer a solid support to the trusses.

With this technique the bond will be reinforced between the facade and the trasversal walls, and between these and the apse wall, in order to prevent the possibility of overturning. The lateral wall will be connected to the internal pillars in such a way to guarantee the synchronous movement as in the the analyzed model. Our intervention achieves the idea that Rondelet recommended, fruit of a more modern culture, but deriving from the architects of the Cathedral.

Vertical tie-rods will not be used. These would completely transform the structural behaviour, changing the system of rocking rigid bodies into elastic frames. The dynamic amplification that today is cut by the kinematic movement would follow. The bent masonry would be overstressed on the compressed edges with little probability of survival; the bars themselves would perhaps not succeed in attaining the necessary bond, between the stones of the wall, to achieve the required traction without disconnecting them.

The masonry of Rondelet is not the modern reinforced masonry. The latter re-proposes, with different material, the experience made with reinforced concrete in the optics of the continuum. The former rationalizes without distorting an ancient technology as old as civilization.

Without attempting to illustrate the entire restoration of the Cathedral, this paragraph is concluded with a warning of an excess in the connections. Kinematic liberty must be controlled but not stopped.

CONCLUSIONS

The ancient masonry structures merit respect, because they have in themselves all the safety that we expect today. In order to preserve them we must understand them and learn how to use their language.

[1] *Atlas of Isoseismal maps of italian earthquakes*, CNR, Progetto Geodinamica, Bologna 1985

[2] G.A.BREYMANN, *Trattato di costruzioni civili. Costruzioni in pietra e strutture murali, 4° ed. it., Milano 1926*

[3] D. DONGHI, *Manuale dell'architetto. Materiali, elementi costruttivi e finimenti esterni delle fabbriche*; vol. I, parte I, ristampa stereotipa, Torino 1925.

[4] A. GIUFFRE', *La meccanica nell'Architettura, cap. VI, Roma 1987*

[5] M. COMO - A. GRIMALDI; *An unilateral model for the Limit Analis of Masonry Walls*, in "atti del congresso: "Unilateral Problem in structural Analysis" Ravello, 1985", ed. Springr 1985

[6] G. RONDELET, *Trattato teorico e pratico dell'arte di edificare*, I Trad. it. a cura di B. Soresina, Mantova 1832, libro VI, Vol III, parte II, p. 77.

FIG. 1
The Cathedral of Sant'Angelo dei Lombardi after the earthquake of 1980

FIG. 3
isoseismal maps of the earthquake of Sep. 8th., 1694

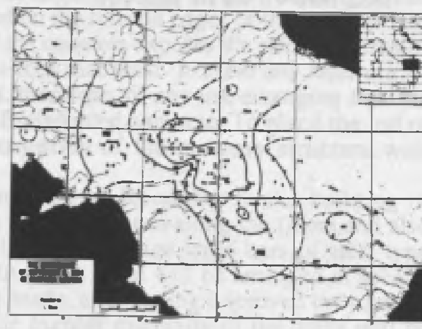


FIG. 2
The structure of the Cathedral with nave and two aisles

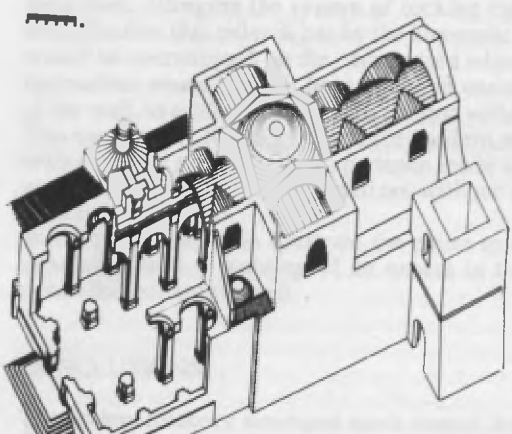


FIG. 4
Isoseismal maps of the earthquake of Nov. 23rd, 1980

The isoseismal maps of the two strongest earthquake show the same focal mechanism.

FIG. 1

The piller turner over towards the inside undre the effect of hammering

FIG. 2

The wall turned over towards the outside

FIG. 3

The original metallic chains in the corners have avoided the detaching of the wall

f) Avvertenze. — La costruzione dei muri di pietrame greggio va affidata ad operai esperti i quali sappiano convenientemente scegliere le pietre per formare corsi, per fare legamenti e per bene assestarle.

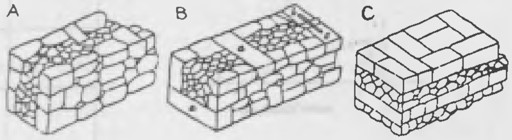


Fig. 549. — Con pietre frontali grosse.

Fig. 550. — Con pietre di legamento a, o leghe, o con pezzi incatenati mediante grappa di ferro.

Fig. 551. — Con corsi di pietre grosse.

FIG. 4

The drawings of an Italian author of treatises of 1906:

A) masonry little resistant to seismic action;

B) and C) masonry suitable to realize kinematism

FIG. 1
The trasversal structure

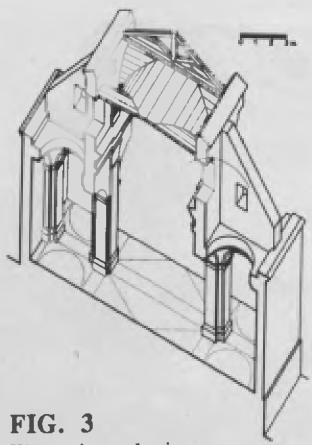


FIG. 2
Scheme of analysis of the trasversal structure

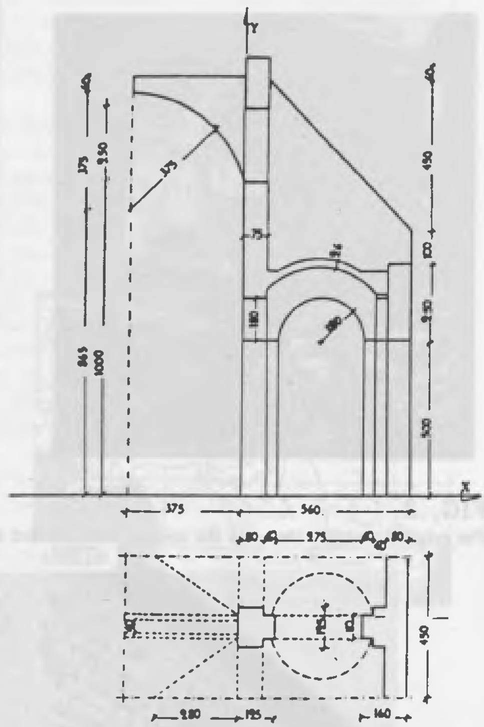
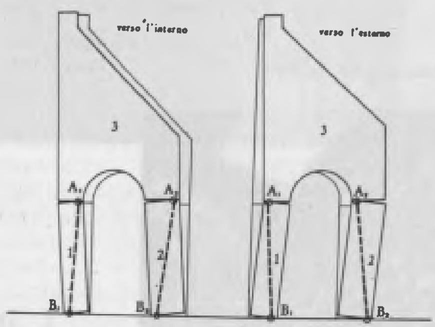


FIG. 3
Kinematic mechanisms



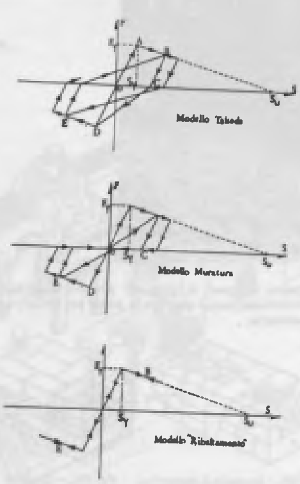
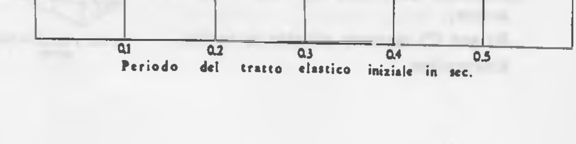


FIG. 4
Theoretical model Force-Displacement. The "turnover" model describe the oscilletion of the rigid system with an initial elasticity

FIG. 5
At the variation of the initial elastic constant, and therefore of the period, the three models give maximum displacements contained between 2 and 6 cm.



RENFORCEMENT ET RESTAURATION DE MONUMENTS EN MAÇONNERIE EN PIERRE D' EPOQUE ROMAINE DANS LA ZONE DE LA SABINE (LAZIO).

STEFANO GIZZI (*)

SUMMARY

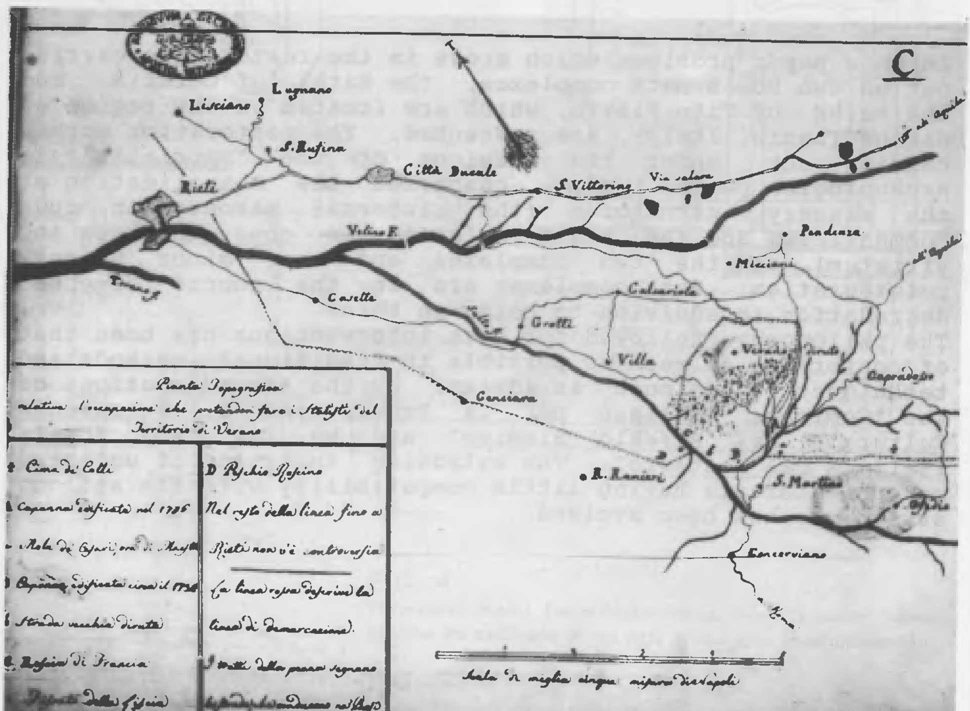
In this paper problems which arose in the restoration carried out on two Roman bath complexes, the Baths of Cotilia and the Baths of Tito Flavio, which are located in the region of Sabina (Lazio, Italy), are presented. The restoration works, carried out under the auspices of the "Soprintendenza Archeologica per il Lazio", concerned the consolidation of the masonry structures (the internal masonry in opus coementicium and the masonry facing in opus incertum and vittatum) of the two complexes and some minor masonry reintegration. Both complexes are in the course of great degradation in addition to being in ruins.
The philosophy followed for both interventions has been that of conforming as much as possible to traditional methods and techniques in so much as advised by the recommendations of the "Comitato Italiano per la Prevenzione del Patrimonio Culturale dal Rischio Sismico" and by the 1987 "Carta Italiana del Restauro". The extensive insertion of untested modern materials having little compatibility with the antique structures has been avoided.

(*) Architecte de la Soprintendenza Archeologica per il Lazio - Ministero per i Beni Culturali e Ambientali - Roma, Italia.

Juillet 1989

Nous désirons présenter la restauration de deux ensembles architecturaux en maçonnerie; situés dans la région de la Sabine (actuellement province de Rieti) et faisant partie de thermes remontant à l'époque de la Rome républicaine; le premier connu sous le nom de "Thermes de Cotilia" (Cittaducale), le deuxième appelé de façon erronée "Thermes de Titus Flavius Vespasianus", ces deux ensembles étant distants l'un de l'autre de quelques kilomètres seulement (cinq).

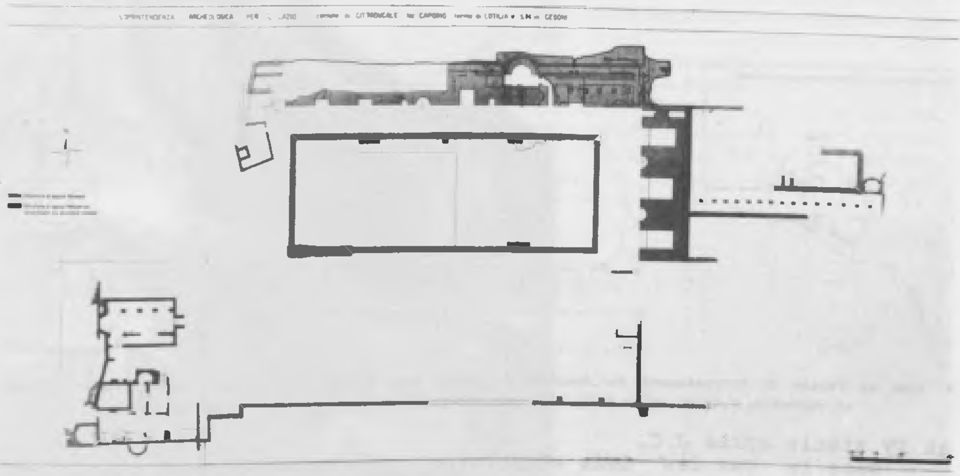
Le nom du deuxième lui vient de ce que les empereurs flaviens l'ont fréquenté assez souvent et l'ont probablement modifié et agrandi. Entre les deux constructions se trouve une série de terrassements et substructions de la Via Salaria qui s'articulent dans une zone de petits lacs karstiques et de sources d'eau sulfureuse où l'on pense que se situait le Vicus Aquae Cutiliae implanté près du lacus Cutiliae.[1]



1. La zone des thermes romains de Cotilia et de Castel S.Angelo dans une gravure du XVIII.ème siècle; on y remarque que les petits lacs et les sources y sont signalées. Archivio Segreto Vaticano,Confini-Buste Bianche 21

Les deux ensembles étudiés, qui sont reportés sur la Table de Peutinger avec le symbole des thermes, ont eu différentes phases de construction se développant du IIème siècle avant J.C. jusqu'au IVème siècle après J.C. et présentent des similitudes typologiques et morphologiques. De plus, sur le site du premier ensemble, on atteste la présence, à l'époque proto-médiévale, d'une petite église dédiée à Sainte-Marie in Cesoni.[2]

Les thermes de Cotilia sont composés d'une série de quatre terrasses descendantes soutenues par des murs de substruction, certains avec des niches et des absidioles, qui s'articulent sur plus de 300 mètres de longueur. Il s'agit d'appareils muraux avec un parement en opus incertum plutôt grossier, uni à des sections de murs à sec. Dans les angles on trouve des morceaux de mur qui sont construits parfois directement sur la roche, d'autres fois sur des blocs équarris plus régulièrement, et présentent une situation similaire à celle d'autres édifices thermaux et de "villae rusticae" sabines (par exemple la Villa di Assio à Colli sul Velino).



2. Relevé planimetrique d'un secteur des terrassements des thermes de Cotilia avec les structures de la petite église de S.Maria in Cesoni construite au Moyen Age. (Relevé de F.Graziani-G.Carconi, Soprint.Archeol.Lazio).

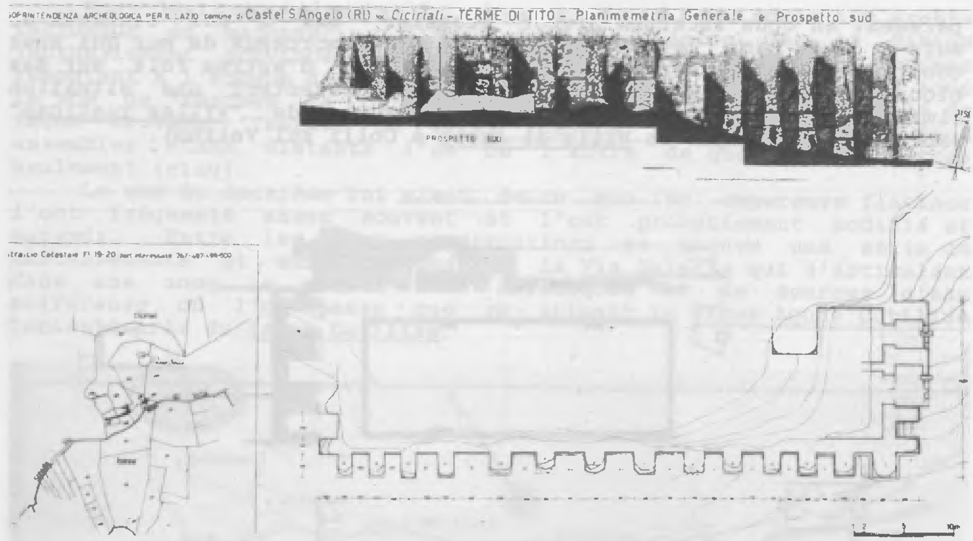
A d'autres endroits l'appareil en opus incertum montre également les lignes d'arrêt des banchées comme on peut le voir aussi dans certaines des "villae rusticae" sabines déjà citées (par exemple dans le parement de la villa avec un cryptoportique à Vacone).

Une des structures murales les plus articulées, avec des niches, donne sur une grande piscine rectangulaire de 60 m par 24 m comprise entre le 2ème et 3ème terrassement, creusée dans la roche et retrouvée grâce aux interventions de la Soprintendenza Archeologica per il Lazio.

Le deuxième ensemble présente lui aussi deux terrasses descendantes, la première soutenue par une série de murs et d'arcades et la seconde par des murailles en appareil mixte bordé et réticulé, et par des contreforts dans lesquels s'ouvrent de nombreuses niches rectangulaires tandis que dans la partie supérieure se trouve l'accès à différentes pièces en forme de T.[3]

Sur la partie orientée vers la campagne, à des hauteurs régulières, il reste des traces de canalisation avec des empreintes de formes en bois.

On a déjà mentionné plus haut que les types de maçonnerie et les matériaux utilisés dans les deux ensembles semblent similaires. Tandis cependant que pour les "Thermes de Cotilia" la pierre utilisée pour l'opus incertum, ainsi que pour l'opus quadratum, est un calcaire local très dur et compact, travaillé en morceaux presque réguliers de grandeur moyenne au cours de la même période de construction, en ce qui concerne en revanche les thermes dites de Titus Flavius Vespasianus à Castel S. Angelo, le matériau calcaire est plus spongieux et d'une couleur différente; on peut noter également sur la partie orientée vers la campagne des reprises relatives à des restaurations d'époque successive, remontant au moins



3. Plan et façade du terrassement des thermes de Castel Sant'Angelo en cours de restauration. (Dessin et relevé de F.Graziani et G.Carconi, Soprintendenza Archeologica per il Lazio, 1989).

au IV siècle après J.C.

Dans le cas des deux structures, les éléments sont constitués par un carbonate de calcium riche en substances végétales conservées par les incrustations calcaires.

Au moment où ont commencé les travaux de restauration, actuellement en cours sur les premiers lots, l'état de dégradation est apparu de façon plus évidente. Dans les deux cas, les interventions de restauration ont été plus difficiles que prévu à cause de l'instabilité du sol étant donné que les constructions se trouvent sur le point de conjonction de différents types de terrain, l'un caractérisé par des éboulis de faille et l'autre par des alluvions (avec des dépôts lacustres marécageux et tourbeux). A cause de la solubilité des roches, on constate encore des phénomènes karstiques et des glissements de terrain.

On remarque aussi dans les deux cas que les murs ont subi, au cours des temps, des restaurations et "réparations" souvent dépourvues de pierres d'attente.

D'autre part le terrain et les éboulis accumulés au dos des murs de substruction et de terrassement exerçaient (et exercent aujourd'hui dans les parties qui ne sont pas encore dégagées et consolidées) une poussée considérable provoquant une certaine "rotation" et un renflement aussi bien au niveau des constructions qu'entre les successifs terrassements descendants.

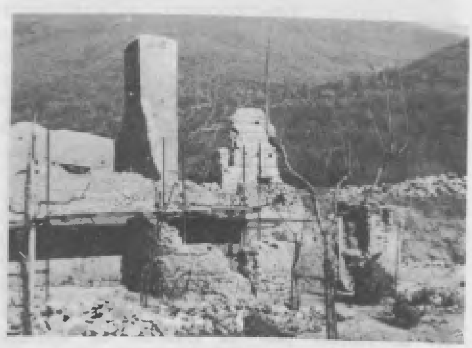
Les problèmes de statique ont été encore accentués par les derniers tremblements de terre de 1974, 1978 et 1979, et surtout par ceux de novembre 1980 et 1982. Il faut ajouter que les deux sites étaient envahis par la végétation qui avait agravé le processus de détérioration auquel était soumis la maçonnerie. A cause des racines qui " digèrent " 'les matériaux sur lesquels elles végètent, des effritements et des fissures se sont produits dans les murs entrainant la pénétration de l'eau qui, outre à dissoudre et délaver lentement les liants, provoquent en gelant des cassures et le détachement de morceaux de plus en plus larges de structures. Cette végétation empêchait entre autre la lecture des détails architecturaux (des exèdres et des demi-colonnes en brique dans le

4. Les thermes de Castel Sant'Angelo dans une gravure publiée dans le livre de G.A. Guattani, Monumenti Sabini, Tipografia C.Pinelli, Roma 1827-1830, II, entre p. 272 et p. 273.

cas de Cotilia et des enduits avec des pigments jaunes dans le cas de Castel Sant'Angelo).

La méthode suivie pendant la restauration des deux ensembles a

5. Thermes de Cotilia. Le terrassement inférieur et l'absidiole avant la restauration. (Photo mars 1987).



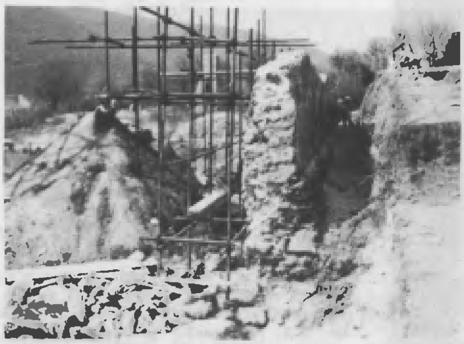
6. La consolidation du même secteur au moyen de tecniques traditionnelles. (Photo février 1988).

été unitaire, s'agissant de deux structures urbanistico-architecturales semblables non seulement du point de vue de la typologie architecturale, comme on l'a déjà dit, mais encore du point de vue des problèmes de détérioration présents. Une telle méthode peut être résumée en deux points fondamentaux:

a) choix de techniques de consolidation de type traditionnel (pre-industriel);

b) intégration de maçonnerie strictement liée à la stabilité pour les noyaux et les parements en pierre, s'agissant de restauration de type archéologique.

En ce qui concerne le premier point, on a essayé surtout d'alléger le plus possible l'action de la poussée des terrains situés à l'arrière au moyen de fouilles (en partie à ciel ouvert, en partie "à section fixe" et en partie stratigraphiques) destinées, outre à la vérification des niveaux originaux de la couche de base des murs, également à la création de véritables "boutonnières"



7. et 8. Thermes de Cotilia. L'allègement de la pousse du terrain au moyen de la création d'une "boutonnière" derrière le deuxième terrassement (à gauche) et avec le vidage de l'absidiole du terrassement inférieur (à droite). (Photos 1988).

d'isolement réenterrées partiellement avec un drainage. Ceci a cependant créé des problèmes dans un secteur des Thermes de Cotilia où, pendant la phase de fouille pour la création du bassin nécessaire pour le drainage et pour le vide d'allègement prévus à l'intérieur de tout le front du mur de terrassement en <u>opus listatum</u> et <u>opus incertum</u>, on a découvert des structures thermales de grand intérêt, telles que des <u>suspensurae balneorum</u> d'argile et brique qui

9. La fouille pour la création du drainage. (Photo janvier 1989).

10. Le terrassement inférieur réintégré partiellement. (Photo mars 1989).

soutenaient le pavement en bipedales d'une pièce chauffée sans aucun doute par hypocauste. Pour cette raison le système d'allègement et de drainage prévu a été réalisé d'une façon plus simple, avec des pierres à sec arrangées manuellement. En ce qui concerne le problème plus particulier de la consolidation des murs, la question plus importante était comment et dans quelle mesure reconstituer les morceaux manquants de maçonnerie.

En particulier dans le cas des thermes dits de Titus Flavius Vespasianus à Castel Sant'Angelo, la partie frontale et surtout les contreforts présentaient d'importantes lacunes soit dans la maçonnerie elle-même soit dans les parements au pied de la section de la base de telle sorte qu'avec l'aggravation progressive de la détérioration le phénomène de gauchissement à cause de la charge de pointe aurait pu se produire.

11. Castel S.Angelo (Rieti), thermes romains: état de l'ensemble avant les restaurations.(Photo mars 1987).

12. Etat de détérioration et amincissement de la section de base des éperons. (Photo mars 1987).

On a décidé de réintégrer avec la technique "découdre et coudre" les parties de la maçonnerie qui montraient les pertes plus importantes éliminant les parties les plus instables et reconstituant le parement superficiel là où il est apparu nécessaire comme protection de la maçonnerie elle-même, presque comme une "surface à sacrifier" et non pour recomposer une unité figurative hypothétique.

Dans un premier temps on avait pensé à faire une consolidation "massive" avec une intervention structurale comportant aussi des injections avec insertion d'armatures. Ceci aurait pu, considérant le mur en opus caementitium avec une maçonnerie et des parements poreux à cause de la qualité de la pierre et de la détérioration des mortiers, garantir une bonne absorption du coulis. Par la suite on s'orienta vers une autre solution étant donné également la présence d'enduits avec des pigments jaunes, apparus après le désherbage, qu'il aurait été coupable de percer.

De plus, dans un secteur des Thermes de Cotilia au nord de la natatio, on a pu vérifier la façon dont laquelle les armatures métalliques insérées au cours des restaurations faites par la Soprintendenza dans les années 70 ont provoqué des fissures dans la maçonnerie à la suite de l'augmentation de volume due à l'oxydation.

C'est alors que, comme on l'a dit plus haut, le choix s'est porté sur des techniques traditionnelles, refusant des matériaux et des technologies incompatibles avec les maçonneries anciennes ou différents par leurs caractéristiques chimico-physiques (des élongations unitaires différentes dues aux dilatations thermiques, et des comportements divers suivant les lois de Hooke). Dans ce sens on a tenu en considération les Recommandations du Comité national pour la prévention du risque sismique et la Carta Italiana del Restauro de 1987, dans lesquelles on met l'accent sur la nécessité d'éviter au cours des restaurations l'utilisation "non réfléchie" de matériaux nouveaux sur l'utilisation desquels les idées ne sont pas très claires, et de préférer au contraire, même si elles ont une apparence visive étrangère à l'oeuvre, des actions de restauration de type traditionnel (contreforts et colmatages) facilement controlables et remplaçables.

Ainsi, dans le cas des Thermes de Cotilia, on a préféré intégrer les murs de terrassement en opus incertum avec un type similaire d'appareil, distinct du précédent d'une manière discrète ou au moyen d'une fine rainure de séparation (comme près de l'absidiole proto-médiévale) ou au moyen encore d'un retrait de 2 ou 3 cm de la nouvelle surface (comme dans le deuxième terrassement nord-oriental près de l'exèdre avec des colonnes) élevée juste le nécessaire pour servir de retenue au terrain situé derrière.

A Castel Sant'Angelo les contreforts qui soutiennent le grand front orienté vers la campagne ont été réintégrés et renforcés avec la régularisation des plinthes de la base et ensuite réenterrés partiellement. Le premier contrefort nord-oriental d'angle a été presque totalement réintégré, la maçonnerie ainsi que le parement, comme exemple didactique afin de montrer quelle pouvait être la configuration originale. On a essayé d'autre part d'utiliser une pierre provenant d'une carrière de la zone même si cela a posé de gros problèmes vu la rareté des carrières encore existantes en Sabine. L'approvisionnement s'est fait dans la carrière de Contigliano, à une quinzaine de kilomètres de distance des Thermes de Titus Flavius Vespasianus.

13. Castel S.Angelo, thermes romains: réintegration partielle -en cours- des plinthes et des sections de base qui auparavant étaient détérioreés. (Photo 1989)

14. Le prémier éperon oriental réintégré dans le noyau de la maçonnerie et dans le parement dans un but statique ainsi que didactique. (Photo j. 1989).

En ce qui concerne la bibliographie du XIX.ème siècle sur le deux ensemble, on peut consulter: G.A.Guattani, Monumenti Sabini,Tip.C.Pinelli, Roma 1827-1830,II,pp.272 et suiv., et N.Persichetti, Viaggio archeologico sulla via Salaria nel circondario di Cittaducale, E.Loescher ed., Roma 1893, pp.162-168.

1. M.Santangelo, Aquae Cutiliae, dans Fasti Archeologici, XXX-XXXI, 1975-'76, 2, n.11661.

2. A.M.Reggiani, Le terme di Cotilia, dans Archeologia Laziale, ed.CNR, III,1977, pp. 91-98, et G.De Palma, Terme di Cotilia, dans Archeologia Laziale, ed.CNR, VII, 1985, pp. 185-192.

3. F.Coarelli, Lazio, Laterza ed., Bari 1982, pp. 24-25.

TITLE: STUDY OF THE CAUSES FOR DILAPIDATION OF A TYPE OF MEDIEVAL HINDU TEMPLES - CALLED, 'KAKATIYAN TEMPLE'S IN SOUTH INDIA.

AUTHOR: LAKSHMANA MURTHY, KHANDRIKA.*

SUMMARY

The Hindu Temples built by a Dynasty called Kakatiyas in South India (12th & 13th Cent.A.D.) were found dilapidated. Defective foundations and poor workmanship were thought as possible reasons for dilapidation of these temples by some experts. The possibility of damage by earth quakes was mooted in some quarters. While all these studies were based on observation, a team of Engineering professors carried out Scientific studies on these temples (1986).

Recently two dilapidated temples were dismantled completely (for reerection by way of Anastylosis) which gave an opportunity to see the concealed parts of the Kakatiyan temples for the first time. The Author is directly associated with the dismantling of one of the temples. Basing on the information obtained after dismantling, an attempt is made to understand the causes for dilapidation of the Kakatiyan Temples.

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3.4.1989

I. INTRODUCTION

A Hindu Dynasty named Kakatiyas ruled the Upper parts of South India (Approximately the area of present Andhra Pradesh State lying between 13° to 20°N Lat; and 72° to 85° E-Long.) for about 160 years from A.D. 1163 to A.D. 1323. During their rule a good number of Hindu Temples (more than 35) were constructed in stone Masonry. A Kakatiyan Temple in plan generally consists of a Cella (Sanctum) facing east, with a vestibule in front. This vestibule is connected to a wide square pillared Hall, which has entrance porches on the other three sides. In variation to this plan, is another design consisting of triple shrines (each consisting of a cella and vestibule) connected to a pillared Hall on its three sides and an entrance porch into the pillared hall, provided on the fourth side. Some of the Kakatiyan temples have a pyramidical spire called Sikhara above the roof of cella, while others are flat. The Sikharas of Kakatiyan temples are made of bricks, and these bricks are quite lighter and called floating bricks, as they really float in water.

Kakatiyan temples were built in trabeate technique, without using any mortar for binding, by placing the stones, one after other in layers from foundation to pinnacle. The bond or key is obtained by perfect cutting and fine dressing of stones, with hair thin joints, as found in Greek temples of classical Age. Iron clamps were used to keep the stones bonded. The walls were formed by placing stone slabs in two rows, leaving a cavity of about 0.20 M in between, and this cavity was filled with brick bats or rubble mixed with mud. The pillars that support the ceiling are not of a single block, but formed by joining 5 separate pieces. The entire ceiling was divided into square bays formed by running beams on all four sides, and these bays were closed with 4,8 or 12 triangular slabs and a central square slab, fixed by way of corbelling Technique.

At present all the Kakatiyan temples are found in dilapidated condition, while some temples have actually collapsed. Scholars who tried to understand the problem by observation pointed to intrinsic causes like defective foundations and poor Engineering knowledge of the builders, (Yazdani - 1927). Another such study mentions earth quakes among the possible reasons for dilapidation. (EACRI - 1982).

II. SCIENTIFIC STUDIES

The National Organization for conservation of Ancient Monuments in India (A.S.I.) entrusted a project work to Regional Engineering College (REC), Warangal in A.P. State, to study the causes for dilapidation of Kakatiyan temples and suggest remedial measures. Geological and Geophysical investigations together with remote sensing technique, and Geo-technical surveys including Engineering surveys of two important Kakatiyan temples, were conducted by a team of professors of REC in 1986. Structural analysis of these temples was also done to understand the causes for dilapidation. For want of space, it is not possible to discuss in detail, the studies made by REC. The REC Report summarises the following causes responsible for dilapidation. (P-311-REC Report - 1987).

1. *Inadequate foundations.*
2. *Over stressed rock floor beams.*
3. *Loss of confinement of sand in the base foundations.*
4. *Use of dry non-rigid joints incapable of sustaining lateral loads.*

5. *Leakage and seepage.*
6. *Lack of maintenance.*
7. *Pilferage and vandalism.*

III. STUDY AFTER DISMANTLING

The Archaeology Department of Andhra Pradesh State that has the responsibility of preserving 90% of the Kakatiya temples, decided to completely dismantle and re-erect, two dilapidated temples by way of anastylosis (1985), which gave an opportunity to see the concealed parts of Kakatiyan temples for the first time.

One of the temples chosen is a triple shrine at Nagulapadu village. Granite stone was used in temple structure and black dolorite was used for pillars, beams, and door frames, on which profuse ornamentation was done, the shining lustre of which has not dimmed even today. The details of the temple are as under:

(1) Location: Nagulapadu village, Nalgonda District. Andhra Pradesh State. (INDIA). 17°N.,Lat; 79°-36' E - Long.

(2) Site conditions: The Temple is located on the left bank of River Musi (about 200 M from the river bed). Before construction of a Reservoir in 1920, floods were occuring now and then to this River. According to local villagers, the floods in 1908 have inundated the temple site. When the temple was built, the site around was probably habitation site, but after its abandonment, the area might have become agricultural fields and continues to be the same at present.

(3) History of construction: The temple was built in A.D. 1234. and enjoyed worship atleast upto A.D. 1303. After the fall of Kakatiyan Dynasty (A.D. 1323), this temple might have been damaged by Muslim invaders and from that time abandoned.

(4) Condition of the temple before dismantling:-

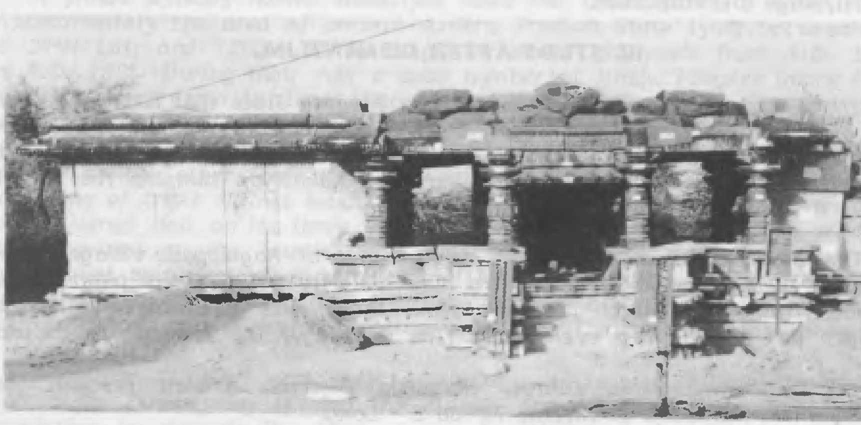
(i) The Northern shrine of the temple was completely missing. The chipped off faces of the sculptures and the presence of temple slabs in the Mosque near this temple, points to vandalism and despoilation.

(ii) The basement of the temple upto 0.60 M was concealed under the debris and earth accumulated alround. Extensive growth of vegetation was observed on top and around the temple. Some of the stones at basement level were missing, while some were found broken.

(iii) The walls around cellas have moved out of plumb, and the core filling has flown away and plants have grown in the cavities which further widened the joints.

(iv) The Pillars also moved out of plumb. The floor beams on which pillars rest were found sunk and broken, making some of the pillars stand without much support.

(v) Wide opending were seen in ceiling and there was evidence of profuse leakage. The lime concrete on top of the roof has worn out completely. A good number



FRONT VIEW OF THE DELAPIDATED TRIPLE SHRINE AT NAGULPAD VILLAGE, ANDHRA PRADESH, INDIA

of ornamental eve slabs (Placed at cornice level) were missing, leaving the wall top open to sky.

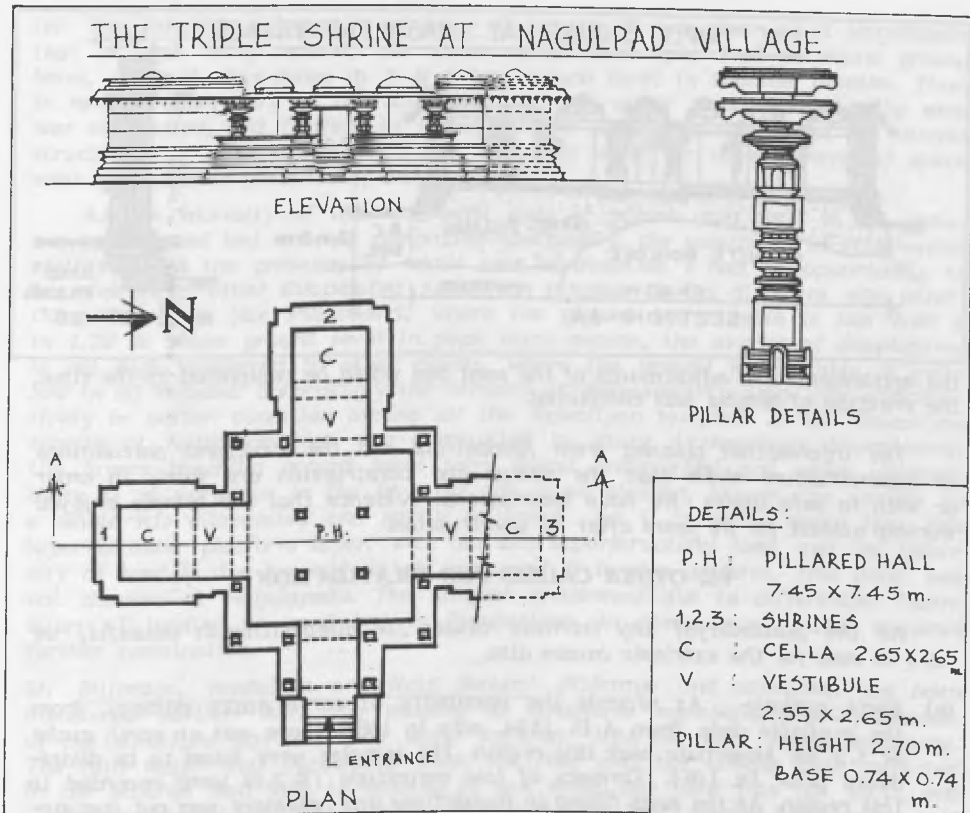
As the tilt of the structure, or the cracks do not show any pattern, and as the structure was in precarious condition, it was not possible to carry out any further studies before dismantling.

V. INFORMATION GLEANED AFTER DISMANTLING.

The top soil in the temple site is black cotton soil. The trench for foundation was dug upto hard Murrum level on all sides. As hard Murrum could not be found earlier, the trench in the Northern shrine was taken upto 2.40 M below ground level, while on the eastern side where a granite boulder was met, the depth of the trench was about 1.20 M. The entire trench was filled with river sand. The temple structure was raised on this sand field bed. The lowest layer of the temple called Foundation Layer, went up 0.40 M below ground level and directly rested on sand bed. From this foundation layer, upto basement top, there were 5 layers of stone blocks, placed one over the lower. The layers of basement around looked like a retaining wall (width 1.10 M at bottom level and 0.75 M on top, and height about 1.50 M), and the superstructure covering 3 shrines is resting on this retaining wall, and the space with in the retaining was was filled with sand. The floor beams, supporting the pillars, were fixed on this sand bed in a grid pattern and the space in between the floor beams was covered with floor slabs.

VI. TECHNICAL DETAILS AND ANALYSIS

The total weight of the structure is about 1850 Tonnes. The load of the 3 shrines is transmitted to sand bed, by the walls running around the shrines. The intensity of load at foundation layer level of these walls (i.e.,on sand bed) works out to 1.53 Kgs/Cm^2.

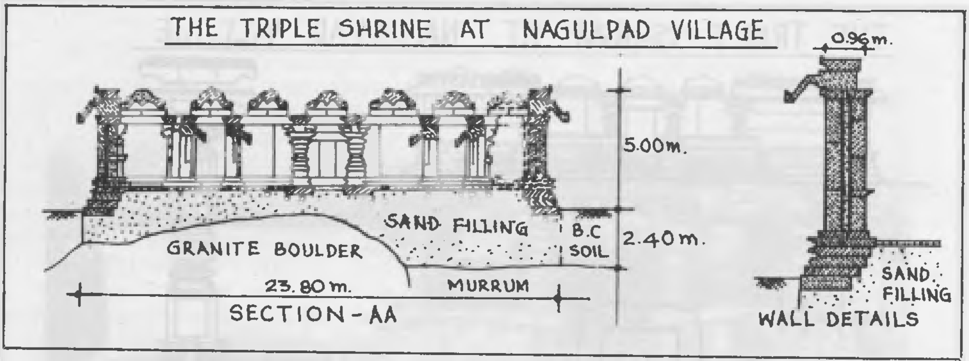


The load of the temple structure in pillared Hall is transmitted to sand bed through the 16 pillars resting on floor beams, and the intensity of load is 2.73 Kgs/Cm^2

The foundations of walls around shrines resemble strip foundations and that of pillars also strip foundations with pointed loads. But either of the shallow foundation (of the walls or pillars) is not resting on soil directly, but resting on a sand filling through which, load is transmitted to soil. There was no enlargement of the size of the sand filled trench. The area of the temple at foundation layer level was about 2780 Sft. (258.54 Sq. M) and the sand filled area was also same.

The temple resembles a framed structure, in which the load of the roof is transmitted to roof beams, which in turn transmit the same to floor beams through the pillars. The span of the beams was not more than 2.50 M and the sizes of beams, pillars, and roof slabs are with in safe limits.

If load distribution is considered, the behaviour of the sand filling has also to be considered. The sand was of fine ungraded type and as it lies into a trench, it is prevented from spreading. The bearing capacity of sand filling depends upon its compaction. According to REC Report, a sand sample examined was found to be compacted upto 40% (0% means loose sand filling, while 100% means thoroughly compacted sand fill). As is the character of sandy soils,



the settlements and adjustments of the sand bed would be completed by the time, the erection of temple was completed.

The information gleaned from foundations and the structural calculations of superstructure show that the design and construction are fairly in order or with in safe limits. We have inscriptional evidence that the temple enjoyed worship atleast for 69 years after its construction.

VII. OTHER CAUSES FOR DILAPIDATION

As the possibilityof any intrinsic causes for dilapidation is doubtful, we have to look for the extrinsic causes also.

(a) Earth quakes:- As regards the possibility of earth quake damage, from the available data from A.D. 1594, only in 1969, there was an earth quake of 5.3 Mb Magnitude near this region. The temples were found to be dilapidated prior to 1969. Tremors of low magnitude (≯ 2.0) were recorded in this region. As the sand filling in foundations and basement was not compacted thoroughly, there is a possibility of the sand filling getting compacted by the low tremors and the ensuing readjustment of the structure can disturb its equilibrium leading to dilapidation. This assumption requires confirmation by further studies.

(b) Breakage of Floor Beams:- Some of the floor beams supporting the pillars were found broken. On structural Analysis REC found that, the floor beams were subjected to a high bending force, which resulted in a tensile stress of 150 Kgs/Cm^2, which the stone beams were not able to bear and thus broken.

As we have seen earlier, such high tensile stresses would not develop in floor beams, as long as the temple structure was intact.So, the breakage of floor beams might have occured, after the process of dilapidation has begun due to other causes. After the loss of under bed sand filling, the floor beams stressed by pillars resting on them, readjusted by sinking at one end, and protruding at the other end or sinking totally, and where such readjustment was not possible, they were broken as suggested by REC. In some dilapidated Kakatiyan temples, I chanced to observe that the pillars and even their bases were found broken due to high compressive stresses, which points to a total change in the load distribution pattern, after distortion of the original equilibrium.

(c) The behaviour of sand in presence of water:- It was observed at Nagulapadu that in peak rainy season, the water table rises upto 1.20 M below ground level, while it goes down to 3 M below ground level in summer months. Thus, in rainy season, part of the sand filling comes under water level. As the sand was compacted, and further, as there was the superimposed load of the temple structure, the possibility of flowing away of sand, or development of quick sand conditions in presence of water is meagre.

As the intensity of load (on sand bed) of pillars, and wall is not same, and as the sand bed was not compacted thoroughly, the possibility of differential settlement in the presence of water can be doubted. I had an opportunity to examine eight other dilapidated Kakatiyan temples having different site conditions. In places like Pillalmarri, where the ground water table is just 0.60 M to 1.30 M below ground level in peak rainy season, the degree of dilapidation is not high compared to other places, where the ground water table is quite low in all seasons. Surprisingly the temples at Pillalamarri village are comparatively in better condition among all the Kakatiyan temples. In the Kakatiyan temple at Nidikonda that was dismantled by State Archaeology Department, the layers touching ground level were found undisturbed and when checked, there is no variation in the level of the layer alround, though the temple as a whole has dilapidated and partly collapsed. But it is to be noted that this layer is outer platform layer, with out any superstructural load, and the intensity of load is due to self weight only and it is also uniform. This point was not checked at Nagulapadu. The unequal settlement due to differential intensities of loading on sand bed in foundations, in presence of water requires further examination.

(d) Pilferage, Vandalism and Rain Water:- Pilferage and vandalism has been mentioned earlier. With the missing of structural members, the equilibrium of the structure was lost, and water entered into the core of the structure. The rain water that percolated from the roof and the oblique rain collected in the pillared Hall (due to missing of eve slabs) has gone into the walls, and washed away the filling in the cavity walls. The iron clamps that were used to keep the stone blocks in position have oxidised (in presence of water) and having lost their strength, caused the stones to move asunder:

(e) Botanical and biological causes:-In the humid and tropical climate of this region, vegetation growth is extensive. The trees and wild plants that have grown in and around the temple have penetrated inside the walls and further widened the joints. The roots of some trees like babul, found spread deep inside the foundations. Among the biological agents, the rats and bandicoots that dug holes around the temple caused the spreading of sand.

(f) Flowing out of sand from basement level:- At the basement level, the fine sand particles have flown out along with water through the crevices of widened joints. As the temple was neglected, this process went on unabated for centuries. After flowing out of sand, the floor beams settled and the pillars lost their support and the temple structure as a whole lost its equilibrium and the process of dilapidation started.

VII. CONCLUSION

Alike all Ancient Monuments, the Kakatiyan temples dilapidated due to age and age old neglect. With the available evidence from Nagulapadu, it can be said that, there were no inherent structural defects as such, in design and construction of Kakatiyan temples. But, the technique of sand filling in foundations and basement requires some more improvement, like use of graded and well compacted sand filled bed. It is also necessary to prevent the flowing *out of sand from basement level. Following are some of the important causes for dilapidation.*

1. *Low intensity tremors of the region.*
2. *Ground and Rain water (by way of seepage and leakage etc).*
3. *Loss of confinement of sand at basement level.*
4. *Lack of maintenance and total neglect.*
5. *Pilferage and Vandalism.*

** * **

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2. My article is based on the information obtained while dismantling the Kakatiyan temple at Nagulapadu. I am very much thankful to Director, Archaeology & Museum Department Government of Andhra Pradesh State (INDIA) for permitting me to utilize the information.

3. I express my thanks to NGRI (National Geophysical Research Institute Hyderabad) for the valuable information given by them on earth quakes in the region under consideration.

** * **

STRUCTURAL PROBLEMS OF CONSERVATION AND REINFORCING WORK OF A MONUMENTAL BUILDING IN THE OLD PART OF SALERNO (ITALY)

AMALIA SCIELZO

SUMMARY *

The illustrated works to Palazzo d'Avossa, a XVIII century building situated in one of the oldest part of Salerno, chosen by the Superintendent Architect Mario De Cunzo as permanent offices of the Superintendence of Salerno, have been made by the Superintendence of Environmental, Architectural, Artistic and Historical Properties of Salerno.

The tamperings to the original building, the consequent lack of stability in the thickness of the walls and the unbalance in conditions of loadings together with the deterioration of the mortar are the prime causes of the structural damage.

Reinforcing project: Prof. Giovanni Castellano

Supervision to the works: Arch. Amalia Scielzo

* Architect, Superintendence of Environmental, Architectural, Artistic and Historical Property of Salerno and Avellino, actually at Superintendence of Environmental and Architectural Property of Naples.

April, 1989

SALERNO, PALAZZO D'AVOSSA

Historical and critical notes

Palazzo d'Avossa, which in its present architectonic form is a building of the first half of the XVIII century, it is found in one of the oldest parts of Salerno inside the roman walls to the west dating from the third or fourth century, in the part of the town called "dei Barbuti" since the Longobards occupied and settled in Salerno. The building consists of a conglomeration of a lot of parts built in different periods.

The oldest part, which surrounds a large courtyard which can be accessed from Via Botteghelle, was originally composed of only two storeis, other than the ground floor, where the stables for the horses and coaches were kept. In front of the entrance on the west side there was the only part at the same level that ended in a large terrace. The two portals in sculptured stone belong to the first stage of construction, carried out in the mid-seventeenth century. They form the entrance to the yard and to the stables which are characterized by vault roofs supported by limestone columns.

It is thought that the building was bought by the D'Avossa family at the beginning of the XVIIIth century as previously the family lived in another building not far from this one.

From the land registry of the mid-eighteenth century one can see that in 1754 Saverio D'Avossa, a rich Salerno merchant owned this "Casa Palaziata" in Salerno, "recently built" in the "district called Santa Maria dei Barbuti (1) together with a lot of agricultural lands in Capriglia, a small village north of Salerno.

The expression "Casa Palaziata" which is often found in land registry documents of the XVIII century, indicates a building of notable dimensions used in part by the owner and his family usually the first floor and the premises on the ground floor and the rest of the building was divided into several flats which were rented out.

Following the disastrous earthquakes which struck the area in 1685-1688-1694 the original buildings was notably damaged. As opposed to a completely new building, it was more a reconstruction and an enlargement of the original building. The first and the second floor were increased in size so that they took up a complete block to the south, extending over a narrow street by means of an arch. Another narrow street, parallel to the first, was bridged in the same way with a terrace which extended the seventeenth century building to the south until it reached the corner of Via Mercanti. The terrace, decorated with marble busts, forms together with the low building on which it rests, one of the most interesting backdrops of the "Drapperia"(2). The position of the terrace was so far from the central part of the building, because it was prestigious to have a house, or even just the terrace, facing the most important commercial street of the town.

Valuable elements are the two portals in sculptured stone, the one of the main entrance and the other one of the entrance to the stable. The first portal, unlike the other seventeenth century portals of the town, does not have tympanum, but it has a round arch built on low piers. On the keystone of the arch there is the coat-of-arms of the D'Avossa family.

A corridor with a barrel vault puts in communication the elegant entrance portal with the courtyard decorated with five statues belonging to the end of the seventeenth century or the beginning of the eighteenth century. One of these much older than the other has been identified as a roman copy of an important greek masterpiece of the latter part of the fifth century b.C. Unfortunately the decorative stuccoes that ornamented windows and balconies were destroyed during previous works of replacement of the floors and of the flat arches.

The present restoration of the building is directed towards using it as permanent offices of the Superintendence of Salerno and Avellino, according to the terms of the agreement with the Provincial Administration of Salerno, that owns more than half of the usable surface area of the building.

<u>State of sites and structural damage to building</u>

Palazzo D'Avossa consists of two distinct buildings (besides the terrace that forms another building) connected by the upper floors. The area on which the building is built is of about 1330 m with a height of about 20 m. with four floors and a roof. The supporting structure is made of tufa masonry made of a conglomeration of river sediment.

The structure presented clear signs of adjustments and tamperings to the original eighteenth century building, built on previous structures (as for example "the stable").

The consequent enormous lack of stability in the thickness of the walls and the unbalance in conditions of loading, due to the deterioration of the mortar during the centuries are the prime causes of the structural damage following described:

v. Imp

- the rotation of the front part of the building in Via Botteghelle, probably due to the earthquake of 1980, and proved by the cracks in the hortogonal walls and in the floors and the lack of connection between the walls and the floors.

- many cracks on the vaults of the old stable: the lesions in the floor of the first floor show the crushing of the pillars of the ground floor. All this is confirmed by the fact that the pressure created on the walls is of

30 Kg /cm.

The wall of reinforcement already built between the arches of the ground floor, demonstrates that the damage had already been noticed in the past.

The new floors built during previous restoration, heavier than the previous wooden ones, contribute to making the situation worse.

This is one of the cases, in which the original structure, consisting of ground floor and first floor, has been raised arbitrarily later;

- the crushing of the walls, at the corner between Via Botteghelle and the vicolo Grimoaldo. (3)

The cause of all this is the abnormal division of the rooms due to the changes to the building made in the past and similar abnormalities in the two floors built later.

- the bad condition of the masonry on the ground floor, in the walls of the courtyard. These phenomena depend on the anomalous position of the empty spaces of the two stone portals and are made worse by the vertical extension works of eighteenth century;

- damage to the vaults of the main staircase.

This phenomenon is more evident in the highest part of the staircase, that is an extension built without respecting the correct rules of construction.

The foundations of the building are of a continuous type sunk two metres below street level, characterized by big offsets. They are in good condition and there is no damage to the building due to failure of the foundations (4).

Principles of intervention

The intervention in progress, made to reinforce the walls against the seismic risk, consists in linking the horizontal structures to the vertical ones. The old wooden floors, before the work started, had been replaced with iron floors and brick beams. These floors fit no more than 20 cm into the walls, even the large floors (of about 10 m.) thus offering an insufficient tie between the floors and the walls. There are even some floors placed parallel to the front, without any connection with it.

To overcome this enormous shortcoming due to the scarce depth of the connections, some couples of chains were put between parallel fronts and some flat pieces of iron have been welded at right angles to the beams with dimensions of 30x5 mm in the central parts and of 40x80 mm along the walls.

Moreover some fascias have been made with cement gun and with electrically welded iron netting (Ø 4 cm 10x10). These fascias are 110 cm high outside and inside they are divided into two parts: one above floor level the other below it. To fix these fascias the plaster has been removed and the walls have been pickaxed for about 4 cm.

These fascias linked together with iron rods Ø 6 every 50 cm anchor to the external front some iron rods Ø 16 which on the inside are welded either to the sections or to the hortogonal plates. this kind of intervention, done to all the floor, is reinforced in the case of bigger floors. In the big floors with hollow brick tiles on the iron beams was necessary to demolish a part of the floor of about 150 cm on the four sides and replace it with a frame of iron plate, fixed to the beams and to the cement slab with pegs (Ø 10). Some iron rods Ø 16 that go into the walls with a slope of about 45° are fitted into the cement slab. They are linked to the external front with cement gun. a second range of iron rods with the same slope are welded to the cement gun fascia, under the floor to the two sides of the iron-girder core. Usually used for curved surfaces (domes and vaults rooves), or for entire walls, for this intervention the planner has chosen to build linking and strengthening beams that the building did not have before.

The reinforcement of the old stable

The intervention on the crushed pillar of the stable was particularly difficult. As we have already said, the loads burdening it, due to the building of further floors, are heavier than the loads on the other pillars of the stable. For this reason, the intervention done to lighten the load of the upper vaults and their filling with light clay, the ligthening and the rebuilding of the wall of the low part of the building, all this reduced the weight on the pillars by 15%, brinking back the load to the minimum, but was not sufficient for the pillar in question. In fact the reduction of weight carried out on it scarcely affects the loads burdening it.

It was necessary to carry out a stronger intervention, that is to say, the realization of a reinforced concrete frame with three pillars incorporated into the perimeter walls and another pillar inserted into the stone pillar.

The phases of the intervention were:

1) The digging of trenches for the beams of the foundations in the lines 1-2, 1-3, 1-4. Around the pillar no digging was carried out. Building of the central parts of the beams, leaving the iron reinforcement ready to be linked with the pillar.

2) Shoring of the arches with the 80 cm thick masonry, with tufa bricks and non shrinking mortar. Some wooden wedges have been fitted to the arches to link them to the shoring.

3) At the same time as the building of the wall (direction 2-4) the top of the pillar has been cut into the impost of the arches to fit into it two sections HE 180 linked together in the hortogonal direction, with T beams 120x120, welded to the base.

4) Shoring with wooden props of the vaults of the ground floor.

5) The cutting away of the basament walls and of the walls corresponding to the base of the pillars. The cut was made outside for the pillars 2-3 and inside for the pillar 4 because there was a vertical wall outside which prevented external works, the cut inside was divided into two parts: one from the ground floor to the arch and another on the second floor.

6) Finishing of the basement beams corresponding with the external pillars, leaving the iron beams "ready" to accept the linking with the pillar core.

7) Pillar positioning and their welding to the foundation pillars.

8) Before the casting, the reinforced concrete structure has been isolated from the walls by using a sheath, so as to avoid reciprocal actions between the two structures.

9) On the second floor, after having closed all the existing spaces into the arches to be worked on, two flat arches have been built at the same height of the walls burdening the central pillar.

10) The demolition of the crossing walls, for about 150 cm above the pillar.

11) The dismantling of the drums of the pillar and their repositioning on a horizontal platform previously erected in the yard of the building. The central part of the pillar has been bored with a drill Ø 40 (36 holes).

12) After dismantling the central part of the pillar between the converging arches was bored and after was emptyed.

13) The completion of the central part of the foundations linking the iron beams with the ones previously prepared in the beams already made.

14) Before the casting of cement and after the welding of the structure, a zinc sheet has been inserted between the reinforced concrete pillar and the wall, to separate the two structures.

15) The making of the beams of the second floor between the four pillars. Every beam, composed of two smaller beams placed side by side along the sides of the walls, measuring 40x40, has transversal links through reinforced borings with Ø 18 iron rods.

16) When work had been completed the parts of the pillar that were detached, and before the intervention were linked with iron cramps, were reconnected using epoxy-resin.

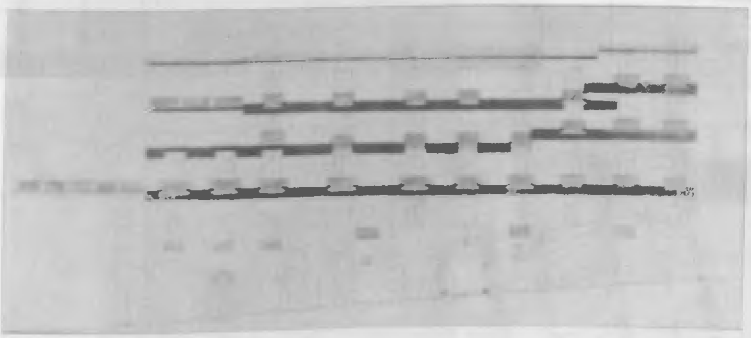
17) 30 days after the completion date of the restoration work, all the shorings and the iron supporting beams were removed. The wall which had been cut away was rebuilt and all the finishing touches were carried out. During the restoration work, cracks in the arches and in the vaulted rooves were checked by having pieces of glass inside them. No measurable movement was noticed in the building. The interventions described, together with other works such as the repositioning of the empty spaces corresponding to the alterated prospects removing the causes of the

anomalous concentration of the weights, together with the reinforcement of the masonry, form the base of the static restoration, indipendently from the damage due to the earthquake. The further work, necessary for the reinforcement of the vertical structures are concentrated on the crosses and the angle-walls to the chaining of the masonry and in the rebuilding of the jutting wooden roof.

1

1- The main front of the building facing Via Botteghelle before its restoration.

2- Project of the reinforcing fascias and the reinforced iron seams as seen from Via Botteghelle.



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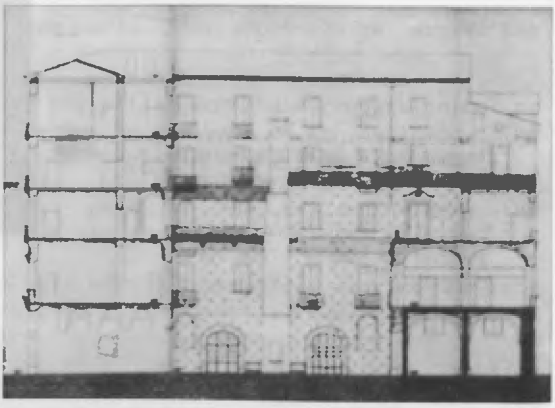
Footnotes

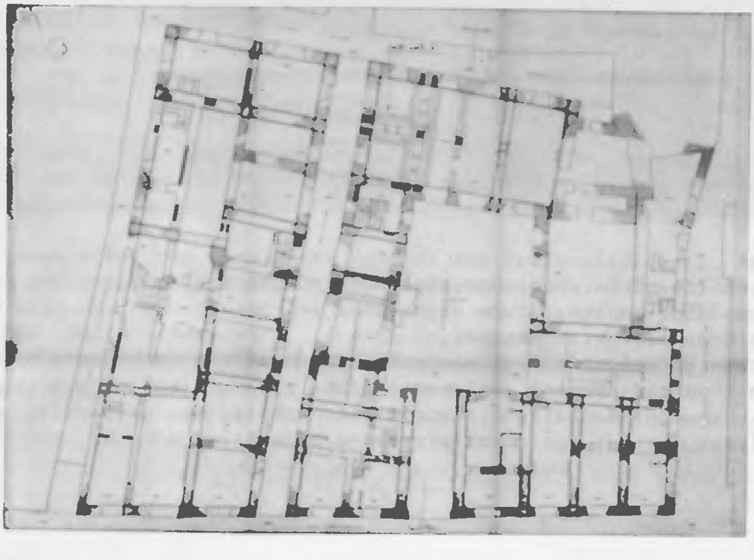
1) Land office of Salerno, Apprezzo Vol. n.3946 f.499 n.4; Vol. n.3953 (Naples, State archives).
2) The old name of Via Mercanti.
3) A narrow Street.
4) Taken from the report accompanying the reinforcing project prepared by Prof. Giovanni Castellano, Faculty of Architecture University of Naples (Italy).

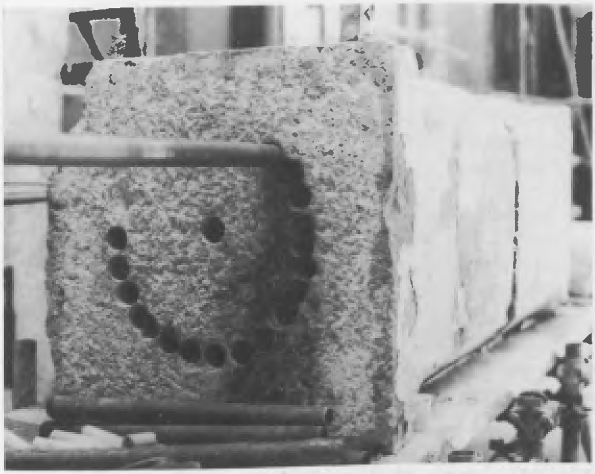
3- Operation of reinforcement with cement gun. Also for thickness of 4-5 cm, the cement gun gives a strong resistence thanks to the force with which it is fired onto the surface to be covered.

4- Project of the reinforced fascias, the reinforced iron seams and demolitions on the main parts of the building. In the bottom right hand corner can be seen the stable which has had reinforcement work carried out on it.

5- Plan of the ground floor with future plans shown.







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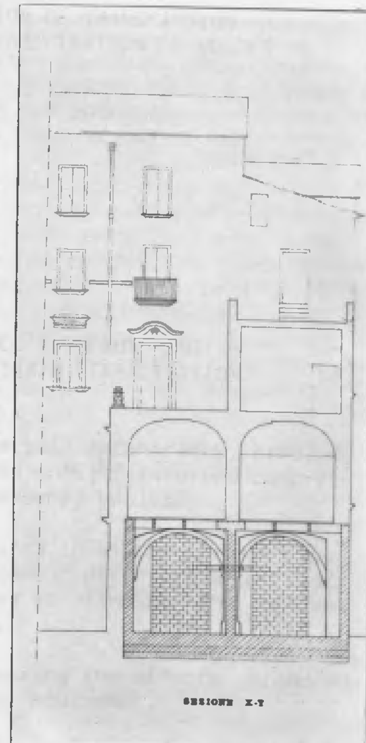

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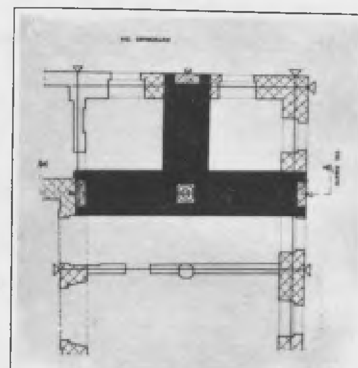

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6- The central part of the pillar bored with a drill Ø 40 mm with 36 holes.

7- Remounting of the pillar and iron rods of its scaffolding.

8- The old stable and the work carried out to lighten the crushed pillar.

9- Plan of the reinforced concrete spacial frame, with three pillars inserted into the walls and a fourth one inserted into the stone pillar.

8 the building. In the bottom right hand corner can be seen the

9

METHODES ET EQUIPEMENTS DE REGULATION DE TRANSMISSION DES CONTRAINTES A INSTALLER POUR LA STABILISATION DE VOUTES

Jean-Louis TAUPIN

VAULTS STABILIZATION : PRINCIPLE AND EQUIPMENT AIMING AT MANAGING A REGULATED STRAIN STABILIZATION

Researches worked out to cope with material incoherency problems that arouse when inserting, for consolidation ends, reinforced concrete rigid structures into complex structures of masonry vaults.

Researches for the Saint-André-le-Bas church in Vienne (France) were conducted on the principle that the reinforcement structure would react in a flexible and adjustable way, and aimed at finding the optimal location for applying corrective constraints.

Putting up such an installation and measuring the effects produced on masonry, require an appropriate measurement equipment.

Description of a vault consolidation equipment endowed with an adjustable constraints generator and flexible transmitters and a measurement outfit (under experimentation now).

Advices to define a logical system adapted to works site conditions.

Architecte en chef des Monuments Historiques

13 Juin 1989

Introduction

Conception d'un équipement de stabilisation de Voûte doté d'un générateur de contrainte réglable, de transmetteurs souples et d'un appareillage de mesures. Recommandations d'optimisation du système logique et instrumental (complément d'équipement de mesures).

Description

Contrôler avec précision en temps réel les forces effectivement mises en jeu lors de la confortation d'une voûte serait très utile pour connaître mieux les effets de cette opération et améliorer la performance des actions de ce genre entreprises sur les monuments.

L'enjeu est d'autant plus intéressant que la sensibilité moderne souhaite en abordant le Patrimoine s'écarter des actions radicales, privilégier des interventions minimales. La démarche suggérée, de viser seulement ce qu'on pourrait appeler un simple retour à la marge interne du noyau d'équilibre, contribuerait à donner des instruments aux options théoriques de la Charte de Venise.

L'opération, mise à l'étude en 1983, mise en chantier en 1986 et aujourd'hui réalisée sur 2 doubleaux de voûtes de l'église St-André-le-Bas de Vienne, nous fournira en termes réalistes mais généraux, une vue sur la nature du problème posé.

Le cas traité est celui d'une nef sans bas-côtés de 25 mètres de longueur par 12,40 mètres de largeur intérieure, recouverte au XIIe siècle par 3 travées de voûtes pourvue d'arcs diagonaux sur une silhouette fortement domicale, séparées par 2 doubleaux, et dont les arcs-boutants ont été créés ou aménagés postérieurement. Ce voûtement, a été établi de façon très audacieuse sur les murs d'un édifice charpenté pré-roman, renforcés par des piles et des arcades intérieures.

L'ouvrage est parcouru par un réseau de fissures dont certaines sont grossièrement parallèles aux murs gouttereaux à 1,80 m en moyenne de ceux-ci, et d'autres orientées sur une direction NE/SW.

L'analyse des désordres de St-André-le-Bas et une appréciation réservée de l'efficacité de ses arcs-boutants modifiés jadis, a conduit à l'objectif, limité, de ramener sur un cheminement plus rassurant la ligne des pressions que le calcul montre au niveau du sol, déjà dangereusement décalée jusqu'au parement extérieur des murs gouttereaux.

L'Architecte en chef des Monuments Historiques a choisi d'intervenir en partie haute au niveau de la formation des poussées des voûtes, plutôt que d'essayer de renforcer la fondation des organes de contre-butée logée dans un terrain difficile à connaître. Cela permet de ne pas menacer par des injections de coulis de mortier à l'aveugle, ou même par des forages de micro-pieux, le gisement archéologique dense et inconnu qui forme l'environnement immédiat de l'édifice.

L'étude développée en collaboration avec l'Ingénieur Michel Bancon a recherché la possibilité d'ajuster le plus finement possible la puissance du confortement à la dérive pathologique à traiter. Ce pilotage est appuyé sur un compromis stratégique nourri de considérations diverses : disponibilités financières, possibilité ou non d'une action et d'un suivi à long terme, degré de certitude, attendus doctrinaux, etc.

Un projet célèbre de consolidation de voûtes gothiques a été conçu et réalisé par l'Inspecteur Général Architecte Jean-Pierre PAQUET dans les années 1950 à l'église de Saint-Leu-d'Esserent (nef encadrée de bas-côtés), en application d'un modèle d'analyse théorique des conditions d'équilibre des structures de ce type. On peut s'inquiéter des dispositions de certaines variétés de pinces de béton, rejetons plus ou moins fidèles de ce prototype très novateur, qui, coulées généreusement mais trop haut, ne sont que des super-structures greffées sur les voûtes dans de mauvaises conditions.

On se donnera les principes suivants :

Principe de limitation de rigidité

Théoriquement : écarter la notion d'un "blocage des mouvements de la voûte", en faveur d'une notion de "gestion d'une ligne des poussées" prise comme une entité complexe non figée.

Concrètement : ne pas admettre à l'interface entre matériaux de nature et de comportement aussi différents que le sont le béton (ou l'acier) et les maçonneries, des connections autres que des appuis ou tractions simples.

Principe de quasi-réversibilité

Limiter les apports de masse, dans les parties hautes de l'édifice (dérangement des descentes de charges, risque sismique, quasi impossibilité de démontage).

Se défendre de l'ambition d'apporter un traitement immuable et définitif. Sans répondre totalement au voeu angélique de n'opérer sur le patrimoine que par actions parfaitement réversibles, on apporte une suggestion dans un champ d'activité qui est bien loin d'être effectivement converti à la réversibilité, en se donnant la possibilité de réglages. On suggère ensuite, bien au-delà, la possibilité d'un équilibrage permanent selon un concept idéal de "pince active" (dans cette perspective se présenterait un problème de cohérence entre ces formules techniques et les modalités de l'entretien).

Ces principes sont interprétés dans un tableau de spécifications :

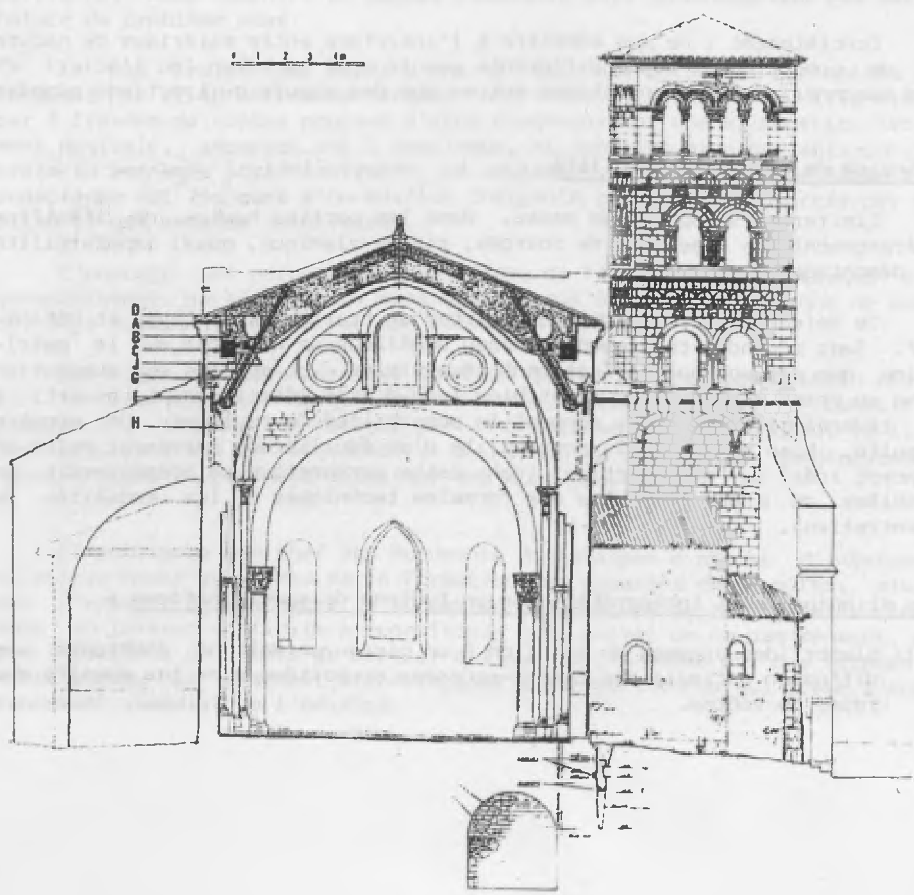
1/ placer des organes de réaction à un niveau permettant d'obtenir une diffusion optimale des contre-poussées souhaitées dans les massifs des reins de voûtes.

2/ interposer dans les organes de contre-poussée des éléments élastiques chargés d'assurer une régulation spontanée des efforts, quelles que soient les variations de comportement des matériaux. On pense ainsi diminuer sérieusement les risques inhérents à l'insertion de structures "rigides" (béton armé) dans des structures "friables" (maçonneries),

3/ disposer d'un appareillage permettant, après une discussion et un calcul d'opportunité, de régler à l'intensité assignée, la force de contre-poussée qui doit affronter la composante horizontale des poussées de voûte,

4/ conserver la faculté d'accentuer ou d'atténuer la force de contre-poussée, en fonction de l'évolution de la situation durant l'opération elle-même ou ultérieurement, selon les évènements pouvant survenir dans l'ensemble du système (fluage, action sur une travée voisine),

5/ l'édifice ici est couvert d'une toiture à faible pente : les fermes de béton armé seront l'élément rigide de la confortation des voûtes et auront également à porter la couverture qui reposait précédemment sur des arbalétriers et des pannes et assez souvent sur la voûte elle-même!



Eléments de la structure mise en place :

* Une poutre-ferme en béton armé, portant de gouttereau à gouttereau,
(A) remplit toute la hauteur disponible sous la couverture, traverse l'édifice dans toute sa largeur en restant séparée de l'extrados de voûte par un large joint.

* Des patins en profilés et leurs massifs de béton armé sont fondés sur
(B) la crête des murs gouttereaux (complètement indépendants des quartiers
(C) transversaux des voûtes), et servent de supports flexibles à la poutre-ferme dans le sens transversal.

* Des boucliers de béton armé, d'une surface de l'ordre de 1 mètre
(E) carré, insérés verticalement de part et d'autre des têtes d'arcs-boutants sous les parements des murs gouttereaux, à peu près au niveau de séparation des 2 tiers inférieurs des arcs doubleaux. Cela est relativement facile à réaliser ici puisque ces parements sont constitués de panneaux de pierre d'une épaisseur moyenne de 20 cm dissimulant un blocage de gros cailloux. La contre-poussée se diffuse sur les massifs des reins de voûtes à partir de ces 4 plateaux soumis à une pression de l'ordre de 1 kg/cm2.

* Des leviers (profilés métalliques longs de 5 mètres baptisés "avirons"
(G) par analogie avec l'effort du rameur) plongent dans l'épaisseur des voûtes (complètement indépendants de celles-ci) jusqu'à un point proche du centre de ces boucliers, qu'ils rejoignent par un court tirant
(H) horizontal. Ces leviers sont calés en leur milieu et à leur extrémité supérieure sur des profilés traversant la ferme de béton armé.

(G) * Des fourreaux de tôle délimitent un vide permanent autour des leviers, Extérieurement la continuité est rétablie entre fourreau et blocage des voûtes par un coulis de mortier de faible résistance.

(J) * Un vérin est placé à mi-hauteur de chacun des leviers pour installer la contre-poussée dans le système. On utilise des vérins plats mis en charge progressivement à un niveau de pression ou à un niveau de force fixé d'avance.

* Des vis de blocage sont destinées à relayer les vérins par une butée permanente. On élimine ainsi la relaxation du système hydraulique sans avoir à geler irrémédiablement celui-ci par injection de résine. Les vérins restent prêts à être réactivés si l'opportunité s'en fait sentir.

Appareillage de mesures

Il est important que l'opération de mise en contrainte puisse être conduite dans une connaissance très fine des effets subis par le monument lui-même, en termes de contrainte reçue et en terme de réactions de déformation propre.

(a) Des informations sur la contrainte appliquée peuvent être attendues soit de la mesure de pression dans le réseau hydraulique des vérins, soit de la mesure directe des forces dans la charpente de confortement,

(b) Des informations sur les réactions de déformation peuvent être fournies par un appareillage apte à détecter et mesurer de très fines variations de longueur en parement (à la base des murs), ou d'écartement (entre murs gouttereaux opposés ou entre sol et voûte).

La première source d'informations (a) devrait permettre d'évaluer le résultat obtenu dans l'action extérieurement appliquée au monument, la consistance de l'action au sein de l'édifice (l'essentiel !) pouvant dans une certaine mesure être supputée à partir de là, à travers un modèle théorique de comportement des murailles sous les contraintes.

La seconde source d'informations (b) devrait ouvrir une possibilité de jauger l'évolution réelle de la courbe des pressions notamment à la base des murailles, durant et après la mise en contrainte, donc de connaître plus directement la véritable efficacité de l'opération entreprise.

L'opération de confortement a été divisée en 2 étapes (1986/87 : doubleau intermédiaire Est, 1988 : doubleau intermédiaire Ouest). Ces étapes se distinguent par des approches différentes de certains problèmes, problème du forage dans les voûtes et problème des mesures notamment.

La préparation de la seconde phase s'est vue assignée de suivre les conclusions catégoriques issues de la 1ère phase :

- d'abord disposer d'une méthode de mesure directe et instantanée, mobilisant des capteurs de force en compression LOGéS EN SéRIE et de façon définitive dans la chaîne de contrainte,
- disposer de capteurs que leurs dimensions et leur configuration physique permettraient d'intégrer durablement et sans nuire à l'efficacité de cette structure de confortement,
- disposer de capteurs assurés par leur constitution d'une durabilité fonctionnelle certaine en tant qu'instruments de mesure consultables à volonté.

l'Entreprise a reconnu parmi plusieurs propositions possibles les capteurs de force fabriqués par la firme SCAIME d'Annemasse, couramment utilisés en pesage industriel. Le principe de ces appareils consiste à mesurer avec une très grande précision la variation de résistance d'un circuit électrique adhérent aux parois d'un bloc d'acier déformé de manière élastique sous l'effet de la contrainte mécanique subie.

Les données recueillies par ces appareils sont présentées instantanément par des lecteurs à diodes lumineuses. Depuis le 14 Décembre 1988 des mesures de suivi des contraintes sont faites régulièrement avec une extrême facilité grâce à cet appareillage.

Courbes de mesures

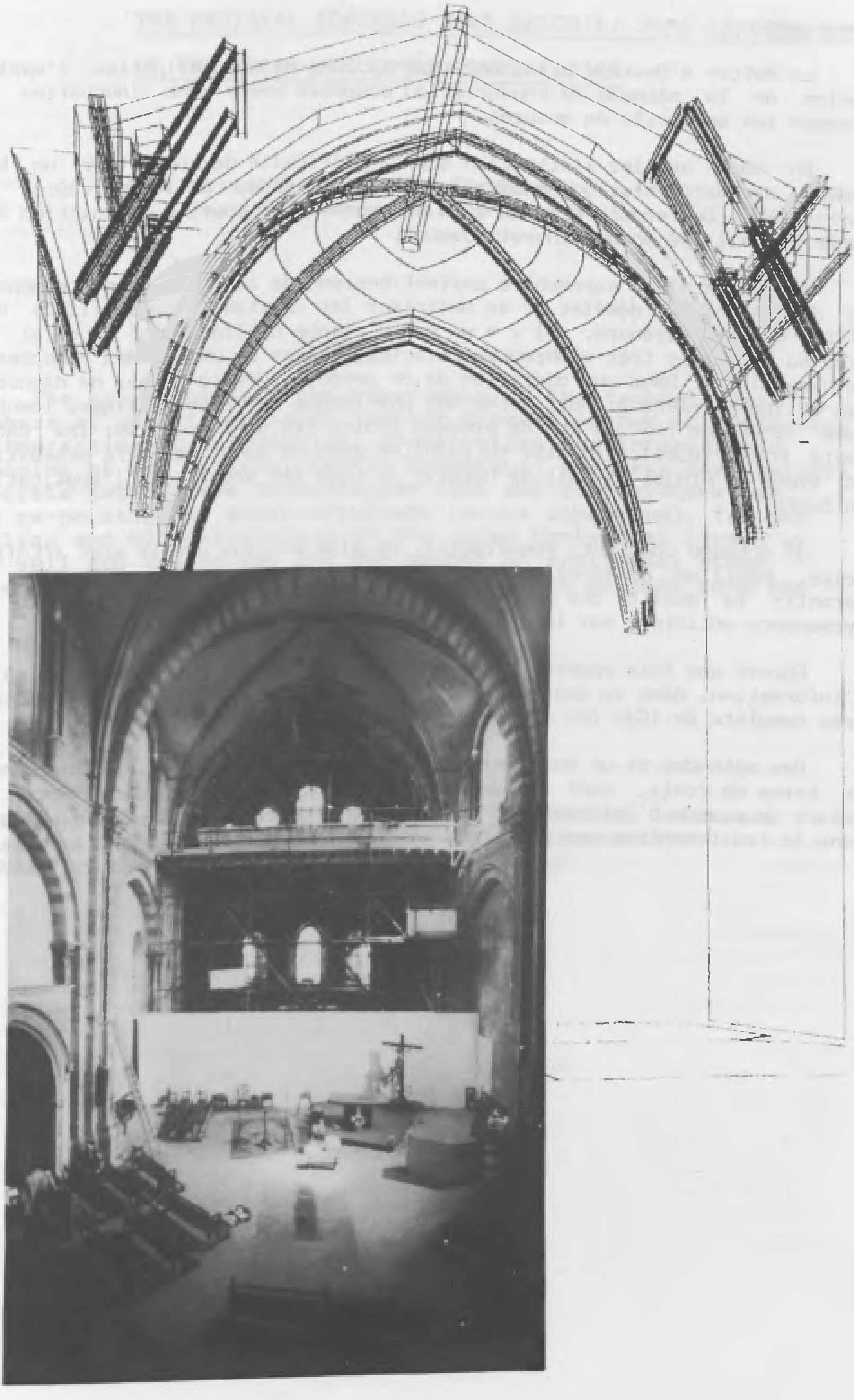
Une finesse accrue dans l'observation fait apparaître des faits et des questions ordinairement insoupçonnables. Les courbes de suivi sur 3 mois montrent une atténuation des contraintes, dont l'amplitude en fait est de peu d'importance au regard de ce que serait pour l'édifice une modification au sein d'un système rigide en "tout ou rien".

Il s'agit probablement, pour une partie, dans une phase de "rodage", d'un réarrangement de la structure fine des maçonneries par tassement de micro-éléments "friables" dans les régions de ces complexes hétérogènes les plus sollicitées par les contraintes créées. On ne peut à l'heure présente affirmer qu'elle ne tend pas à rejoindre une asymptote horizontale.

Deux facteurs au moins peuvent intervenir :

- une incidence thermique agissant sur la structure de confortation.
- une évolution de l'ensemble arcs-boutants/culées vers un nouvel état d'équilibre.

Le réchauffement ambiant joue sur la chute des forces en accroissant l'écartement entre plaques à cause d'une disparité de valeur des coefficients de dilatation (béton+acier d'une part, maçonnerie d'autre part) et surtout, d'un processus d'échauffement (ou de refroidissement) retardé par effet de volant thermique dans la maçonnerie par rapport au béton+acier.



Remarques

Le Maître d'Ouvrage a soutenu tout au long de ses péripéties l'application de la méthode de confortation proposée comme une innovation et financé les appareils de mesures.

On doit appeler l'attention sur la difficulté de constituer un bon modèle de représentation géométrique des structures, celle des voûtes en particulier. Un recours précoce à un Photogrammètre pour la description des voûtes aurait été un bon investissement.

Au risque d'une surenchère perfectionnisme de la conception, s'oppose la difficulté de démêler et de maîtriser les véritables possibilités des offres des fournisseurs. Il y a un grand risque d'allongement du délai de mise au point des très nombreuses caractéristiques de chacun des composants du dispositif. Dans une opération de ce genre, inédite et qui ne débouche pas obligatoirement à court terme sur une longue série de répliques identiques (c'est en fait le cas de presque toutes les opérations sur les bâtiments historiques), la mise au point ne peut en aucun cas être exhaustive au stade du projet et doit se nourrir de tous les acquis de l'application en cours.

Un échange constant, constructif, de plus en plus précis avec l'Entreprise, mobilise celle-ci et renforce le sentiment de son obligation de garantir la qualité des prestations. L'Entreprise performante doit en permanence anticiper sur la mise au point des détails d'exécution.

Encore une fois apparaît essentiel un progrès dans la communication de l'information, dans sa qualité et sa rapidité, pour assurer une information très complète de TOUS les membres de l'Entreprise.

Des méthodes et un matériel adaptés à la situation concrète, notamment en terme de coûts, sont indispensables. Faute de quoi, condamnées à se priver de source d'informations de qualité, toutes initiatives de progrès dans le traitement de nos problèmes seront condamnées dans l'oeuf.

THE MEDIEVAL FORTRESS NEAR BALCHIK: SOME ASPECTS OF THE STONE-STRUCTURE CONSERVATION

By: Valentin Nedelchev Todorov *
Ioanida Ivanova Nikolova **

SUMMARY

The paper outlines the experience of the technologists members of an International Complex Expedition for investigation photographing, documentation, conservation, restoration and exponing of the medieval (early Byzantine) fortress near Balchik. Concrete data of the technologies used and compositions for the re-pointing of stone-structure (acnus implectum), for the sealing and hydro-isolation of the upper horizontal layers of the wall and towers and for the making of artificial stone, necessary for the partial restoration of the western gate and its adjoining towers, are outlined.

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** Head of Stone-structure Restoration Group

April 5, 1989

1. INTRODUCTION

The medieval (early Byzantine) fortress near Balchik was built in the V century A.D. and has since undergone a number of reconstructions. Soon after the foundation of the Bulgarian state in 681 it was seized by the Bulgarians and its fate ever since has been closely connected with the different events taking place in Bulgarian history. From the XV century on its remains (mainly of stone) were systematically plundered for the purpose of constructing houses and public buildings in the ever-growing town of Balchik. Nowadays only the substructure of the fortress remains (Dimitrov, 1986).

From 1981 to 1985 an International Complex Expedition comprising specialists from different socialist countries was organized to carry out excavation works on site. It was made up of archaeologists, architects, photogrammetrists, chemists-restorers. Archaeological investigations, architectural design and conservation-restoration works were carried out simultaneously. The general aspects of the activities of the different groups of specialists were reported at an International Conference held in 1985 at the Black Sea resort of Albena and were published in the "Museums and Monuments of Culture" magazine (Palichev, Ovcharov, Pyrovska, Popov, Todorov, 1988). This paper will attempt to outline some aspects of the activities carried out by the chemists-restorers group.

2. PRESENT-DAY STATE OF THE FORTRESS WALLS

The unearthed parts of the western wall are 3 m thick. It is built according to the "Opus implectum" technology, characteristic for the early Byzantine era. The wall is protected by numerous towers of different shapes - circular, rectangular, pentagonal, triangular, most of which are situated along the more vulnarable to attacks western wall and least in number are those erected at the eastern wall, which overlooks a steep slope. The main gate to the city, flanked by two pentagonal towers, was built at the western wall.

Nowadays only the substructure of the fortress remains. Archaeological excavations have been carried out since 1970. The unearthed parts are subject to the destructive effects of atmospheric factors due to a disruption in the stationary temperature-humidity regime and conditions of conservation in the soil. The mortar solution of the implectum and the joints of the front part of the wall were in a deplorable state. It was possible to preserve only a small part of the unearthed original joints by treatment with lime water (70 times). Since the results were unsatisfactory treatment with a 12% solution of Wackersilicon 290L was carried out.

The technical state, degree of destruction and the reasons for the destructive processes of all parts of the wall were investigated and analysed. Normal photographic and stereogrammetric documentation were carried out. Samples of original mortar solutions and joints were chemically and mineralogographically analysed in laboratory conditions for the purpose of studying the construction history and periodization of the fortress.

On the basis of the results from the technological investigations, the following main technological problems were formulated:

- preparation of optimal compositions for re-pointing of the unearthed parts of the wall dating back to the Byzantine and Bulgarian periods;
- sealing of the original implectum and making a protective-decorative shield of the upper horizontal layers of the wall, around the western gate and the adjoining towers;
- protection from atmospheric waters - hydro-isolation and leading away of waters. Chemical protection of surfaces - hydrophobization;
- fighting off destructive activities of biological factors.

All these questions were put to broad discussion and practical steps were outlined. Taking in mind the existing degree of destruction, unstable construction and complex construction periodization, the following principles of conservation were outlined: an in-depth strengthening of original binding solutions, marking of the new intervention, minimal conservational super-structure, arhitectural interpretation free of hypotheses.

3. RE-POINTING

Imitative, close to the original in structure and colour mortar solutions of high-quality were used in re-pointing the walls.

All formulas of compositions for re-pointing, sealing of the implectum, hydrophobic putty of the gate and for artificial stone were worked out on the basis of mineral binding agents - lime and white cement. The use of polimeric binding agents, epoxic resin in particular, was categorically discarded, since they are not durable in the open air. The choice of fillerswas payed special attention to. The desired aesthetic effect and the regulation of the strength qualities of the restoration works were derived through them.

After careful experimental work in laboratory conditions and on site, it was decided re-pointing to be carried out with:

lime putty - 1 vol. part
limocaseinic glue - 0,5 vol. part
white cement M 350 - 0,5 vol. part
grinded brick (2-5 mm fraction) - 2 vol. parts
bank sand (yellow) - 4 vol. parts

3.1. Preparing the Surface Layer for Re-pointing

The old eroded joints were cut-out with a pint chisel and cleaned by means of a wire brush until reaching a stable layer. Dusting was carried out by means of a soft brush and simultaneous flushing with water. Deep joints were filled twice. The first coat was layed 1 or 2 cm below the front part of the stone surface and several days later the final coat was layed. This prevented cracking. Special attention was payed to pasting the solution on the stones. Five or six hours after re-pointing the surface was flushed with water. Thus unpleasant shine was removed and the separate components of the mixture surfaced.

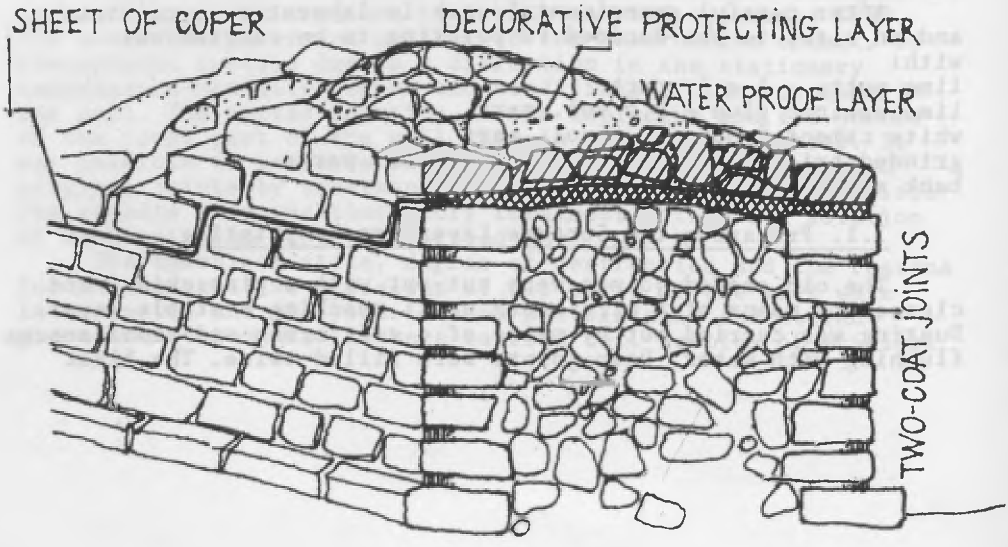
3.2. Preparation of the Limocaseinic Glue

To 1 vol. part of previously swollen in water casein (24h), 3 parts of lime putty are added and stired for 20 min. To increase the solubility of casein several drops of a 30% spirit of hartshorn are added. This thin sticky mess is then sieved through several layers of gauze and the unsoluble casein particles are thus removed. A 1:1 ratio of polivinilacetate polimeric dispersion Mowilith DM1H is immediately added to the sieved solution. This glue is added to the previously prepared dry mixture of the remaining components until the necessary plasticity to acquire workability is reached. The solution thus prepared must be used within 3 to 4 hours, otherwise setting will begin.

4. PROTECTIVE DECORATIVE SEALING OF THE SURFACE LAYER OF THE IMPLECTUM

Initially it was proposed that the uppermost parts of the wall be sealed by coating an isolation layer of clay and epoxic resin to which stone slabs be glued. It was, however, decided to stick to the original way as much as possible. For the purpose of sealing of white cement and sand (1:2), 5 to 6 cm thick was layed on the surface of the wall, upon which stone slabs, similar in form and dimensions to those of the implectum, were fixed. Only part of the surface was coated with solution. Small stone fractures were scattered on top thus imitating the semi-ruined implectum. The most important procedure is the pasting of the stones with solution and of keeping slants for draining water.

A graphic representation is given on Fig. 1.



5. WORKING OUT A TECHNOLOGY FOR ARTIFICIAL STONE

The architectural restoration design envisaged anastylosis and partial reconstruction of the western gate and its adjoining towers. After it became evident that quarry stone for the conservational super-structure would not be provided, the technologists were faced with the task to work out a technology for artificial stone. Exponing to atmospheric conditions necessitated the use of unlimited binders - limestone, white cement and differing in fractional make-up fillers.

Bearing in mind these limitations a technology was worked out and tests for establishing the indices of the stone were made at the Central Research Laboratory with the Institute for Monuments. The following mixture was chosen to be put into practice:

White cement 350 - 1 vol. part
Mowilith DM1H - 0,1 vol. part
Marble dust 2mm - 1 vol. part
Marble fraction 2-5 mm - 1 vol. part
Ground limestone (fraction 15-40 mm) - 1 vol. part
Water to acquire necessary workability

Comparitive testing of samples of real and artificial stone showed similar physico-mechanical indices. Data shown on Table 1.

Table 1

Indices	real stone	artificial stone
Tensile strength MPa	3,05	2,21
Bending strength MPa	4,70	5,19
Compressive strength MPa	28,30	24,85
Volume g/cm	2,200	2,104
Water absorbtion, %	7,90	7,03
Content of water soluble salts, %	traces	0,12

Testing was done 4 months after the samples were worked out. Samples for tensile strength - figures of eight; for bending strength - prisms with measurements 40x40x160 mm (distance between supports 100 mm); for compressive strength - cubes (40x40 mm).

A separate wooden casing was made for each stone, where the previously prepared mixture was cast and vibrated. After 5-6 days, the time it takes to acquire the necessary mechanical strength, the casts were freed of the casings. The final form of the stone and structure of its surface were reached by mechanical treatment with jumper and scarp. Thus artificial threshold stone of the gate, corner stones (70x40x50 cm) and many missing parts of the stone blocks of the northern tower adjoining the gate were worked out. The colour of the artificial stone is a shade whiter than the original, but artificial patining was consciously avoided. A year later this difference in shades was almost unnoticable.

6. HYDROPHOBIZATION

All parts of the wall, the towers and western gate which were restored (re-pointed and with hydro-isolating seal of the implectum) were hydrophobized with the purpose of increasing their resistence mainly to water entering and the destructive processes following therefrom especially in winter time.

Hydrophobization was carried out with Wackersilicon 290L, diluted with mineral turpentine at a ratio of 1:14. The effects from the hydrophobization could be seen from the photos.

Photo No. 1 - Western gate - mechanical treatment of artificial stone

Photo No. 2 - Hydrophobically and non-hydrophobically treated part of the wall after rainfall

7. ACKNOWLEDGEMENTS

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7. ACKNOWLEDGEMENTS

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Photo No. 1 - Western gate - mechanical treatment of artificial

[illegible]ophobically and non-hydrophobically treated part of the wall after rainfall

METHODS OF CONSOLIDATION IN THE RESTORATION OF MONUMENTS

GIUSEPPE TOSTI

SUMMARY

The operations presented here illustrate techniques of consolidation designed to bring the geometric configurations of the structures back to their initial condition in order to restore, albeit with certain necessary technological insertions, their original static conception.

In the Church of S. Tommaso in Terni the verticalness of a wall which had rotated several centimeters was restored; in the case of the "Odeo Cornaro" in Padua, once the geometrical lay of the structure was restored, it was proposed to transfer the stresses to a metal structure inserted inside the masonry.

In the Monastery of San Pietro in Perugia, the technique making it possible to raise a 25 x 12b meter vault more than 40 cm. was illustrated.

- ENGINEER - President of the "Sisto Mastrodicasa" Foundation
- The Architect Michele Bilancia contributed to the text

Perugia, April 1989

CHURCH OF SAN TOMMASO - TERNI

The restoration of the Church of San Tommaso was designed essentially to save the important Early Medieval frescoes endangered by the collapse of the roof vault and in particular by the sharp rotations (63 cm.) of the inner right wall of the single nave.

Fig. 1, which shows the church roof well before it collapsed, clearly indicates a state of unbalance which should have served as a warning immediately to an attentive technician, since the parabolic lesion visible in the photo extended from capital to capital of the two visible pillaster strips. At the same time the separation at the level of the arch in the wall between the pillaster strips, and also the horizontal cracks in this same wall, clearly indicate the state of rotation in progress.

At this point it would have been possible to prevent the collapse by completing some simple works of consolidation. It was possible for the entire wall to rotate at the time of the collapse because the horizontal thrust of the vaulted structure was unexpectedly increased.

On the outside, the rotated part showed marked deflections, while the inside, in correspondence with these deflections, there were sharp cracks and breaking up of the masonry (Fig. 2).

At this point, the problem was to decide on the future of the wall, the frescoes, and the church. The courses that could be followed were:

1) Detach the frescoes, losing the wall and the church.
2) Detach the frescoes, reconstruct a false wall to replase the one destined to collapse and the reattach the frescoes to the wall once the roof was rebuilt.
3) Save the wall, the frescoes, and the church at the same time.

This last course was selected and while certainly not the easiest, it was best from the point of view of a correct restoration

It was a matter of recovering the verticalness of the rotated wall by creating, so to speak, an unbalance equal and contrary to the unbalance the wall had undergone, taking along with it the frescoes.

The most obvious difficulties in an operation of this type were represented by the fact that one had to work on a wall that was already strongly compromised in stability, and, relying on the elastic hysteresis of the whole, invent a temporary structure capable of distributing all the thrusts absolutely uniformly, which would be transmitted by micrometric movements diluted in time.

Naturally, before beginning this straightening operation, a new, extremely rigid rotating fulcrum has to be created at the base of the wall for its entire length (see "Restauro e cemento in architettura" AITEC, vol.II, page 147).

Consequentely, a dual series of telescopic props was set up (Fig.3) placed in pairs inside and outside the wall in specular fashion; these props could rotate at the base in order to back up the subsequent movement of rotation necessary to make the wall return to its original position.

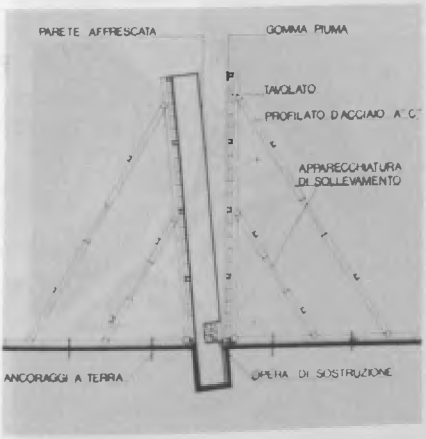
A wooden supporting scaffold was then fastened to both series of props.

This set-up, from the inner part of the church, kept perfectly vertical and appropriately covered with foam rubber to protect the frescoes, had the task of waiting for the wall, once the planned rotation was completed.

At this point the resulting strainghtened wall, compressed between the two scaffolds and therefore subjected to a vertical down-to-up thrust, only needed

F.1

F.2



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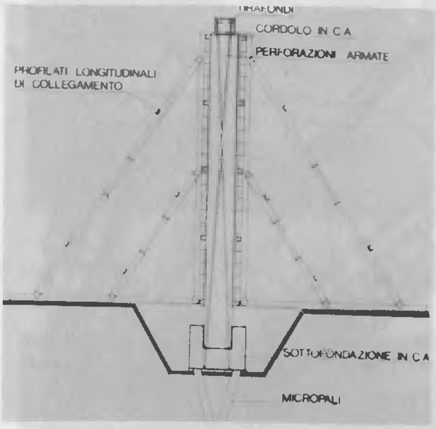


F.4

a new base (Fig. 4).

The base was made by installing micropiles that were slanted and intersecting on the barycentric axis of the wall. This widened the surface on which the walling was laid and provided adequate basal rigidity.

The connection of the head of the micropiles immediately below the church floor permitted creating a longitudinal structure of rigid concrete on which to anchor the bracing provided for in the project.

This bracing, made by inserting strands inside holes drilled slantwise over the entire height of the wall so that the bracing too would intersect along the barycentric axis of the wall, was, after being stretched, anchored to a curbing recessed with respect to the wall so it would be invisible once the roof-frame was rebuilt on it.

THE "ODEO CORNARO" OF PADUA

The building, a 16th-century work by Falconetto (Fig.1), which was built as a symbol of the city of Padua, is a rare example of the synthesis of Renaissance architecture and rich, refined decoration, both on the front facade as well as on the intrados of the skiff vault, which dominates the entire inside of the colonnade and where the encaustics are particularly precious (Fig. 2).

This operation, though planned as the previous two in order to restore and preserve the historical and formal identity of the monument, differs because of the particular function of the building, designed as an odeum and therefore conceived substantially as a stage for theatrical performances, and demands it should only be restored in order to serve its original purpose.

The form of restoration, in this case, was only concerned with freeing the building of serious problems of instability which could be corrected by consolidation designed to eliminate the aggravating causes of degradation, while keeping unaltered the original structural logic and even the evidence of unbalance since it was caused substantially by an error of design in the original project.

The condition of the monument, as stated previously, is currently highly endangered since, because of the particular geometrical conformation of the installation, there are no internal connecting walls nor continuous basal wall and all the orientations are thrusting (Figs. 3-4-5).


F.1


F.2


F.3


F.4

In particular the skiff vault of the first level is badly deformed along the longitudinal faces of the building in correspondence with the reins (Fig. 6). These deformations are discordant, for the vault is squeezd along the back side on the intrados while along the front it is considerably depressed.

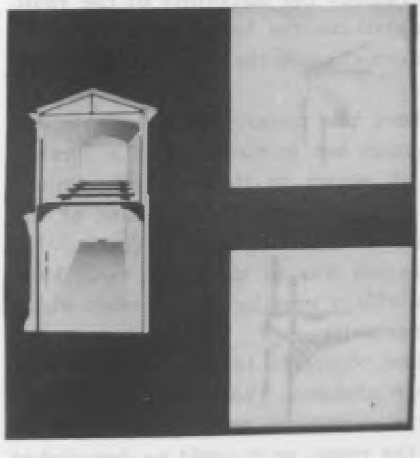
An original system of chains, under the action of horizontal thrusts, which because of the geometrical conformation of said chains, has set off in turn a moment which has produced further rotation of the entire front facade (Fig.6). This facade is certainly the most vulnerable because it consists of a colonnade, it too degraded by the weather which easily attacks the sandstone material of the structures.

Basically, the unbalances found in the building and their relative causes may be attributed to the following:

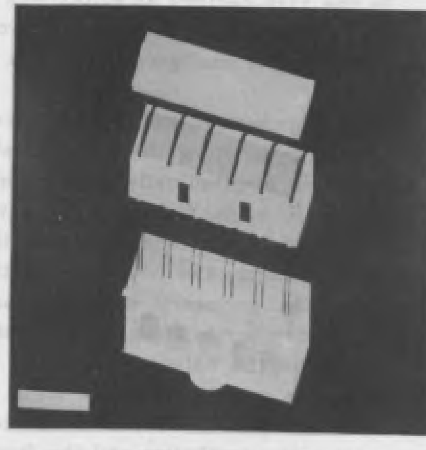
a - the rotation fo the front facade. The aggravating causes are represented

F.5

F.6



F.7



F.8

by reduction, through corrosion, of the resistant section of the columns on the outside, by the thrust of the vault, and by the non-existent transversal connection of the walling.

b - the depression of the vault, in correspondence with the front facade, caused by its rotation.

c - the intrados squeezed by the vault, which ca be seen in correspondence with the reins in the part opposite the turned facade, caused by the different static behaviour of the longitudinal walls.

The operation, because of the rigorous need not to alter the characteristics of the monument in any way, was designed so as to insert inside the structure, through cores, a series of invisible frames, capable of assuring the structure with the connection it lacked and at the same time absorbing all the stresses which the original structure, because of its degradation, was no longer able to resist (Figs. 7-8).

Thanks to the renewed inalterability of the geometric configuration of the original structural complex, guaranteed by the suggested methods, it seems right to think of maintaining the deformations of the vault without their compromising its stability.

MONASTERY OF SAN PIETRO IN PERUGIA

The subject concerns the consolidation and restoration of the Benedictine refectory built in the 15th century in the Monastery of San Pietro in Perugia and is currently the assembly hall of the University of Perugia School of Agriculture.

The refectory, 25 m x 12 m, is covered by a barrel vault with lunettes, which is set on two longitudinal walls which were built at different times (Fig. 1).

The different setting of the two walls, because of the horizontal thrust of the vault, has caused the rotation of the more recent wall, with consequent large transversal deformation of the vault.

In the 17th century an effort was made to remedy the instability of the vault by building below it a series of arches to distribute the load. These followed the geometrical configuration of the vault deformation in the intrados area (Fig. 2).

These arches, placed at distances of about six meters from each other, succeeded in stopping the transversal deformation but in turn set off a further deformation longitudinally in the vault (Fig. 3) shown by the parabolic lesions in the upper walls. Collapse of the vault, in view of the funicular action of the existing loads, was avoided thanks to the presence of the lunettes (Fig. 4).

At this point, the objective of the restoration was to bring the vault back to its original geometric configuration. This in effect was the only solution which could satisfy a series of important design requirements:

1. Reuse of the refectory was the true prime objective to be reached if the structure was to be saved from permanent abandon. The alternative to the final collapse of the vault was to create a permanent prop system, thereby ruining the aesthetics and the function of the room, or it could be demolished thereby losing a monument of historical interest.

2. At the same time, bringing the vault back to its original state meant restoring its original static function. The funicular loads which were not close to the intrados of the vault would once again be returned inside its thickness.
3. Finally, this load capacity fo the vault would permit using the space on the upper floor again, space which for functional requirements would house a library with 1000 Kg/cm2 effective load.

In view of these requirements, certain methods of operation were adopted designed to achieve the desired purpose.

Firstly, the vault in six meter sections was propped with wooden ribs reproducing the original form of the curves. These ribs were supported by a series of jacks capable of bringing to the right height the points in the intrados of the vault (Fig. 5).

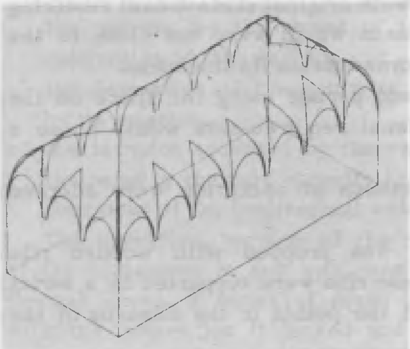
This lifting operation, which lasted for a few months, was carried out slowly to permit, thanks to the elastic hysteresis of the material, the gradual resetting of the ashlars of the vault (Fig. 6).

Once section of the vault was restored to its original height (Fig. 7), all the crushed mortar between the ashlars had to substituted by inserting a series of steel wedges and then casting reinforced concrete, the reinforcement being appropriately connected to a perimeter of reinforced concrete in the form of a curbing very rigid at the corners (Fig. 8).

By repeating this same method for all the other sections, it was possibile to lift up the entire vault, finally freed of the 18th century supporting arches (Fig. 9).

The demolition of the arches brought the Benedictine refectory back to its original architectural dignity (Fig. 10).

Finally, the greater thickness of the vault cross section, made up now of the original brickwork and overlying reinforced concrete casting, produced a corresponding rise of the funicular load, permitting using the upper floors as set out in the design phase.



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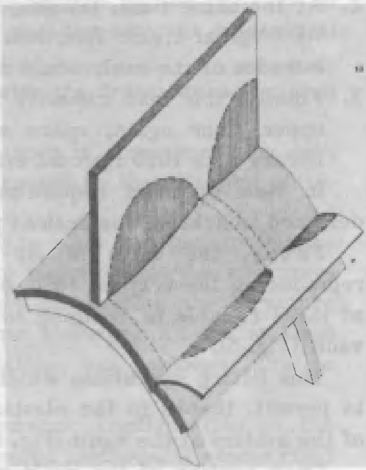

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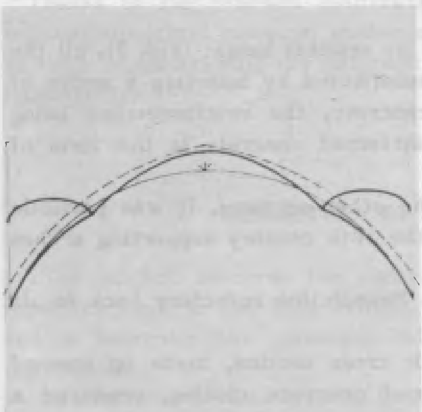

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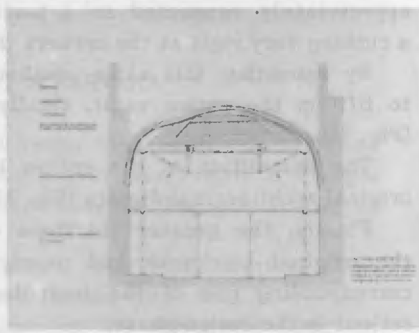

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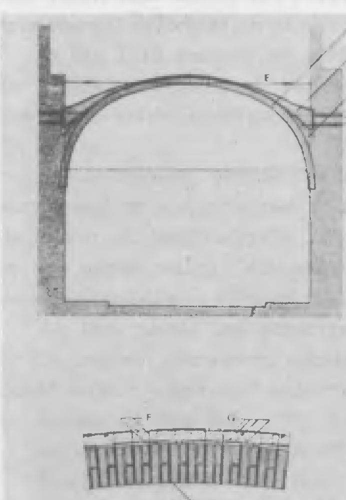

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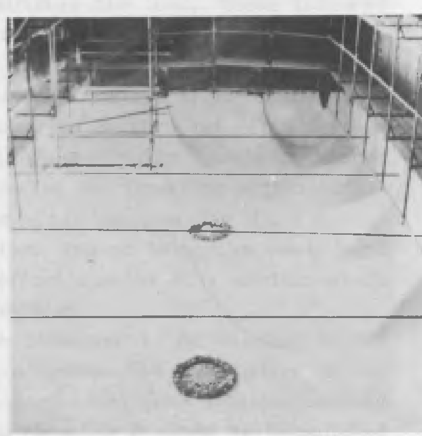

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CONSOLIDATION CRITERIA AND INTERVENTIONS ON THE STONE BEARING ELEMENTS OF THE CHURCH OF THE ARCADI MONASTERY (RETHYMNON, CRETE)

Despina KYRIAZI*, Michael TROULLINOS** and Nicholas BELOYANNIS***

SUMMARY

The very bad deterioration of the building stones in the interior of the church (arches, vaults), as well as at the external front, have made necessary an extensive intervention, including reinforcement of the central arches and slowing down of deterioration effects at the exterior. The criteria and process of intervention are described in detail.

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Canea, 11.5.89

HISTORY OF THE MONUMENT;PREVIOUS INTERVENTIONS

The Monastery of Arcadi is historically known to exist at its present state,concerning the order and position of buildings since the middle of the 16th century.When the church was finished (1587),the Monastery became communal and developed into a very important historic and cultural centre (fig.1).

Its foundation dates in the second Byzantine period and in the following first years of Venetian occupation.It was probably named after its founder,the monk Arcadius.

The church was built from 1562 to 1587,during the abbey of Clemes Hortatzis.Of its two wings,the northern is dedicated to the Almighty Christ Saviour,and the southern to the Saints Constantine and Helen.The front is following designs of Sebastiano Serlio (fig.2,3),modified by the addition of the belltower,as demanded by the Orthodox Church.

During its long history,the church has suffered from many destructions,as well as from destructive interventions.In 1646 the Turks imposed their sovereignty by burning the templon and the icons.During the revolution of 1866,the church was burnt again,and it almost collapsed,since the Turks brought down the largest part of the central supported wall.

The abovementioned destructive interventions started in 1933,when the flagstone paving was replaced by marble,and continued in 1958,when the vaults and the arches were covered with cement coating and,in the interior of the church,the stones of the surrounding eaves were cut and completed again with mortar and then painted with plastic.The stones of the arches that were destroyed by fire were completed with mortar supported by nails. Those latter very soon started rusting and swelling,thus causing cracks in the stones and therefore structural problems at the whole monument(fig.4-10).

During dismounting of the arch,an internal arch was discovered,playing probably a role of relieving mechanical stresses. It generates at the centre of the supported wall,0.9m higher than where the external arch generates.

Consolidants

PROBLEMS IN CHOOSING THE METHOD OF INTERVENTION

1.Loosening,cracking,deterioration or even complete lack of mortar.

2.Different degree of deterioration of every building stone according to its position in the monument and also to its geochemical composition:crust formation,disaggregation of the surface and loss of material,cracking,detachment of pieces,are the effects that cause structural problems.

3.The continuous function of the church.

4.Fluctuations of temperature and humidity inside and outside the monument.

5.Importance of the monument (historic and cultural centre of the whole island).

CRITERIA FOR CHOOSING THE METHOD OF INTERVENTION

1.Reversibility.

2.Increase of mechanical strength.

3.Reduction or complete suppression of water circulation in the walls.

4.Cohesion of the material.

5.Cooperation between stone and mortars.

6.Intrusion ability of the consolidants.

7.Esthetic effect.

CONCLUSIONS CONCERNING THE DEGREE OF DETERIORATION AFTER HAVING UNCOVERED THE MASONRY;INTERVENTION EXECUTED SO FAR

Analysis of the main building material by X-ray diffractometry and scanning electron microscopy-electron probe X-ray microanalysis has shown it to be a sandstone composed of calcitic clastic material and calcitic (slightly marly) cementing material.Porosity is of the order of 15-20%.Pore size distribution is very uneven,with diameters from 20 to 400 microns.The microcrystals of the cementing material vary in size from 1 to 10 microns

The mortar was composed of inert material,lime and an argillaceous earth,the two latter in a 3:1 proportion.

Considering the former data,the following mechanisms of deterioration are proposed:

1.Dissolution of the (even small quantities of) argillaceous material in rainwater.

2.Dissolution of the calcitic cementic material and (to a lesser extent) of the calcitic clastic material in rainwater containing carbonic acid.

Calcium carbonate is then precipitated on the surface of the stone forming a hard crust,which retains the internally disaggregated material.This latter is poured out if the crust breaks because of a random mechanical strain.

3.Capillary rise of water containing soluble salts causes intense deterioration,when the salts crystallize inside the pores and when they pass from anhydrous to hydrated forms.The large increase in volume causes breaking of the pore walls.Macroscopically the effect is expressed as complete disaggregation of the material into thin powder.

4.The combined action of humidity,salts and carbonic acid has provoked deterioration of the mortars.

All the above effects are more intense at the arches,where the stones are completely destroyed (fig.11,12),and the mortars have practically disappeared.The arches are practically hanging in the air.

From in situ tests of materials already tested in the laboratory,we chose the stones and mortars for filling and replacement.The fresh stone that replaced the decayed one (fig.13,14) has similar chemical composition and behaviour in accelerated aging tests.The new mortar has to perform as following:

1.Intrude as deeply as possible into the masonry.
2.Not to change the physicochemical composition of the initial mortar.
3.Similar thermal expansion coefficients between stones and mortars.
4.Similar water absorption coefficient between old and new mortar.
5.The mixture can be diluted indefinitely and does not crack when dry.
6.Good adhesion on stone.

The intervention included reinforcement of the central arches and slowing down of deterioration effects at the ornamented front external part of the church.The reinforcement of the arches was achieved by the removal of the old mortars and the replacement of all the destroyed stones.The internal filling of empty spaces inside the walls was decided,since the mortar there had completely disappeared (fig.15).

The following works have been executed:

1.Insertion of 52 beaks at the S side and 37 at the N side (the work was executed separately for each side)(fig.14).
2.Application of compressed air (5-7 atm),in order to remove dust.The existence of very large empty spaces was verified during this process.
3.Washing with water,so that the injected material would have better adhesion.3 tons of water were thus consumed.
4.Two days later,acetone was injected,in order to remove most quantities of water.80lt of acetone were thus consumed.
5.Application of compressed air.
6.Preparation of the filling material with 1 part of lime and 3 parts of an argillaceous material that exists in the neighbourhood and is locally known as "lourokhoma";it resembles very much to the respective material of the old mortars.
7.Tests of injected filling material,in order to decide about its viscosity.The addition of water was decided,starting with 40% and ending with 60%.
8.Removal of unnecessary beaks.
9.Injection of filling material from the remaining beaks, under atmospheric pressure.
10.Sealing of some beaks and injection of material under pressure of 3-5atm.
11.Preparation of a suspension of barium hydroxide octahydrate in water and injection through the beaks.
12.Construction of a piping system to inject carbon dioxide, so that the barium hydroxide produces carbonate.In this stage, most of the beaks were removed and only 16 used.Carbon dioxide was then injected from bottles.

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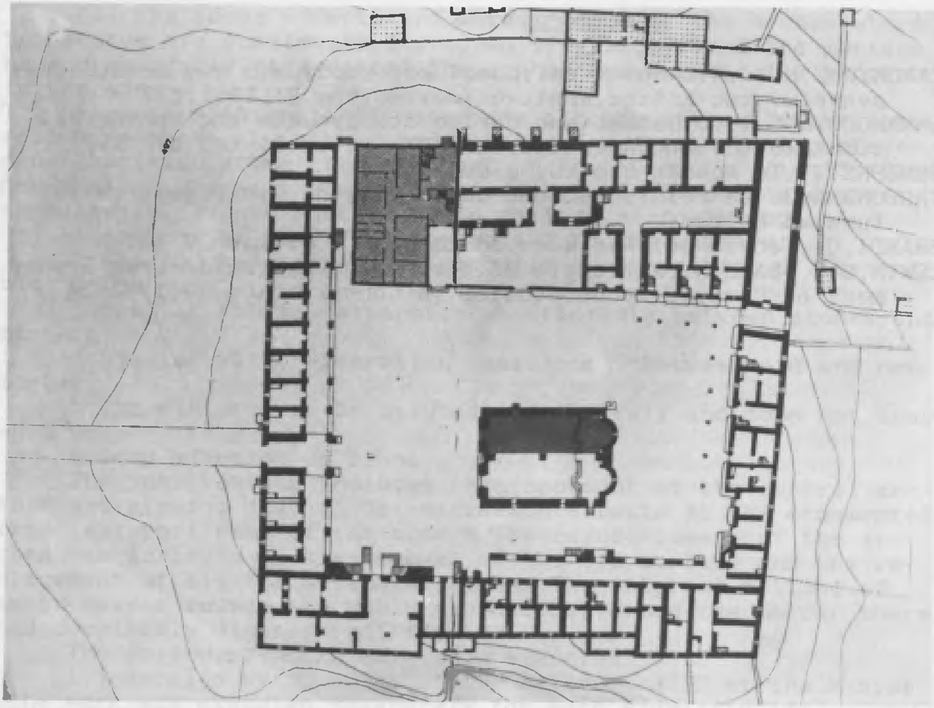
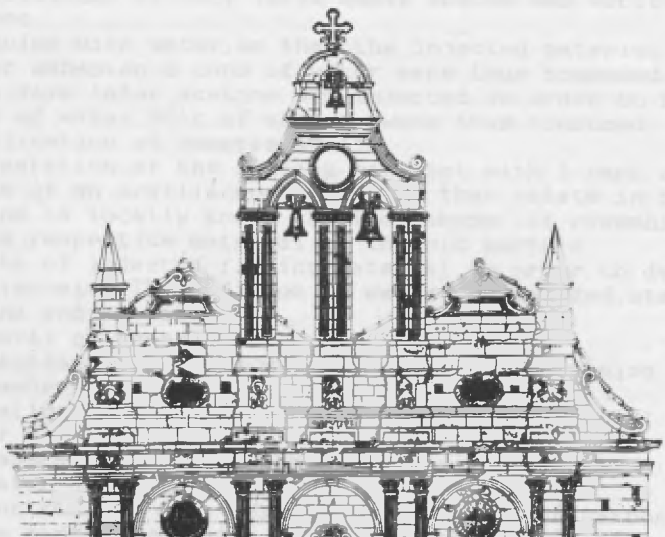
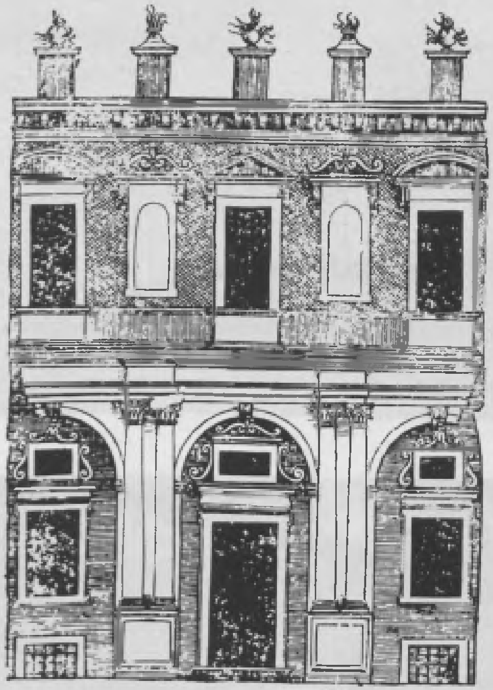
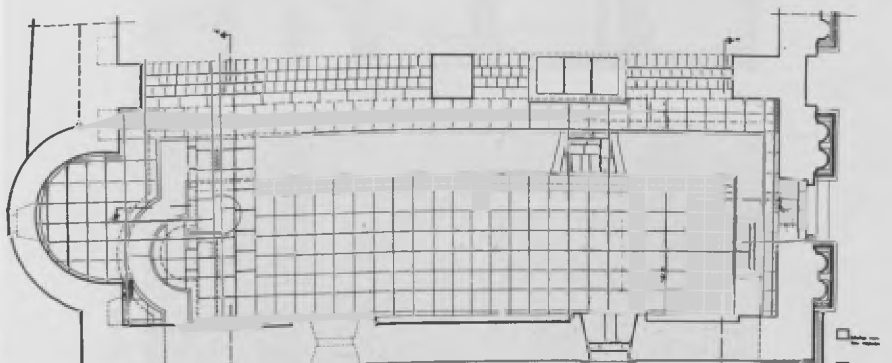
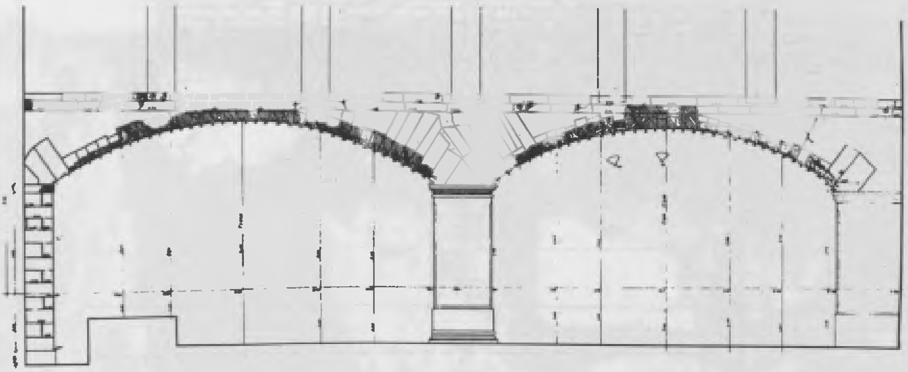


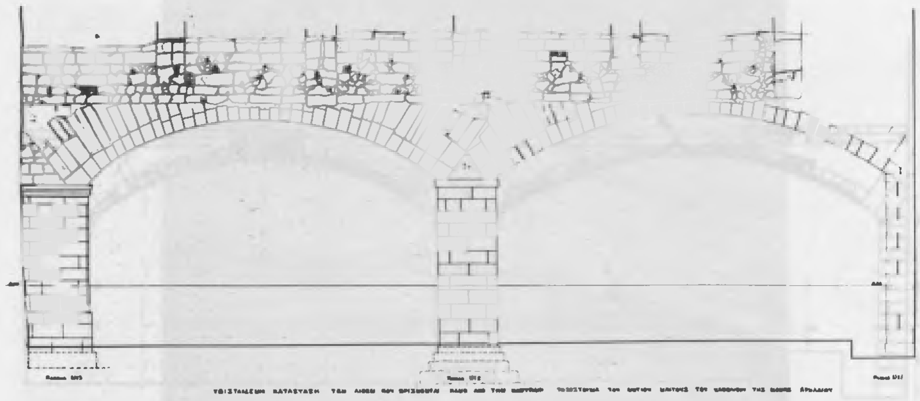


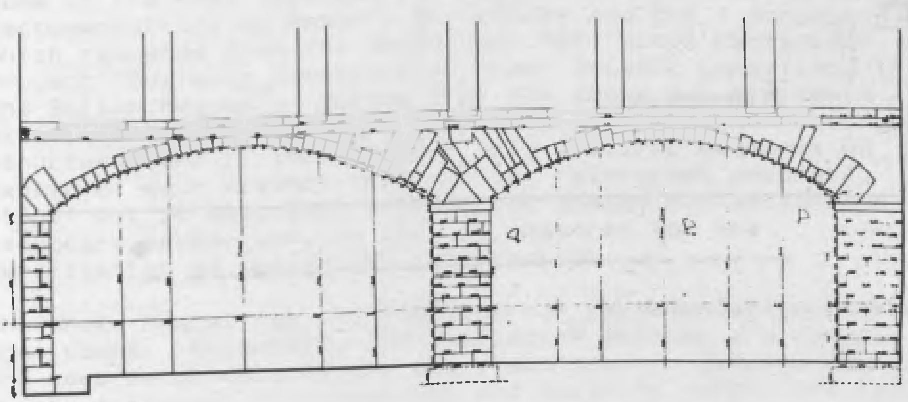


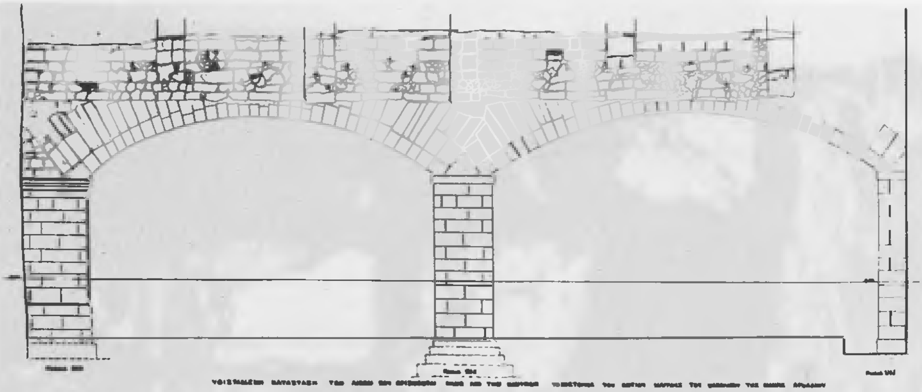


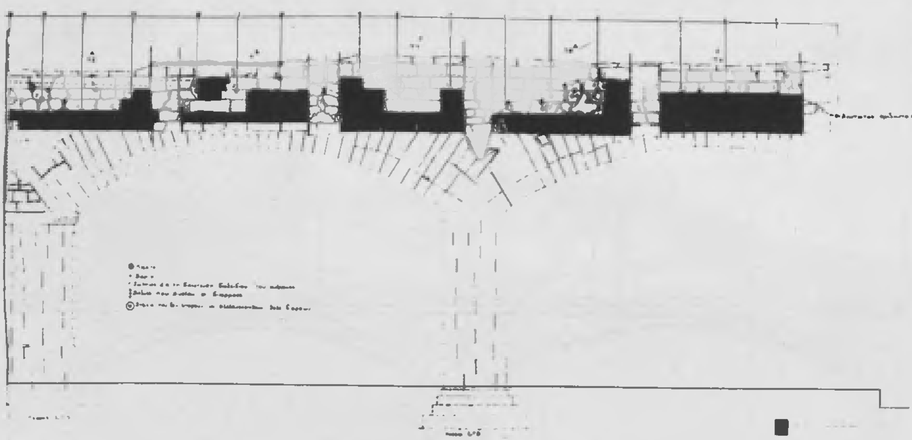


ΥΦΙΣΤΑΜΕΝΗ ΚΑΤΑΣΤΑΣΗΣ ΤΟΞΟΣΤΟΙΧΙΑΣ – ΒΟΡΕΙΑ ΟΨΗ









CODES, RECOMMENDATIONS & SPECIFICATIONS: A GENERAL REPORT

N. Kalogeras
M. Tzitzas N.T.V.A.

The recent interest expressed for masonry structures as a design of technological issue, is due to three main reasons:

a. It is one of the oldest methods of building construction globally, examples of which are abundant in many countries.

b. Major earthquakes have shown how inherently vulnerable are load masonry bearing structures and particularly those that have been altered through remodelling or additions.

c. During the last 15-20 years public interest and awareness for our cultural heritage and vernacular architecture have increased significantly - Principles, rules and guidelines for conservation and restoration of historical buildings or groups of buildings have been established and accepted internationally.

Within this environment new technologies have developed for repairing and reinforcing existing building elements like walls, floors or roofs. Furthermore a number of publications appeared on the subject whether outlining recommendations or reporting on research.

Some of the most interesting were the CIB International Recommendations on Masonry Structures and the 7 Manuals which resueted from the UNIDO/UNDP.RER/79/015 Regional Project "Building Construction Under Seismic Conditions in the Balkan Region". Volume 5 of the above manuals deals exclusively, with the repair and strengthening of masonry structures and it includes not only specific measures of repair of each masonry type (brick, reinforced concrete, stone) but it describes methods for damage evaluation and temporary protection, as well as measures for the verification of design and construction.

The whole Manual has the character of recommendations rather than codes. Especially the evaluation methods are concise and need detailed analysis and elaboration. Old structures due to different environmental and building conditions have their own "damage characteristics" and patterns.

Furthermore the great variety of finishes in vernacular structures hide the damage in different ways, making it difficult to standardise of typify.

On the other hand, as mentioned in the Manual, this situation leads to ".... Unexpected conditions .... and it is essential that the design engineer make the necessary changes to suit the actual field conditions". This means that design revew and verification although, an important step of the design process for seismic repair and strengthening projects, is not a definite or final design stage. We can again assert that even the design of such projects is an open ended procedure which need continuous revisions.

A more ambitious document is the Eurocode 6. Although it is based on the CIB recommendations, it is intended to establish a set of common rules as an alternative to the different rules in force in the various member States. It is also hoped that it will serve as a guide for the development of National codes.

The document deals with unreinforced masonry structures and provides principles for its design as well as rules for application. The principles comprise general statements, definitions requirements and models for which no alternative is permitted. The application rules follow the statements and satisfy the requirements given in the principles.

The basic principles apply to all types of unreinforced masonry structures, they do not however cover the special requirements of a-seismic design.

Comparing the above recommendations and rules with the equivalent rules comprised in the Greek National regulations we can identify a different approach. The Greek documents deal with seismic forces affecting all types of structures and more specifically reinforced concrete ones. Masonry structure is an integral part of the whole structural system as an element either of the external envelope of the building or of the internal partitions. Therefore the principles for its design cannot ignore seismic load. Futhermore national regulations suggests that all masonry structures, even when non loadbearing, should be reinforced in order to help the main structural frame of the building in receiving the dynamic loads from seismic forces. Earthquakes and dynamic loads deriving from them, cannot be ignored when principles of design are stated and therefore affect directly all application rules, as far the Meditterenean members states are concerned, with more or less severity.

Besides the above mentioned documents countries with a rich architectural heritage like Great Britain and Italy have developed Building codes which cover the use of masonry structural systems. The Italian experience is presented by professor B. Casiello's paper which highlights the different steps and difficulties in developing comprehensive normative documents. We think that the distinction of "light" and "heavy" intervention measures is very relevant in describing the present stage of restoration approaches. It would be of great interest to compare these two approaches and draw conclusions on the respective merits and limitations.

In Greece two draft Regulations and Codes of Practice both on the calculation of masonry structures (by Tassios, Vintzilaiou and Trochanis, 1983) and on the earthquake resistant building construction (by Karydis et al 1986) have been elaborated and are pending approval by the relevant authorities. Both documents are dealing with new construction and do not include specific instruction, for reinforcing and repairing stone masonry building.

With the assumption that we really need at the present moment normative documents for repair and strenghthening of masonry structures, which is a debatable issue, and by studying the existing related texts we can identify the following problems:

a. despite the knowledge acquired in recent years we still need more research and case studies on old masonry structures.

b. we need more information in order to identify the limitations of new building materials or techniques used for the repair of masonry building elements. This includes studies on the compatibility or not of certain materials.

c. we must try to formulate repair measures and recommendations of old masonry structures on the base of the action not only of one agent (namely earthquakes) but also of other agents like humidity or environmental pollution.

d. there is a need to propose repair and strengthening recommendation which are compatible with established restoration and conservation principles.

This last point, because it involves cose collaboration between architects and engineers, need further elaboration. Stone masonry structures form parts of historical and vernacular buildings of high aesthetic value. Restoration principles expressed in many internationally accepted documents, like the Charter of Venice or the Amsterdam and Barcelona declarations, form the basis of design restoration. Whatever strengthening measures are needed must be subservient to those principles. This means that recommendations like the one included in the already mentioned UNIDO/UNDP Manual about "... closing openings in existing walls ..." or "... reducing the building weight ... by removing ... stone canopies, balconies, parapets e.t.c.." if followed to the letter can lead into widespread destruction of irreplaceable architectural members.

A case in point is being presented in the paper by G. Helms and N. Baer. The paper describes the effects of the application of a local law in New York which was drafted in order to protect pedestrians. It seems at the beginning the law lead proprietors to strip their buildings of all decorative motifs in stone. Fortunately because of the prosperous economic environment the city enjoyed during the period the law was applied, and most probably because of the fact that there were restoration icentives offered by the local government, most of the buildings were not disfigured. Nevertheless in other situations such a law could become the origin for the loss of historical and architectural heritage. That is why it would be interesting to have more information about other possible reasons for the success of this law in New York.

Another issue deriving by the problems discussed above is the fact that the last few years a major effort has been directed towards evaluating earthquake damage, defining behaviour characteristics and specifying repair and strengthening measures in earthquake situations. Because these situations are not new in regions suffering from earthquakes we find, when studying old buildings protection measures already incorporated in tomasonry structures. In other words there is what we can call an "a-seismic culture" in building construction which provided solutions not only of great easthetic value but also of excellent structural quality. Therefore it is necessary in our opinion to study and analyse these solutions in order to understand and then follow their reasoning, when proposing repair and strengthening measures. In this way the designer will avoid to chose measures which although correct in structural terms, will impinge on the integrity of the existing structure.

Furthermore the above analysis will help to form an all-inclusive picture of the structure of the building. Old buildings have developed complex structural interelationships between differnt building elemnts and have undergone, in many cases, extensive alterations or additions. These alterations have altered the structural behaviour of the whole building increasing its seismic vulnerability.

The study and analysis of an old building's seismic behaviour is inadequate if it is not supplemented with the study of the whole building block of which the building is a part. The nuckeus of many historical towns present a complex configuration of interrelated building volumes. The stiffness of each unit depend in great degree on the stiffness of the whole block. The comprehensive analysis of these volumes and their interrelationships can give us useful clues on which to base our interventions.

In this paper we tried to highlight some of the major problems facing the effort to formulate recommendations, codes or specifications for the repair and reinforcement of masonry structures. It is not, by any means, an exhaustive or comprehensive survey of existing efforts and does not pretend to present guidelines. Nevertheless we think that the present situation needs a revision of some of the approaches and a concentrated and coordinated interdisciplanary effort, if we want to have useful on-the-field codes of practice.

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PROPOSED STANDARDS FOR THE DESIGN OF THE REHABILITATION OF MONUMENTS OR IMPORTANT URBAN NUCLEI

C. Vlahou, P. Kremezis, (*)
C. Pavlatos, P. Plainis
C. Floros

SUMMARY

This proposal has been prepared by the working group set up by the Technical Chamber of Greece and aims to provide engineers and associated professionals with a Code of Practice for the Survey and the Design of the Rehabilitation of Monuments and Urban Nuclei. The proposals are based on the experience of the group, experience gained on the Design of the Rehabilitation of Monuments and modest buildings of historical value.

The paper contains the main points of the Introduction and the proposed standards concerning Structural Design covering the following aspects:

- Structural Survey including Dimensional survey, Damage survey, Historical survey, Environmental survey, Soil survey, Building-materials properties and characteristics.

- Appraisal of Structural Adequacy including assesment of causes of existing damage.

- Structural Rehabilitation Design.

All International conventions refering to the conservation of Monuments have been taken into account in this proposal and the main effort of the group has been to take into account both the special problems inherent in the Structural systems and the need to minimize intervention. Since the working group's member practice in Greece, the special inherent characteristics and features of traditional construction in the Hellenic region have been taken into account and due to the fact that the structural system most commonly found in this region is Structural Masonry, the standards have been drawn up taking this fact into account.

(*) Civil Engineers, members of the Study Group of Technical Chamber of Greece

1.- INTRODUCTION

A large number of Monuments and individual buildings in Important Urban Nuclei that are in need of Rehabilitation exist in Greece.

Recently a coordinated effort to this effect has been taken up by the Ministry of Culture and other Organisations. These efforts have stumbled on the lack of standards governing such interventions, a fact which seems to exist also in many other countries.

The principal source of this problem is that special problems inherent in the structural systems of existing Historic Buildings and particularly Monuments, are not covered by the criteria stated in codes of practice and specifications dealing with the design of modern structures.

The outline of the differences in criteria that should apply to the Design of Traditional Buildings or Monuments as opposed to Modern constructions is:

- Modern Constructions.

Essentials for the design and construction are known in advance, loading history is known.

The designer works with materials of determinate qualities covered by Codes of Practice or Building Regulations.

The designer works by using established calculating methods, formulae and codes of Practice.

A lot of experience exists both in the Design, Construction and Post construction interventions.

- Traditional Constructions

Documentary evidence is usually not available and loading History is unknown. The designer will have to asses loads, forces construction details properties and strength of Materials, which are not readily assesed or determinable.

Application of methods and materials of restoration are not widely known since little specialisation or experience in their use exists.

Design philosophy, as applied to the rehabilitation design, differs from design philosophy applied to design of modern constructions in the following aspects.

(a) Instead of working with materials of determinate qualities and properties covered by codes of practice, the designer has to determine the properties of the materials of construction and define the quality of the structural system as a whole.

(b) Essentials for the design of the structural system are not covered by Codes of practice and the designer must determine the real overall safety of the structural system under an assumed loading history and similtaneously the structure's real behaviour (i.e. signs of overstress or other defects).

(c) While designing the Rehabilitation one has to comply to the principles of Restoration and Protection of traditional buildings while, at the same time, taking into account the specific problems and original qualities inherent in each Monument. It is the rule to find original quality in a Monument and the exeption to find it in a modern construction.

Taking into account the issues identified, the main aspects of the Code of Practice (Standards) should be:

(a) Extensive procurement of Information is required, including a Structural survey and any other on or off site investigation required that is feasible to carry out.

(b) Analysis of the load bearing capacity of the structural system of the Monument in its present and initial condition, including the interpretation of all existing damage and deffects.

(c) Structural Rehabilitation - Upgrading Design.

Some of the main issues of the Standards are presented in this paper and particularly:

(a) Setting up an Investigation programme
(b) Structural behaviour calculation
(c) Seismic Design (Assessment of Buildings under seismic conditions).

2.- INVESTIGATION PROGRAMME

Collecting information required for the Design of the Structural Rehabilitation of a Monument or Urban Nucleus is the product of Investigations.

The proposed Investigations are
- Dimensional and Historic surveys
- Determination of the load carrying capacity of the components of the structural system

The later are usually refered to as Investigations together with all other surveys leading to the determination of dimensions and characteristics of non-accessible structural elements.

The number of Investigations that can be carried out is very large, as large, as the number of characteristics that have to be determined.

In each particular case, one has to decide on the characteristics needed to be determined and thus set up an Investigation Programme which will provide data for the determination of the required characteristics.

Setting up an Investigation Programme depends on the particular project but the basic issues can be summarised as:

- Importance of the Project
- Present degree of structural adequacy and/or need for immediate Intervention
- Cost effectiveness

The main effort of the engineer responsible for setting up an Intervention programme should be the recognition of existing conditions which require evaluation, the results of which, will be possible to be used in the process of the Design.

The Investigation Programme will have to be conducted according to the testing capabilities at hand.

The difficulty in carrying out Investigations, due to the original qualities and special problems inherent in traditional buildings and Monuments, in conjuction to the scarcity of specialised testing laboratories, sometimes make the determination of structural characteristics very difficult or non cost-effective.

In this case, concerning small privately owned traditional buildings, not of Monumental value, characteristics for the strength of materials - structural systems can be determined in the relevant bibliography or Codes of Practice that were current at the time of construction with special care being taken in the use of safety factors as applied to the present condition of Materials and Structural systems. Concerning Monuments, or more important building it is deemed necessary to carry out a full investigation exhausting every possibility of research.

SPECIAL INVESTIGATION PROBLEMS

Thick structural masonry elements are very difficult to investigate. This applies both to their construction mode and to the materials used in building the core. The only possible way to access them, visually, is by creating holes of some magnitude which jeopardize the structural stability of the element and are not compatible with the principles of Restoration.

Technology today, presents us with not a so effective alternative which does not wound the Msonry to such an extent. This Method consists of drilling 5-10 cm diameter boreholes and permits sample taking or by using even smaller diameter boreholes optical inspection and protography. Such an investigation of the core of a massive Structural Masonry element is considered necessary when such an element is not structurally independent but it is a part of a structural system and the result of the investigation will be used both in the structural analysis and in the structural rehabilitation design.

The use of x-rays is not generally justified, due to the inherent problems of the method and its great cost. The method y can be used in very special circumstances as in the restoration of the Erechtion and the statue of Hermes of Praxitelis, where the method was used with great success.

Finally, reference has to be made to the methods of chemical analysis which can be incorporated in the investigation methods since they provide information concerning the composition of materials. Concerning certain methods of intervention (i.e. grouting) the chemical analysis must lead to the determination of the compatibility of the materials of the intervention, with the materials used in the construction of the Monument.

In general one can conclude that detailed Investigation is always useful and that every piece of information determined can be utilised.

INVESTIGATIONS TO DETERMINE BUILDING AND STRUCTURAL MATERIALS CHARACTERISTICS

This part of the Investigation, aiming to the determination of strength and other characteristics of the Structural Materials, can be extremely extensive since it incorporates, in addition to the strength characteristics of the Structural Elements, every other factor that influences the overall stability of the Monument, as for example seismicity and the immediate natural or artifiical environment. Existing experience has proved that considerable difficulty exists in carrying out the Investigation to determine Building and Structural Material Characteristics and utilize its results.

The principal material characteristics that are usefull to the process of Design are: Nature of Material, Chemical properties, influence of water, surface properties (resistance to weathering) resistance to aging, insulative properties, and mainly strength characteristics i.e. tensile and compressive strength, deformability (Modulus of Elasticity etc).

Of far greater importance to the process is the composite behaviour of structural elements, a behaviour which cannot be determined directly through its mechanical characteristics but indirectly through the individual properties of its constituent materials and the methods through which they are connected.

Determination of the characteristics is usually carried out either through laboratory tests (sampling is necessary) or through in situ non-destructive tests.

Apparatus used to carry out in situ non destructive tests (i.e. rebound hammer) must be callibrated specifically for the material being tested. If this is not the case, calibration must be carried out on samples which will then be tested in the laboratory.

Setting up an Investigation programme, must combine destructive sampling with in situ non destructive testing so that reliable results can be obtained through the minimum intervention in the fabric of the structure.

The principles for setting up the Investigation programme and specifying the number of tests are:

- Location of sampling or in situ testing must be representative of the various parts of the structure
- Minimal number of each kind of tests but satisfactory for statistical evaluation
- Compatibility of testing methods with methods of analysis or evaluation so as to provide data usefull in the process of Design.

Such an Investigation becomes prohibitive if one or more of the following conditions are applicable:

- Occurence of a great variety of materials and/or methods of construction within a Structural System.
- Possibility that sampling will cause irreversible damage to structural elements or valuable finishes.
- Cost as compared to the importance of the building is prohibitive.

If the Investigation has become prohibitive it is possible to determine structural characteristics necessary to the process of, Desing either through existing codes of practice and regulations or through available bibliography by translating through them results obtained by the structural survey and/or the macroscopic investigation of Materials and construction method of the structural system.

3.- STRUCTURAL - SEISMIC DESIGN

The design for the structural appraisal of an existing building is a completely different process from the desing for the construction of a new building. In the design of a new building dimensioning and choise of materials of construction precede stress calculations. Stress must not exceed allowable stress for a specific material. In the structural appraisal design dimensions of structural-elements are definite and there can be no chose of materials to fit the specific problem. The designer will have to determine the stresses which result from a given loading condition and decide on the level of safety of the structural system.

The term "Structural Behaviour" covers the appraisal of the behaviour of the structure under past-present or future loading conditions. Structural behaviour can be expressed either through calculated stress and its comparison to acceptable values of stress or through the estimation of safety factors and damage caused by loading the assesment of existing damage forms a part of the Structural Behaviour Design.

Researching the "History" of the structure is of great value to the Structural behaviour design, since loading of the structure can be determined.

Lack of information concerning loading History and uncertainty concerning strength characterisitcs of structural materials reduce considerable the validity of conclusions drawn from the structural behaviour design, so that these conclusions are considered qualitative and not quantitative. The use of parametric solutions taking into account hypothetical loading conditions and variable values for the strength characteristics can make up for the lack of data and the uncertainties inherent to the structure.

Conclusions drawn from the structural behaviour design can be considered valid if they are in agreement with actually occuring damage or signs of overstress (cracks, deflections, etc).

The validity of the Structural behaviour design depends to a great degree on the adopted structural model. Structural models can be either individual for each mode of loading or structural elements or they can incorporate the whole structural system and are directly linked to the uprposed method of structural analysis. The choise of highly complex methods of calculation offering a high degree of accuracy, is possible but not always effective, unless supplied with very accurate information. It is sometimes more appropriate to use simpler methods of calculation checking on the load bearing capacity of individual structural elements of the structure and drawing qualitative conclusions on the behaviour of the structure from such calculations. The criterion for the successful choise of the model is the agreement of the calculations conclusions with the actual behaviour of the structure.

The Structural Investigation can provide invaluable information to the structural behaviour design since it can provide answers to the questions of phases of the construction of the structural system, successive interventions, causes and time of the occurence of damage. The structural behaviour design can also assist the Investigation by helping to interprete its results thus leading to working formulations for individual structural elements of the structural system where overstress should be expected and the expected time of appearance.

A special chapter of the structural behaviour design is the study of the behaviour of the structure under Earthquake loads.

The existence of a Monument in a seismic area is proof of its ability to withstand Earthquakes. However, it is desirable to obtain proof of its ability through scientific methods.

It is worthwile to keep in mind that the ability of structural masonry to withstand seismic loading diminishes considerably if the masonry is not maintained properly.

Earthquakes are a propabilistic phemomenon and characteristics of the next expected Earthquake can be radically different from those expected. Due to this fact, advances in technology permitting the use of more accurate and refined methods of Investigation and Calculation of characteristics of the structural system should not lead to the neglection of simple but effective rules of maintenance and preservation of Structural Masonry and the research for measures leading to the protection of Monuments from irreperable damage.

Care should be exercised when the planned rehabilitative or strengthening interventions effect or change the structural characteristics of the Monument since then its Historically proven Earthquake resistance can be adrersely affected. In such cases Investigation can be extended after carrying out the intervention through in-situ tests which will verify the results and accordingly lead to corrective measures.

PROPOSED CONTENTS OF THE CODE OF PRACTICE

1.- Introduction
2.- Data Collection
  2.1. Dimensional Survey
  2.2. History
  2.3. Investigations
3.- Structural Behaviour Design
4.- Intervention Design

NEW YORK LOCAL LAW 10: PRESERVATION IMPLICATIONS

Gertrude M. Helms* and Norbert S. Baer**

SUMMARY

The background and provisions of New York Local Law 10, a law enacted to protect pedestrians by requiring periodic inspection of building facades, are described in the context of potential effects on historic buildings. Despite early concerns that owners would simply remove non-structural building ornament rather than undertake more costly restoration, anecdotal evidence suggests that restoration has, in general, been the preferred course of action. The full text of the law is presented as an Appendix.

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Figure 1. Overall view and details of the Mayflower Hotel, New York City, showing non-structural architectural decoration stripped in response to Local Law 10. Photo courtesy of the New York Times.

NEW YORK LOCAL LAW 10: PRESERVATION IMPLICATIONS

HISTORY OF THE LAW

On February 21, 1980, New York City enacted Local Law 10,[1] an amendment to the administrative code of the city, that required the "periodic inspection of exterior walls and exterior appurtenances of buildings" followed by necessary repairs. This law was the result of public outcry over the 1979 death of a college student caused by masonry that fell off an improperly maintained building. The law stipulated that every building over six stories was to undergo a 'critical examination' within five years of the law's enactment.

The law, intended to reduce the risk of injury to pedestrians, had a strong initial effect on the fabric of the city (Belkin, 1984; Goodman, 1982). Several well known buildings, among them the Mayflower Hotel on Central Park West (Fig. 1), were stripped of their potentially unstable "exterior appurtenances", in that case purely decorative terra-cotta balconies and other non-structural ornament, rather than restored. Newspaper and journal articles predicted that stripping, because it is economically more viable than restoration, would soon become the norm (Belkin, 1984; Goldberger, 1983). Preservationists, alarmed by this possibility, lobbied to have the law re-written to decrease the cost of compliance, hoping thus to save the facades of as many buildings as possible (Rossbach, 1983).

The direst predictions that many important buildings would be denuded of ornament because of the expense of restoration seem not to have come to pass. Possibly because New York has had unparalleled economic prosperity in the last ten years, many building owners have opted for preservation. This may also be the result of the early publicity about the negative effects of the law. In fact, real-estate professionals and restoration architects agree that the law has led to safer structures and spurred much needed maintenance and restoration (Daniels, 1984).

THE LAW IN PRACTICE

Under Local Law 10, every building over six stories in New York must have a 'critical examination' every five years. This 'critical examination', supervised but not necessarily carried out by a Registered Architect or Licensed Professional Engineer, can be made using any methods that the pro-

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[1] The text of Local Law 10 is given in the Appendix to this paper.

fessional deems appropriate. Although the Buildings Department suggests the use of a scaffold or other observation platform in the examination, it does allow examination by remote observation equipment and photographic magnification techniques. Most professionals believe that except in the case of terra-cotta and sheet metal ornament, where the most problematic areas are usually hidden, a fully competent facade survey can be done using high power binoculars. In most cases the liability costs associated with examination platforms are considered to outweigh their potential benefits.

The New York City Buildings Department provides a short computer generated check-list of items that must be considered in the 'critical examination' for each building to be surveyed. After the examination, the professional must submit that form and any necessary accompanying information documenting the condition of the building and any recommendations for necessary repairs. All of the unsafe conditions are to be corrected within 30 days of the filing with the Department of Buildings. Once the conditions are corrected, an ongoing maintenance program that pays special attention to the areas subject to repair in the past is suggested.

During the whole process the Buildings Department acts primarily as a records center. Only in unusual cases does the Department become actively involved before it inspects the building after all violations have been corrected. The Buildings Department relies on the individual architects and engineers to properly supervise on-going projects. This policy of non-involvement is due in part to several factors of significance in New York City: the desire for less government intervention, the lack of adequate funding, and the potential for corruption associated with more governmental involvement.

This of course places an additional responsibility on the supervising architects and engineers. In practice those who undertake Local Law 10 examinations seem to have sorted out into two groups: those that are responsible, careful professionals who are dedicated to making sure that buildings are safe and those who, as one architect put it, "operate out of the back of their cars" and have little sense of responsibility and generally no insurance. Unfortunately, the more responsible group is usually the more expensive. As of now, the Buildings Department has only two choices of disciplinary action against professionals who have falsified information: they can be reported to the State Board of Education which has the power to suspend or revoke licenses or they can lose their Buildings Department privileges under Directives 14/1975 and 2/1975. Under these directives BNs (Building notices) and ALTs (Alteration permits) receive only cursory Building Department examination; a loss of these privileges would mean that all of the practitioners work would be seriously held up in the system.

Although there is a great potential for liability in Local Law 10 work, there have been few instances where a reputable practitioner has signed off on a building that has had a subsequent problem. Most practitioners protect themselves by suggesting more frequent re-examinations of problematic areas than are called for by the law and by providing stringent maintenance schedules that should also pick up on any changes in building condition. Under the New York City Building Code all responsibility for a building ultimately rests with the owners, so stability and maintenance is also in their best interests.

Immediately after the passage of the law there was such reluctance on the part of building owners even to schedule examinations that the Buildings Department ultimately held three warrant days when egregiously non-complying owners, whose buildings posed real dangers, were arrested. The Police Department lent their warrant squads and the owners were taken to criminal court. By the third warrant day the publicity was such that a fourth warrant day was not needed.

New York City is now into the second round of Local Law 10 compliance. During round one, 1982-1987, over 1,200 violations were corrected to safe or precautionary status. In this second round, 1987-1992, there have been 460 dismissals and there are now only 117 violations out-standing. The Buildings Department interprets these figures to mean that the law is really working.

There are no statistics available on how owners chose to correct violations; obviously projects run the gamut from complete stripping to total restoration. As is frequently the case in New York City, any construction or restoration job costs much more than comparable work elsewhere. Side-walk bridges to protect pedestrians can cost as much as $4-5,000 a month, and even basic preservation compliance may cost in excess of a million dollars (Belkin,1984).

LOCAL LAW 10 AND LANDMARKS PRESERVATION

While Local Law 10 has had an unexpectedly beneficial effect in certain sectors, it has had more equivocal results in others. New York City has an extremely vigorous Landmarks Preservation Commission that since 1965 has had a mandate to designate and regulate buildings and districts of special historical, cultural or aesthetic value (New York Charter, 1973). Currently there are over 16,000 properties under Landmarks jurisdiction. Following designation, each property owner has an affirmative obligation to keep that property "in good repair". Designated property can not be altered in any way without appropriate certification from the Landmarks Preservation Commission. This means that even minor maintenance work such as small repairs or restorations that do not usually require a building permit must be approved.

In the case of facade defects discovered through Local Law 10, the owners have no choice but to restore the buildings following Landmarks Preservation guidelines. In a few early cases Local Law 10 compliance was used by some building owners as a pretext for altering buildings so severely that they would not be landmarked. Recently, members of the Landmarks Preservation staff have noticed a decided change in attitude from the early 1980s. In the years immediately following the passage of Local Law 10 many owners seemed to resist the concept of restoration. Now owners are interested in restoration but have a real problem with financing.

Although largely undocumented, current New York Real Estate wisdom suggests that the value of buildings within Historic Districts has been adversely affected because of such factors as increased facade maintenance costs and restrictions on alterations and demolition (Listokin, 1982). However, because of the pragmatic, flexible character of the New York Landmarks Commission, restoration architects in New York believe that buildings in Historic Districts have not lost value.

CONCLUSION

For New York City, Local Law 10 has had unexpectedly beneficial results. The only two potential negatives that seem to have become associated with the law, stripping and liability, have not been as severe as originally expected. In fact, besides improving the physical condition of much of the fabric of the city, the law has actually spurred a substantial amount of restoration work. In case after case, building owners who were confronted with building facade violations have come to realize the importance of restoring and even more importantly, maintaining facades.

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APPENDIX

LOCAL LAWS
OF
THE CITY OF NEW YORK
FOR THE YEAR 1980

No. 10

A LOCAL LAW

To amend the administrative code of the city of New York, in relation to requiring periodic inspection of exterior walls and exterior appurtenances of buildings and requiring a record of such inspection to be kept on the premises.

Be it enacted by the Council as follows:
Section 1. Sub-article 105.0 of article one of part two of title C of chapter twenty-six of the administrative code of the city of New York is hereby amended by adding thereto a new section C26-105.3, to read as follows:

C26-105.3. Exterior walls and appurtenances thereof.--In order to maintain a building's exterior walls and appurtenances thereof in a safe condition, the following additional requirements shall apply to all existing buildings or buildings hereafter erected which are greater than six stories in height:

(a) Inspection requirements.--A critical examination of an applicable building's exterior walls and appurtenances thereof shall be conducted at periodic intervals as set forth by rule or regulation of the commissioner, but such examination shall be conducted at least once every five years.

(1) The initial examination for any existing building shall be conducted within two years of the effective date of this local law and the initial examination for any building hereafter constructed shall be conducted in the fifth year following the erection or installation of any exterior wall and/or enclosures.

(2) Such examination shall be conducted and witnessed by or under the direct supervision of a licensed architect or licensed professional engineer by or on behalf of the owner of the building.

(3) Such examination shall include, in addition to an inspection, a complete review of the most recently prepared report.

(4) Such examination shall also be conducted in accordance with applicable rules and regulations promulgated by the commissioner.

(b) Report of examination.--Such architect or engineer shall submit a written report certifying the results of such examination to the commissioner, clearly documenting the condition of the exterior walls and appurtenances thereto. The report shall include a record of all significant deterioration, unsafe conditions and movement observed as well as a statement concerning the watertightness of the exterior surfaces. Such report must be signed by and bear the professional seal of such architect or engineer.

(c) Necessary repairs.--Upon the filing of the architect's or engineer's report of unsafe condition with the commissioner, the owner, his agent or the person in charge shall immediately commence such repairs, reinforcements or precautionary measures as may be required to make the building's front walls or appurtenances thereof conform to the provisions of this code. Such ameliorative work shall be completed within a time period to be established by rule or regulation of the commissioner.

(d) Exceptions.--The additional requirements imposed by this section shall not be applied to:

1. Exterior walls set back more than twenty-five feet from the street and/or any paved pedestrian walkway.

(2) Buildings having an on-going maintenance program subject to rules and regulations promulgated by the Department of Buildings for the exterior walls and appurtenances thereof, under the supervision of a licensed architect or licensed professional engineer retained by or on behalf of the owner.

(e) Violations.--Any person who shall violate, or refuse, or neglect to comply with any provisions of this section shall, upon conviction thereof, be punished by a fine of not more than one thousand dollars, or by imprisonment not exceeding six months, or both; and any such person shall, also, for each offense, be subject to the payment of a penalty in the sum of two hundred fifty dollars for each month there is non-compliance, to be recovered in a civil action brought in the name of the commissioner.

2. This local law shall take effect immediately.

Enacted 21 February 1980.

APPENDIX

LOCAL LAWS

OF

THE CITY OF NEW YORK

FOR THE YEAR 1980

No. 10

A LOCAL LAW

[illegible] code of the city of New York, in [illegible] periodic inspection of exterior walls [illegible] appurtenances [illegible] buildings and [illegible]ing a [illegible] to be [illegible] the premises.

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[illegible]ation [illegible] and [illegible]iodic intervals as [illegible] forth [illegible] rule or [illegible] of the [illegible] examination [illegible] conducted [illegible] every five years [illegible] examination [illegible] existing [illegible] shall be conducted within two years of the [illegible] of this local law and the initial [illegible] for any building hereafter constructed [illegible] conducted in the fifth year following the [illegible] installation of any exterior wall and/or [illegible]

(2) Such examination shall be conducted and witnessed by or under the direct supervision of a licensed architect or licensed professional engineer by or on behalf of the owner of the building.

(3) Such examination shall include, in addition to an inspection, a complete review of the most recently prepared report.

(4) Such examination shall also be conducted in accordance with applicable rules and regulations promulgated by the commissioner.

PRESENTATION CRITIQUE DES NORMES ET CONSIGNES EN VIGUEUR EN ITALIE POUR LA CONSOLIDATION DES BATIMENTS EN MACONNERIE

STELLA CASIELLO*

RESUME

The Italian legislation regarding the consolidation of brick buildings and the technical regulation for constructions in seismic area were framed in a system, in Italy, just since 1986.
Actually two laws were passed: the decree 1/24/1986 regarding "Technical rules for constructions in seismic areas. Interventions on existing buildings" and the decree 11/20/1987 titled "Technical rules for the planning and the testing of the brick buildings and for their consolidation". Therefore both these laws are also applied to the Architectural Heritage with historical-artistic and environmental value.
About the architectural heritage the "Recommendations concerning the interventions on the monumental heritage with a specialistic typology in seismic areas" (6/17/1987) were elaborated by a national committee. This paper shows how in the restoration praxis many difficulties arise trying to apply to the cultural wealth all those rules regarding buildings in seismic areas.

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NAPLES, le 15 - 4 -1989

Législation et patrimoine culturel

Les contrastes et les polémiques qui ont caractérisé le débat culturel sur la restauration des monuments au cours des vingt dernières années en Italie, tout en ravivant l'intérêt commun pour le patrimoine historique et artistique, ont néammoins engendré, sur le plan pratique, une certaine désorientation chez ceux qui sont appelés à opérer dans le secteur due entre autre à l'absence de lois adéquates.

C'est en 1986 que furent promulguées pour la première fois des Normes ministérielles (Décret 24/1/86 - Normes techniques pour les constructions en zones sismiques. Interventions sur les bâtiments existants. - Décret 20/11/87 - Normes techniques pour le projet et le contrôle des bâtiments en maçonnerie ainsi que pour leur consolidation) concernant la consolidation des bâtiments, en posant les jalons pour un nécessaire approfondissement de la matière. Tout le monde sait, désormais, qu'afin de protéger convenablement les mémoires du passé, les consignes et les lois ne suffisent pas, mais il faut sensibiliser tous les citoyens vis à vis du patrimoine d'histoire, d'art et de nature dont ils sont dépositaires. Mais il est vrai aussi que pour toute raisonnable action de sauvegarde il faut disposer d'un ensemble de normes et de lois, coordonné et précis, où s'appuyer.

Il faut tout d'abord rappeler qu'en Italie sont encore à présent en vigueur deux lois de tutelle: celle du 1er juin 1939, n. 1089, concernant ce qui relève de l'intérêt artistique et historique, et celle du 29 juin de la même année, concernant les beautés naturelles et le paysage. Les deux lois relèvent, en tant que mise à jour des précédentes, du vaste débat culturel qui s'était développé au niveau européen aprés la première guerre mondiale et qui avait apporté à la Conférence d'Athènes de 1931, lors de la promulgation de la fameuse "Charte d'Athènes", d' heureuses normes pour la conservation et la restauration du patrimoine culturel d'Etat.

Ce que je veux traiter en particulier ici c'est la règlementation en vigueur en Italie pour la consolidation des bâtiments en maçonnerie qui, au cours des dernières années, à la suite du séisme du 23 novembre 1980 qui a touché les régions de la Campanie et de la Lucanie, a enregistré une formulation nouvelle.

Les lois sismiques depuis les années Trente jusqu'à aujourd'hui

Avant de faire quelques considérations critiques sur les lois en vigueur et sur leurs applications, il va falloir examiner le rôle des bâtiments d'intérêt historico-artistique à l'intérieur des lois sismiques promulguées depuis les années Trente.

Dans le R.D.L. 22/11/1937 n. 2105 ("Normes techniques pour le bâtiment avec prescriptions spéciales pour les zones affectées par les tremblements de terre"), les bâtiments "à caractère monumental ou ayant de toute manière un intérêt archéologique, historique ou artistique, qu'ils soient publics ou privés" (art. 39) sont traités selon les dispositions de la loi n. 364 du 20/6/1909 qui était la loi de tutelle de l'époque. Cela veut dire que l'on ne promulguait pas de normes techniques spécifiques, dans la mesure où le législateur était surtout soucieux de garantir la sauvegarde du monument en fonction de ses caractéristiques historico-artistiques. De plus, pour les bâtiments affectés par le séisme, on suggérait des oeuvres d'amélioration, sans aucune indication relative aux calculs de vérification statique pour les interventions sur les fondations, sur les voûtes, sur les escaliers en porte-à-faux, sur les maçonneries lézardées, etc., la loi dictant des dispositions à caractère constructif pour ce type de structures (art. 36). Dans l'art. 35 il est notamment précisé: "La restauration des bâtiments endommagés doit viser à obtenir des conditions de stabilité meilleures par rapport à celles qui préexistaient au tremblement de terre, afin qu'elles puissent résister à un éventuel renouvellement des secousses".

Et même dans la loi successive n. 1684 du 25/11/1962 ("Mesures pour le bâtiment avec prescriptions spécifiques pour les zones sismiques") on insiste sur la nécessité de sauvegarder les monuments d'après les lois en vigueur en matière de tutelle, c'est-à-dire la loi n. 1089 de 1939 et la loi n. 1492 de la même année; on indiques aussi les critères de consolidation en ce qui concerne les escaliers, les voutes, les maçonneries lézardées, les fondements, etc., sans pour autant faire, une fois de plus, la moindre référence aux calculs de vérification.

Par la suite, en 1974 fut promulguée la loi n. 64 du 2/2/1974 ("Mesures pour les constructions avec prescriptions spécifiques pour les zones sismiques") qui met l'accent sur l'importance des interventions sur les monuments renvoyant, pour ce qui concerne leur consolidation, aux normes contenues dans la précédente loi n. 1684 et affirmant de façon générale, au sujet des interventions sur les bâtiments, qu' "elles doivent viser à obtenir un meilleur degré de sécurité vis à vis de l'action sismique".

Le D.M. 3/3/1975 ("Normes techniques pour les constructions en zones sismiques") indique au point C.9 les "réparations des bâtiments en maçonnerie", rapportant une fois de plus, les critères techniques d'interventions à appliquer lors du consolidement de fondations, escaliers, voûtes, planchers, etc.

A la suite du tremblement de terre qui toucha les régions de l'Italie du Sud en 1980, l'année suivante fut promulguée par D.M. 2/7/1981 la "Règlementation pour les réparations et la consolidation des bâtiments endommagés par le séisme dans les régions Lucanie, Campanie et Pouilles" avec les relatives "Consignes pour l'application" (Circulaire ministérielle 30/7/1981). C'est dans ce décret que pour la première fois sont affrontés les problèmes de l'ajustement antisismique" ainsi que ceux de la "vérification sismique" pour les bâtiments en maçonnerie, soulevant ainsi d'inquiétants interrogatifs sur la possibilité d'appliquer le décret aux bâtiments d'intérêt historico-artistique. En effet, les travaux nécessaires pour l'ajustement sont tels que le plus souvent, au cas où ils seraient mis en oeuvre, provoqueraient de graves dommages esthétiques aux immeubles à préserver.

Interventions d'"ajustement" et d"amélioration" sur les bâtiments existants.

La situation d'incertitude a duré pratiquement jusqu'à 1986,lors de la promulgation le décret du 24/1/1986 contenant les normes techniques relatives aux constructions sismiques et aux interventions sur les bâtiments existants; cela fournit pour la première fois, comme on disait ci-dessus, une référence normative sur la façon dont il fallait effectuer les interventions aptes à augmenter la capacité de résistence aux actions sismiques des bâtiments existants.

En vertu de ce décret deux types d'interventions sont prévus:

a) l'ajustement, en tant qu'"exécution d'un ensemble de travaux nécessaires à rendre le bâtiment apte à résister aux actions sismiques";

b) l'amélioration en tant qu'intervention apte à atteindre un degré de sécurité supérieur pour des éléments structuraux isolés, "sans modifier de façon fondamentale le comportement global" du bâtiment.

L'ajustement est obligatoire lorsque l'on prévoit une surélévation ou un agrandissement du bâtiment; une incrémentation des charges d'origine pour un changement d'affectation, ou encore si l'on doit effectuer des travaux de trasformation globale visant à modifier même les parties structurales du bâtiment.

Les interventions techniques d'amélioration, prévues pour les bâtiments en maçonnerie, sont en réalité celles qui intéressent aussi bien l'architecture à valeur ambiante qui caractérise la quasi totalité des centres historiques, que les ensembles monumentaux, pour lesquels on avait trouvé dans le passé les plus grandes difficultés d'intervention, justement à cause de l'incertitude contenue dans la règlementation.

Dans les lois récentes on remarque un plus grand intérêt et surtout une sensibilité plus accentuée vis à vis des bâtiments en maçonnerie, tout en évitant, dans la mesure du possible, la "vérification sismique". Cela vaut pour les bâtiments destinés à l'habitation et, évidemment, pour les monuments aussi. Pour ces derniers il faut toutefois préciser que le défaut de la loi réside justement dans l'obligation de l'ajustement en cas de changement de destination d'usage de l'immeuble. En effet, comme l'on sait, dans la Charte d'Athènes (1931) l'art. 4 affirmait que: "pour les monuments soi-disant vivants doivent être admises seules les utilisations qui ne s'éloignent pas trop des affectations primitives, afin de ne pas apporter, lors des adaptations necessaires, des altérations essentielles au bâtiment"; dans la Charte de Venise (1964) le même article a subi une modification substantielle, et cela grâce surtout à la contribution apportée par Roberto PANE et Piero GAZZOLA dans la rédaction du nouveau document. (Pane, 1967). En effet dans ce dernier on affirme: "La conservation des monuments est toujours favorisée par leur utilisation pour des buts utiles à la société: une telle utilisation est souhaitable à condition qu'elle n'altère pas la distribution et l'aspect du bâtiment. Les adaptations pretendues par l'évolution des us et des coutumes doivent donc être maintenues dans ces limites". (art. 5). Dans les deux Chartes on souhaite que l'adaptation pratique ne nuise pas au complexe organique du bâtiment, cependant dans le document de Venise on souligne que la nouvelle fonction peut aussi bien s'éloigner de la primitive, ce qui n'avait pas été prévu précédemment. En effet Pane écrit que "des utilisations tout à fait différentes des originaires peuvent s'avérer beaucoup plus respectueuses de l'intégrité de l'oeuvre par rapport à la répétition de l'affectation initiale". (Pane, 1967, p. 27).

Voilà donc que des normes techniques relatives à des constructions sismiques (24/1/86), tout en préconisant l'"ajustement antisismique" pour des bâtiments qui changent de destination d'usage, en fait rendent obligatoire cet ajustement pour beaucoup de monuments - notamment des hôtels particuliers - dont la conservations dans les péculières formes architecturales est souvent garantie exclusivement par une utilisation différente de l'originaire.

Il est évident que pour les monuments, même en présence de modification de destinations d'usage, il ne faudrait qu'effectuer les interventions de consolidement minimales, aptes à garantir une amélioration sismique. D'ailleurs, dans les "Consignes pour l'application du D.M. 24/1/86" on affirme que la règlementation, étant donné la péculiarité de la matière qui ne peut être assujetie à règles rigides, n'a indiqué que des concepts fondamentaux "où chercher la solution plus adaptée au cas spécifique", tout en laissant "un vaste éventail de choix parmi les solutions projectuelles et les modalités techniques opérationnelles, en fonction des caractéristiques du bâtiment et par rapport aux interventions prévues".

Il convient d'ajouter que c'est peut-être à cause d'une telle liberté qu'au cours de ces dernières années l'on a assisté, notamment en Campanie, au recours à techniques d'intervention (injections de béton, grilles cimentées), murs armés, insertion de structures non visibles à l'intérieur de maçonneries) produisant soit l'élimination physique de la matière dont le monument est constitué, soit son bouleversement structural.

Interventions "lourdes" et interventions "légères". Consignes.

C'est justement dans la conscience que suite aux événements qui se sont produits ces dernières années en Italie, les interventions sur les ensembles monumentaux ont souvent été inutilement "lourdes", trop couteuses, pas toujours efficaces, exigeant parfois le recours à nouveaux matériaux dont on n'a pas encore expérimenté la durabilité et l'intéraction avec les matériaux d'origine, que le Comité Nationale pour la Prévention du Patrimoine Culturel du risque sismique a approuvé les "Consignes relatives aux interventions sur le patrimoine monumental à typologie spécialisée".

Dans ce texte on souligne l'importance de prêter, pendant les travaux de restasuration des monuments, "une attention particulière aux matériaux d'origine"; de faire une anlyse particulière de l'histoire sismique de la fabrication, en se rapportant aux prècédentes interventions effectuées après les tremblements; de faire appel aux contributions d'experts de différentes disciplines et, surtout, de faire "recours à techniques et à matériaux qui soient le plus proches possible des originaux, avec un examen critique interdisciplinaire sevère sur d'éventuelles interventions qui diffèrent de celles ci-dessus".

A ce propos on a récemment écrit (Croci, 1989) que le moment le plus important de toute l'oeuvre de restauration c'est la vérification de la sécurité dans l'état actuel "car c'est à ce moment-là que s'évidentie s'il faut effectivement intervenir et au même temps se définit l'entité des interventions elles-mêmes: faute de cette phase de l'étude, toute solution ne peut être qu'arbitraire et souvent même trop lourde, pour combler par l'éxubérance de la consolidation les carences de la connaissance".

La connaissance du monument en tant que premisse pour une restauration correcte.

Le moment de la connaissance est sans aucun doute le plus important au cours du projet de restauration et, notamment, la connaissance historique du bâtiment fournit une série d'informations utiles pour un diagnostic correct et, par conséquent, pour une proposition d'intervention qui soit valable.

La même loi (20/11/1987) relative à la consolidation des bâtiments en maçonnerie souligne que les critères à adopter dans le choix du type d'intervention doivent ressortir d'une étude préliminaire de l'ensemble de la construction qui concerne:

"a) les caractéristiques sous le profil architectural, structural, et de la destination d'usage dans la situation existante;

b) l'évolution des caractéristiques précitées rapportées notamment au bâtiment originaire et aux principales modifications apportées dans le temps;

c) l'analyse globale du comportement structural afin de vérifier les causes et le mécanisme d'éventuelles déformations en acte, notamment au niveau du soussol intéressé."

C'est ainsi que l'on sanctionne dans la loi elle-même ce qu'avaient suggéré les consignes précitées. Il est d'ailleurs intéressant de remarquer que pour le projet de consolidation, la loi indique aussi les opérations à effectuer en voie préliminaire; il s'agit d'opérations qui prévoient, pour l'investigation de l'état de fait, soit le relief qui permette d'individuer le schéma structural, soit l'évaluation des mesures de sécurité ainsi que des caractéristiques de résistence des éléments structuraux sur lesquels il faut intervenir, soit le choix projectuel des types d'intervention; enfin, la vérification de sécurité du nouvel organisme structural. C'est ainsi que l'auteur du projet doit se confronter aux problèmes de choix des sistèmes de consolidation les plus appropriés au bâtiment à restaurer - qu'il s'agisse d'une urgence architecturale ou d'un bâtiment à valeur ambiente situé dans une aire sismique - sans se limiter à la recherche de solutions qui garantissent une amélioration sismique, mais tenant compte aussi de l'importance de la conservation de la matière elle-même dont l'immeuble est construit. Le restaurateur doit d'autre part garder à l'esprit la nécessité de maitenir la stratification historique ainsi que la valeur esthétique du bâtiment, choisissant par conséquent les systèmes qui tendent à maintenir au maximum le schéma structurel originaire avec toutes les modifications qu'il a subies dans le temps.

Conclusions.

Au lendemain-même du tremblement de terre de 1980, des experts de sciences et techniques des constructions et de restauration des monuments avaient affronté le problème de la consolidation des structures en maçonnerie, soulignant dès lors la nécessité de la connaissance de l'histoire sismique du monument et indiquant une série de mesures certainement utiles pour l'amélioration statique des structures. Mais, malheureusement, neuf ans après le séisme, lors d'un bilan général, on relève que les interventions "lourdes" depassent, et de loin, celles qu'on avait appelées "légères". De plus, à la solution de garder autant que possible seuls les témoignages autentiques, en laissant même des bâtiments à l'état de rouine, on a préféré la voie la plus facile, qui est certainement la moins correcte, celle de la réconstruction intégrale, vanifiant ainsi tous les principes qui sont à la base des modernes restaurations.

Cependant, d'après ce qui a été jusqu'ici exposé, on en déduit qu'au niveau législatif et de consigne dans le domaine de la consolidation des bâtiments en maçonnerie situés dans des zones sismiques, l'Italie a apporté au cours de ces dernières années une considérable contribution, entreprenant entre autre une série d'études spécifiques qui couvrent dans l'ensemble la matière de la restauration.

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PHILOLOGY AND SAFETY IN VULNERABILITY MITIGATION OF HISTORICAL TOWNS IN SEISMIC AREAS: THE PRACTICE CODE FOR CASTELVETERE VILLAGE

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SUMMARY

This note is the methodologic introduction to the formulation of a "**practice-code**" for the conservative seismic protection of historical urban centres. The practice-code, compiled by M.Zampilli, V.Ceradini, F.Iacovoni, A.Pugliano, concerns the village of Castelvetere sul Calore, in the central region of Italy.

The analysis of the historical construction techniques, as well as the analysis of their seismic resistence and the detailings for a filological and safe restoration, have been dealed with, referring to the seismic expected intensity.

This short report has the aim to present an articulate procedure meant to achieve the knowledge of the historic masonry centres; knowledge directed to the purpose of attaining seismic safety within the preservation of the historical values. The way to chose the proper techniques, suitable to fulfill such double requirement, will be presented too.

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1. *LOCAL SEISMICITY*

The probabilistic definition of the seismic risk is proper to the cost-benefit analysis. It can be carried out only if an effective relationship between loss and seismic intensity has been formulated.
That is not always the case when an historical centre not recently struck by the earthquake is examined.
If a program for a preventive strengthening is wanted the relationship loss-intensity should derive by a numerical analysis of the structural seismic response, but all of us know that such result is completely irrealistic. From the other side, the statistical analysis carried out somewhere after the earthquake can't be applied to a different town, as we find evident later.
It follows that, putting on one side the probabilistic approach for a possible subsequent implementation, a proper research shall be carried out, directed to collect the most quantity of local informations. It is the same analysis that the seismologists use to carry out for the macro-seismic assessment. Documents could be found useful for the microzoning of the urban area, but at least the historical maximum and the most frequent intensity are to be pointed out.
The italian village of Castelvetere was very seldom struck by the earthquake, in spite it is within a strongly seismic area. Not only the documents reporting damages suffered by the nearby villages often don't mention it, but its buildings are the oldest of all the area. The strongest earthquake it experienced was the VIII degree and this information is enough to specify the goal of the preventive strengthening.
If a maximum of X degree had been found we should have needed a more frequent value in order to split in two levels the interventions: no-damage for the frequent earthquake and no-collapse for the expected maximum.

2. *ANALYSIS OF THE URBAN AREA AND DEFINITION OF BUILDING TYPOLOGY*

Who has experience of reading an urban web does not hesitate recognizing on the planimetry of landed property the tracing of the original courts, the most ancient form of surrounding a 15 by 30 metres land and building a house inside. To recognize the courts is also to point out the original building, usually oriented toward South-East, and then the following of the later building.
That gives data on the bond of the walls.
Of course the matrixes of the urban web are not only the courts; aligned houses about 5 - 6 metres large, disposed along the route constitute an other well recognizable ancient typology. The frame of the streets is one of the most persistent testimony of the original layout. Today it plays an important seismic role during the emergency traffic.
The little streets without outlit that creep within the ancient courts today filled with houses are not safe ways for runaways. They are forced on the main street.
It could be useful to connect two streets through a court: in this case it is better to open a barrel-vault that keeps the idea of the original court, than to demolish a little house loosing that memory. But in order to avoid such error, the architect needs to distinguish a court by a barred street.
The history of the expansion of the urban area shall be studied too. Very often it is possible to point out, within the urban centre, areas built in well defined temporal periods. Materials, techniques, detailings, within the area, are quite homogeneous, and that is an extremely useful information regarding the possibility of generalizing the observations.
In addition to the notes on the growth of the urban centre, the house typologies will be pointed out, with their synchronic variants and the diachronic evolution. The changements introduced in the original feature by the improved requirements in the inhabiting follow typical ways, easy to single out.
The apparent tangle of lanes and little houses built the ones on the others is disentangled and

it discloses its simple, foreseeble, repeated logic.
The actual structure features are quite correlated to that logic, and can be easily singled out following the typological analysis.

3. *READING OF THE LOCAL CONSTRUCTIVE TECHNIQUES AND THE RELATED MECHANICAL CHARACTERISTICS*

From the analysis of the urban morphology the study goes on down to the detailing of the structural elements.
We need to observe from the inside masonry, floors, roofs, ... The surveyor must look for yards with work in progress.
The masonry walls are always characterized by the local stones.
In Italy we find frequently masonry made by big pebbles cut in half, or walls made with stone chips. In Castelvetere we found a generalized usage of the same kind of stone, irregularly cut always with the same dimensions. That usage makes the characteristic of the wall dependent in a strange way by its thickness.
The main criterion for a well made masonry, the "rule of art" of the historical masonry is the following:
1. More big stones than little ones.
2. Careful bond among the stones through the thickness of the wall.
3. Filling of the space between the stones with little stones and bricks.

In a good masonry the mortar plays an inessential role. The strength is achieved by the wise joint of the stones. The mortar is called to contribute to the strength when the masonry is lacking in bond, but in such case it is very difficult that it succeeds in making up that lack.
The rule of art, therefore, in substance concerns the laying of the stones. It has been so clearly formulated in the treatises of the end of the last century that we can assume that definition as an objective reference.
The main mechanical characteristic of a masonry wall is not its compression strength, but its ability to get the collapse through the realization of kinematic mechanisms, with concentrated rotations.
A bad quality masonry is unable to make active the kinematism: the lack of trasversal bond makes it to disintegrate at the beginning of the motion.
The different quality of the three kinds of masonry walls found in Castelvetere is esaily pointed out in comparison with the "rule of art", and their different mechanical behaviour is evident. The best wall is 50 cm thick.
The anlysis made by the surveyor recognizing the typology of the masoney has an objective meaning in terms of mechanical behaviour, and then it is scientifically valid for the safety verifications.
Of course the floor texture shall be observed, and the roof too.
Usually we found in the most hauses the same detailings, all of them with the same defect. All of them can be strengthned using the same procedure.

4. *ANALYSIS OF THE SEISMIC VULNERABILITY: DAMAGE MODES AND SAFETY.*

First of all some preliminary remark on the seismic behaviour of masonry buildings is based on the definition of macroseismic intensity as reported in the Modified Mercalli Scale.
The VIII degree intensity is measured by the "partial collapse of some houses".
The IX degree intensity, instead, gives rise to "total collapse of some houses; many others remain so strongly cracked than they can't be inhabited any more".
The urban centres referred to by De Rossi and Mercalli, are just the historical towns we are studying now. Two or three stories houses, with a network of masonry walls of 5 or 6 metres; the same masonries we know how to examine and to judge from a mechanical point

of view comparing them with the rule of art, the same floor texture, the same roofs. But above all the same quality that, in average, we find rather far from the good rule of art. Giving credit to those experts we have to conjecture that the bad masonry walls, transformed into heap of stones by the X degree earthquakes at Sant'Angelo dei Lombardi, or at Venzone, only in a very little part would have failed for an VIII degree event.
According with Mercalli's definition, it is just the IX degree that, more deeply than our expert surveyor, points out the worst walls and knocks them down. Only the worst ones, infact only "some houses" suffer "total collapse". The "many others" which will be strongly cracked by the IX degree are not structures built according to the rule of art, if we have noticed that the most part of our ancient towns significantly defect the rule of art.
It can be asserted that the IX degree is the threshold of resistence of the mean historical masonry, that is to say the bad quality masonry. A masonry structure, built according to the rule of art gets over, uninjured, the IX degree, and suffers significant damage only by the X degree, as it has been demonstrated by the cathedral of Sant'Angelo dei Lombardi.
These assertions have an experimental origin that must not be underestimated.
In addition they are extremely important as orientation in the seismic verification of historical centres, but too statistical to be used as reference for the single building.
We need further mechanical considerations:
the masonry walls of the historical buildings are, regarding seismic action, in a position similar to the unstable equilibrium.
If the seismic intensity is less than VIII degree, if the quality of the masonry is not the worst and the structural typologies have been not too alterated, their stability is not affected.
Beyond this threshold the safety can not be guaranteed: it is most probable that cracks appear and kinematic mechanisms arise. Nevertheless as it happens in the unstability phenomena, if the arising of such mechanisms is prevented, the first threshold of resistence is easy overcame. The bulding is now able to resist an higher value of seismic action, and it will fail according to a new "mode". The first mode of collapse concerns the rotation of the walls out of their plane. The second mode engages and breacks the walls in their proper plane. The first mode leads to a ruinous failure, the second one provokes diagonal cracks through the whole walls, but very seldom such dislocations, as far as they are heavy, go onill the collapse.
It is some more useful to notice that in the first mode, like for the unstable equilibrium, the cause of the failure is not the masonry strength, but the lack of connections, whereas the second mode is condicioned just by the resistence.
Now following statements can be pointed out.

a) the analysis of the building shall be directed to point out the masonry quality and the collapse mechanisms;
b) the restoration must introduce ties as necessary to control the arising of such mechanisms;
c) if a X degree earthquake is expected the good quality (according to the rule of art) must be achieved.

The analysis of the possible damage follows the typologic characteristic of the buildings and in some way it can be generalized. Will present in the following some examples.

1. The roofs not organized in trusses can produce damage on the top of the walls, and can pull out them.
2. The urbanistic increase of the town, as the infilling of the courts, often goes on putting houses
mear preexistent houses. In such cases the masonry box of the late house is uncomplete. The external wall is subject to come off along the non bonded edge.
3. Joining more original cells often the new owner puts down one of the transversal walls in order to achieve a larger room. The external wall remain free for more than 10 metres and its stability is strongly reduced.

In a similar way, looking at the actual consistence of the buildings, it is not difficult to point out dangerous defects, and, at the same time, to find the way of controlling them.
What has been said is sintetically contained in the table I

TABLE I

1. DAMAGE FORECAST

seismic intensity	description of expected damage
VIII (MM)	- Some collapse of very bad masonry; - No damage in ordinary buildings.
IX	- Total collapse in masonry strongly far from the "rule of art"; - Important cracks in well made buildings where the "first mode" mechanisms are make easier by local defects; - No damage in building according to the "rule of art" and without local defects; - No damage in ordinary buildings if the "first mode" collapse has been controlled;
X	- Nearly certain collapse for buildings not reinforced, even if built according to "rule of art"; - If the "first mode" collapse has been controlled more or less damage can arise according to the quality of the masonry, but total collapse is generally avoided.

2. EXPERIMENTAL DATUM

The quality of masonry is, in average, rather far from the "rule of art", and many local defects are present in the structures

3. STRENGTHENING STRATEGY

max expected intensity	preventive measures
<VIII	Nothing
VIII	control the "first mode" vulnerability in cases of bad masonry or local defect
IX	control the "first mode" vulnerability in every buildings
X	control the "first mode" vulnerability and make sure the good quality of the masonry

5. *THE DETAILINGS FOR RESTORATION AND SEISMIC PROTECTION*

The actual feature of the historical buildings offers two kinds of problems.

a) Defect of egreement with the "rule of art"

b) uneffectiveness of the same "rule of art".

The first case is frequent in masonry. How poor the building of the poor people has been, is pointed out by the earthquakes. They have used unsuitable stones, like the little pebbles ill-assorted, or have put stone over stone without bond. In such cases the restoration is impossible. The only way, really effective, would be to demolish the wall and rebuild it according to the rule of art.

That is an historical formula, it is still used in some village of Italy, where some people are practising the ancestral aptitude to build their own house.

To make a masonry wall again is not different from making a roof again, and that requires the same skill than to make a trough. Today, nobody is interested in trough, or roof, or wall, and these workmanships are no more included in the popular culture. But today we have got the ability to organize every productive process. If we are interested in preservation of masonry towns and safety of their inhabitants, we can organize the production of stone or brick walls, making in advance the opportune experimentation, like we did for the vaults of the cathedral of Sant'Angelo dei Lombardi.(*)

Nevertheless, not always rebuilding the wall is the only way to improve its quality. In some cases a good result can be achieved by injecting cement. When it is possible. That is when the masonry has been built with two good exterior faces badly infilled by poor material and many voids. In such case, injecting cement after a good washing it is possible to achieve some connections between the two faces, and to introduce, like bonding stones, hard masses of cement.

In different cases, like in the masonry of little pebbles, the injections are uneffective, and the bad quality can't be improved.

The second problem arises when the "rule of art" is originally uneffective. It seems strange that the "rule of art", deriving by millennia of experimentation, can be uneffective. Against all the natural actions, except the earthquake, the construction rule of art had got the best solutions, as it is demonstrated by the ancient buildings we have still in use. But the earthquake has been always removed by the conscience of the technicians, as an unnatural calamity.

Only in XVIII century, after the disasters of Lisbon in 1755 and of Calabria in 1783, the architects started looking for the remedies.

It is interesting to notice that after some attempt to design the "anti-seismic" house, as Vivenzio did in 1785 in Naples, the authors of architecture treatises did not want specify the anti-seismic measures. The good "rule of art" must be able to make the house resistant against all the natural actions, included the earthquake.

The use of steel ties to connect the walls each other is widespread for all the XIX century. It is the application to the strengthening of a new rule introduced in the building technique during the XVIII'century. The new rule consists in inserting steel bolts within the masonry during the construction, in order to improve the cross connections. At the beginning of the XIX century Rondelet recommends such technique, spread to all the length of the walls.(**)

Since today we are not able to think any techniques but reinforced concrete, we use to apply that suggestion transforming it in reinforced concrete tie beams.

It is evident that there is not utility in inserting reinforced concrete, with its stirrups and the steel bars arranged at its four corners near the surface, exposed to the corrosion, when we can put the steel inside the masonry. It will be more protected and then more effective.

(*) Ortolani,F; *Effetti del terremoto sulle strutture laterizie e volte,* in Palladio N. 2, 1988,pag. 143.

(**) Rondelet,G.; *Trattato teorico e pratico dell'arte di edificare*, Mantova 1732, trad. it. di B Soresina.

The way to control the kinematism is just that. The walls collapse toward outside, because toward the inside they are contrasted by the floors, then it is enough to keep them, enchaining each other the opposite fronts.
The whole XIX century has reinforced the masonry buildings by this way. It is a device beyond the structural lexicon of the Middle Ages, but it is its natural extension; it has been consacrated by an historical usage, and then it can be philologically adopted.
There is not a rigorous criterion for dimensioning this ties. The wood beams of the floors, which contain the rocking of the wall toward inside, will be able to keep it toward the outside if they will be transformed in tie-rod with steel stirrups through the masonry.
The steel ties applied in XIX century very seldom are more than 20 mm in diameter: this dimension can be kept in current buildings. The steel bars can be arranged within the floor, in the filling between wooden boards and flooring.
When the roof is to be renewed it is very useful to improve the top of the walls replacing the last 40 or 50 cm with a brick-work laied on lime mortar, containing inside a steel tie anchored at the ends, according to the suggestion of Rondelet.
In this way a good support is provided to the roof beams too.
The wooden floors shall be kept, or rebuilt in wood again.
Their elasticity, the insulation provided by the infilling, the pleasant ceiling give to the way to live at home a precious quality there is no reason to loose.
The idea that reinforced concrete floors provide solid diaphragms able to distribute the seismic force to the vertical elements comes from the practice of elastic framed structures, and it is valid only in that field.
Fragile concrete slabs would not be able to control the stiffness and the mass of walls 50 cm thick, as in historical buildings, and their connection to the masonry walls provides more damage than improvement.
As it has been demonstrated the masonry box, connected with ties, is resistant enough.
The criterion for the restoration is elementary and general: to prevent the motion toward the outside by steel or wood ties; to prevent the motion toward the inside by wooden or masonry contrasts.
As long as the rule of art has not been heavily disregarded, such devices make the buildings vulnerable only by more than IX degree earthquakes, with "second mode" type damage. That means that cracks and sliding on the plane can arise during a very strong earthquake, but a ruinous collapse will be avoided.
The requirements of a healthy politics of seismic prevention are fulfilled, and that can be achieved acting within the constructive lexicon we want to preserve.

FIG. 1

MASONRY OF ELEVATION

DESCRIPTION:

Hand made with local stones

dimension of the stones: 25-35 x 12-18 cm.

Thichness: a) cm. 40; b) cm. 50; c) cm. 60

BEHAVIOUR:

a) trend to extrude the external little elements
b) trend to maintain the geometry of the cross section and consequent rigid rotation
c) trend to separate in dabble walls and consequent streugth reducion

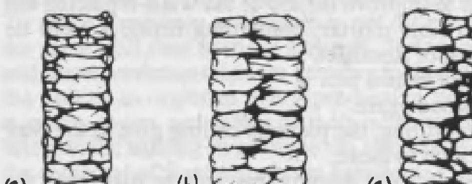


FIG.2

TYPOLOGY: "profferlo":

archetypal rural house with outside stairs

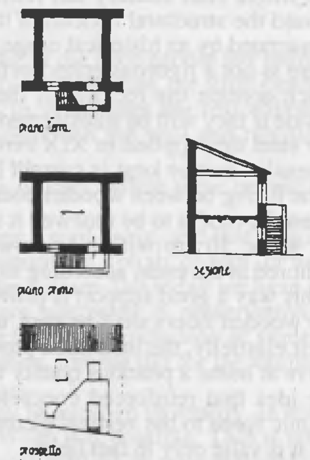
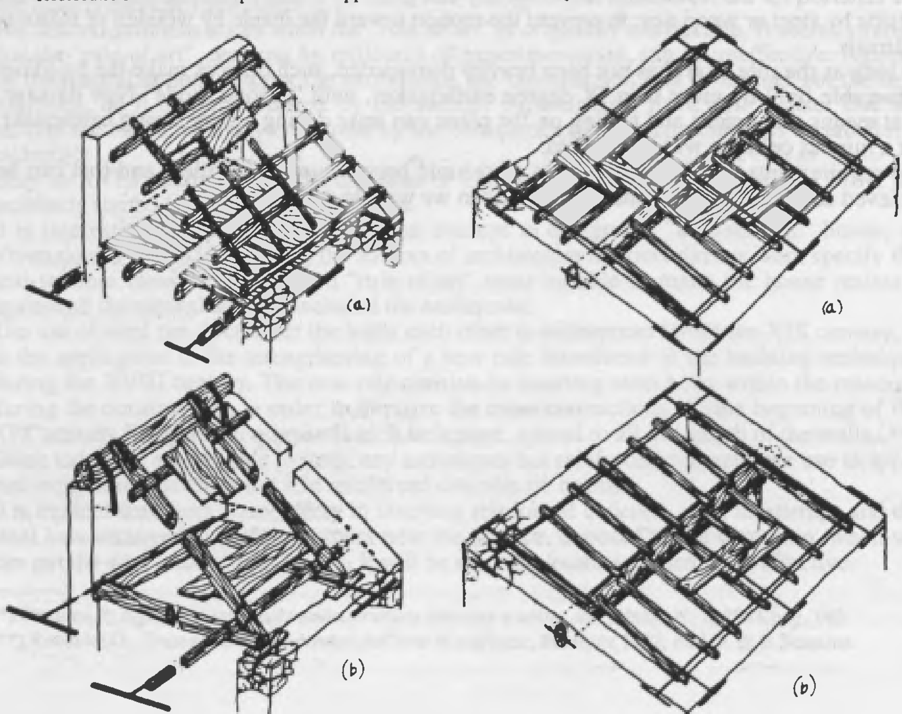


FIG.3

Constructive details roofs and flors typical in Castelvetere.

Restoration and seismic protection: opposite walls connected by steel ties or wood ties



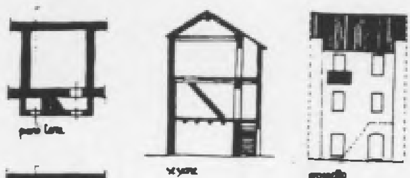


FIG.4

<u>TYPOLOGY:</u> "profferlo" ortiginal house with additions: closing the outside stairs

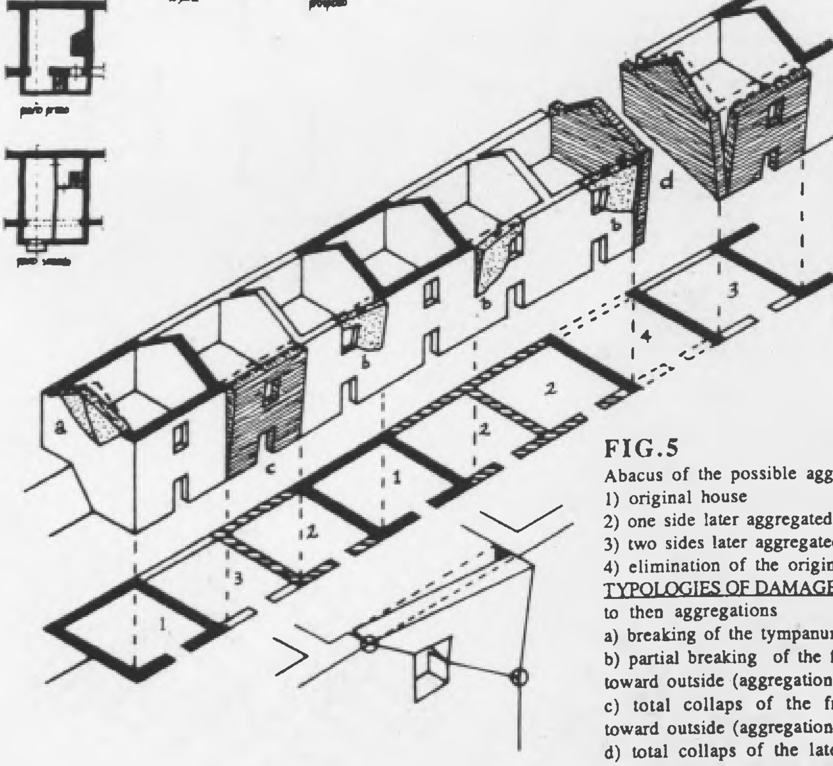


FIG.5

Abacus of the possible aggregations
1) original house
2) one side later aggregated house
3) two sides later aggregated house
4) elimination of the original house

<u>TYPOLOGIES OF DAMAGES</u>: related to then aggregations
a) breaking of the tympanum wall
b) partial breaking of the front wall toward outside (aggregation 2)
c) total collaps of the front wall toward outside (aggregation 3)
d) total collaps of the lateral walls toward the passage (aggregation 4)

FIG. 6

Masonry tie beam reinforced by one steel bar connected to the wooden structures of the roof by steel clamlps

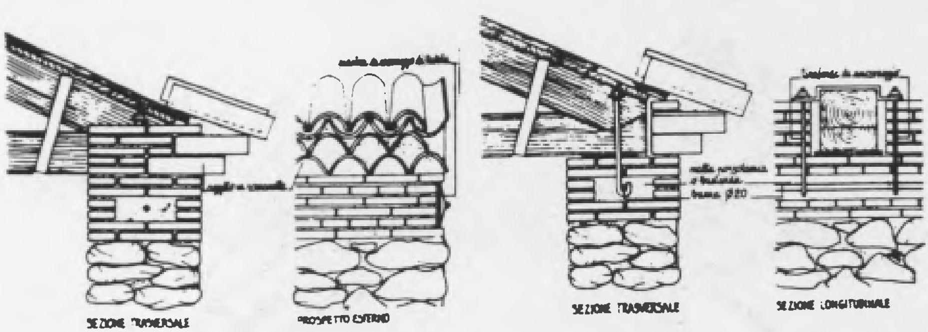


FIG. 1

DESCRIPTION:

stone made with local stones

dimension of the stones: 25-35 x 12-18 cm.

Thickness: a) cm. 40; b) cm. 50; c) cm. 60

BEHAVIOUR:

a) ... little elements

b) ... try of the cross

c) ... and consequent

FIG. 3

Restoration and ...

(a)

(b)

32 $

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