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Session 4

Remedial Measures

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Compatibilité de la thérapie avec le monument

Verträglichkeit Instandstellungsmassnahmen mit dem Monument

Appropriate Measures for the Preservation of Monuments

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RÉSUMÉ

Le monument, héritage que l'on doit transmettre à la postérité mais soumis aux vicissitudes du temps est une substance vivante et vulnérable dont la durée de vie ne peut être éternelle. L'utilisation de techniques et de matériaux nouveaux de plus en plus performants constitue un précieux auxiliaire pour la conservation de l'édifice. Leur mise en oeuvre ne peut cependant s'envisager qu'à partir d'une connaissance approfondie du monument et dans le plus grand respect de son authenticité. Par prudence à l'égard du monument, le caractère de réversibilité à donner à ces interventions doit s'imposer.

ZUSAMMENFASSUNG

Das Baudenkmal als ein der Nachwelt zu erhaltendes, aber den Wechselfällen der Zeit ausgesetztes Erbe, ist ein lebendiges, verwundbares Gebilde, dessen Lebensdauer nicht ewig währt. Ein wichtiges Hilfsmittel für die Gebäuderhaltung bilden die modernen, laufend verbesserten und leistungsfähigen Baustoffe und Techniken. Deren Einsatz erfordert jedoch gründliche Kenntnisse des Bauwerks und das Bewusstsein seiner Einmaligkeit. Aus Vorsicht sollten notwendige Eingriffe auch rückgängig gemacht werden können.

SUMMARY

A monument is a part of our cultural heritage to be preserved for posterity. Yet being a living and vulnerable object, submitted to time and weather influences, its life expectation cannot be forever. The use of new techniques and constantly improved materials represent useful tools for the preservation of buildings. Their application requires, however, a sound appraisal of the monument and respect for its uniqueness. A careful treatment of the monument requires of all work undertaken the character of reversibility.

Tout monument est soumis au cours des âges aux mutations de son affectation et de l'usage qui en découle. Il est également perpétuellement mis au goût du jour suivant l'évolution de la mode et des convenances, des principes d'habitabilité, des notions d'utilisation et de confort. Il en résulte que chaque édifice est l'objet de constantes modifications tributaires des vicissitudes du moment et de la destinée de son proche environnement. Constitué de surcroît de matériaux altérables dont la durée de vie est limitée par les données climatiques -nous restaurons aujourd'hui en France, les restaurations de la fin du siècle dernier !- l'assiette même de l'édifice peut reposer sur un sol dont la force portante n'est pas celle existant à l'origine de sa construction soit à cause d'un caprice de la nature, soit à la suite des effets induits par l'aménagement de ses abords.

De même que tout corps vivant renouvelle ses cellules, le monument est, hier comme aujourd'hui, indissociable de l'histoire contemporaine dont il reflète l'évolution. Créé par la main de l'homme, il ne peut être éternel pas plus que le paysage naturel ou construit qui l'environne. Situation paradoxale dès lors que notre civilisation commande l'impérieuse conservation du monument dont il convient d'assurer pleinement la pérennité afin de transmettre aux générations futures le précieux témoignage d'une culture universelle intégrant celle de tous les temps, depuis l'époque la plus reculée jusqu'aux temps contemporains.

Incarné par le monument, le leg culturel du passé prend une valeur d'autant plus inestimable que celui-ci se rapporte à un unicum portant la marque du créateur et de toute l'équipe qui, autour de lui, a réalisé l'oeuvre. La sauvegarde de l'authenticité ou de ce qu'il en reste, essence même du témoignage transmis à la postérité, est assurément le paramètre le plus délicat à prendre en compte et le plus difficile à traiter pour tous ceux qui ont la charge de la conservation du monument. Comment en effet conserver le leg culturel du passé incarné dans la matière alors que celle-ci est vulnérable, voire éphémère ?

L'entretien portant plus particulièrement sur les problèmes relatifs à la structure des monuments, nous n'engagerons pas le débat sur tout ce qui touche à la restauration et à la réutilisation du monument. Nous nous cantonnerons seulement aux conditions de l'indispensable maintien des structures et exigences de stabilité du monument sans lesquelles celui-ci ne peut être viable. Nous ne perdrons pas de vue en outre qu'il convient d'obtenir la meilleure adéquation possible entre les modes d'intervention à mettre en oeuvre et le rigoureux respect de l'authenticité de l'édifice à conserver.

Aucune intervention ayant pour objet d'assurer la stabilité d'un édifice existant ne peut se justifier sans bien sûr mener au préalable une reconnaissance approfondie de celui-ci et de sa structure, sans quantifier les contraintes qui l'affectent, sans analyser, à partir des documents d'archives et examen de la construction, l'évolution des conditions de son équilibre au cours du temps et au gré de son affectation. La mise au point des méthodes d'analyse et de calcul les plus appropriés restent fondamentales pour obtenir la meilleure compréhension possible du monument et déterminer le jeu des forces et contraintes qui l'animent.

Il est aisé de comprendre que les anciens, en l'absence d'instruments d'analyse performants et d'une approche scientifique du problème, ont dû se fier à leur propre intuition et expérience. Les méthodes de consolidation et restauration alors adoptées ont consisté en un remplacement d'une matière devenue inopérente par une nouvelle autant que possible identique. Les exemples sont multiples. Ils concernent pratiquement toutes les réparations réalisées sur les édifices au cours des siècles précédents par l'intermédiaire du procédé des incrustations de pierres à l'identique dans les maçonneries en remplacement des parties hors d'usage ou fissurées.

Mais lorsqu'un renforcement de structure s'avère indispensable, à l'élément défaillant insuffisamment calibré, se substitue un autre élément d'une nature ou d'un aspect qui n'est pas forcément identique. Le meneau en pierre placé dans l'axe de la rose du bras nord du transept de la cathédrale de Tours, les renforts sous la forme d'arcs complémentaires en pierre soutenant les grandes arcades du carré du transept de la cathédrale de Wells, les chemisages de maçonnerie qui entourent les points d'appuis fissurés comme à l'abbatiale de Saint-Avit Senieur ou Saint-Seurin à Bordeaux sont autant d'exemples très significatifs dont l'efficacité ne répond pas forcément aux problèmes à traiter. Il s'agit en fait de palliatifs dont le résultat sur le plan du respect de l'intégrité du monument est contestable et aujourd'hui considéré comme inacceptable tant les modifications apportées à l'aspect de l'édifice sont importantes, dénaturent celui-ci et portent atteinte à son authenticité.

L'apparition et l'utilisation de matériaux nouveaux plus performants ouvrent la voie à de nouvelles perspectives. Déjà l'utilisation du fer pour armer la maçonnerie traditionnelle en pierre était largement pratiquée aux XVII° et XVIII° siècles. Les exemples de la colonnade du Louvre par Perrault et du Panthéon par Rondelet et Soufflot sont célèbres. Ils avaient à l'époque provoqué des controverses et un long débat doctrinaire. Bien loin de présenter le même statut d'éternité qui était alors attribué à la pierre, le renfort métallique intégré à la maçonnerie apparaît être un expédient constructif jouant le rôle d'un simple moyen de sauvegarde et non pas constituer un élément fondamental de la construction. C'est en somme l'utilisation d'un artifice considéré à l'époque comme impur à l'égard de la maçonnerie traditionnelle.

Fort intéressante est l'apparition à partir de cette même époque de l'utilisation de matériaux de substitution venant en remplacement des éléments constitutifs de la construction traditionnelle. C'est ainsi que la mise au point de mortier et mastic permettant de composer des "pierres factices" évite de mettre en oeuvre les nécessaires incrustations de pierre à l'identique sur les maçonneries malades. De la même manière, l'utilisation de la fonte de fer semble apporter une aide bénéfique pour les restaurations monumentales. La reconstruction de la flèche de la cathédrale de Rouen en 1823 par Alavoine en est un premier exemple. La matière est changée mais les formes de l'architecture gothique préservées. Venant à l'appui de son projet, Alavoine précise "qu'on aurait tort de s'astreindre à n'employer que les procédés d'exécution en usage dès le XIII° siècle et de se priver des ressources nouvelles que nous procure le perfectionnement des arts industriels".

Cette réflexion d'Alavoine est toujours d'actualité. Prenons l'exemple de l'usage du béton armé ou de celui du métal qui sont apparus non seulement comme des techniques novatrices pour l'architecture contemporaine mais encore comme de précieux auxiliaires pour la conservation des monuments anciens.

La réfection des charpentes avec ces matériaux que ce soit par exemple pour la cathédrale de Reims ou celle de Chartres illustrent bien les utilisations bénéfiques de ces techniques lorsqu'elles ne portent pas atteinte à l'aspect du monument. Les avantages que procure l'usage de ces matériaux et leurs performances par rapport à la maçonnerie traditionnelle ont par ailleurs très rapidement mis en évidence tout l'intérêt de leur emploi pour traiter les problèmes de stabilisation de l'équilibre d'un édifice.

Les reprises en fondation par pieux en béton ou métal ou association de ces deux matériaux en sont un exemple. La mise en oeuvre de prothèses actives ou passives en béton armé afin de recentrer les charges et mettre ainsi un terme aux désordres provenant d'une modification de l'équilibre du monument est naturellement utilisée dans le souci d'une consolidation durable modifiant le moins possible la nature et l'aspect du monument. Il va sans dire que ces techniques délicates à mettre en oeuvre exigent un projet très élaboré fondé sur une connaissance approfondie de l'édifice à traiter.

Mais l'utilisation et l'adaptation de ces procédés se heurtent aux difficultés d'appréciation liées aux problèmes de vieillissement et d'assimilation par la maçonnerie existante.

Nous connaissons aujourd'hui les redoutables problèmes qu'il importe de traiter au Panthéon pour conserver l'oeuvre de Soufflot. Pour la flèche de la cathédrale de Rouen, les avantages que devait à l'époque offrir la fonte (matière homogène, indestructible, d'un poids relativement faible et d'un coût modéré) sont apparus très rapidement si illusoires qu'une vingtaine d'années plus tard, Viollet le Duc s'est opposé à son achèvement. Cela l'a convaincu que la restauration d'un édifice ne pouvait s'envisager que par l'intermédiaire d'une intégrale restitution de sa matière originelle impliquant un démontage complet et une reconstruction de l'ouvrage altéré.

Dès les premières utilisations du béton armé, de prudentes réserves étaient émises quant à l'insertion dans un édifice dont la structure est élastique et déformable, d'éléments rigides susceptibles de désorganiser son équilibre au lieu de le renforcer.

Nos connaissances en matière de structures anciennes et dans le domaine des matériaux nouveaux et traditionnels, avancent fort heureusement chaque jour. Sont-elles suffisantes pour autoriser une intervention faisant appel à des techniques non traditionnelles et que l'on veut efficaces, sans effets ultérieurs nocifs, aussi longtemps que le monument restera debout ?

C'est bien parce que des consolidations et restaurations mises en oeuvre dans le passé et qui paraissaient parfaitement adaptées se sont avérées ultérieurement décevantes et même périlleuses pour la sauvegarde du monument que la notion de réversibilité s'est progressivement imposée. A ceci, doit s'ajouter l'impossibilité de connaître à l'avance le devenir d'un édifice. Prenons l'exemple des temples pharaoniques de Nubie qui allaient être cycliquement inondés à la suite de la construction du premier barrage d'Assouan en 1902. Pouvait-on prévoir que toutes les consolidations faites alors avec beaucoup de soin mais faisant appel à un large emploi de ciment, deviendraient un handicap préjudiciable à la conservation de leur intégrité lorsqu'il a fallu démonter les édifices menacés d'une noyade définitive par la mise en eau du nouveau barrage commencé en 1960 ?

Nous pourrions citer également les toutes récentes interventions sur certains édifices prestigieux comme Saint-Sernin de Toulouse où les dispositions arrêtées par Viollet le Duc en 1860 et effectivement exécutées, n'ont pas été reconduites. Une telle décision a alimenté de longs débats sur les problèmes de la "dérestauration" qui en l'espèce, pour Saint-Sernin, permettait de répondre en outre à d'impérieures raisons techniques. Il n'était pas en effet envisageable de maintenir en place des dispositions vicieuses préjudiciables à la conservation du monument.

C'est bien toutes ces incertitudes qui militent pour la réversibilité des interventions rendant possible un retour aisé à l'état antérieur ou la réalisation d'une modification technique plus pertinente rendue possible par le progrès de la science. On comprend dès lors immédiatement à quel point les solutions légères qui peuvent facilement être soustraites pour être remplacées par d'autres plus performantes, sans porter atteinte au monument, restent avantageuses et ne peuvent avoir que la préférence des restaurateurs.

L'apport de nouvelles techniques sera à cet effet toujours recherché et le bienvenu, dans la mesure où ces dernières permettent des interventions douces non seulement parfaitement respectueuses de l'intégrité du monument mais aussi susceptibles d'être bien assimilées par la morphologie et la substance de ce dernier sans faire l'objet d'un rejet ultérieur.

A cet égard, l'utilisation et la précontrainte de la maçonnerie pour le recentrement des charges, ou de techniques sophistiquées d'injections des maçonneries pour augmenter leur résistance doivent faire l'objet de recherches approfondies afin de les rendre plus efficaces et mieux adaptées à la restauration des édifices anciens. Garantir la respectueuse survivance du matériau originel et les conditions de stabilité grâce à de nouvelles approches scientifiques reste une ambition légitime pour ceux qui ont la charge de la conservation des monuments.

Les progrès apportés dans le traitement de la statuaire et extrapolés aujourd'hui à l'ensemble d'un portail de cathédrale puis à la totalité d'une façade d'église comme celle de Saint-Mexme de Chinon ou de Notre-Dame la Grande à Poitiers sont autant d'encouragements pour parvenir à la mise au point de thérapies nuancées et complexes mais de plus en plus performantes.

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Decision Process for Monuments

Procédure de décision pour les monuments

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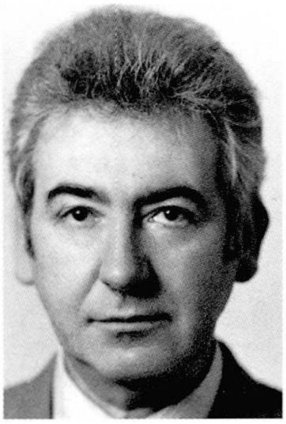

SUMMARY

The authors report on the main problems examined by the National Committee for the Seismic Protection of Monuments and Italian Cultural Heritage; such problems concern the seismic vulnerability of the whole cultural heritage and the guidelines to adopt for the measures directed both to the seismic protection and the preservation.

RÉSUMÉ

Les auteurs exposent les problèmes principaux qui ont été récemment discutés dans le Comité National Italien pour la Protection Séismique des Monuments. Il s'agit de la vulnérabilité séismique de tout le patrimoine monumental et des critères à appliquer pour assurer aussi bien la protection séismique que la conservation.

ZUSAMMENFASSUNG

Die Autoren berichten über die Hauptprobleme, welche kürzlich im nationalen Komitee zum Schutz der Baudenkmäler vor Erdbeben behandelt wurden. Diese Probleme betreffen die seismische Verwundbarkeit des gesamten italienischen Kulturguts und die Kriterien, nach welchen Eingriffe zum Schutz vor Erdbeben und zur Erhaltung der Bauwerke vorzunehmen sind.

1. INTRODUCTION

Italy is a country where neither earthquakes nor monuments are lacking. They are often present in the same area, with interferences, as amply documented by the History.

By now there is a general conviction that the Cultural Heritage our ancestors have handed down to us through Architecture, has to be conserved for our successors as integral as possible. It is equally evident that the seismic protection of constructions avoids the often irreparable damages that the earthquakes may cause. These two statements seem to mean an obvious syllogism, according to which the interventions directed towards the highest degree of seismic protection should also assure the greatest possibility of preservation. Yet we know it isn't so, since the interventions are often heavy, intrusive and irreversible and they may alter remarkably the signs of material culture present in the building.

These already complex problems are further complicated by the fact that by now we still know very little about the real vulnerability of ancient buildings and about the actual seismic hazard of sites where they are situated.

Two facts, quite different from each other but both equally relevant, must be mentioned. The first regards the ancient constructions whose reserves of resistance are progressively diminishing due to the decay, which in a state of complete neglect and general carelessness is made more accelerated. The second concerns the high risk present in some monuments which, either because of their capacity to give rise to interest or due to their present functions which often differ from the original ones, are subjected to great and uncontrolled concourses of persons.

The debate about the monumental heritage risen in Italy in recent years, is based upon these and other considerations. But in order that such debate would lead to really tangible consequences, it is necessary that the already existing knowledge will be enriched with a vast amount of researches and experiences on the field.

While describing such researches and experiences and developing some critical and methodological considerations, we shall here pay a particular attention to researches aimed at supplying information on a territorial scale, indispensable to define policies for the conservation of the cultural heritage.

The presentation will be divided in two parts, dealing with:

a) the methodologies for the assessment of seismic vulnerability and risk of the architectural patrimony;

c) the intervention strategies on the patrimony.

In Italy the National Committee for the Seismic Protection of Monumental Building and Italian Cultural Heritage (CNPPCRS) has the institutional task of promoting researches aimed at the systematic collection of data on architectonic, cultural heritage exposed to seismic risk and at the individuation of evaluations on its seismic vulnerability and on the intervention provisions.

This description of the present State of the Art and of the tendencies admitting of developments, even if it reflects personal opinions of the authors, intends to present some results and trends of the Committee.

2. SEISMIC VULNERABILITY AND RISK OF MONUMENTAL BUILDINGS

It is long since the necessity of collecting systematically all the possible information relevant to the Italian monuments has been felt, so much that there exist a Central Institute of the Ministry of Cultural and Environmental Heritage, ICCD (Central Institute for the Cataloguing and the Documentation) founded for this purpose; such an Institute has prepared and uses a series of forms regarding the knowledge relevant to the architectonic heritage as well as to the other kind of cultural property. Anyhow, what has been done till now is not yet a System, able to permit the management of the information with such a systematical way and efficiency that is required today and is possible using the modern computing instruments.

It is important to observe that the above-mentioned problem cannot be solved simply by proceeding to the computerization of the existing instruments, even if they are valid; what is necessary to do is quite more difficult in that one should put together conceptions, needs, different disciplinary languages, mostly in a large context of different institutions, organizations and conditions. That is why first of all it is necessary to conceive a national system, on the level of a conceptual form, in order to verify, through a discussion relevant to a concrete planning hypothesis, if what one is proposing corresponds to the different needs and expectations.

The mentioned National Committee has carried out a discussion regarding the general lines of such a problem [02,03,10,13,14] and moreover has predisposed a first instrument of work consisting in a form for the first level survey of the seismic vulnerability of monumental buildings [16,17]; such a form should be considered only as a part of a much larger series of operations, precisely the above-mentioned National System.

The fundamental elements to propose for the System can be the following:

- systematic collection of data by means of coordinated forms or equivalent surveying methodologies, possibly computerized and easy to load; among these forms and methodologies are included the existing ones;
- hierarchical ordering of such instruments and their coordinated use (forms of I level, of II level, of expert levels, possible monitoring, etc.);
- systematic loading of data on computer, in order to create a national data bank easy to use by the operators;
- setting up of a practice of programmed control, that follows the phase of surveying, for the continuous updating of the data;
- study and preparation of programs for the management of the data in order to supply for:

  - "friendly" systems of access and readout of the data;

  - general intervention plans, indicating the priorities and estimations of the costs;
  - punctual evaluations of hypothesis of intervention;
  - ...
- systematic updating of data, once the interventions have been carried out, accounting for a general planned maintenance of the monumental patrimony as a whole.

On the ground of the above described definition the proposed system has been called SISTEMA NAZIONALE PER LA **CA**TALOGAZIONE, IL **RI**LEVAMENTO, LA **S**ORVEGLIANZA E LA **MA**NUTENZIONE PROGRAMMATA DEI MONUMENTI: CA.RI.S.MA. (National system for the cataloguing, the survey, the control and the planned maintenance of monumental buildings) [17].

The form of the first level for the monumental buildings, both churches and other kinds of buildings, applicable in seismic and non-seismic areas, conceived in the optics of the System CA.RI.S.MA, is directed to gather information which can be subdivided in 12 classes:

- data for the identification of the monument;
- synthetic description;
- present destination of use;
- position in the environmental context;
- soil and foundations;
- state of maintenance;
- crowding;
- structural seismic history;
- geometric and material description;
- presence of cracks;
- decay;
- interventions carried out.

The form is completed with some figures, usually 2. The first reports a general planimetry and the second reports plans and significant sections of the monument.

Later on, the gathered data can be loaded in a database by means of a suitable program, including the use of a scanner for the drawings and possible vectoring. A software for the loading of the data directly on site by means of a Laptop is a useful alternative; such a software, called CADING in the English version, has already been prepared.

The data management and elaboration present quite difficult and delicate problems. Among the most important exigencies of the possible users we can mention at least the following ones:

- consultation of the forms, one by one or according to topics, for generic information;
- simply to work out and to compare historical, typological, quantitative, numerical, functional data;
- assessments of vulnerability;
- risk evaluations;
- proposals of priorities of intervention;
- development of intervention strategies;
- studies of hypotheses for intervention plans;
- singling out of situations requiring close studies;

- singling out of situations requiring monitoring;
- studies regarding the coordinated use of monuments.

Most, if not all, of the above problems, can be solved developing methodologies and/or models and/or decision processes to be implemented as Expert Systems. A first System of this kind has been proposed to evaluate the seismic risk and supply the elements that serve to create some scales indicating the priorities of intervention [20,21]. The proposed method is based on the following elements:

- maximum importance given to the "structural seismic history";
- in the "most fortunate" cases the same history leads directly to the evaluation of the risk, graduated only in three levels: high, medium, low;
- in other cases the evaluation of the risk requires the previous assessment of the seismic vulnerability of the monument (also this latter on three levels);
- assessment of the above vulnerability by means of typological comparisons with other monuments included in the database;
- maximum attention to the decay and its development;
- distinct consideration of the risk regarding the people (RP) and of the risk regarding the conservation of the monument (RM);
- consideration of an external risk (RE), connected with the position of the monument in the environmental context;
- consideration of the "value" of the monument, again on three levels (high, medium, low), for the evaluation of the priorities;
- definition of the priorities by means of graduated classes (three: high, medium, low, or otherwise five: very high, high, medium, low, no intervention) by cross comparison of the data relevant to seismic risk, decay and value;
- continuous distinction, up to the final output (included), of the single factors that concur to the definition of the priorities, in order that the mechanisms which take to the proposals for decisions were always clear.

The implementation of the method as Expert System [20,21] has been conducted using the shell NEXPERT OBJECT of the Neuron Data.

3. GUIDELINES FOR INTERVENTIONS ON MONUMENTAL BUILDINGS

The discussion within the Committee on how to increase the seismic safety of monuments and ensure the goal of preservation has both produced many individual contributions and papers [01,04,05,08,09, 11,12,15,18,22,23] and two official consecutive documents, first a text of "Recommendations" [06], then a text of "Guidelines" [07].

The main consideration which is at the basis of such documents is the following: today the engineers are used to operate on new constructions, that they themselves design, verify and construct according to principles and technical codes aiming to assure, first of all, the security; but when the engineers are called to deal with a monumental building they must confront two specific circumstances that should modify completely the approach to the problem:
a) the monument is an existing building;
b) the monument is a building with an "identity" and a "value" that should be conserved. The architects that deal with the

monuments are well acquainted with such circumstances: the science of restoration, in fact, has to do with the relevant problems; but when the security is in question, the different exigencies often come into conflict that can be substantial and not likely to be resolved. Such conflicts can also depend on the fact that the above principles and technical codes, thought as they are in view of the new constructions, do not consider the specific problems relevant to the existing buildings in general and the monumental buildings in particular. That is why it is necessary that the interventions on monuments, whenever they imply problems of security, will be confronted with specific principles and regulations, studied and defined on purpose.

In the Guidelines [07] such principles are the following:

1) The estimate of seismic risk concerning values exposed in the monuments should be part of an overall risk estimate that examines all the important hazards.

2) The estimate of risk should take into account, rationally, all the available sources of knowledge about the monuments and their environmental and sociocultural context with special reference to the historical source.

3) The balancing of the seismic risk concerning cultural values, as that of other risks, should be considered not as an insolated objective but as one of the exigencies to satisfy in the multidisciplinary context of monument restoration.

4) The restoration of monuments should be inserted in a continuous temporal process of programmed monitoring and maintenance

5) Interventions on monuments should tend, in principle, to abate vulnerability added by degradation.

6) Decisions concerning interventions which modify the intrinsic vulnerability of monuments must emerge from a very strict interdisciplinary confrontation.

7) If the diminution of risk to human lives through the reduction of the vulnerability of a monument involves interventions detrimental to the cultural values, this diminution is to be instead found through uses of the monument which reduce the exposure of those lives.

8) Interventions on monuments should allow our successors to express restoration and preservation cultures different from ours.

9) Ways to intervene implying a wide use of traditional techniques are firmly counselled for monuments.

Such principles seem to be very similar to those proposed in Skopje [24] and by the State of California [25], and they have been proposed inside a Project Team working for the preparation of the seismic EuroCode 8 [19].

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Strengthening of Masonry Structures by Lateral Confinement

Renforcement de construction en maçonnerie par remplissage latéral

Verstärkung von Mauerwerksbauten durch seitliche Umfassung

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SUMMARY

The research deals with the quantitative evaluation of the safety increment obtainable through a lateral confinement of masonry structures, made with steel plates and bars. It is expressed as a function of parameters, such as dimensions of bars and plates and mechanical properties of masonry. Specimens from ancient masonry walls have been tested, obtaining different failure modes and variations of strength, deformation and energy absorption capacity. Suggestions are made on strengthening criteria to be used in practice, in relation to applied loads, design limit states and required safety factors.

RÉSUMÉ

La recheche traite de l'évaluation quantitative de l'augmentation de sécurité obtenue par un remplissage latéral de constructions en maçonnerie, réalisé à l'aide de plaques et de tirants en acier. Elle est exprimée en fonction de paramètres tels que dimension des barres et plaques et de la propriété mécanique de la maçonnerie. Des spécimens tirés d'anciennes murailles ont été testés, permettant d'obtenir plusieurs modes de rupture, en fonction des variations de résistance, de capacité de déformation et d'absorption d'énergie. Des propositions traitent les critères de renforcement en pratique, basées sur les charges appliquées, les états limites de projet et les facteurs de sécurité exigés.

ZUSAMMENFASSUNG

Die Forschung befasst sich mit dem Quantifizieren der erhöhten Sicherheit, die bei Mauerwerksbauten mittels lateraler Verstärkung aus Stahlplatten und -spangen erreicht wird. Dies geschieht in Funktion von Parametern, wie Grösse und Dicke der Platten und Spangen sowie der mechanischen Eigenschaften des Mauerwerks. Proben von alten Mauerwerkswänden wurden getestet und je nach Festigkeit, Verformungs- und Energieabsorptionsvermögen unterschiedliche Brucharten beobachtet. Praktische Verstärkungskriterien richten sich nach den aufzunehmenden Lasten, Grenzzuständen der Bemessung und geforderten Sicherheitsfaktoren.

1. FOREWORD

1.1 Philosophic choices for strengthening monuments

In recent years numerous problems arised about basic principles and actual techniques for strengthening monumental buildings, having become apparent that the great majority of the materials used in the past have only a very short life with respect to the life of the structure. As a consequence it is now believed by most researchers that the potential reversibility of the intervention has a key role in the evaluation of the appropriateness of the strengthening technique, reversibility being defined as the possibility of restoring the situation preceeding the intervention.

1.2 Strengthening by lateral confinement

The possibility of increasing the strength (or in general the safety against collapse) of elements essentially subject to vertical loads by a lateral confinement constituted by prestressed bars and confining plates is of particular interest, the only permanent (non-reversible) result being some minor hole in the masonry texture.

From a qualitatively point of view it is evident that a confinement will increase the safety of the structure, and the basic mechanisms which will allow such an increment are qualitatively clear, as well. Nevertheless many questions remain without an answer if a precise evaluation of the safety of the strengthened structure is required, as a function of appropriate parameters of the original structure and of the strengthening intervention.

An interesting application of this technique has been applied by one of the authors to the case of a medieval masonry tower: details are presented in [1].

1.3 Objectives and methods

The main objective of the research presented in this paper consists in the determination of the increment of strength, deformability and energy absorption capacity obtained as a consequence of a lateral confinement of a masonry wall, as a function of appropriate parameters, such as ratio between bar and plate area, ratio between distance between two plates and wall thickness, and expected failure modes.

To pursue this objective twelve wallettes obtained from medieval masonry walls have been tested, varying the parameters mentioned above in order to obtain three possible failure mechanisms:

- yielding of the horizontal bars;
- punching of masonry underneath the confinement plates;
- shear-tensile failure of masonry in the less confined zones between plates.

The large scatter of material properties together with the limited number of available specimens would have not assured a dependable interpretation of the results through a correlation of different tests on different specimens. Each specimen has therefore been loaded at maximum load, strenghtened, and loaded to failure, then considering percentual increments rather than absolute values of the properties of interest.

A more detailed presentation of the research can be found in [2].

3. DESIGN OF THE EXPERIMENTAL TESTS

3.1 Properties of materials

The masonry walls to be strengthened had been taken from the debris of a medieval tower failed in 1989 [3,4]. The basic material is rather a conglomerate made with pieces of clay bricks and river stones enbedded in a lime mortar matrix, then a standard brickwork; such material is common in ancient structures, where thick walls present a regular brickwork only in the outer skin.

The ultimate strength was in the range of 2 to 4 MPa, with Young modulus varying between 700 and 4600 MPa and ultimate deformation of 0.3% to 0.5%. A significant cracking process starts at

horizontal deformations of 0.02% to 0.08%, while the ultimate horizontal deformation is between 2.5% and 5%. These values imply that the plastic deformation of confining steel bars should take place in a highly non-linear range of the masonry behaviour, with two positive consequences:
- the initial prestressing tension of the bars should not affect the results, since the corresponding strain is neglegible with respect to the potential lateral expansion of masonry;
- the properties of the steel bars are fully exploited, either in terms of strength and in terms of energy absorption capacity.

The steel used for the reinforcing bars had a yield stress of about 600 MPa and a uniform elongation capacity (i.e. at maximum force) between 3% and 4%.

3.2 Failure modes

As already mentioned three fundamental failure modes have been considered, as shown in figure 1. For the purpose of some preliminary estimation of the failure mode to be expected, a bidimensional behaviour was assumed, imposing a perfect confinement in the third direction. These conditions could correspond to those of a wall with width significantly larger than thickness, and have been reproduced in the tests applying much stiffer bars and plates covering the whole side in one of the horizontal directions (see figure 2).
The relative probability of bar yielding versus punching of masonry depends on the ratio between bar section and plate surface, as well as on material properties.
Assuming a yield strength of steel bars (f_{ys}) of 600 MPa and a punching strength of masonry of 6 MPa (f_{pm}; it has to be obviously higher than compression strength), an equal probability of failure is obtained for a ratio of 100 between area of plate (A_p) and area of bar (A_b). For this reason squared plates with side of 40 and 80 mm have been used in conjunction to 6 mm diameter bars, obtaining A_p/A_b = 57 and 226.
A numerical evaluation of the probability of the third failure mode (i.e. shear-tensile failure of unconfined masonry) is more difficult, since it depends on mechanical properties of masonry which are not sufficiently known; it is nevertheless clear that the ratio between plate distance and wall thickness can be assumed as a fundamental parameter. Values of 0.5 and 1 have been assumed for this parameter, since 0.5 is considered to be a lower limit, hard to prescribe in a real case.

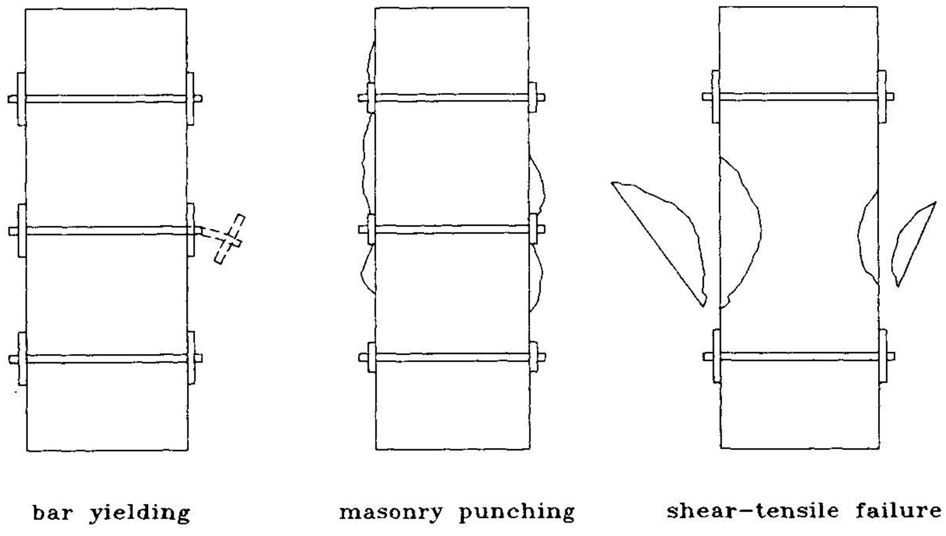

Fig. 1 Possible failure modes. From the left hand side: yielding and fracture of bars; punching of masonry beneath the confining plates; shear-tensile failure in the unconfined region

3.3 Test setup

The approximate dimensions of the wallettes were 650x700x290 mm, with a vertical load applied on the 650x290 sides, and the prevoiusly mentioned "perfect" confinement on the 700x290 sides.
On the 650x700 sides 4 or 9 plates were applied, tied with 4 or 6 mm bars located through holes very carefully drilled. A slight tensile stress (70 to 300 Mpa, depending on number and diameter of bars) was applied to the bars before testing the strengthened specimens, obtaining an average compression stress in the masonry of approximately 0.05 MPa; as already mentioned this was not considered a fundamental parameter.
All bars were instrumented with strain gauges, displacement transducers were applied to measure vertical and two horizontal deformations (see figure 2).
Each specimen had been first loaded to its maximum capacity, then unloaded to 70% of maximum load, confined by tying the plates up and stressing the horizontal bars, and finally loaded to complete collapse. Clearly all the specimens were significantly damaged before being strengthened.
It is worth mentioning that all the bars had originally a diameter of 6.5 mm, and only in the region of application of the strain gauge were milled to 6 or 4 mm; as a consequence the confining force capacities were significantly different in the two cases (of about 50%), but the corresponding total elongation of the bars was approximately the same.

3.4 Expected results

An estimation of the expected increment in strength and deformation capacity has been tempted on the base of energy considerations, assuming yielding in the steel bars and equating their strain energy to the increment of the energy absorption capacity of the masonry panels. Such an approach has been used in the past to evaluate the effects of steel confinement on reinforced concrete members [5], but it is questionable whether other dissipation mechanisms, such as friction in cracks, would not be greatly affected by the presence of confinement, therefore increasing the energy absorption capacity of the specimen. The very low reinforcement percentages contribute to raise this concern.
Assuming the materials behaviour as shown in figure 3, the total energy absorption capacity of the wall can be approximately evaluated as 1400 kN mm, the energy absorption capacity of each reinforcing bar as 140 kN mm. The energy increment should therefore be estimated as 40%, for the case with 4 bars, and 90% for the case with 9 bars, provided that the assumptions mentioned above are applicable and that the final collapse corresponds to a contemporary fracture of all bars. The energy increment could correspond to different combinations of strength and deformation capacity variations.

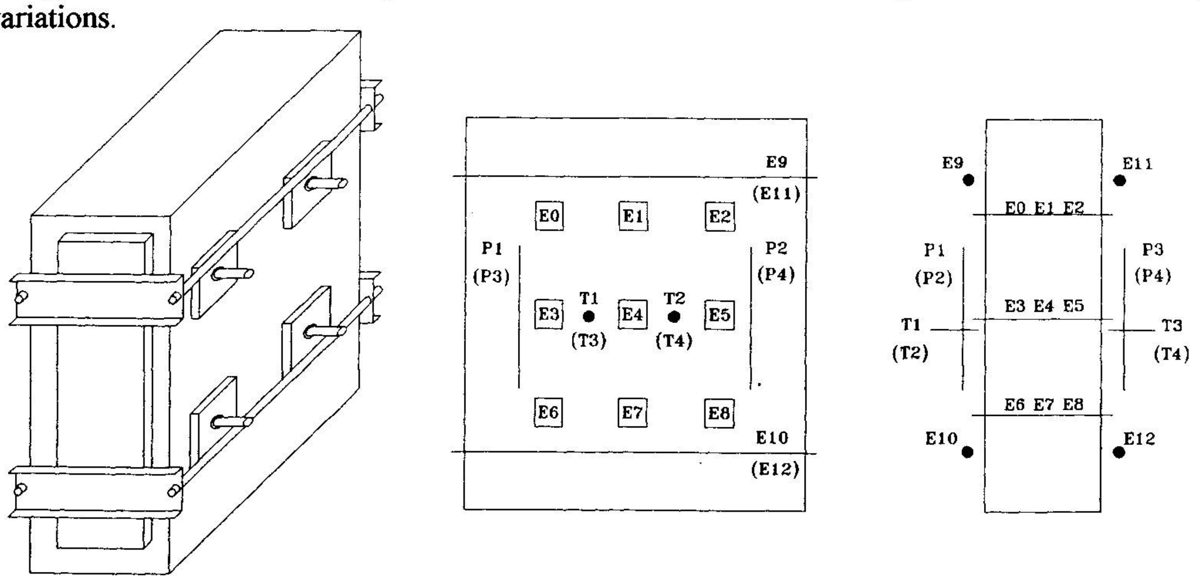

Fig. 2 Test setup (P: linear potentiometers, T: LVDT, E: Strain gauges)

4. EXPERIMENTAL RESULTS

4.1 Failure modes

The failure modes experimentally obtained corresponded substantially to the expected modes:
- in three wallettes strengthened with four bars a shear-tensile failure took place in the central (less confined) region. Only in one case one bar fractured. It has to be noted that the shear-tensile failure is strongly affected by the heterogeneity of the masonry conglomerate, it is therefore logical to expect various response in term of post-peak branch as well as in term of strength, deformation and energy absorption capacity;
- the specimens strengthened with nine 80x80 mm plates and 6 mm bars shew fracture of one or two bars followed by a rapid strength deterioration;
- punching of masonry has been detected in the cases with 40x40 mm plates. The strength deterioration progressed gradually to a final shear-tensile collapse of the no more confined masonry;
- eventually in the case of nine plates and 4 mm bars an apparently contemporary fracture of four bars took place, with an immediate total collapse of the wallette.

4.2 Strength

The unconfined specimens shew a very uniform strength (average 3.66 MPa, c.o.v. 0.13), probably because all of them had been obtained from the same block of material, therefore being characterized by a random distribution of pieces of bricks and stones, but by the same mortar matrix as well.
On the opposite the strength increments varied from 4% to 43%, with an average of 22.7%.
The greater variability was encountered, as expected, in the case of four plates (i.e. shear-tensile failure), with increments between 4% and 25%.

4.3 Deformation capacity

The recorded Young modulus (E_s, taken on the straight line to the point at 70% of the unconfined strength) was also quite uniform (2854 MPa, c.o.v. 0.22). No initial hardening due to compaction of voids and closing of microcracks has been detected.
The ultimate deformation capacity of the unconfined specimens was equal to 0.24% (average, c.o.v. 0.26), with an equivalent "ductility" (μ_u, defined according to figure 4) of 1.76 (c.o.v. 0.12).
The ductility calculated for the strengthened specimens (μ_c) was never lower than 7.18, with an average of 9.92. The ultimate deformation was therefore equal to at least five times the original ultimate deformation, no matter how the walls had been strengthened.

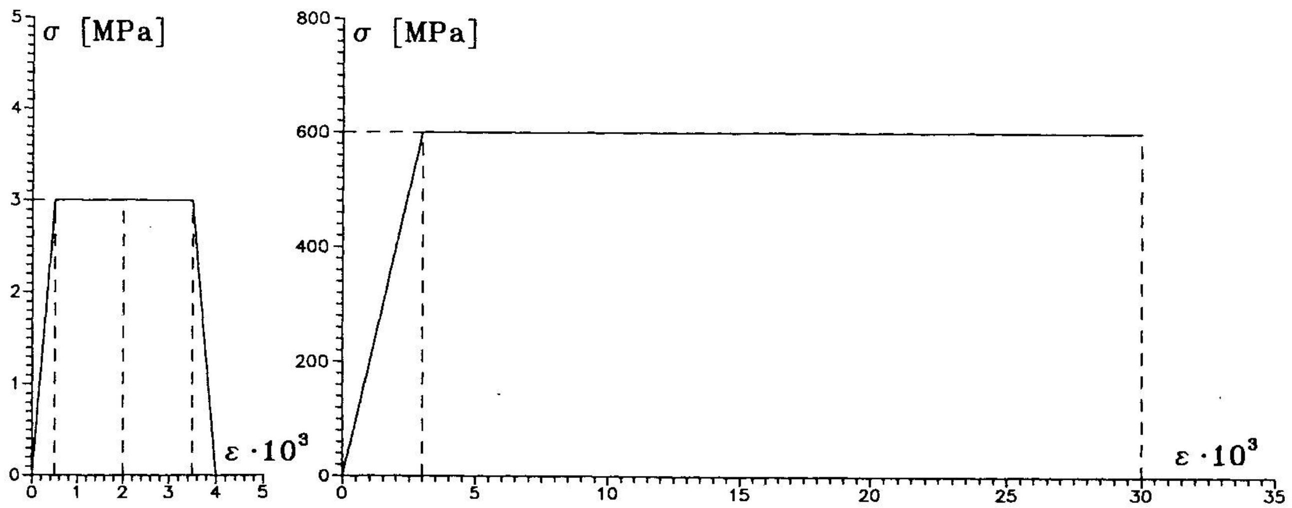

Fig. 3 Assumed material behaviour. Left: masonry; right: steel bar

4.4 Energy absorption capacity

The energy absorption capacity of the stengthened wallettes is, on the average, 5.5 larger than in the unconfined conditions (c.o.v. 0.37), as expected from the figures given on strength and deformation capacity. This results clearly indicates that the strain energy of the bars has little to do with the increment of the total energy absorption capacity. Actually if a detailed evaluation of the energy absorbed by the bars is performed values from 30 to 300 times smaller than the increment in energy absorption capacity are found.
Strengthening through confinement is therefore very convenient from an energy point of view, since a negligible amount of energy effectively added allow the exploitation of resources hidden in the original structure. For the same reason it appears that the increment in the energy absorption capacity is relatively unsensitive to modes of failure and confinemement details.
The increment of energy absorption capacity is probably related to phenomena of friction and aggregate interlock.

5. CONCLUSIONS

5.1 Strengthening criteria

The experimental investigation confirmed the expected failure modes; it is therefore reasonable to conclude that ratios between plate and bar area of about 100 separate cases for which bar yielding and punching of masonry have to be expected. For different materials the assumptions adopted in section 3.2 can be applied.
The experimental results also suggest that a shear-tensile failure is not likely to take place if the distance between plates edge is smaller than the thickness of the wall. This result may be significantly influenced by scale effects and texture of the outer skin of the wall; it has therefore to be applied with caution. For large scale structure and good masonry brickwork skin it is felt that it may result in very conservative solutions.

It has to be recommended to avoid the shear-tensile failure mode, because of the unreliable prediction of the improved behaviour after strengthening.
The punching of masonry failure mode partially compensates a less dependable behaviour up to the maximum stress with a slower strength deterioration.
It can be concluded that it is important to limit the distance between plates edge, while the ratio between plates and bars area can be maintained around the separator value.

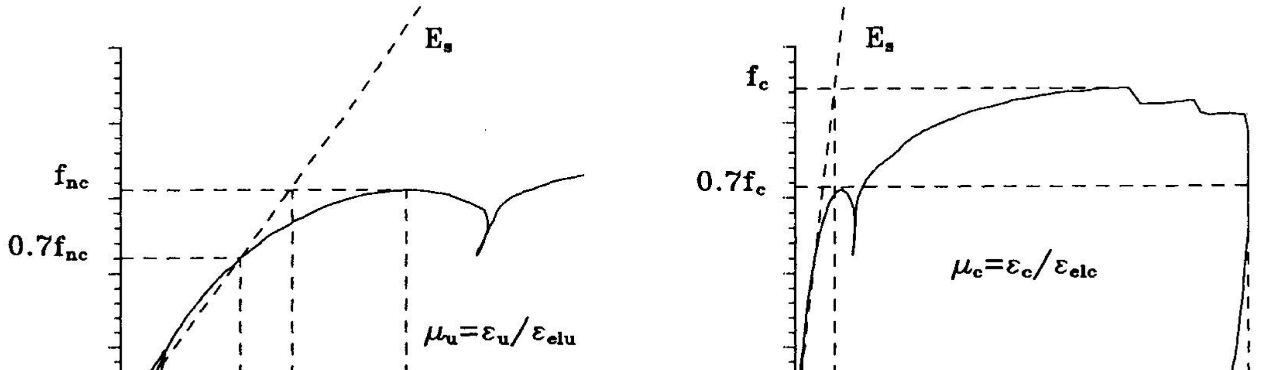

Fig. 4 Typical stress-strain curve obtained from the tests (the definition used for stiffness, equivalent elastic limit and ductility are shown)

The experimental investigation has shown that for the given materials increments of 20% in strength and of 500% in deformation capacity can be assured if proper details are used. It is then necessary to evaluate the variation of the safety of the structure which corresponds to these or other increments. For the seismic case it is common practice to translate available ductility into force reduction factors, this case will not therefore be discussed here in details. It is worth anyway to mention that a ductility of 5 could correspond to force reduction factors of the order of 3 for regular structures, the overall seismic safety for the specific case of the masonry towers from which the specimens were taken could be improved of about four times through a proper confinement.
If long duration or simply gravity loads constitutes the main concern it is still possible to obtain great benefits from ductility if the material strength varyies significantly from point to point. To make the point assume that a tower is made with three perfect materials randomly distributed, with strength equal to 2, 3 and 4 MPa. The benefit of an elastic-perfectly-plastic behaviour versus an elastic-brittle behaviour can then be estimated in an overall strength increment equal to 50 %, as shown in figure 5, which gives a total safety increment equal to 80% when combined with the a local general strength increment equal to 20%.

The most important effect of the strengthening technique here presented lies therefore in allowing stress redistributions without local collapse. An implicit confirmation of its selective effects (function of the local stress and damage level) can be learnt from the results of dynamic identification tests performed on a medieval tower before and after strengthening [6]: while the natural frequencies remain substantially unchanged, significant variations have been detected in the modes of vibration.

5.2 Future research development

The evaluation of the safety increment of a masonry tower deserves more refined studies then what presented above. For this reason extensive parametric finite element analyses are planned. The relevance of the material distribution and the possible definition of "critical volumes" for which abnormal behaviours have to be expected are of particular interest. This study involves probabilistic simulations, or at least statistical sensitivity analyses, with 3D non linear elements.

The abundance of experimental data related to the behaviour of masonry conglomerate under tridimensional states of stress and strain suggests the possible development of a constitutive model based on full strain and stress tensor. It is planned to explore Mohr-Coulomb-type constitutive relations, essentially derived from geotechnical models, as well as relations based on functions of the stress (strain) tensors invariants.

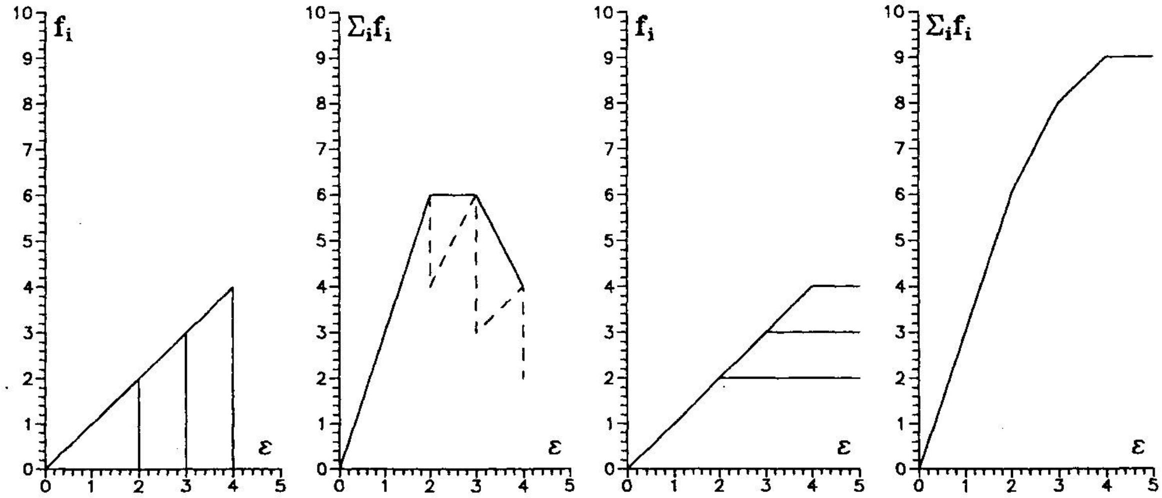

Fig. 5 Comparison of the overall behaviour shown by systems made with materials of different strength working in parallel. Left: elastic-brittle; right: elastic perfectly plastic.

n_p		4	4	4	4	6	6	6	6	6	6
l_p	[mm]	80	80	80	80	80	80	40	40	80	80
f_b	[mm]	6	6	6	6	6	6	6	6	4	4
Df/f_u	[%]	25	23	18	4	27	33	17	43	25	12
μ_u		1.83	1.48	1.71	1.46	1.76	1.87	1.89	1.67	1.71	2.25
μ_c		7.9	9.1	8.2	9.3	9.6	17.6	7.2	11.6	8.2	10.3
E_c/E_u		3.7	5.8	4.2	4.5	5.3	9.9	3.4	8.6	4.8	4.7
$\Delta E/E_s$		167	145	205	29	53	73	106	337	98	131

Table 1 Summary of the experimental results. Each column corresponds to a tested wall, with the indication of (from the top) number of plates, side of the plates, bar diameter, percentual increment of strength, equivalent ductility of the unconfined and confined specimens, ratio between absorbed energy of the confined and unconfined specimens, ratio between increment of absorbed energy and strain energy effectively absorbed by the the bars.

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Consolidation of 'Fill Layer' Masonry Structures

Consolidation des structures en maçonnerie avec matériau de remplissage

Festigung eines 'Füllschicht'-Mauerwerkbaus

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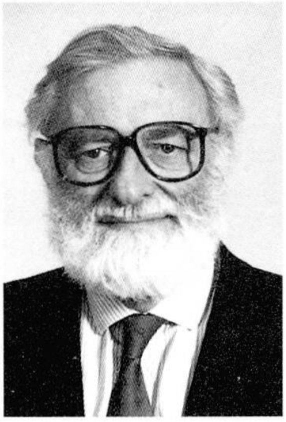

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SUMMARY

A new technology for the consolidation of "fill layer" masonry is proposed: the incoherent material is injected with an inorganic hydraulic binder and is confined by means of connectors with valves. Test results have been satisfactory and encouraging, showing increased strength and essentially unaltered stiffness after consolidation. The type and properties of the materials adopted also ensure the durability of the measure.

RÉSUMÉ

Une nouvelle technique pour la consolidation de la maçonnerie avec matériau de remplissage est proposée: le matériau incohérent est injecté avec des liants hydrauliques à base inorganique, à l'aide de canalisations dotées de vannes. Les résultats des essais sont satisfaisants et encourageants; ils montrent une résistance accrue et une rigidité pratiquement inchangée après la consolidation. Le type et les propriétés des matériaux adoptés assurent également la longévité de l'intervention.

ZUSAMMENFASSUNG

Vorgestellt wird eine neue Technologie zur Konsolidierung von 'Füllschicht'-Mauerwerk: Durch Einspritzen von anorganischem hydraulischen Bindemittel in Verbindungsröhren, die mit Ventilen ausgestattet sind, wird dem Lockermaterial eine Kohäsion verliehen. Testergebnisse waren zufriedenstellend und ermutigend. Nach dem Erhärten konnte eine gesteigerte Festigkeit bei grundsätzlich unveränderter Steifigkeit festgestellt werden. Art und Eigenschaften der verwendeten Werkstoffe garantieren eine dauerhafte Sanierung.

1. AIM OF THE INVESTIGATION

As is widely known, historical buildings undergo a structural deterioration process (micro-climate aggression, vibrations caused by vehicle traffic, heavy utilisation conditions) and it becomes necessary to take action in order to maintain adequate safety coefficients and to improve a building's load-bearing capacity in relation to actual utilisation conditions. When selecting an adequate consolidation technique, an essential factor not to be overlooked, is the need to forestall possible chemical-physical interactions between newly added material and the existing ones. Another prerequisite is not to alter the shape and appearance of a building. Finally, ease and speed of execution should also be taken into account.

In this investigation, special attention has been devoted to the so-called "fill layer" structures (whether piers, columns or walls), that is to say, structures consisting of facing walls (brick courses with joints of mortar, mostly of the air-hardening type) and a fill layer of incoherent material (building site rubble, such as sand, earth, brick fragments). When dealing with these structures, it may prove important to be able to consolidate this fill layer so that it will contribute to the enhancement of overall strength.

Account taken of these needs, a new technique has been developed to meet such needs. It consists of injections of hydraulic mortar (made of suitable inorganic substances) combined with the application of specially designed connectors to serve as confinement elements.

To this end, test pieces were produced from samples of historical materials: following an initial loading cycle carried on till an advanced state of cracking, the specimens were repaired and their strength was assessed through additional tests.

2. SUMMARY DESCRIPTION OF THE HISTORICAL MATERIALS AND TEST PIECES

2.1 Bricks

As was to be expected, the bricks, taken from the baroque structure of the Castello della Venaria Reale in Piedmont, revealed highly scattered geometrical, physical and mechanical characteristics which, however, were comparable to those gathered from the material in the State Archives of Turin (see [5]). Significant test results are listed in Table 1.

Compressive strength N/mm^2	nominal dimensions of area in compression 60 x 250 mm	nominal dimensions of area in compression 125 x 125 mm	
		dry	soaked
No. of specimens	8	8	4
mean value	14.79	10.78	6.01
s. d.	5.06	5.55	—

Table 1. Compressive strength [N/mm^2] (strength values were calculated by assessing the effective area in compression of each specimen).

2.2 Joint mortar

To simulate as closely as possible the composition and behaviour of historical materials, mortar was produced with sand and binder in a 4 to 1 ratio, the binder consisting of air-hardening lime and hydraulic lime in 1 to 1 proportions.

Table 2 lists flexural strength values as measured on specimens sized 40x40x160 mm and compressive strength values as obtained on each pair of test piece produced by the bending test.

Mortar strength N/mm^2	Bending		Compression		
	at 14 days	at 28 days	at 7 days	at 14 days	at 28 days
No. of specimens	3	3	6	6	6
mean value	0.46	0.63	0.41	0.45	0.59
s. d.	–	–	0.015	0.020	0.085

Table 2 Mortar strength [N/mm^2]

2.3 The test pieces

Fig. 1 clearly shows the shape and geometry of the columns. The fill layer was reproduced with rubble taken from the Castello della Venaria. Grain size is described in fig. 2.

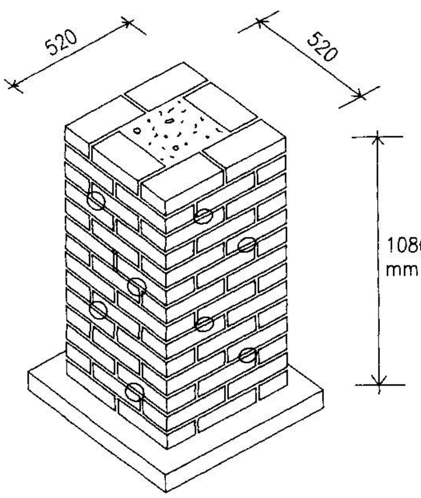

Fig. 1 Test column showing arrangement of connectors

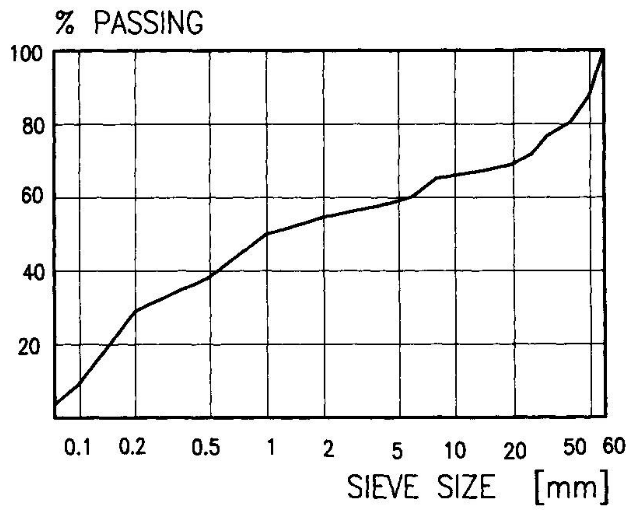

Fig. 2 Grain size of the fill material

3. DESCRIPTION OF THE CONSOLIDATION TECHNOLOGY AND THE MATERIALS EMPLOYED.

The consolidation process takes place in two distinct stages. The first consists in making holes in the structural element by means of a 50 mm diameter core drill. Through holes, spaced 45 cm apart, are made in the brickwork, and, to make sure they are not obstructed by fill material when the drilling equipment is pulled-out, a temporary pipe-shaped support is inserted, enabling the special connectors with valves to be inserted. After that, the elements are wetted with a mixture of water and surface-active agents so as to avoid the dilution and dissipation of existing mortar and to prevent soluble salts from surfacing. At this point, injection operations get underway, the aim being: a) to bind together the incoherent fill layer material, b) to provide anchorages for the connectors in the outer brickwork. Finally, the heads of the holes are filled with the material taken out, so as to conceal the damage in the best way possible.

3.1 Materials employed

The injected material is a hydraulic binder with an inorganic base which is chemically compatible with the original materials, especially with regard to the possible presence of sulphates, since the latter may trigger the deterioration/destruction of the brick-joint-binder chain.

Their salient physical-mechanical properties are:

- mix water: 37% - weight/volume: 1900 kg/m3
- pot life: > 2 h
- cone emptying time (according to the Italian Motor-way Authority's method for grouting material):
  at the time of mixing: 24.5 seconds after 30 minutes: 24.5 seconds
- hardening time:
  hardening begins: 16 h hardening ends: 20h
- mechanical strength (N/mm^2) at

	1 day	3 days	7 days	28 days
in compression	15	30	38	46
in tension by bending	2.8	4.0	4.4	5.6

- elastic modulus at 28 days: 1500 N/mm^2
- pull-out test (performed through the connector adopted): 7 N/mm^2

3.1.2 Connectors with valves

These consist of:

a) a pipe with valves, of DURVINIL PVC, 21 mm in diameter and long enough to match the element's size, fitted with three manchette-type 30 mm long polyester elastomer DUR-O-RING valves, with opening pressure < 1 bar;

b) two circular cross-section bars with nominal diameter of 5 mm, made of parallel aramidic fibres impregnated with an epoxy-based compound and coated with the same compound with the addition of fine silica sand (grain size between 0.125 and 0.25 mm);

c) two (or more) spacers-centering pieces of polypropylene, 37 mm in diameter and 60 mm in length.

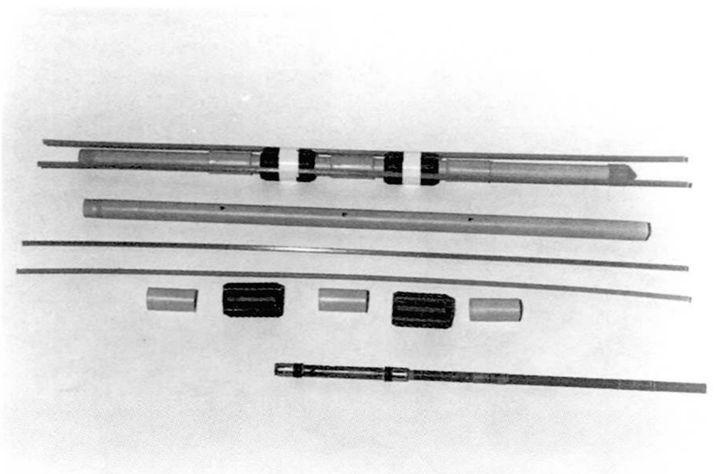

Fig. 3 Detailed view of connector

The physical-mechanical properties of the connecting bars are:

- specific gravity: 1250 kg/m^3

- tensile strength (referred to the section measured): 1150 N/mm^2
- elastic modulus: 56.3 kN/mm^2.

3.2 Specific consolidation technologies

The need to bind together the incoherent rubble filling the inside of the column (which includes fine grained material) made it necessary to adopt specially developed grouting techniques. The operation was performed from the bottom up in two stages. During the first stage, macro cavities were saturated with a 0.4 ratio water-binder mixture; after that, micro cavities were filled with 0.50 water-binder proportions.

Fig. 4 Inside view of consolidated column with the connectors in place.

The alternate use of individual valves, obtained with the aid of a special hydraulically sealed piston sliding inside the duct, made it possible to reach high pressures (up to 3 atm.) at the injection point without making any further damage to the test pieces.

Individual valves were used and - thanks to the grouting mixture's long pot-life - the injection time was graduated so as to be able to diffuse the mixture better and to control the quality of the materials introduced.

Confinement is obtained through a pair of bars - made of aramidic fibres impregnated with an epoxy based compound - featuring high mechanical strength. These bars are arranged on the outside of the injection pipes and are anchored by bonding to the two opposite facing layers. The presence of reinforced connectors is shown in fig. 4, where it is also possible to observe the effects of the consolidation of the fill material.

4. TESTING PROCEDURE

4.1 Specimen manufacture

The materials described above were used to manufacture 20 specimens sized 52x52x108 cm, hollow inside and subsequently filled with the material as per para. 3.3.1. The heads of the columns were then smoothed with mortar before the application of metal plates for the distribution of the test loads.

4.2 Damaging cycle

After about 30 days of ageing at 20°C temperature and 65% humidity, the specimens were subjected to centred compressive tests up to advanced cracking. For each load level, strains were measured by means of 16 potentiometric transducers on 250 mm gauges, arranged as shown in fig. 5.

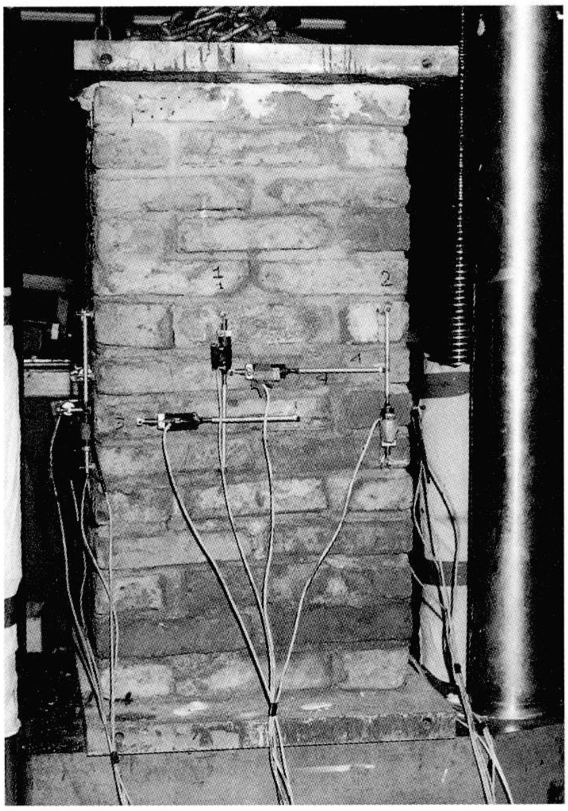

Fig. 5 Arrangement of the strain gauges

A load-strain diagram is shown in fig. 6. Transverse strains are shown on the left, and it can be seen that vertical cracks appeared at a load level of about 220 kN and then continued to increase in width with increasing load. It should be noted that at the maximum load reached at this stage (400 kN) the width of the main crack was about 3.75 mm: in actual practice, this corresponds to the specimen's maximum bearing capacity, as was also confirmed by the value of residual strain after the removal of the load. A subsequent loading cycle showed that the slope of the axial load-strain diagram, shown in the right-hand side of the diagrams in fig. 6, was greater than in undamaged specimens. This aspect must be taken into account when comparing the deformability of consolidated specimens.

4.3 Load tests on consolidated masonry

Consolidated specimens were tested until failure by the same procedure employed in the earlier series of tests. Figs. 7 and 8 reproduce the diagrams for two representative specimens. Lines a) refer to the damaging cycle and lines b), c) and d) refer to the subsequent application of the load to the consolidated specimens.

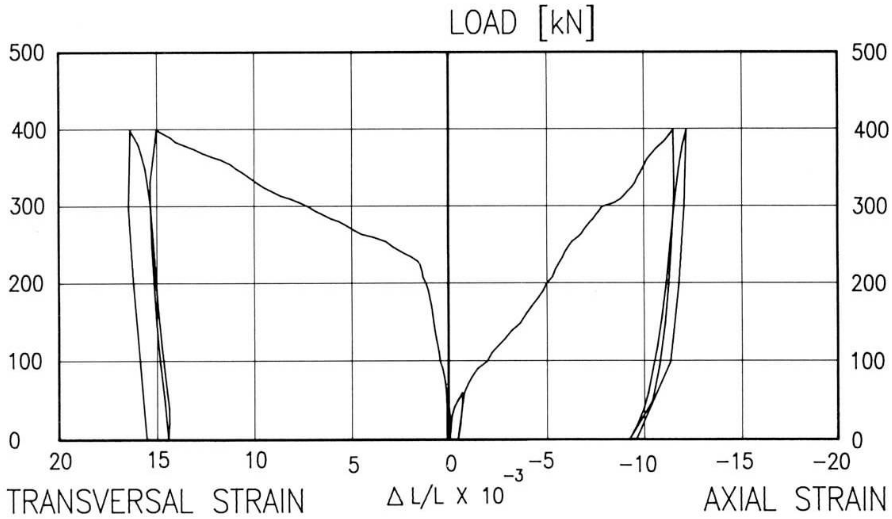

Fig. 6 Load-strain diagram, specimen No. 12

5. DISCUSSION OF TEST RESULTS

All consolidated test pieces displayed a significant increase in their load-bearing capacity, from 70 to 75% higher than the initial values.

The analysis of the load cycles, denoted by letter b), makes it possible to discern an elastic relationship between axial and transverse strains at the initial stage. The appearance of significant lesions, as revealed by transverse strains, takes place under loads about 60% higher than those applied in the first loading cycle.

The slope of the load-strain curves obtained from the consolidated structures can be compared to the load removal curves obtained during the damaging cycle. This means that the stiffness of the structural elements remains substantially unaltered after the intervention.

Finally, it can be pointed out that the action of the connectors makes it possible to obtain wide load-strain cycles; this suggests that the chosen consolidation method may be able to cope with considerable energy dissipation and hence can be effectively applied in seismic areas.

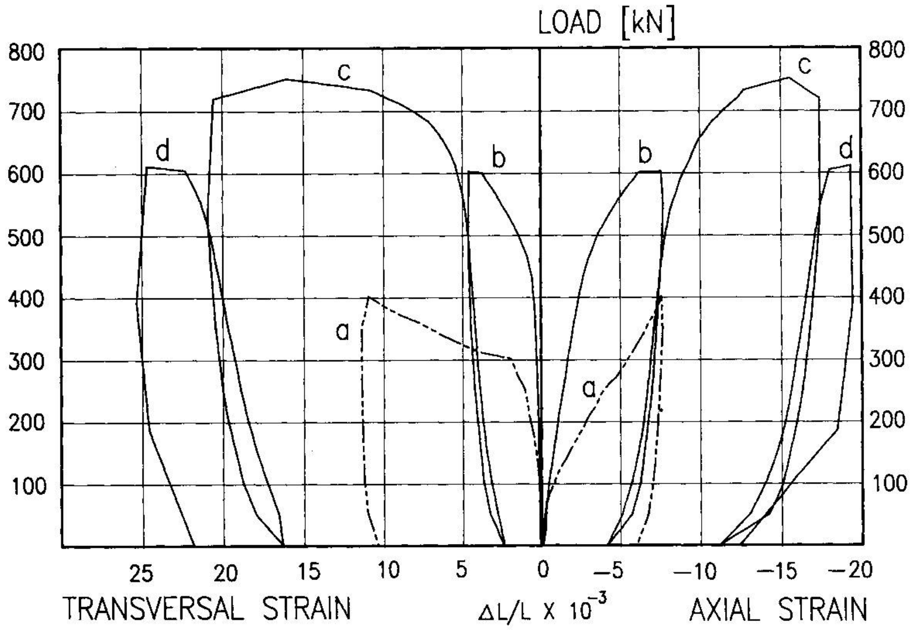

Fig. 7 Load-strain diagram, specimen No. 6

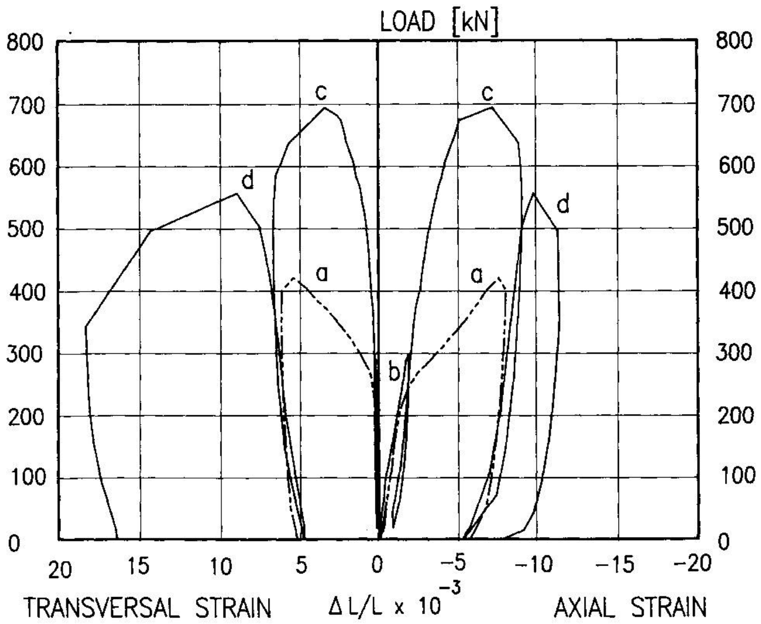

Fig. 8 Load-strain diagram, specimen No. 7

6. CONCLUSIONS

The investigation was meant to verify the validity of a new fill layer masonry consolidation technique by which the incoherent inner material is bound together through the injection of chemically compatible mortar and confined by fitting special high mechanical strength connectors designed to ensure satisfactory bond with the grouting material and not to react with the micro-climate in serviceability conditions.

The results obtained are quite encouraging: strength is seen to increase by about 65-70% compared to the initial values and the stiffness of consolidated elements remains essentially unaltered compared to that of the cracked specimens; energy dissipation and durability are also satisfactory. These results confirm the validity of the proposed technology

AKNOWLEDGMENTS

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Long-term Strength of Mortars and Grouts Used in Interventions

Contrôle à long terme de la résistance des mortiers de réparation

Langzeitkontrolle der Festigkeit von Reparaturmörteln

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SUMMARY

Measurements are presented of mechanical properties (compressive strength, tensile strength and modulus of elasticity) for a great number of mortar and grout specimens as much as four years old. Lime, natural pozzolana, brick powder, crushed full bricks and sand are the main constituents of these mortars and grouts.

RÉSUMÉ

L'article présente les mesures relatives aux propriétés mécaniques (résistance à la compression, à la traction, module d'élasticité) d'un grand nombre de mortiers et de coulis ayant jusqu'à quatre ans. Les matériaux principaux pour la préparation des mortiers utilisables dans la réparation des bâtiments historiques sont la chaux, le pouzzolane, les fragments de tuiles et de briques, et le sable.

ZUSAMMENFASSUNG

Im vorliegenden Bericht werden die Ergebnisse der im Labor durchgeführten Untersuchungen zur Bestimmung der Druck- und Zugfestigkeit, sowie das Elastizitätsmodul von Reparaturmörteln, die bis zu vier Jahre alt sein können, aufgezeigt. Die für Reparaturzwecke am besten geeigneten Mörtel bestehen hauptsächlich aus Kalk, Puzzolan, Sand und Backstein- oder Ziegelsplitter.

1. INTRODUCTION

Since the mortars and grouts used in repairing old masonries should be compatible to existing ones, "traditional" materials such as lime, natural pozzolana, brick powder and sand prevail among other materials found at market for these mortars' preparation. In addition cement is often added to the mixtures in small quantity in order to fulfill strength requirements. The criteria for the selection of these mortars or grouts are usually based on short time strength tests although pozzolanic reaction to which is mainly due the binding capacity of these mortars follows a slow and time-dependent process.

The aim of this work is to determine the long-time mechanical strength of mortars and grouts. Laboratory tests on 4x4x16 cm specimens according to DIN 18555 were made for the calculation of flexural and compressive strength while the dynamic modulus of elasticity was detrmined by ultrasonic measurements performed on the same specimens.

2. LABORATORY TESTS

The materials used for the preparation of tested mortar and grouts mixtures, as mentioned above, are lime, natural pozzolana from Santorini and Skydra (a place close to Thessaloniki), brick powder, crushed full bricks and river sand as well as cement. The mixing proportions of the mortars prepared in laboratory is given in Table 1 and of the grouts in Table 2.

A great number of specimens were crushed at different ages during a long period (over four years) after mixing to evaluate flexural and compressive strength and modulus of elasticity. The test results for mortars are indicated in Table 3 and for grouts in Tables 4 and 5.

Mortar No	Constituents (by weight)										
	Lime	Santorin earth max size		Skydra earth max size		Portland cement	Sand max size		Crushed bricks max size		Water
		0.1mm	0.25mm	0.25mm	6mm		2mm	6mm	2mm	6mm	
M1	1	1	-	-	-	-	6	-	-	-	2.20
M2	1	-	1	-	-	-	6	-	-	-	2.50
M3	1	-	1	-	-	-	3	-	3	-	2.73
M4	1	-	1	-	-	-	3	-	-	3	2.42
M5	1	-	0.8	-	-	0.2	6	-	-	-	2.17
M6	1	-	-	-	1	-	6	-	-	-	2.25
M7	1	-	-	-	0.8	0.2	6	-	-	-	2.17
M8	1	-	0.8	-	-	0.2	3	-	-	3	2.14
M9	1	-	-	-	0.8	0.2	3	-	-	3	2.08
M10	1	-	-	-	0.2	0.8	3	-	-	3	2.17
M11	1	-	0.2	-	-	0.8	3	-	-	3	2.18
M12	1	-	1	-	-	-	-	6	-	-	1.92
M13	1	-	1	-	-	-	-	6	-	-	1.86
M14	1	-	1	-	-	-	-	5	-	1	1.18
M15	1	-	1	-	-	-	-	3	-	3	1.35
M16	1	-	-	-	1	-	-	6	-	-	1.05
M17	1	-	0.8	-	-	0.2	-	6	-	-	1.10
M18	1	-	0.6	-	-	0.4	-	6	-	-	1.07

Table 1 Mix proportions of mortars

Grout No	Constituents (by weight)							Expanding agent
	Lime	Santorin earth	Skydra earth	Portland cement	River Sand	Brick powder	Water	
		max size <0.25mm			max size <1mm	max size <0.25mm		
G1	1	0.8	–	0.2	1	1	1.869	–
G2	1	–	0.8	0.2	1	1	1.875	–
G3	1	–	0.6	0.4	1	1	1.875	–
G4	1	–	0.6	0.4	2	–	1.530	–
G5	1	0.6	-	0.4	-	2	2.187	–
G6	1	2	–	0.4	–	-	1.720	0.034
G7	1	2	–	0.4	–	–	1.788	-
G8	1	2	–	0.8	3.8	–	2.250	0.038
G9	1	0.8	–	0.2.	1	1	1.670	0.020
G10	1	–	0.8	0.2	1	1	1.610	0.020

Table 2 Mix proportions of grouts (by weight)

Age	28 days			1 year			5 years		
Mortar No	Strength		Modulus of Elasticity	Strength		Modulus of Elasticity	Strength		Modulus of Elasticity
	Tensile N/mm^2	Compressive N/mm^2	N/mm^2	Tensile N/mm^2	Compressive N/mm^2	N/mm^2	Tensile N/mm^2	Compressive N/mm^2	N/mm^2
M1	0.32	0.54	1709	0.28	0.87	1756	0.39	0.83	1827
M2	0.23	0.34	1594	0.29	0.74	1643	0.28	0.71	1545
M3	0.25	0.84	1340	0.24	0.97	1375	0.22	0.77	1106
M4	0.26	1.32	1807	0.32	1.46	1936	0.23	0.81	1290
M5	0.31	1.44	2431	0.21	0.61	2677	0.21	0.62	1330
M6	0.22	0.54	1271	0.32	1.55	2487	0.36	1.16	2419
M7	0.30	0.52	2103	0.36	0.88	2096	0.36	0.95	2263
M8	0.52	2.60	3038	0.53	2.03	2819	0.54	1.89	2421
M9	0.71	3.07	3462	0.56	2.26	3473	0.52	2.00	2420
M10	1.58	6.26	6812	1.25	5.69	6465	1.25	4.48	5138
M11	2.01	6.84	6740	1.35	6.73	6713	1.76	5.95	5715
M12	0.24	0.70	1505	0.22	0.93	2018	0.28	0.83	1919
M13	0.26	0.93	2390	0.27	1.08	2235	0.32	0.83	2069
M14	0.27	1.25	2446	0.30	1.42	1241	0.24	1.10	2348
M15	0.20	0.78	1802	0.29	1.12	2297	0.32	1.04	2437
M16	0.14	0.65	1640	0.23	1.26	1996	0.20	1.10	1900
M17	0.24	1.27	2774	0.42	1.79	3754	0.42	1.77	3221
M18	0.48	1.76	4184	0.56	2.57	5045	0.74	2.68	4616

Table 3 Mortar strength and modulus of elasticity

Age	7 days		14 days		21 days		28 days		90 days		1540 days	
Strength	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive
Grout No	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
G1	0.10	0.21	0.11	0.34	0.16	0.64	0.22	0.72	-	0.80	0.56	1.01
G2	0.12	0.25	0.10	0.35	0.20	0.42	0.26	0.61	-	-	0.43	1.25
G3	0.20	0.44	0.37	0.75	0.45	0.98	0.66	1.55	0.71	2.00	0.82	2.46
G4	0.12	0.38	0.39	0.67	0.37	0.81	0.34	1.18	0.49	1.60	1.06	2.75
G5	0.34	0.42	0.45	0.97	0.64	1.91	0.50	2.65	0.62	2.62	1.20	2.62
G6	0.07	0.28	0.20	0.53	0.40	0.90	0.42	1.30	0.67	1.47	1.04	1.62
G7	0.09	0.32	0.23	0.52	0.44	1.00	0.53	1.44	0.64	1.53	0.98	2.16
G8	0.43	1.13	0.75	2.14	1.13	3.46	1.16	2.96	1.43	3.47	1.64	3.63
G9	0.04	0.20	0.25	0.44	0.48	1.06	0.44	1.39	0.39	1.32	0.87	1.58
G10	0.05	0.24	0.29	0.49	0.37	0.88	0.39	1.20	-	0.98	0.66	1.31

Table 4 Strength of grouts at different ages

Dynamic modulus of elastisity [N/mm^2]						
Age / Grout No	7 days	14 days	21 days	28 days	90 days	1540 days
G1	527	1015	1266	1461	1547	1675
G2	508	1050	1456	1541	1616	1572
G3	1766	–	1942	2544	2548	2549
G4	1392	2013	2248	2325	2761	2939
G5	1210	1729	2242	3400	3456	2537
G6	960	1049	1832	1801	1731	1743
G7	–	1310	1918	2071	2182	1922
G8	3930	4390	5088	4852	5082	5066
G9	787	1533	2216	-	2262	2130
G10	670	1238	1847	1873	1857	1993

Table 5 Dynamic modulus of elasticity of grouts

3. DISCUSSION

3.1 Mortars

Regarding the test results of Table 3 and the mortar composition given in Table 1 it can be said in general that the addition of ceramic material as well as of cement increases the strength of the mortars. The aggregate gradation influences also the mechanical properties (mortar M12 has a different aggregate gradation than mortar M13 mentioned in Table 1). These remarks were commended in a previous work of the authors [1].

In relation to the strength development with time it can be said the following: Mortars without ceramic material or cement (M1, M2, M6, M12, M13, M16) show a slight increase in flexural strength from 28 days up to 1 year. After this age the strength remains almost unchanged except for the one of mortar M6 and M16 which are made with pozzolana from Skydra and showed a decrease at five years. The development of the flexural strength increases up to 60% from 28 days to 1 year. The compressive strength of mortars M1, M2, M6, M12, M13, M16 increases with the time and after one year decreases slightly. The dynamic modulus of elasticity follows almost the same process as the compressive strength, but its values vary strongly.

Mortars M3, M4, M14, M15 with ceramic material show an increase of strength and modulus of elasticity till one year and after this point a light decrease. These results are in agreement with those found by other researchers [2] in relation to modern mortars.

Mortars M5, M7, M8, M9, M17 in which 20% of natural pozzolana has been replaced by cement do not show significant variations in flexural strength after 28 days. Mortar M18 has almost the same strength as mortar M8, although the amount of cement is double (40% of natural pozzolana has been replaced by cement). This may be owing to the latent hydraulic activity of ceramic material. Mortars M10 and M11 with 80% cement and 20% pozzolana show a decrease of strength and modulus of elasticity after 28 days.

3.2 Grouts

Cement and ceramic material seem to have the same influence on mechanical properties of grouts as in the case of mortars. Strength measurements of grouts were made from 7 days up to 4 years. Grouts with 20% cement and 80% pozzolana (G1,G2,G9,G10) and the ones with 40% cement and 60% pozzolana (G3, G4, G5, G6, G7) show generally an increase in strength with time (Table 2, Table 4, Table 5). The strength development of these grouts is shown in Fig.1. The same behaviour presents G8 with 80% cement and 20% pozzolana.

The increase of grout strength with time has also been observed in previous works done at the Laboratory of Reinforced Concrete [3,4].

4. CONCLUSIONS

The long-time strength of mortars and grouts must be taken into account in the analysis of bearing capacity of under restoration masonry as well as the masonry deformations. Therefore, a slight decrease in mortar strength must be expected after one year. In contrast, grout strength is slightly increased after years. This leads to the suggestion that in order to achieve the require strength for repairing a masonry a higher strength mixture should be designed.

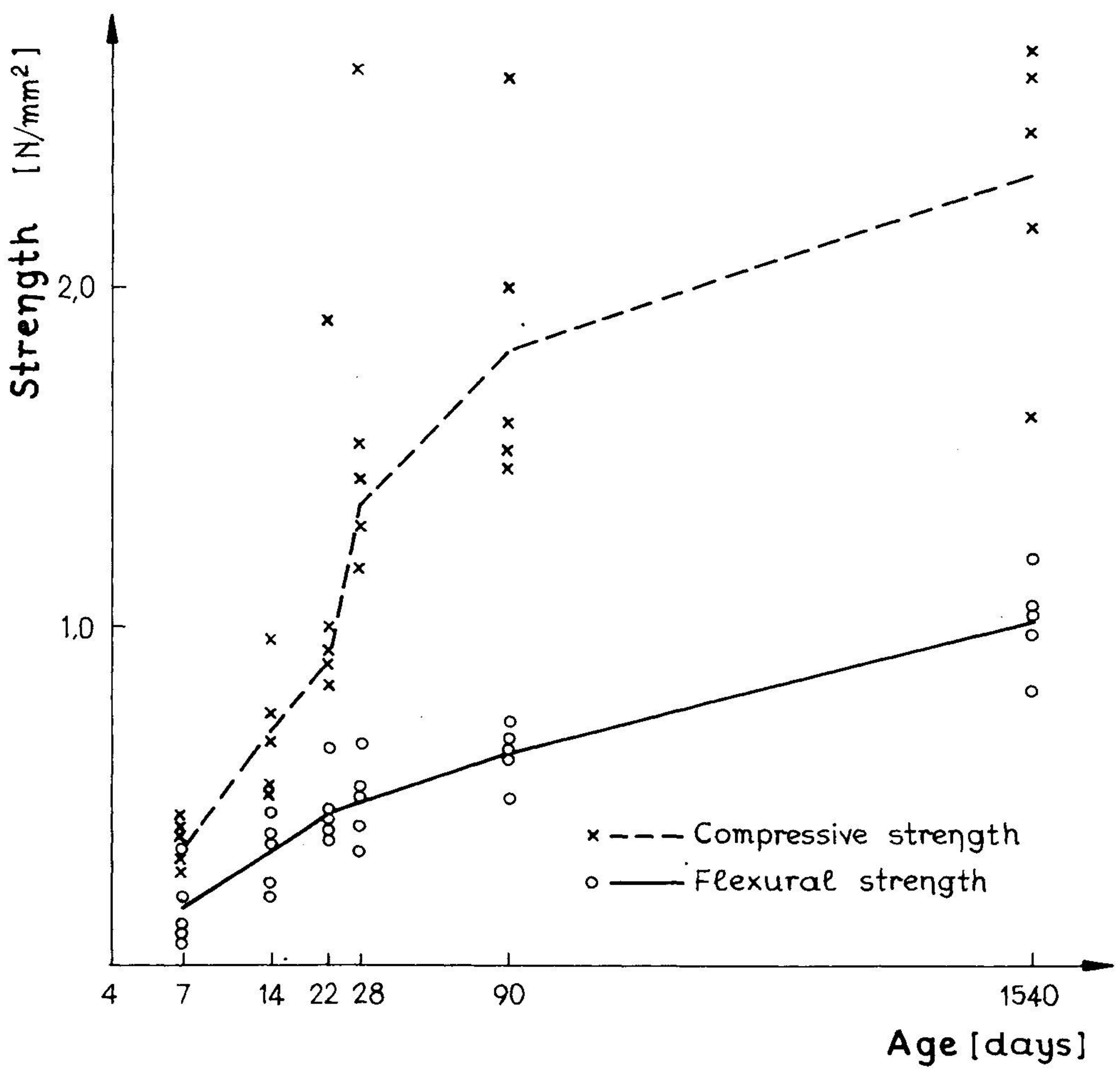

Fig. 1 Strength development of grouts G_3, G_4, G_5, G_6, G_7

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Rendering Products for the Protection of Historical Buildings

Enduits pour la protection des bâtiments historiques

Putzmörtel für den Schutz von historischen Gebäuden

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SUMMARY

The protection of artistic and historic buildings must be among the first commitments of a nation, even if restoration problems are often complicated by the compatibility with the existing substrates. This paper studied the performances of different types of rendering mortars in order to furnish the operators with a proposal of methodologies for an 'appropriate' choice between various products.

RÉSUMÉ

La protection des bâtiments historiques et artistiques doit être une des premières responsabilités d'une nation, bien que les problèmes des réparations soient souvent compliqués par la compatibilité avec les matériaux déjà appliqués. Ce rapport étudie les performances de différents types d'enduit afin d'offrir aux utilisateurs une méthodologie pour un choix approprié parmi divers produits.

ZUSAMMENFASSUNG

Der Schutz historisch und kulturell wertvoller Gebäude muss eines der wichtigsten Ziele jeder Nation sein, wenn auch Unterhaltsarbeiten oft mit Verträglichkeitsproblemen behaftet sind, die zwischen den neuen und bereits verwendeten Baustoffen auftreten. Dieser Bericht studiert die Eigenschaften und Einsatzmöglichkeiten verschiedener Produkte, damit Anwendern die Wahl erleichtert wird.

1. INTRODUCTION

In Italy there is the highest concentration of hystorical and artistical monuments in the world. The problems of maintenance or repair of this heritage are very large. In the past years there was some cases in which the repairs caused serious damages to the existing structures. For these reasons a true scientific approach to the ancient structures repair problems has been considered necessary. The aim of this paper is principally to show example of this scientific approach in a particular and very important field: the rendering mortars for hystorical buildings. This is in order to give to the field operators a proposal of methodologies for an appropriate choice between the commercial products.

2. EXPERIMENTAL PART

The following systems have been evaluated:

- EMACO RESTO (MAC SpA - Italy) + sand
- Plastocem (Italcementi - Italy) + sand
- Hydraulic lime (Lafarge - France) + sand
- Hydrated lime (Ceprovip - Italy) + pozzolana

2.1 Mixes preparation

Mortars with typical workability conditions have been made; the measure of the workability has been performed according to UNI 7044. Water/binder ratios have been varied in the systems in order to have approximately 90% of flow (Tab.1).

2.2 Tests

The above mentioned systems were cured in standards conditions (20°C and 95% R.H.), execpt for the hydrated lime+sand system (room conditions). Thechnological, physical and chemical properties have been evaluated according to the relevant standard methods.

2.2.1 Compressive strength

The compressive strength was measured according to UNI 7102 standard (D.M. 03.06.1968) after 7,28 and 90 days of curing.

2.2.2 Sulfate resistance

According to Anstett test, the expansion of the hydrated binders in presence of 50% of weight of gypsum has been detected.

2.2.3 Capillarity water absorption : according to Normal 11/85.

2.2.4 Water absorption by total immersion : according to Normal 7/81.

2.2.5 Water vapour permeability : according to Normal 21/85.

2.2.6 Porosity and mercury porosimetric distribution: according to Normal 4/80

2.2.7 White paint quantity requested to cover the mortar colour

The same white paint, silicates based in water solution, has been applied on the samples until the same whitness degree, measured by a reflectance colorimeter.

2.2.8 Water vapour permeability on the painted mortars

According to Normal 21/85 the same samples of previous test were used; the surface of evaporation was the painted surface.

2.2.9 Resistance to the salts crystallization

Samples have been kept at cycles of 24 hours in Na_2SO_4 solution and 24 Hours in a windy oven at 60°C until the formation of visible cracks on the samples. After which the samples were washed in order to eliminate the residual sodium sulfate, and further the capillary and total water absorption tests were carried out.

2.2.10 Soluble salts content

The soluble salts content was measured after acqueous extraction, according to Normal 13/83, by ionic chromatography.

2.2.11 Efflorescences formation tendency

This test has been performed on samples of 4x4x16 cm size, partially dipped in deionised water, with the lateral faces covered by a paraffin layer. The efflorescences have been determined qualitatively and quantitatively after 28 days.

2.2.12 Shrinkage

The appearance of cracks on the mortars, applied on brick walls of 1.30 m x 2 m size, previously water saturated, has been observed in the first days, in order to make evidence on plastic shrinkage, and after approximately 2 years, to verify the hydraulic shrinkage. Also the surface appearance after an external exposition was observed.

3. DISCUSSION

3.1 Mixing water

EMACO RESTO, Plastocem and the Lafarge+pozzolana systems requested approximately the same mixing water quantity, as indicated in Table 1. In this table, we have to consider that hydrated lime was about 40% of solids.

3.2 Mechanical strength

The four systems were very different as far as it concerns this property, as showed in Table 1. In all systems the compressive strengths were not very high, in order to have a better mechanical compatibility with the hystorical structures. Emaco Resto showed the highest compressive strength, so indicating the possibility to have an efficient restoration.

	bleeding	*water added*	*flow UNI*	*compressive strength (MPa)*		
	%	%	%	7 days	28 days	90 days
EMACO RESTO+sand	absent	15	90	2.7	10.5	13.0
Plastocem+sand	absent	15	100	1.7	4.6	7.8
Lafarge+sand	absent	17.2	100	1.6	2.7	3.9
Ceprovip+pozzolana	0.1	8	95	1.5	2.9	3.5

Table 1 Mechanical performances of renderings

3.3 Anstett test

EMACO RESTO had the lowest expansion (0.75%). Hydrated lime and pozzolana gave an expansion of 3.8%; Plastocem expanded of 31% after 28 days, with the appearance of large cracks on the surface of the sample.
Lafarge lime showed an initial low expansion and, after 7 days, suddendly arrived until the total destruction of the sample (Tab.2).

	EXPANSION (%)			
renderings	1 day	3 days	7 days	28 days
EMACO RESTO+sand	0.50	0.75	0.75	0.75
Plastocem+sand	1.31	9.06	14.56	31.25
Lafarge+sand	0.71	1.00	1.42	destr.
Ceprovip+pozzolana	0.40	3.10	3.50	3.80

Table 2 Anstett test expansion

3.4 Water absorption by capillarity and total immersion

Ceprovip and pozzolana systems had the highest values of absorption in terms of capillarity coefficient and plateau values; the other products had a similar behaviour between them.
It has to be noted from the analyses of the absosption curves that EMACO RESTO and Plastocem show a behaviour similar to porous materials treated with hydrorepelling products.

3.5 Water permeability

Data obtained in this test are are consistant with those detected in the water absortion test: as higher the Imbibition Coefficient and the capillary absorption, as higher the vapour permeability values.

3.6 Water vapour permeability of painted mortars

The presence of a paint, that is necessary when you need to vary the colour of a rendering mortar, reduces the permeability, but maintains the initial relative difference.

3.7 Demand of white paint

All the systems obtained approximately the same whitness degree (close to whitness degree of pure paint) by applying the same quantity of colour and the same application conditions (Tab.3).

	EMACO RESTO + sand	Plastocem + sand	Lafarge + sand	Ceprovip + pozzolana	Plain
Y (%)	88.17	88.3	88.88	87.01	89.91
paint weight (g)	0.84	0.72	0.62	0.45	--

Table 3 Brightness test

3.8 Crystallization resistance

The systems were placed in Na_2SO_4 solutions at 70 g/l and 700 g/l, in order to expose to very hard conditions the materials. In the more diluted solution Ceprovip was resistant to 2 cycles and Plastocem to 8 cycles; in the more aggressive conditions (700 g/l) Lafarge+sand overpassed 3 cycles.
Finally, only EMACO RESTO was resistant to all the 10 cycles.

3.9 Interaction with water after weathering

The hydrated based and Lafarge systems were destroyed during the washing. The values obtained with EMACO RESTO and Plastocem are generally consistant with the values on the "fresh" samples, even if higher. This is reasonable as a similar artificial weathering induces surely some structural internal modifications, increasing the porosity.

3.10 Soluble salts release

The chemical analyses made on the aggregates indicate, as expected, lower concentrations of free ions in the sand and higher quantities of sodium and potassium in the pozzolana.
The anhydrous binders contain principally sodium and sulphates. These ions generally decrease in all the samples after the setting.
EMACO RESTO had, at the beginning, the highest concentration in those ions, confirmed by a tendency of this system to an efflorescences formation (mirabilite).
In cooperation with the formulator, the origin of this compound was defined.

So, the verification of the same mix, in wich the cause of the origin of the ions was eliminated, confirmed better results. In fact, there was not efflorescences formation (Tab.4).

	sulfate	*sodium*	*potassium*	*magnesium*	*calcium*
EMACO RESTO	0.50	0.14	< 0.05	< 0.05	2.26
Plastocem	1.40	0.10	0.30	< 0.05	1.70
Lafarge	0.40	0.12	< 0.05	< 0.05	8.10
Ceprovip	not availab.	not availab.	not availab.	not availab.	not availab.
Pozzolana	< 0.1	0.12	0.11	< 0.05	0.70
Sand	< 0.1	0.04	< 0.05	0.06	0.53
EMACO RESTO+sand	0.27	0.14	< 0.05	< 0.05	0.64
Plastocem+sand	0.05	0.10	0.44	< 0.05	0.40
Lafarge+sand	0.08	0.03	0.07	< 0.05	1.10
Ceprovip+pozzolana	0.07	0.10	0.12	< 0.05	0.62

Table 4 Soluble salts content (% by weight)

3.11 Porosimetric characteristics

The three hydraulic binder based systems have a similar total porosity, between 23 and 30%. Otherwise, the mortar made with Ceprovip+pozzolana has a total porosity higher than 50%.
Following these results the Ceprovip system, that has a higher porosity with finer pores, shows the highest water absorption.
Quite unexpectedly the mix with Lafarge, having a porosity lower than Plastocem and EMACO RESTO, has a higher water absorption and permeability.
Since the last three mixes were made with the same quality and quantity of sand, the differences observed in the porous structure and in the behaviour with water, seems to be due to the different chemical behaviour of the binder.

3.12 Plastic shrinkage

The plastic shrinkage of EMACO RESTO is of about 100 microns/m, that of Plastocem reaches 700 micron/m.
Otherwise the shrinkage of Lafarge system and moreover of Ceprovip system is much higher, showing also the presence of many cracks.
The samples are still under observation for the evaluation of the behaviour in open air conditions.

4. CONCLUSIONS

The analytical methodology choosed in this study gave the possibility of distinguishing between the examined rendering mortar systems and making evidence of the peculiarity of each one especially with regard to the possible use in architectural restoring works.
The hydrated lime based mortar system must be considered separately between studied systems. In fact it was taken into consideration as reference material, because already largely used since ancient times.
The comparison between the two "modern" binders having an hydraulic lime base shows that the two products are quite similar but not equivalent. They are different especially with regard to compressive strength, Anstett test and salt crystallization resistance.
The comparison between those two systems and the "cementitious" product specifically formulated indicates that this one provide generally better performances.
The grey colour of this product should often require the application of a paint of different colour. The higher content of sodium and sulfate and the consequent formation of efflorescences is a negative feature that it was possible to eliminate thanks to the results of this study.
However, we should wish that the employ of mortars for restoring purposes would always be tested first by the experimenter.
The cooperation on a scientific base between the "operator" and the "formulator" could allow an improvement of the product to the aim of meeting the specific need of monumental repairs.

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Advanced Composites for Strengthening Historic Structures

Composites d'avant-garde pour le renforcement de structures historiques

Verbundwerkstoffe zur Verstärkung historischer Bauwerke

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SUMMARY

The paper deals with the application of uni-directional fibre reinforced plastic (FRP) tendons for reversible strengthening of masonry monuments. The tendons, anchored to the masonry only at the ends, are circumferentially applied on the external face of the structure and post-tensioned to provide horizontal confinement. The relevant properties of FRP prestressing systems are summarised, concepts for their application in masonry structures are presented and a design procedure is proposed.

RÉSUMÉ

L'article s'occupe de l'application de câbles en fibres plastiques unidirectionnelles (FRP) dans le renforcement réversible des monuments en maçonnerie. Les câbles qui sont ancrés sur la maçonnerie à leurs extrémités seulement, sont appliqués le long du périmètre de la structure. Ensuite, ils sont tendus, afin de ceinturer horizontalement la structure. Les propriétés significatives des systèmes de précontrainte sont présentées, ainsi que les concepts de leur application. Une procédure de calcul est proposée.

ZUSAMMENFASSUNG

Der Beitrag behandelt die Verwendung von Spanngliedern aus einsinnig faserverstärktem Kunststoff für die vorübergehende Tragfähigkeitserhöhung historischer Mauerwerksbauten. Die Spannglieder werden um das Bauwerk herumgelegt, nur an ihren Enden verankert und vorgespannt, so dass sich eine horizontale Umschnürungswirkung einstellt. Die massgebenden Eigenschaften von Vorspannsystemen aus faserverstärktem Kunststoff werden kurz geschildert, Anwendungskonzepte für Mauerwerksbauten vorgestellt und Bemessungsverfahren vorgeschlagen.

1. INTRODUCTION

In recent years the importance of structural interventions in the architectural heritage (monuments, historic buildings and bridges, etc.) for repair and strengthening has increased considerably. Such interventions often follow special principles, e.g., those of the Charter of Venice (1964). Very important among these principles are the requirements that interventions should not adversely affect the *character* of the monument and must be *reversible*, especially when the techniques applied have not be proven by a very long in-service performance.

Among the methods used to upgrade historic structures are grout injections, stitching of large cracks with metallic elements or concrete zones, application of reinforced grouted perforations, external jacketing by shotcrete or by cast-in-situ concrete, and external or internal post-tensioning with steel ties [1, 2]. Unlike other methods, external post-tensioning with steel ties combines efficiency, simplicity and reversibility. It has been applied in many historic structures, such as the Rotunda and the San Andreas domes in Thessaloniki, and the Martinego rampart of the Old Castle in Corfu [2, 3], but presents some practical difficulties in protecting the strands against corrosion and handling them at the construction site (due to their considerable weight). As an alternative, the steel ties can be replaced with advanced fibre reinforced plastic composite materials, which offer excellent physical and mechanical properties, and are lightweight and immune to corrosion. Last but not least, they may be applied to historic structures in a reversible manner, in the form of external tendons in a colour matching that of the external surface of the structure.

In this paper, the authors establish the applicability of composite materials in strengthening of masonry-type monuments. The relevant properties of these materials are summarised, concepts for their application in masonry structures are presented (including attachment), and a design procedure is proposed.

2. ADVANCED COMPOSITES AS STRENGTHENING MATERIALS

Fibre reinforced plastics (FRP) have been used extensively in a variety of industries, including aerospace, automotive, ship-building and sports. They are increasingly becoming important in the construction industry too, with great potential in many areas, offering the designer an outstanding combination of properties not available from other materials. Fibres such as glass, aramid and carbon (with diameter in the range 5-25 μm) can be introduced in a certain position, volume and direction in a binding matrix (e.g., epoxy, polyester, vinylester) for maximum efficiency. When the fibres are continuous, parallel and at high volume fractions (typically more than 50%), a unidirectional material is produced with a strength and stiffness close to that of the fibres and with the chemical resistance of the matrix [4]. Among other properties, unidirectional FRPs (advanced composites) offer high strength and stiffness, lightness and immunity to corrosion. Therefore, use of these materials for special applications in construction is highly attractive and cost effective, due to improved durability, reduced life-cycle maintenance costs, savings from easier transportation and improved on-site productivity.

Because of their advantages over conventional materials (low and high strength steels), unidirectional FRPs have found their way in numerous construction applications, including: (a) development of tendons for prestressing [5-11]; and (b) strengthening of concrete and wood structures with non-prestressed or prestressed composite sheets, bonded externally on the tension faces using epoxy adhesives [12-17]. In all these applications, the composites are manufactured by highly automated processes such as pultrusion, in which fibres are pulled through a heated die into which resin is injected, and a fully cured element is produced with good dimensional stability.

The concept proposed here for the application of advanced composites as strengthening materials of masonry-type historic structures involves the introduction of circumferential externally attached ties, post-tensioned on horizontal planes. There is quite a large number of FRP materials manufacturers and suppliers around the world, and several companies provide complete tendon-anchorage systems. From a variety of products, basic information (including physical properties) about the most widely known commercial systems is given in Table 1, and a comparison of the mechanical properties of these systems is given in Table 2 [5-11, 18].

Table 1 Representative tendon-anchorage FRP post-tensioning systems.

Manufacturer	Product name	Shape	Fibre, matrix	Fibre volume fraction, V_f	Density, ρ (kg/m^3)	Coeff. of thermal expansion, α (x10^{-6}/°C)
Bayer AG & Strabag Bau AG (Germany)	Polystal	Round	E-Glass, polyester	0.68	2000	7.0
ICI Linear Composites Ltd.(England)	Parafil G	Rope	Aramid (Kevlar 49)	1.00	1400	-5.7
HBG & AKZO (Holland)	Arapree	Both round & strip	Aramid (Twaron), epoxy	0.44	1400	-1.8
Mitsui Constr. Co. (Japan)	FiBRA	Round	Aramid (Kevlar 49), epoxy [1]	0.65	1300	
Teijin Co. & Sumitomo Constr. Co. (Japan)	Teijin Rod	Round	Aramid (Technora), vinylester	0.65	1300	
Tokyo Rope Mfg Co. & Toho Rayon Inc. (Jap.)	CFCC	Multi-wire cables	Carbon (Besfight), epoxy	0.60	1500	0.6
Mitsubishi Kasei Co. (Japan)	Leadline	Round	Carbon (Dialead), epoxy	0.65	1600	

[1] Braided Aramid, impregnated with epoxy

Polymeric composites are often subjected to environmental effects such as attack by chemicals, moisture uptake, temperature fluctuations and irradiation with ultra-violet light (UV), which may lead to deterioration and premature failure. In general, carbon fibre reinforced plastic (CFRP) is highly resistant to these effects, glass fibre reinforced plastic (GFRP) is sensitive, while aramid fibre reinforced plastic (AFRP) displays an intermediate behaviour. The detrimental action of moisture and chemicals (e.g., alkalis) to GFRP and that of UV to AFRP deserve special mention. Finally, it is worth mentioning that in some of the prestressing systems above the rods are protected: in Polystal, a 0.5 mm polyamide coating is employed; in Parafil G, the continous fibres are contained within a thermoplastic sheath (protected from UV); and in CFCC the wires are overwrapped with a polymeric yarn.

3. CONCEPTS AND ANCHORAGE

Masonry structures can be consolidated and strengthened using FRP ties as illustrated in Figure 1. The tendons, in the form of either round rods or strips attached to the masonry only at their ends, are circumferentially applied on the external face of the structure and post-tensioned to provide horizontal confinement.

Table 2 Mechanical properties of various post-tensioning elements (according to manufacturers).

Product	Young's modulus, E (GPa)	Tensile strength, $f_{FRP,t}$ (GPa)	Ultim. strain, ϵ_u (%)	Poisson's ratio, ν (-)	Creep / elastic strain	Relaxation (%)	Stress rupture[2]
Polystal	51	1.57	3.3	0.27	0.03 (2 yrs)	1.4 (100 hrs) 3.5 (100 yrs)	0.70[3]
Parafil G	120	1.95	1.6		0.04 (1 day)	4.0 (100 hrs) 8.0 (100 yrs)	0.40
Arapree	55	1.35	2.4	0.38	0.002[1] (100 yrs)	7.5 (100 hrs) 15 (100 yrs)	0.60
FiBRA	64	1.35	2.2	0.62		10 (100 hrs) 20 (100 yrs)	
Teijin	55	1.90	3.6	0.35		8 (100 hrs) 20 (100 yrs)	
CFCC	137	1.80	1.6		0.0004 (100 hrs, 180 °C)	1 (100 hrs)	
Leadline	147	1.80	1.3				

1 This value appears to be too low for AFRP, which is known to creep considerably more than GFRP and CFRP
2 Projected residual strength as a fraction of the short-term strength, after stressing the elements at about $0.5f_{FRP,t}$ for 100 yrs (and testing after unloading)
3 This value appears to be too high for glass/polyester, which displays very poor stress rupture behaviour

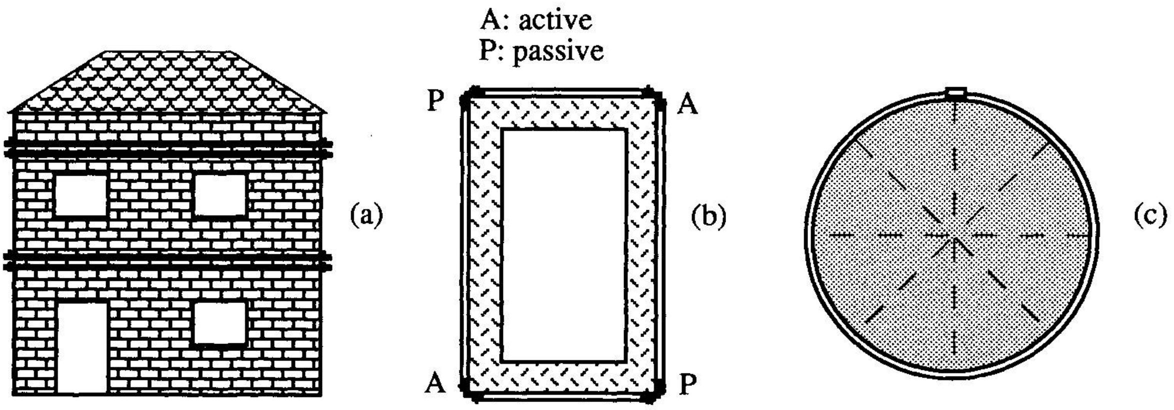

Fig. 1 Application of external FRP ties: (a) elevation; (b) plan view of rectangular structure; and (c) plan view of spherical dome.

Due to their anisotropic nature, unidirectional composites have relatively low transverse compressive strength (approx. $0.1f_{FRP,t}$) and even lower (interlaminar) shear strength. Furthermore, because of their brittle nature, the materials are sensitive to stress concentrations and hence cannot be pierced or threaded. Finally, their abrasion resistance allows only limited frictional stresses. Thus, conventional anchoring solutions (upset heads, threads, wedges, etc.) are not applicable, and relatively large anchor lengths are required. Strip-like tendons may be better than round ones for external post-tensioning of masonry, because they minimize anchor lengths (due to their large surface area) and simplify the attachment of anchorages on the masonry walls. Proposed concepts for anchorages and their attachment on masonry are illustrated in Fig. 2.

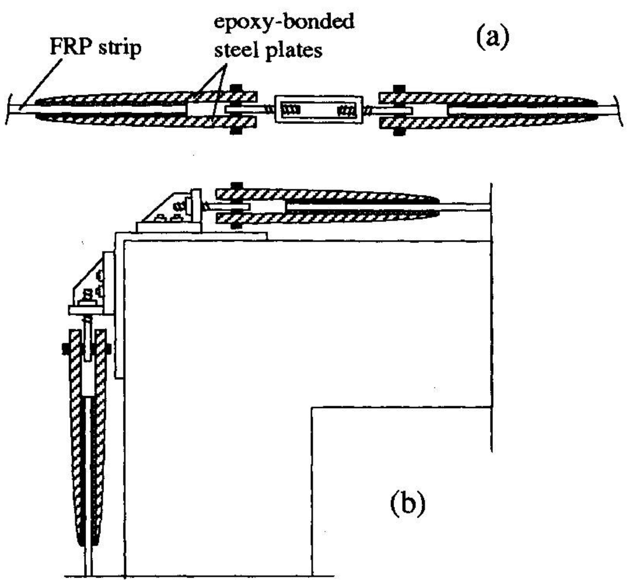

Fig. 2 Conceptual FRP anchorage/attachment for (a) circumferential prestressing of circular domes; and (b) masonry structure corners.

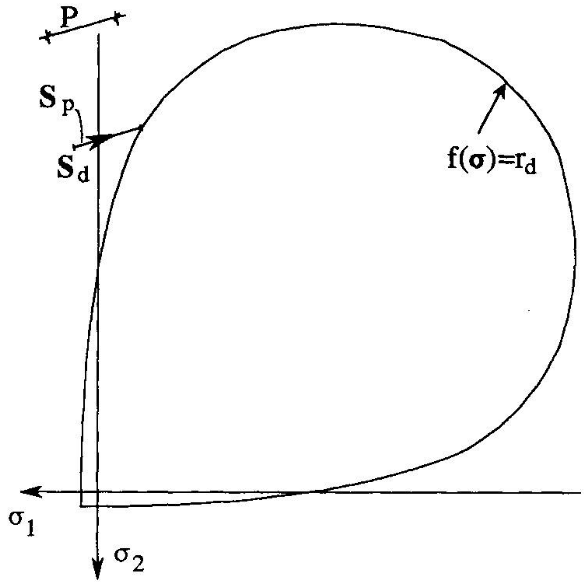

Fig. 3 Illustration of design procedure.

Figure 2(a) refers to circumferential prestressing of structures with a circular plan and involves a single FRP tendon around the perimeter, gripped at each end between a pair of steel plates to which it is epoxy-bonded. The two pairs of plates extend into a corresponding threaded steel bar and are coupled by a usual threaded bar coupler. Because FRP tendons cannot be bent to a large curvature, they cannot turn around sharp corners of the structure and have to be individually anchored there. For this latter case the anchorage in Fig. 2(b) is proposed herein, involving a structural steel angle weakly attached to the corner of the wall and transferring prestressing forces to the masonry through bearing stresses. The two tendons anchored at the same corner angle have to be prestressed gradually, by alternate turning of the nuts at their end anchorage, so that at each corner the moments of the individual tendon forces with respect to the corresponding wall mid-surface counterbalance each other. For no such end moment to develop during tensioning at a dead or passive anchorage (P in Fig. 1(b)), the two pairs of tendons actively anchored at two diametrically opposite corners (A in Fig. 1(b)) have to be tensioned simultaneously.

4. DESIGN PROCEDURE

Considering that at the generic point **x** of the structure the masonry is in a biaxial state of stress, the strengthening effect of circumferential prestressing by FRP is due to the in-principle beneficial effect of introducing additional compression to each normal stress component. If the ultimate strength condition of the masonry, shown in Fig. 3 in biaxial principal stress space for the case of isotropic stone-masonry, is expressed as:

$$f(\sigma(\mathbf{x}))-r(\mathbf{x})=0 \tag{1}$$

in which for biaxial stresses $\boldsymbol{\sigma}$ stands for $(\sigma_1,\ \sigma_2)$, or, in general, for $(\sigma_x,\ \sigma_y,\ \tau_{xy})$ for anisotropic (e.g. brick) masonry, then the design ultimate limit state under biaxial stresses is given by:

$$f(\sigma(\mathbf{x}))-r_d(\mathbf{x})\equiv f(\sigma(\mathbf{x}))-\frac{r(\mathbf{x})}{\gamma_m}=0 \tag{2}$$

In Eqs. (1) and (2) the "constant" r(**x**) of the ultimate strength condition is a measure of the as-built strength of the masonry, e.g. its uniaxial compressive strength in the horizontal or in the vertical direction, and depends, in general, on location **x**. γ_m in Eq. (2) is the material partial safety factor for the old masonry, which may be different than the one for new masonry (its value may be taken higher, depending on the historical importance of the structure, or lower, due to better knowledge of the as-built strength properties).

The ultimate strength condition of isotropic masonry can be fitted by the failure criterion proposed by Ottosen [19] for concrete:

$$\alpha J_2 + \lambda\sqrt{J_2} + \beta I_1 = 1 \tag{3}$$

in which I_1 is the first stress invariant, J_2 is the second deviatoric stress invariant and

$$\lambda = c_1 \cos\frac{\cos^{-1}(c_2 \cos 3\theta)}{3} \qquad \text{if} \qquad \cos 3\theta \geq 0 \tag{4a}$$

$$\lambda = c_1 \cos\left(\frac{\pi - \cos^{-1}(-c_2 \cos 3\theta)}{3}\right) \qquad \text{if} \qquad \cos 3\theta < 0 \tag{4b}$$

In Eqs. (4a, b) $\cos 3\theta = 3\sqrt{3} J_3 / 2 J_2$, with J_3 the third deviatoric stress invariant. To fit biaxial test data for stone masonry with a ratio of uniaxial strengths in tension and compression equal to 0.085 and of equal biaxial to uniaxial compression strength ratio equal to 1.65, the following parameter values can be used: $\alpha = 0.665 / f_m^2$, $\beta=3.84/f_m$, $c_1=13.8/f_m$ and $c_2=0.959/f_m$ (f_m=uniaxial compressive strength). For anisotropic (brick) masonry, the models by Ganz and Thuerliman [20] including tensile strength of bed joints, or by Koenig et al [21] or Dialer [22] may be used.

The state of stress in Eqs. (1) and (2) equals:

$$\sigma(\mathbf{x}) = S_d(\mathbf{x}) + \sum_{i=1,n_p} P_i \, S_{pi}(\mathbf{x}) \tag{5}$$

in which $S_d(\mathbf{x})$ is the value of $\sigma(\mathbf{x})$ due to the ultimate limit state design combination of actions, factored with the appropriate load partial safety factors, γ_F, and combination factors, ψ_o. P_i is the uknown value of the prestressing force of FRP tendon or group of tendons i and $S_{pi}(\mathbf{x})$ the state of stress at **x** due to P_i=1, and n_p the number of tendons or groups of tendons with independently different and unknown prestress force values. For simple geometries, such as rectangular in plan structures or spherical domes, analytical expressions for the stresses $S_d(\mathbf{x})$ and $S_{pi}(\mathbf{x})$ can be obtained, while for complicated three-dimensional structures with openings, finite element analyses will be required.

The n_p unknown prestressing forces are determined by satisfying the nonlinear in these values design ultimate limit state condition, Eq. (2), at n_p representative locations **x**. Alternatively, we may seek to minimise a linear functional of P_i, which expresses the total cost of prestressing, subject to the nonlinear constraints $f(\sigma(\mathbf{x}))-r_d(\mathbf{x})\leq 0$ at more than n_p locations **x**. Usually in good approximation the state of stress within an area of the structure is affected only by the value P of the prestressing force of a single group of FRP tendons, typically located within the same area:

$$\sigma(\mathbf{x}) \approx S_d(\mathbf{x}) + P \, S_p(\mathbf{x}) \tag{6}$$

The meaning of the combination of Eqs. (2) and (6) is shown in Fig. 3. P is the distance of the stress point $S_d(\mathbf{x})$ due to the design actions from the design ultimate limit state, Eq. (2), measured along the direction defined by the stress vector $S_p(\mathbf{x})$ through the point $S_d(\mathbf{x})$. This problem has a solution only if the arrangement of the tendons is such that the line through $S_d(\mathbf{x})$ along the direction of $S_p(\mathbf{x})$ intersects the design ultimate limit state, Eq. (2).

Once the values of the P_i have been determined, the cross-sections of the tendons are computed on the basis of the FRP design strength, $f_{FRP,d}=f_{FRP,t}/\gamma_{FRP}$. The value of the partial safety factor γ_{FRP} depends on the dispersion of the FRP strength (typical coefficient of variation 2-5%), and the ratio between its mean and characteristic values, taking into consideration the consequences of its (brittle) failure. For the design of the anchorage by epoxy-bonding to the steel plates, the ratio of the anchorage length to the thickness of the FRP strip should (roughly speaking) not be less than one-half the ratio between the design strength values of the FRP in tension and the epoxy in shear. Finally, for the anchorage detail of Fig. 1(b), the bearing strength of the masonry should be checked, taking into account the beneficial effect of stress triaxiality under the contact plates.

5. CONCLUSIONS

Fibre reinforced plastics offer many advantages as strengthening materials of historic structures: they have excellent physical and mechanical properties, are lightweight and immune to corrosion, and may be applied in a reversible manner in the form of circumferential externally attached tendons in a colour matching that of the external surface of the structure. Concepts for the application and attachment of post-tensioned FRP ties are developed, along the lines of providing horizontal confinement to masonry structures and minimising anchor lengths. The design of the strengthening scheme can be accomplished on the basis of the general design procedure proposed here, which is applicable to any type of masonry structure and material.

6. ACKNOWLEDGEMENT

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Effectiveness of Seismic Strengthening Measures

Efficacité des interventions de renforcement parasismique

Wirksamkeit von Verstärkungsmassnahmen gegen seismische Einwirkungen

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SUMMARY

The effectiveness of alternative techniques for seismic strengthening of stone-masonry buildings is studied by linear elastic Finite Element Analyses, using thick-plate/plane-stress combination elements to model the in-plane and out-of-plane behaviour of the walls. A biaxial failure criterion is developed for stone-masonry and used for safety checking. Alternative structural measures are compared on the basis of the resulting average reduction of the 'equivalent' biaxial stress in the masonry.

RÉSUMÉ

L'efficacité de techniques alternatives de renforcement parasismique de bâtiments en pierre est étudiée par des analyses linéaires, élastiques suivant la méthode des éléments finis. Le comportement des murs dans et hors de leur plan est simulé usant des éléments plats, épais sous un état bi-dimensionnel de contraintes. Un critère biaxial de rupture développé pour la maçonnerie en pierre a été utilisé pour la vérification. Les méthodes alternatives d'intervention ont été comparées entre elles à la base de la réduction moyenne des contraintes biaxiales équivalentes de la maçonnerie.

ZUSAMMENFASSUNG

Die Wirksamkeit verschiedener Methoden zur Verstärkung von Mauerwerksgebäuden gegen seismische Belastungen wird mit Hilfe linear-elastischer Finite-Elemente-Berechnungen untersucht. Zur Simulation des Verhaltens von Wänden in Scheiben- und Plattenwirkung, werden kombinierte Elemente von dicken Platten und Scheiben verwendet. Zur Kontrolle der Sicherheit wird ein biaxiales Versagenskriterium für Mauerwerk entwickelt. Die alternativen Verstärkungsmassnahmen werden aufgrund der resultierenden durchschnittlichen Reduktion der äquivalenten biaxialen Spannung im Mauerwerk miteinander verglichen.

1. INTRODUCTION

Stone or brick masonry or combinations thereof is the traditional construction material in Europe. Old masonry buildings, most of them constructed prior to this century, are common in our modern cities and towns, providing them with their traditional character and sense of historical continuity. For this reason old buildings with little individual historical or architectural value are often restored and put into new use. To a certain extent this usually requires structural interventions, to reverse the effects of post structural deterioration and damage and/or to bring the old building up to the safety level required from new structures by modern structural design codes. In seismic regions, e.g. in most of Southern Europe, old buildings often have suffered significant structural damage during past earthquakes and are subject to higher future risk. Therefore in such regions the importance of strengthening interventions is larger. Because of the low architectural and historical importance of most individual old buildings, interventions are not subject to strict requirements of reversibility and absolute respect to the original type and material of construction. Accordingly, the type and extent of structural interventions is usually decided on the basis of cost considerations. In this respect it is useful to have a general idea of the structural effectiveness of the various possible alternative strengthening measures, especially as the latter are often of a non-engineered nature, i.e. they are empirically applied, without design calculations or safety checks.

Field observations from past earthquakes, as well as detailed analytical studies by the first two authors [1], have shown that seismic damage to low-to-medium rise masonry buildings with flexible (e.g. timber) floors is mainly due to out-of-plane horizontal forces on the walls. These forces induce large magnitude nearly horizontal tensile stresses due to out-of-plane bending of the walls, as well as horizontal transfer forces at the intersections of orthogonal load-bearing walls. The former induce nearly vertical cracking, especially over upper storey openings, and out-of-plane overturning of the walls, whereas the latter cause separation of the walls from the transverse ones. This type of action and damage calls for the introduction of horizontal elements, such as reinforced concrete tie-beams or slabs and horizontal prestressing, to resist the horizontal tensile stresses in the walls, and to tie them together. In [2,3] the first two authors have compared analytically the effectiveness of such strengthening devices to that of vertical ones, such as tie-columns and vertical prestressing, and of universal interventions, such as one- or two-sided shotcrete jacketing of the walls. The tool used was the Finite Element linear elastic Analysis in three dimensions, applied to three two-storey (plus basement) stone-masonry buildings in Kalamata, Greece, statically subjected to the estimated horizontal response acceleration of 0.42g of the Kalamata 1986 earthquake, separately in the two horizontal directions but simultaneously with the gravity loads. The effectiveness of the various strengthening measures was quantified by computing the average reduction in the magnitude of the principal tensile stress in the masonry over each individual wall, storey or building, effected by each intervention.

In the present paper the same three buildings, considered typical of Greek and other Southern European stone-masonry construction of the 19th and early 20th century, are studied under a larger variety of strengthening interventions or combinations thereof under a horizontal response acceleration of 0.4g, equal to the design acceleration of stone-masonry buildings (behaviour factor equal to 1.5) in the main seismic-prone area of Greece (Zone 3, with a design ground acceleration of 0.24g). The main difference, though, with the earlier study [2,3] is the failure criterion used: Instead of the principal tensile stress criterion, which is certainly inadequate for biaxial stress conditions involving a significant compressive principal stress as well, an-isotropic multiaxial failure criterion is developed herein and fitted to biaxial test results, and applied further over a denser grid of points over the surface of the masonry walls.

2. FAILURE CRITERION OF STONE MASONRY UNDER MULTIAXIAL STRESSES

Uncoursed rubble stone masonry is the typical material of old masonry structures, especially historic ones, in Greece and other Southern European countries, including the infill of brick-faced walls of

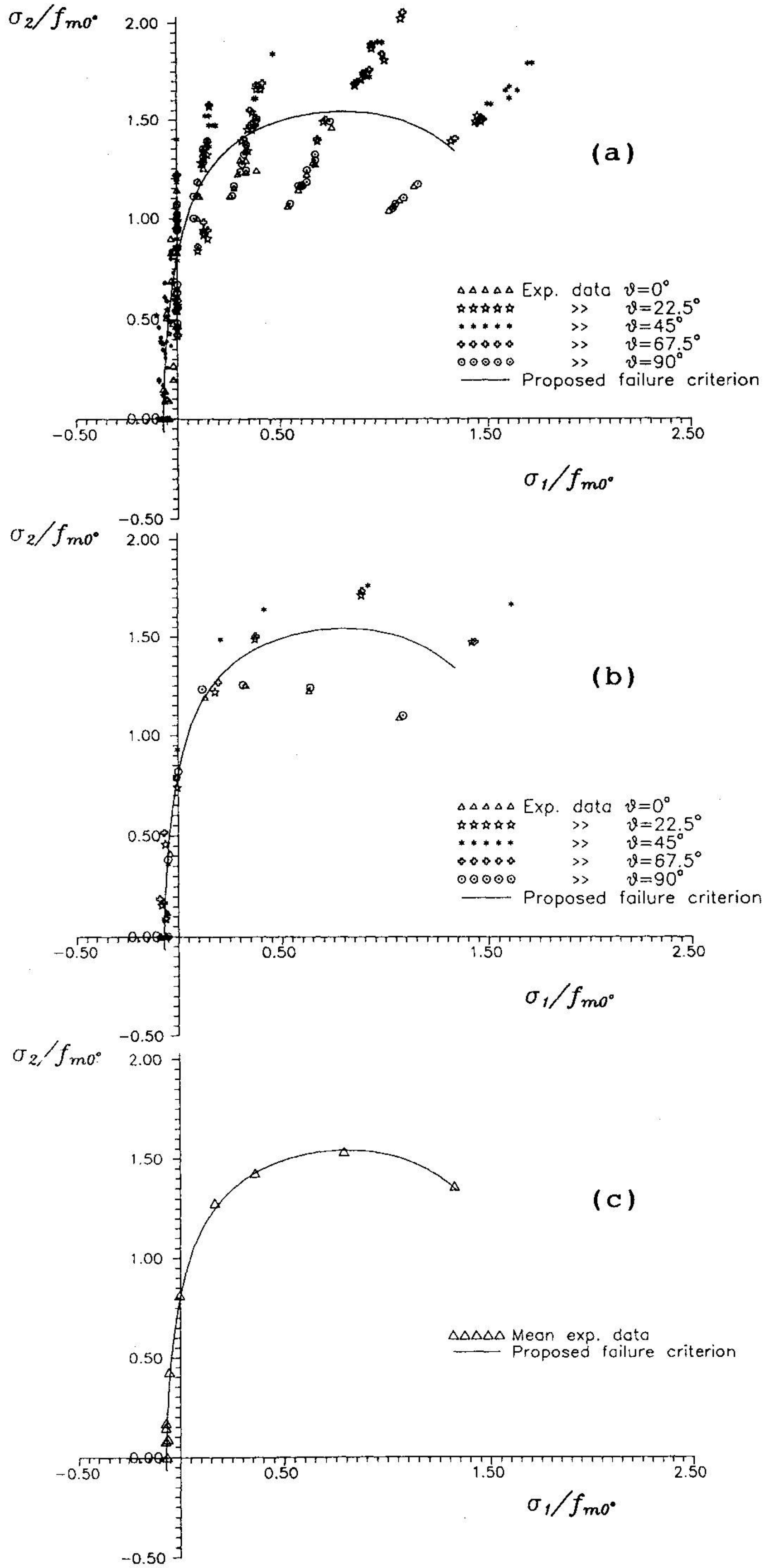

Fig. 1 Test data from [4, 5] and proposed failure criterion. (a) Individual data; (b) average data for each θ value; (c) average data for all θ.

medieval and Renaissance monuments. Despite of its importance, though, it has never been the subject of a systematic experimental investigation, such as those reported for brick masonry under biaxial stresses [4, 5]. In view of this lack of test data the authors had to resort to these latter results on solid brick masonry tested to failure under a variety of biaxial tension-compression and compression-tension principal stress combinations, oriented at various angles θ equal to 0^o, 22.5^o, 45^o, 67.5^o and 90^o with respect to the bed joints, and to fit a biaxial failure criterion to them after removing the dependence on the angle of θ.

The individual test data in [4, 5] are presented in Fig. 1(a) in biaxial principal stress space, using a different symbol for each of the five values of θ above. Fig. 1(b) shows the average of the test data separately for each value of θ. It is clear from Fig. 1(b) that in the compression-compression range strength is systematically and significantly lower for principal stresses parallel and normal to the bed joints ($\theta=0^o$ and $\theta=90^o$), whereas in the range from 22.5^o to 67.5^o the exact value of θ is not very important. It is assumed herein that the behaviour of isotropic masonry, such as stone masonry of the type considered herein, will be close to the average of all data, regardless of the value of θ. Accordingly a failure criterion was fitted to these average data as shown in Fig. 1(c). This criterion follows the four-parameter model proposed by Ottosen [6] for the failure of concrete under triaxial stresses:

$$\alpha\frac{J_2}{f_w}+\lambda\frac{\sqrt{J_2}}{f_w}+\beta\frac{I_1}{f_w} = 1 \tag{1}$$

in which $I_1=\sigma_1+\sigma_2+\sigma_3$ is the first stress invariant, $J_2=[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2]/6$ is the second deviatoric stress invariant, f_w is the uniaxial compressive strength of masonry and parameter λ equals:

$$\lambda=c_1\cos\frac{\cos^{-1}(c_2\cos 3\theta)}{3} \qquad if \quad \cos 3\theta\geq 0 \tag{2a}$$

$$\lambda=c_1\cos\left(\frac{\pi-\cos^{-1}(-c_2\cos 3\theta)}{3}\right) \qquad if \quad \cos 3\theta<0 \tag{2b}$$

in which $\cos 3\theta=3\sqrt{3}J_3/2J_2$, with $J_3=(\sigma_1-I_1/3)(\sigma_2-I_1/3)(\sigma_3-I_1/3)$ the third deviatoric stress invariant.

For given value of the "shape" parameter c_2 parameters α, β and c_1 can be computed from:

$$\alpha=\frac{9-(1+\frac{\lambda_1}{\lambda_2})\frac{3}{b}+(1-2\frac{\lambda_1}{\lambda_2})\frac{3}{f}}{3-(1+\frac{\lambda_1}{\lambda_2})b+(1-2\frac{\lambda_1}{\lambda_2})f} \tag{3a}$$

$$\beta=\frac{1}{3}\left(\frac{1}{f}-\frac{1}{b}+\frac{b-f}{3}\alpha\right) \tag{3b}$$

$$c_1=\frac{1}{\lambda_2\sqrt{3}}\left(\frac{2}{f}+\frac{1}{b}-\frac{2f+b}{3}\alpha\right) \tag{3c}$$

in which $\lambda_1=\cos((\pi-\cos^{-1}c_2)/3)$, $\lambda_2=\cos((\cos^{-1}c_2)/3)$, $f=f_{wt}/f_w$ is the ratio of uniaxial tensile and compressive strengths (equal to 0.085 on the average in Figs. 1) and b is the strength ratio in equal biaxial to uniaxial compression (1.65 on the average in Figs. 1). The best fit to the average data in Figs. 1 is obtained for $c_2=0.959$, in which case Eqs. (3) give $\alpha=0.665$, $\beta=3.84$ and $c_1=13.8$

The four-parameter model by Ottosen provides a very good fit to the failure data for concrete under triaxial stress conditions, $\sigma_1, \sigma_2, \sigma_3\neq 0$, albeit with parameter values very different from the ones fitted herein to the average biaxial strength data for masonry. Due to the similarities between concrete and uncoursed rubble stone masonry, it is expected that Eqs. (1) and (2) along with the above values of the four parameters will provide a good fit even to the triaxial strength of masonry ($\sigma_3\neq 0$).

The proximity of a biaxial stress state (σ_1,σ_2) to the failure criterion of Eq. (1) is quantified herein by computing the proportionality factor σ^* such that the stress point ($\sigma^*\sigma_1$, $\sigma^*\sigma_2$) lies on the failure envelope, Eq.(1). So the scaling factor σ^* has the meaning of an "equivalent" stress under biaxial conditions, normalised to its failure value, and the value of $1/\sigma^*$ can be considered as a safety factor against failure of the masonry, with $\sigma^*<1$ signifying a safe stress condition inside the failure envelope and $\sigma^*>1$ implying failure due to fictitious elastic stresses σ^*-times beyond failure.

3. INTERVENTION MEASURES AND THEIR FINITE ELEMENT MODELING

The intervention measures considered in the present study cover the entire range of techniques commonly applied in Greece and in other Southern European countries for seismic strengthening of old masonry structures. As these techniques have been described in detail in [2, 3] they are only listed here, along with some remarks regarding their F.E. modeling.

1. Introduction of through-thickness 0.3m-deep horizontal **reinforced concrete (R.C.) tie-beams** at the levels of the floors and at the top of all load-bearing walls.
2. Construction of vertical **R.C. tie-columns** at the corners and intersections of all load-bearing walls, with horizontal dimensions equal to those of the common area in plan of the intersecting walls.
3. Replacement of timber floors by rigid within their plane **reinforced concrete slabs**.
4. Application of a 60mm-thick shotcrete layer on both sides of the wall, to create a **two-sided or double shotcrete jacket**.
5. As in 4. above, but on the external or the internal face of the exterior walls, to create a **one sided or single shotcrete jacket**.
6. Concentric **horizontal prestressing** of the spandrels over openings of the walls, at a level of prestressing force corresponding to an average horizontal compressive stress in the spandrel equal to 10% or 20% of the uniaxial compressive strength of the masonry, f_w.
7. Concentric **vertical prestressing** of all the piers of the wall, at a prestressing force level corresponding to a mean vertical compressive stress in the pier equal to $0.1f_w$ or $0.2f_w$.

Two-way or three-way combinations of the individual interventions are also considered:

8. 3 plus 1, i.e. R.C slabs at floor levels and a 0.3m deep circumferential tie-beam at the top of the wall.
9. 2 plus 1 at the top, i.e. reinforced concrete tie-columns at the intersections of load-bearing walls and a 0.3 m deep circumferential tie-beam at the top.
10. 7 plus 1 at the top, i.e. vertical prestress of the piers at an average compressive stress level of $0.1f_w$, along with a 0.3 deep horizontal R.C tie-beam at the top of the wall for anchorage of the tendons.
11. 6 plus 7, i.e horizontal and vertical prestressing at the two prestress levels mentioned above, i.e. at nominal average stresses of $0.1f_w$ and $0.2f_w$.
12. 1 plus 2 plus 3, i.e. reinforced concrete slabs at the floor levels, R.C. tie-columns at the wall corners, etc., and a 0.3m deep R.C. tie-beam at the top of the wall.
13. 4 plus 1 plus 3, i.e. one-sided shotcrete jacket combined with R.C. slabs at floor levels and with a R.C. tie-beam at the top.
14. 5 plus 1 plus 3, i.e. two-sided shotcrete jacket along with R.C slabs and with R.C tie-beam at the top of the wall.

Walls are modeled using a dense grid of thick (Midlin) plate bending - plane stress combination 4- to 8-node isoparametric Elements. Element dimensions are about 0.5 to 0.6 m on the average, and over a thousand Elements are used for each building. Reinforced concrete tie-beams and tie columns are modeled by assigning the Elastic Modulus of concrete to the corresponding elements of the F.E. model. As the main effect of reinforced concrete slabs is their diaphragmatic action, they are modeled by kinematically constraining all nodes of the F.E. model at the level of a floor into a rigid body motion within a horizontal plane. Shotcrete jackets are modeled by considering the thick-plate Elements as layered, with the 60mm outer layer(s) assigned the Elastic properties of concrete and inner core assigned those of the masonry. Finally, prestressing forces, horizontal or vertical, are introduced as consistent line loads along those F.E. boundaries where tendons are anchored.

4. RESULTS AND CONCLUSIONS

From each F.E. Analysis nodal stresses σ_x, σ_y and τ_{xy} within the plane of the wall are computed at both surfaces by surface-extrapolation from those at the Gauss points of the Element, and then averaged over the Elements connected to the node. Principal stresses σ_1 and σ_2 computed thereof are used to compute the value of the "equivalent" nondimensional stress σ^*. At each nodal point the maximum value of σ^* on either surface of the wall over all combinations of interest of the gravity load with the seismic action provides a measure of the most adverse biaxial stress conditions there. Contours of this maximum value of σ^* give a picture of the distribution of seismic demand over each wall. At a given point in the wall of the strengthened building the ratio of the maximum σ^*-value as above to that in the unstrengthened building measures the reduction in masonry biaxial stresses due to the intervention, and provides a local measure of the effectiveness of strengthening. The mean value of this ratio over the entire wall, over a storey of the building or over the entire building provides an average measure of the effectiveness of the intervention. The average value of this ratio over the three buildings is listed in Table 1 for each strengthening technique, separately for the walls which are normal to the seismic action to show the effectiveness of strengthening for the most-important out-of-plane behaviour, then for those which are parallel to it for the less-important in-plane one, and finally independent of the direction of the wall relative to the seismic action, i.e. for the most adverse direction of the latter. In the first line, denoted by "everywhere", the average ratio of the σ^*'s over all nodal points in the wall is listed, whereas in a second line, denoted as "critical regions", the average is taken only over those nodal points where the value of σ^* in the unstrengthened building exceeds 0.9. The second line results bear more gravity regarding the effectiveness of intervention than those of the first, and almost invariably show larger effectiveness in the critical regions than overall.

Among the individual interventions not-surprisingly the two-sided jackets come out as most effective, reducing biaxial masonry stresses by about 60% in the critical regions and by more than 40% overall. R.C. slabs, R.C. tie-beams and one-sided shotcrete jackets are almost equally effective, reducing stresses by about 1/3 in the critical regions and by 20 to 25% overall. Prestressing at a mean nominal stress of 0.2 f_w reduces critical region biaxial stresses by about 25%, when applied in the horizontal direction or by 20% when applied along the piers. Reducing the level of prestressing force by half has a less than proportional effect, as critical region stresses are reduced by about 1/6, with horizontal prestressing being slightly superior. R.C. tie-columns have a minor impact on the level of stresses.

Among the two-way combinations the difficult-to-construct horizontal and vertical prestressing at a nominal average stress of 0.2 f_w in both directions is very effective, reducing biaxial stresses in the critical regions by more than 50%, and with a high degree of repeatability among the buildings. This is the result of the beneficial effect of increasing compressive stresses on biaxial failure, as shown in Fig. 1. Adding a tie-beam at the top of the wall significantly increases the effectiveness of R.C. slabs or R.C. tie-columns, as average stress reduction in the critical regions rises to 45% or to 20%, respectively. The corresponding value is about 35% or higher when vertical prestressing at a nominal average stress of 0.1 f_w is combined with a tie-beam at the top of the wall, or with horizontal prestressing at the same level of average nominal stress. Finally, the three-way combinations do not offer a very significant advantage over their individual constituents, as a two-sided jacket plus R.C. slabs and a tie-beam at the top is a little better than the jacket alone, the same combination with the one-sided jacket is slightly better than the slabs and the tie-beam without the jacket, whereas the addition of R.C. tie-columns to the combination of slabs with a tie-beam at the top does not improve the effectiveness of the latter.

Most note-worthy among the results above are a) the relatively low effectiveness of the one-sided jacket; b) the good performance of prestressing in both directions at a nominal average stress level of 0.2 f_w and of the combination of R.C. slabs with a tie-beam at the top; and c) the relatively limited improvement effected by a three-way combination of interventions.

Table 1. Average Ratio of the Equivalent Stress in the Strengthened to those in the Unstrengthened Building

Intervention		Walls parallel to seismic action			Walls normal to seismic action			Irrespective of seismic direction		
		1st story	2nd story	Building	1st story	2nd story	Building	1st story	2nd story	Building
R.C Tie-Beams	everywh.	0.92	0.76	0.82	0.76	0.56	0.73	0.97	0.61	0.77
	cr. reg.	0.86	0.60	0.72	0.51	0.52	0.57	0.77	0.57	0.68
R.C Slabs	everywh.	0.74	0.78	0.79	0.66	0.86	0.77	0.70	0.82	0.76
	cr. reg.	0.59	0.67	0.63	0.42	0.73	0.70	0.59	0.72	0.67
R.C Tie-Columns	everywh.	1.28	1.13	1.24	0.96	0.89	0.96	1.18	0.97	1.08
	cr. reg.	1.06	1.03	1.05	0.87	0.84	0.84	0.93	0.94	0.96
One-sided (Single) Jacket	everywh.	1.02	0.78	0.95	0.76	0.59	0.65	0.93	0.66	0.81
	cr. reg.	0.79	0.68	0.74	0.60	0.57	0.58	0.74	0.65	0.68
Two-sided (Double) Jackets	everywh.	0.87	0.57	0.76	0.46	0.29	0.39	0.74	0.39	0.58
	cr. reg.	0.57	0.38	0.46	0.23	0.26	0.25	0.49	0.32	0.40
Horizontal Prestressing at 0.1 fw	everywh.	0.83	0.81	0.85	0.88	0.86	0.86	0.89	0.87	0.91
	cr. reg.	0.79	0.70	0.75	0.74	0.75	0.80	0.81	0.79	0.83
Horizontal Prestressing at 0.2 fw	everywh.	0.86	0.88	0.91	0.95	0.86	0.96	0.91	0.82	0.91
	cr. reg.	0.68	0.58	0.66	0.64	0.63	0.71	0.72	0.66	0.75
Vertical Prestressing at 0.1 fw	everywh.	0.93	0.91	0.89	1.06	0.83	0.90	0.89	0.83	0.83
	cr. reg.	0.86	0.92	0.88	0.89	0.88	0.85	0.87	0.89	0.84
Vertical Prestressing at 0.2 fw	everywh.	1.01	1.03	0.98	1.25	0.85	0.97	0.92	0.83	0.83
	cr. reg.	0.81	0.93	0.85	0.89	0.86	0.79	0.88	0.87	0.80
Slabs + Tie-Beam at the top	everywh.	0.75	0.74	0.78	0.63	0.52	0.62	0.69	0.57	0.66
	cr. reg.	0.65	0.69	0.59	0.30	0.49	0.45	0.56	0.50	0.55
Tie-Columns+ Tie-Beam at the top	everywh.	1.24	1.01	1.16	0.88	0.63	0.80	1.13	0.77	0.98
	cr. reg.	1.01	0.82	0.92	0.74	0.55	0.62	0.93	0.69	0.80
Horiz.+Vert. Prestressing at 0.1 fw	everywh.	0.72	0.66	0.69	0.87	0.65	0.74	0.73	0.67	0.69
	cr. reg.	0.60	0.60	0.61	0.58	0.61	0.59	0.65	0.65	0.65

Table 1 (continued)

Horiz.+Vert. Prestressing at 0.2 fw	everywh.	0.74	0.53	0.71	1.00	0.59	0.77	0.69	0.55	0.61
	cr. reg.	0.46	0.44	0.50	0.42	0.43	0.42	0.47	0.46	0.47
Vert. Prestr. +Tie-beam at the top	everywh.	0.91	0.71	0.81	1.03	0.55	0.77	0.85	0.53	0.70
	cr. reg.	0.82	0.60	0.69	0.79	0.49	0.55	0.81	0.54	0.63
R.C.Slabs + Tie-Beam + Tie-Columns	everywh.	1.16	1.01	1.15	0.71	0.65	0.73	1.05	0.74	0.93
	cr. reg.	0.88	0.77	0.84	0.38	0.55	0.51	0.77	0.65	0.72
Double Jack. R.C. Slabs + Tie-Beam	everywh.	0.82	0.54	0.71	0.38	0.17	0.30	0.71	0.33	0.54
	cr. reg.	0.50	0.28	0.39	0.12	0.15	0.15	0.42	0.24	0.32
Single Jack.+ R.C. slabs + Tie-Beam	everywh.	0.99	0.72	0.89	0.54	0.30	0.52	0.88	0.46	0.71
	cr. reg.	0.70	0.40	0.54	0.21	0.29	0.32	0.59	0.36	0.49

ACKNOWLEDGEMENT

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Strengthening Landmarks for Improved Seismic Performance

Renforcement des édifices historiques et amélioration de leur résistance sismique

Erdbebenertüchtigung von Baudenkmälern

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SUMMARY

Historic landmarks in California are relatively modern compared with the landmarks of Europe and Asia, but recent experiences in strengthening such structures for improved seismic performance is applicable to their more ancient counterparts. Evaluation procedures, criteria and strengthening techniques are discussed with several recent case studies cited as examples. Creative strengthening techniques not only improve public safety during future earthquakes but protect historic fabric and preserves our landmark structures for future generations to enjoy.

RÉSUMÉ

Les édifices historiques en Californie sont relativement modernes en comparaison avec ceux d'Europe et d'Asie, mais les expériences récentes pour renforcer ces bâtiments et pour améliorer leur résistance sismique sont aussi applicables à des constructions plus anciennes. Une évaluation des procédés, des critères et des moyens techniques de renforcement est discutée dans plusieurs études citées comme exemples. De nouvelles techniques de renforcement améliorent non seulement la sécurité du public lors de tremblements de terre, mais protègent et conservent aussi les édifices historiques pour la joie des générations à venir.

ZUSAMMENFASSUNG

Historische Wahrzeichen Kaliforniens sind verhältnismässig neu, verglichen mit jenen in Europa und Asien. Trotzdem sind die jüngsten Erfahrungen in der Erhöhung der seismischen Tragfähigkeit solcher Bauwerke in Kalifornien auf ihre älteren Ebenbilder anwendbar. Eine Beurteilung der Verfahren, Kriterien und Verstärkungstechniken bilden Gegenstand der mit kürzlichen Fallstudien illustrierten Abhandlung. Kreative Methoden zur Tragwerksverstärkung verbessern nicht nur die öffentliche Sicherheit im Falle eines Erdbebens, sie dienen auch der Erhaltung historisch wertvoller Bausubstanz zur Freude künftiger Generationen.

INTRODUCTION

Many historic and landmark structures located in regions of seismic activity are vulnerable to extensive damage and possible collapse in severe earthquakes. In order to increase the probability that these structures will survive future earthquakes with minimal or repairable damage, it is frequently desirable to strengthen the lateral force resisting system of these structures for improved performance.

Strengthening historic landmarks presents challenging tasks to engineers as strengthening must be executed to meet life-safety and performance goals while minimizing the impact on the historic fabric of the structure. This creates conflicts which require sensitivity to both historical preservation principles as well as realistic seismic performance. This balance between preservation and seismic safety can be achieved, but it requires careful evaluation of alternatives and acceptance of interventions which greatly improve seismic performance with minimal impact on appearance and historic fabric.

The procedures and examples described in this paper are based on experience in the State of California on the west coast of the United States. The landmark structures of California are generally only about one century old as appreciable western civilization came to the west coast of North America only 140 years ago after the discovery of gold in California. Despite the modern character of these California landmarks when compared to the landmarks of Europe and Asia, most of them contain unreinforced stone and brick masonry similar to the more ancient landmarks.

EVALUATION OF SEISMIC RESISTANCE

The first step in the process is to evaluate the seismic resistance of the existing building. Sometimes this is prompted by some damage from a small or moderate earthquake, as has been the recent case in the San Francisco Bay Area following the Loma Prieta earthquake of 1989. This damage from the smaller earthquake usually occurs at weak links in the lateral force resisting system or indicates deficiencies in its seismic resistance. Lacking an earthquake of sufficient intensity to highlight all the structure's vulnerabilities, it is necessary to evaluate the structure carefully using considerable engineering judgment.

The evaluation process first demands a thorough understanding of the structural and nonstructural elements and how they are interconnected. If drawings are not available, this requires drilling or cutting small holes or performing nondestructive testing to determine weights and composition of elements. Connections between elements are particularly important and must be verified. The strength of key structural elements should be determined by testing. In addition to concrete and steel testing by removing samples, the most common test is an in-place push test to evaluate the strength of masonry mortar in shear. The common test now being used in California for brick masonry involves removing a brick and the mortar from the end of the adjacent brick, inserting a hydraulic jack in the space of the removed brick and determining the in-place shear strength of the mortar.

The evaluation process is always based on a conventional lateral force and gravity analysis of the structure. However, the analysis must also be based on considerable engineering judgment and a sound understanding how similar landmark structures have performed in past earthquakes. In addition to evaluating basic wall and horizontal diaphragm strength, the interconnection between these elements must be carefully considered. A complete stress path must be evaluated throughout the structure. All elements must be tied together, as historic floor or roof arches and walls tend to spread and lose their structural integrity in strong shaking. Discontinuities in the lateral force resisting system need to be carefully evaluated as historic structures seldom have the ability to adequately transfer lateral forces to other bracing elements. Common sense and sound engineering judgment based on experience is needed more than sophisticated analytical procedures.

Criteria for the evaluation is also necessary and important. Building codes in force have been developed for new construction and seismic resistance requires proper material detailing for ductility. Unreinforced masonry is not permitted as it exists in the landmark structure. Thus, a code or criteria recognizing archaic materials is essential. The State of California has developed a State Historic Building Code which gives latitude to engineers to exercise judgment in assigning seismic resistance to archaic materials.

The evaluation will lead to a decision of seismic adequacy of the landmark or if seismic strengthening is required.

STRENGTHENING THE LANDMARK STRUCTURE

Strengthening a landmark structure can take many forms. It can consist of only adding some ties or other elements to correct a specific deficiency. It can consist of strengthening connections between walls and floors or it can be more extensive and add new bracing elements. Whatever solution is selected, it must be selected considering the historic fabric of the landmark and minimizing its impact on the historic features of the structure. Some impact is usually necessary and tradeoffs between structural integrity and historic preservation must be carefully evaluated.

The most common strengthening methods being utilized to strengthen landmarks include:

1. Adding reinforced concrete shear walls or buttresses. These may be new walls or shotcrete walls added against the historic masonry with finishes restored. Sometimes a wythe or two of masonry is removed and replaced with reinforced shotcrete so interior wood trims and ceilings will fit to original dimensions.
2. Adding structural steel diagonal bracing. Unfortunately, this system seldom achieves sufficient stiffness to protect the masonry from damage. Once the masonry cracks, the steel bracing can act and maintain structural integrity.
3. Seismic or base isolation consists of rebuilding the foundation to incorporate isolators that reduce the propagation of ground motion into the building. This method requires adding the isolators near the base of the structure and appropriate strengthening of the structure compatible with the isolation system. Criteria has recently been developed and design is currently underway to incorporate such systems in five or so San Francisco landmarks.
4. Center-coring consists of coring vertically down through masonry and grouting a reinforcing bar in each cored hole to reinforce masonry. The cores are usually wet-drilled although a technology has been developed to dry core so finishes do not have to be removed if susceptible to water damage. This system has been used, although improvements in directional control of the coring would be desirable and further testing is needed to give engineers confidence in design procedures.

The following are several examples of recent California projects:

Ferry Building - San Francisco

San Francisco's Ferry Building was built in the 1890s at the edge of San Francisco Bay on land fill. The three-story structure is about 200 m by 45 m in plan with reinforced concrete floors supported by structural steel beams and columns. The first floor columns are primarily cast iron. The roof is wood sheathing on light steel trusses and the facade is sandstone backed with brick. A tower rises from the center which is about 10 m square and 55 m high. The tower is framed with structural steel including eyebar diagonal X-bracing. The facade is sheet metal over wood above about 35 m. The foundations are timber piles through about 30 m of soft clays to sand bearing below. Figures 1 and 2 illustrate the building.

The building was heavily damaged in the 1906 San Francisco earthquake when eyebars in the tower both failed and permanently elongated about 50 mm. The sandstone on the tower was heavily damaged and replaced with reinforced concrete after the earthquake. Sandstone and brick infill in the low-rise was also damaged and repaired.

With the termination of ferry services in the 1940s after the Golden Gate and Bay bridges were constructed, the Ferry Building was converted to office and commercial space. The high Second Floor waiting room was compromised by adding a full Third Floor. The two sides of the building were altered at different times, the north half in the 1950s and the south half in the 1960s. A more substantial interior concrete wall system was added in the northern half while more of the historic fabric and finishes were maintained in the southern half.

The 1989 Loma Prieta earthquake caused more damage. Several columns in the tower buckled where the X-bracing had eccentric working points. The steel flagpole atop the tower bent as it did in 1906. The front facade of the low-rise aligning with the tower was permanently displaced about 20 mm. There was various damage and cracking of the masonry walls and parapet in the south half and the end wall cracked, failed its few connecting bolts to the roof and separated about 5 mm from the roof. There was no damage in the north half which was stiffer and stronger for lateral forces.

The United States federal government pays for repairs to local government buildings such as the Ferry Building through the Federal Emergency Management Agency (FEMA). Thus, the basic damage to the tower, flagpole and cracked masonry is being repaired. We were able to convince FEMA that the most suitable repair of parapet damage was to brace the parapet with reinforced concrete and steel braces. The sandstone facade in front of the tower that was permanently displaced will be left displaced but the stones will be anchored to new reinforced concrete backing. This includes the decorative facade columns which are a series of round or square stones atop each other with no steel reinforcement. The cracked and slightly leaning south wall will be reconstructed by removing two wythes of brick from the exterior (250 mm thick) and replacing it with reinforced concrete anchored to all floors and the roof. This new concrete both repairs the south wall damage, significantly strengthens the end of the building, preserves the historic interior brick and stone arched window at the Third Floor and allows us to restore previous modifications in the south wall by replicating the original arched windows and trancery. FEMA also agreed to fund these improvements as repairs.

Based on a detailed analysis at the beginning of the project, we also recommended adding some additional interior reinforced concrete shear walls beneath the tower and in the south half of the building. This work is also being funded by FEMA as a reasonable measure to reduce damage in future earthquakes. Care is being taken on locating these walls to minimize their impact on the historic fabric remaining on the interior of the building.

Memorial Church - Stanford University

Stanford University is located about 50 km south of San Francisco. The Memorial Church was built by Jane Stanford between 1899 and 1902 as a memorial to her husband, Leland Stanford, Jr., who founded the University. It was built in the classic cruciform shape with the nave, transepts and chancel meeting in the crossing, an area bounded by four tall stone and brick arches supporting a 15 m diameter dome and four mosaics on plaster of the archangels. The Church was built of unreinforced stone masonry with wood roof and a steel supported tower above the dome of the crossing. The new Church was severely damaged in the 1906 San Francisco earthquake and was completely rebuilt with reinforced concrete walls with stone veneer except for the crossing. The steel tower was removed above the interior wood and plaster dome. Figure 3 is an overall photograph of the Church.

In the 1989 Loma Prieta earthquake, the "new" concrete walls performed well but the original arches of the crossing were damaged. The arches are about 22 m high and each consists of two 1.6 m deep by 550 mm wide stone arches with carved faces. The pairs of arches support a brick wall and are separated by a void about 0.5 m wide where the structural steel for the original tower is located. In the 1989 earthquake, several of the arches moved perpendicular to their plane and caused stones near the center of the arch to crack and drop about 25 mm. At the junction of the arches, the plaster mosaics were damaged with a portion falling to the floor below.

The damage was analyzed, the conditions determined and an analysis was completed. It was decided to strengthen the arches by filling the void in their center with some heavily reinforced concrete, to provide a diaphragm at the top of the walls above the arches consisting of a substantial reinforced concrete cap beam and steel diagonal braces. The added steel included a stiff downward cantilever to stiffen the arches against perpendicular movement. The reinforced core between the arches was also tied horizontally to the reinforced concrete walls of the four projections from the crossing to improve structural ties and the transfer of lateral forces. The wood roof diaphragms over the entire Church were strengthened with new plywood and stronger ties and connections at the concrete walls. All of this work was completed in concealed locations with no visible evidence of the repairs and strengthening. Figure 4 illustrates the strengthening scheme.

The decorative mosaics at the corners of the crossing were found to have partially debonded from their supporting plaster in addition to the portion that fell. An elaborate support system was developed to maintain the mosaics in their location while the plaster backing and a new fiberglass backing to steel supports was installed. Tests were conducted to insure adequate bonding of the fiberglass to the mosaics and the steel supports including epoxy products to insure bonding to some steel supports.

Museum of Art - San Jose

The Museum of Art in San Jose, about 70 km south of San Francisco, was built as the United States Post Office about 1890. It is a two-story plus basement structure with an 18 m high clock tower. The exterior walls are rough cut sandstone with brick backing and are bearing walls. Floor construction consists of brick arches supported by steel beams and girders spanning to cast iron columns. Figure 5 illustrates the building.

The building was extensively damaged in the 1906 San Francisco earthquake when the tower collapsed. The building was rebuilt with minor revisions to the top of the tower.
In the 1989 Loma Prieta earthquake, there was minor damage consisting of cracking in the tower near the bottom of the 1906 collapse. There was no damage in the lower floors, but in a partial Third Floor attic slab, there was significant cracking at locations where the steel framing and brick arches changed directions. At those locations, there were no continuous tension ties in the floor diaphragm to prevent spreading and resist tension forces. In the lower floors, both the steel beams and the girder-to-column connections were bolted for continuity, providing tension capacity for the floors.

The tower and Third Floor attic have been repaired by strengthening, again funded by FEMA. The tower was strengthened by removing two wythes of brick on the interior and installing reinforced concrete applied as shotcrete. The brick was removed to keep room sizes and window recesses unchanged so original wood finishes could be reinstalled without modification. The Third Floor attic was strengthened by welding steel straps to the exposed top flanges of the steel beams in the unfinished attic to provide tensile ties in both directions of the attic slab.

A second project has completed the design phase to strengthen the remainder of the building. This is to comply with a retroactive ordinance of the City of San Jose requiring all unreinforced masonry bearing wall buildings to be strengthened or demolished in the interest of public safety. The strengthening scheme for the remainder of the building also includes reinforcing the exterior walls with reinforced concrete installed as shotcrete to strengthen those walls. Again, two wythes of brick are being removed in most places so original finishes can be restored. In addition, a new interior reinforced concrete shear wall is being added in the First Floor and Basement beneath a brick wall which forms the exterior of the reduced size Second Floor. These walls are being added to minimize horizontal stress transfers in the relatively weak Second Floor diaphragm so it does not have to be strengthened. New foundations are being constructed beneath the new walls and between the original spread footings. The strengthening work is shown on Figure 6.

CONCLUSIONS

Seismic strengthening of landmark structures is often desirable in regions of high or moderate seismicity to provide public safety as well as to preserve our historic heritage of buildings for future generations. Historic and landmark structures are among the most vulnerable in strong ground shaking and often subject to possible collapse.

Strengthening is performed after a thorough evaluation of the seismic resistance and potential performance of the structure. The evaluation is always based on an analysis of the structure but very strongly influenced by engineering judgment based on knowledge of how similar landmark structures have performed in past earthquakes.

Strengthening usually involves adding sufficient strength and stiffness to resist a significant percentage of the lateral forces, to prevent partial or complete collapse and to protect brittle historic fabric so the building will be easily repairable following a major earthquake. Since most landmark structures are at least partially built of unreinforced brick or stone masonry, strengthening most usually consists of adding reinforced concrete shear walls, often installed pneumatically as shotcrete. Other systems, such as adding steel diagonal bracing, providing a base isolation system combined with judicious strengthening or reinforcing the masonry by the center coring technique is possible. Important aspects of most strengthening schemes in insuring that the structure is positively tied together with adequate continuous tension capacity in all horizontal diaphragms and properly bracing all parapets and anchoring exterior stones and ornamentation to the structural system.

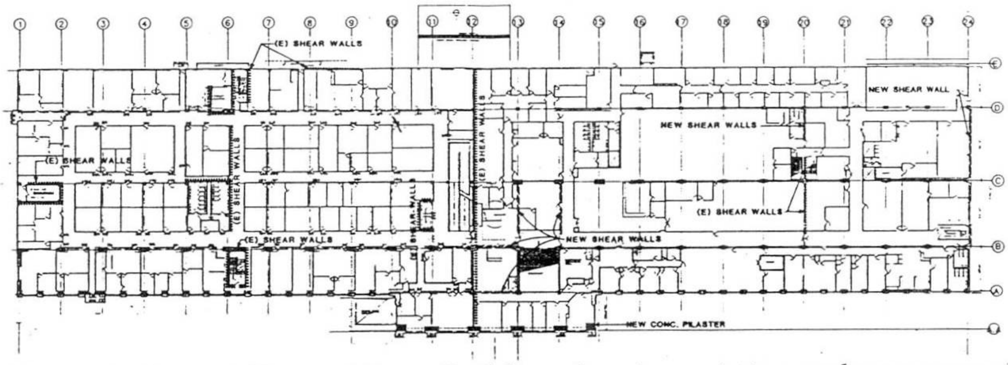

Figure 1. Second Floor of Ferry Building showing existing and recommended locations of shear walls. The tower is between columns 12-13-A-B.

Figure 2. Photograph of Ferry Building tower after Loma Prieta earthquake with bent flagpole.

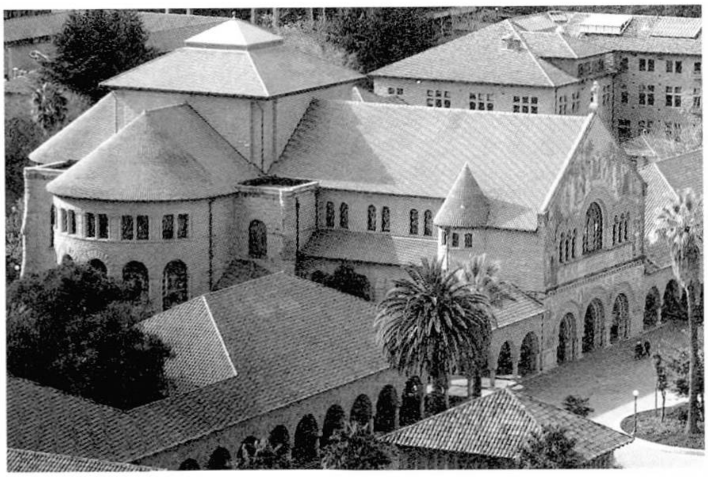

Figure 3. Stanford Memorial Church, Stanford University.

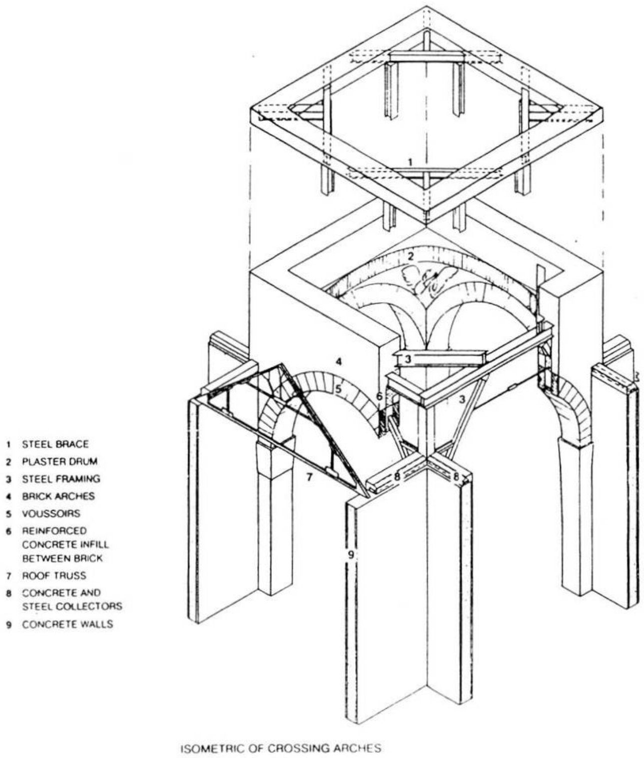

Figure 4. Isometric of strengthening of the Crossing Arches of Stanford Memorial Church.

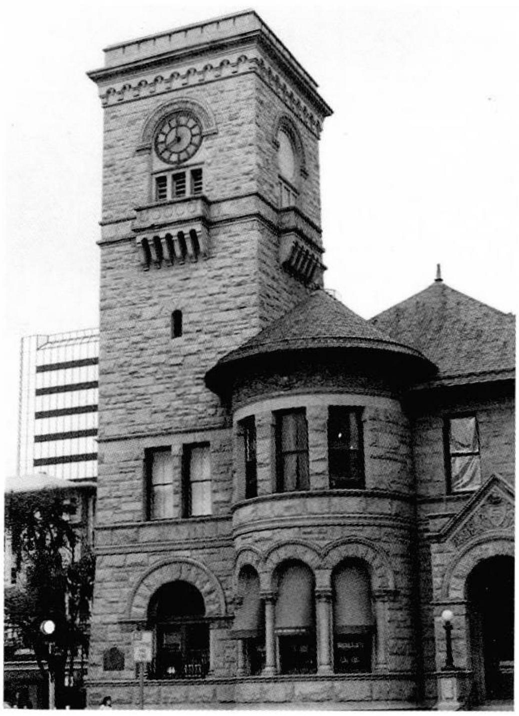

Figure 5. San Jose Museum of Art, formerly the United States Post Office.

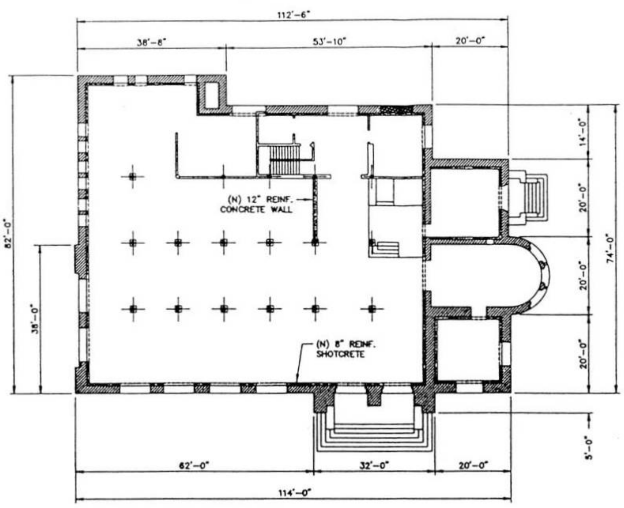

Figure 6. Proposed strengthening of San Jose Museum of Art - First Floor Plan.

Repair and Strengthening of Arch Bridges

Réparation et renforcement des ponts en arc

Ausbesserung und Verstärkung von Bogenbrücken

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SUMMARY

Masonry arch bridges form an important part on the highway network of the United Kingdom. It is essential to enable them to continue to play their part; the cost of replacing them would be enormous and many make a positive contribution to the landscape. This paper describes the common types of deterioration from which they suffer, including the results of a condition survey. Repair and strengthening methods are then examined, based on a survey of their cost and effectiveness. The importance of timely maintenance is demonstrated.

RÉSUMÉ

Les ponts en arc en maçonnerie constituent une partie importante du réseau routier de la Grande-Bretagne. Il est essentiel de leur permettre de continuer à jouer leur rôle; le coût de leur remplacement serait énorme et un grand nombre de ces ponts contribue d'une manière positive au paysage. Cet article décrit les types courants de détérioration qui affectent ces ponts et présente les résultats d'une enquête sur les coûts et l'efficacité de ces méthodes. L'importance d'effectuer l'entretien à temps est soulignée.

ZUSAMMENFASSUNG

Gemauerte Bogenbrücken bilden einen bedeutenden Teil des britischen Hauptstrassennetzes. Die Fortsetzung ihrer Rolle in der Zukunft ist wesentlich. Die Kosten für ihren Ersatz wären enorm. Viele leisten einen positiven Beitrag zum Landschaftsbild. Dieses Referat beschreibt die üblichen Schadensmechanismen an denen sie leiden, und enthält Ergebnisse einer Zustandsüberprüfung. Dann werden Ausbesserungs- und Verstärkungsverfahren im Hinblicke auf Kosten und Wirksamkeit untersucht. Die Bedeutung eines rechtzeitigen Unterhalts wird aufgezeigt.

1. INTRODUCTION

There are about forty thousand brick or stone masonry arch bridges in the United Kingdom, representing about forty percent of the bridge stock. Very few have been built since the first world war and many have reached the end of the present nominal design life for UK bridges of 120 years. It is not however either practicable or desirable to replace them. The cost would be enormous and many make a positive contribution to the landscape or are of historical or architectural importance.

Many of them have deteriorated due to the effects of weathering and traffic: some of the traffic they are now required to carry is much heavier than was envisaged when they were built. There are a variety of commonly used repair and strengthening methods used to maintain their function or to increase their load carrying capacity. This paper discusses the common types of deterioration and the effectiveness and cost of some repair and strengthening methods. Typical construction of masonry arch bridges is shown in figure 1.

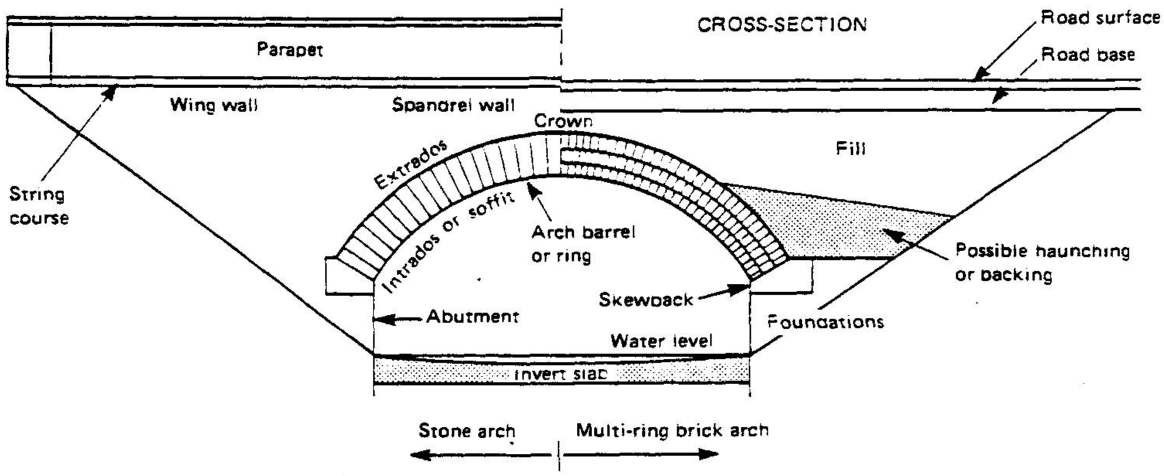

Fig 1 Typical masonry arch bridge construction

2. COMMON DEFECTS

2.1 Scour of river foundations

Scour is probably the most common cause of collapse of masonry arch bridges, because foundations are generally shallow. For example, severe rainfall in Southern Ireland in August 1986 resulted in considerable flood damage to many bridges; nine in County Wicklow alone. In all but one case failure was due to scour of the upstream side. River bed levels were lowered by up to 600mm, whereas foundations only extended to about 300mm below bed level.

Scour is difficult to detect because it is likely to be at its worst when the river is in flood and access is impossible. It may be made worse by fallen trees and other debris catching in the arch when the river is in flood. Scour holes may refill as floods subside and conceal undercutting of foundations.

2.2 Arch ring defects

2.2.1 Problems due to movement of abutments

Arch rings generate outward pressure on their abutments and may lead to outward movement. The fill behind abutments will resist the outward movement and may cause inward movement. The effect on the arch ring will depend on whether the movement is outwards or inwards and whether it is accompanied by rotation of the abutments. It is likely to manifest itself as transverse cracks in the arch ring.

Most arches would settle when the centring was removed during construction but would be expected to stabilise so recent cracks are a cause for concern as they indicate fresh movement.

2.2.2 Splitting beneath the spandrel walls

Fig 2 Crack in arch ring

Spandrel walls stiffen the arch ring at its edges. Flexing of the arch ring due to traffic loads will produce shear stresses in the ring where the relatively flexible part with only fill above it is stiffened by the spandrel wall, and these stresses may result in a crack. A severe example of such a crack is shown in figure 2. This type of failure may be assisted by rainwater getting into the structure at the parapet/surface joint and causing particular damage to the arch ring mortar where the spandrel wall meets the ring.

2.2.3 Ring separation

Ring separation is a common problem with multi-ring brick arches and may be due to deterioration of the mortar or may be load induced. Research [1] has shown that the load capacity is likely to be significantly affected. Tapping with a hammer is the technique commonly used to detect separation.

2.3 Spandrel walls

Spandrel walls probably represent the biggest single problem with masonry arch bridges. They suffer from the normal problems associated with exposed masonry such as weathering and loss of pointing. They are also frequently affected by dead and live load lateral forces generated through the fill or as a result of vehicle impact on the parapet or by freezing of the fill. The effect may be (see figure 3) outward rotation, sliding on the arch ring, cracking of the arch ring, or bulging.

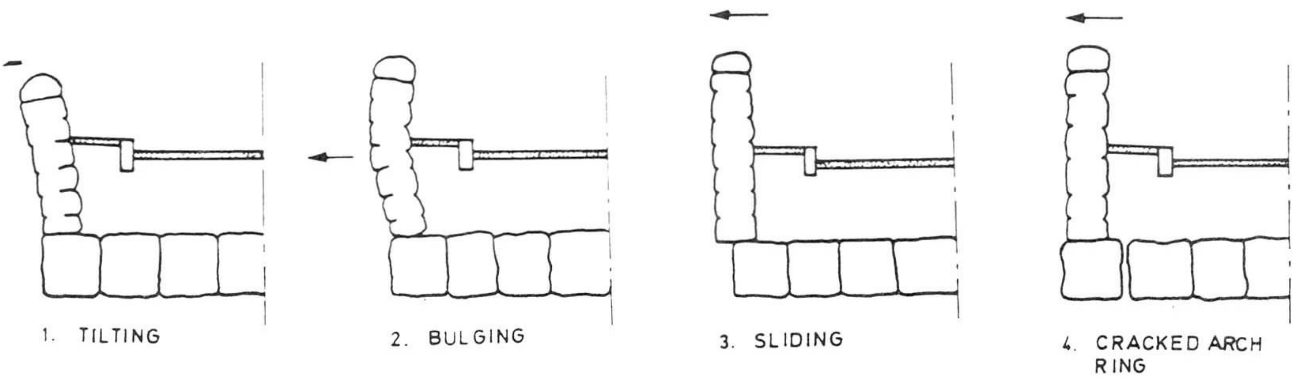

Fig 3 Spandrel wall failures

2.4 Fill

The major problem likely to affect fill is that the road surface or the drainage breaks down and the fill becomes saturated. This is unlikely to affect load capacity of the bridge immediately, indeed the increased weight may increase it. Longer term effects are that fines may be washed out of the fill leading to voids. Water percolating through the arch ring is likely to lead to deterioration of the mortar. Saturated fill will substantially increase the lateral pressures on spandrel walls particularly if the fill freezes in winter, perhaps leading to outward displacement of the wall.

3. FREQUENCY OF OCCURRENCE OF DEFECTS

A survey of 98 masonry arch bridges chosen at random was undertaken by the Transport Research Laboratory (TRL) in 1989. Forty one were in the north of Scotland, forty six in south west England and eleven in south east England. The main conclusions about the condition of the bridges were as follows:

- Only three bridges had no sign of water leakage through the arch ring. Severity of leakage was higher in Scotland.
- Sixty nine of the bridges had some spandrel wall defect, either leaning, bulging, outward movement, or a crack in the arch ring beneath the inside edge of the spandrel wall. Only seven of the bridges had tie bars.
- Forty of the bridges had some arch ring defect, either local bulges, cracks or missing mortar. Eighty four of the arches had been repaired at some time in the past.

Deterioration is therefore the rule rather than the exception.

4. MAINTENANCE

Routine maintenance consists of keeping the road surface in a sound condition to reduce ingress of water into the fill and to minimise dynamic loading from traffic due to potholes etc; removing vegetation growing on the structure; and making good small areas of deteriorated mortar. Maintenance involves modest expense compared with that which may result from neglect.

5. REPAIR AND MAINTENANCE TECHNIQUES

Damage will be caused by chemical, physical or biological sources. Examples are acidic rainwater reacting with lime mortar, water in fill freezing and expanding, and tree roots. These effects are discussed in some detail in reference [2].

It is essential that the cause of deterioration is understood before the most effective repair or strengthening method can be decided upon. For instance there is no point in repairing a deteriorated arch by saddling if the cause of the deterioration is movement of the abutments. The effect of any repair on the behaviour of the existing structure must also be considered. If the inherent articulation of the stonework or brickwork is lost as a result of the repair, it may have a long term detrimental effect on the fabric of the structure, the very thing the repair was trying to save.

The assessment of the structure will include an assessment of its load carrying capacity. There is not space here to discuss the techniques available; the reader is referred to reference [3]. The history of the structure should be checked, although records may be sketchy. Trial pits or cores should be considered to provide more detail of the internal structure.

Repair materials should be compatible with existing materials. For instance it is unwise to use hard engineering bricks to repair a structure built of much softer bricks. New material inserted into a structure, eg brick patching will not at least initially carry dead load stresses, only live load stresses. Care should be taken that the repair technique used does not itself cause further damage to the structure. For example, care needs to be taken with the use of rotary percussive drills.

Table 1 identifies the common faults of arch bridges and the repair and strengthening methods which may be applied: some of the methods will be described in more detail later in the section.

Table 1 Arch bridge faults and repair/strengthening methods

FAULT	REPAIR/STRENGTHENING
Deteriorated pointing	Repoint
Deterioration of arch ring material	Saddle Reinforced sprayed concrete to soffit Prefabricated liner to soffit Grout arch ring
Arch ring thickness assessed to be inadequate to carry required traffic loads	Saddle Reinforced sprayed concrete to soffit Prefabricated liner to soffit
Internal deterioration of mortar - eg ring separation	Grout arch ring
Foundation movement	Mini-pile Grout piers & abutments Underpin
Scour of foundations	Underpin Invert slab
Outward movement of spandrel walls	Tie bars Replace fill with concrete Take down & rebuild Grout fill if it is suitable
Separation of arch ring beneath spandrel wall from rest of ring	Stitch (short tie bars spanning the crack)
Weak fill	Replace fill with concrete Grout fill if it is suitable
Water leakage through arch ring	Seal road surface Waterproof arch ring extrados + improve drainage

An examination of these methods was carried out in 1990 for TRL [4]. Fifty bridges were examined to identify the advantages and disadvantages of the various repair and strengthening methods which had been applied to them, and their relative costs. An assessment of the effectiveness of the methods was made by inspection; however the repairs had all been done quite recently so it was not possible to assess their long term effectiveness. Frequently more than one method is applied to a particular bridge. Initial costs only were identified, data were not available to attempt to identify whole life costs.

No research is known which examines the structural effectiveness of the various methods; at the time of writing, TRL has just begun such a research programme.

5.1 Repointing

Routine maintenance repointing is widely regarded as essential and may improve arch load capacity by restoring the structurally effective arch ring thickness to its full depth. If properly done when it is needed, it may prevent the bridge from deteriorating to the point where it needs more expensive repair work. If incorrectly done it can accelerate deterioration of the structure. The mortar should not for instance be harder than the brick or stone. Repointing can enhance the appearance of the bridge and need not disrupt traffic while being done.

5.2 Saddling

Saddling involves removal of the fill and casting a concrete arch, often reinforced, on top of the existing arch. The new arch may be designed to act compositely with the existing arch or structurally to replace the existing arch, in effect using it as permanent formwork. The work is invisible once completed but it requires a major construction operation to install.

Before choosing saddling as a strengthening method, it is important to ascertain the reasons for the arch deterioration. A common reason is signs of distress in the barrel; these may be caused by movements of the abutments. The addition of a saddle will lift the line of thrust which may increase abutment movement and make the problem worse.

The defects observed in the surveyed bridges strengthened using this method were signs of weathering, discolouration and leachate encrustation on the arch soffit associated with water seepage.

5.3 Arch grouting

Arch grouting is used to fill voids in the arch ring to ensure that the full depth of section is available for load carrying. It is often used to fill voids caused by ring separation in multi-ring brick arches. It should not affect the appearance of the bridge unless grout extrudes from cracks and is not removed (it may be necessary to repoint the arch ring first). The grout needs to be carefully designed to avoid premature setting before it has completely filled the voids and to ensure that its properties are compatible with the existing arch material. High pressure grouting may damage weak structures. It will always take a line of least resistance which may be into fill, service ducts and drain pipes.

Cementitious or resin grouts may be used. Cost considerations will normally dictate cementitious grout.

5.4 Sprayed concrete

Sprayed concrete is widely used as a means of increasing arch ring thickness to increase load capacity, and of stabilising badly weathered masonry. Pre-mixed concrete is sprayed at high velocity and it adheres on impact, filling crevices and compacting material already sprayed. A layer up to 300mm thick may be applied; it is usually reinforced with at least nominal steel. It is quick to apply and does not involve disruption to traffic or services. It reduces the size of the arch opening and it does not enhance the appearance of the bridge although careful design can reduce its visual impact.

All the cases investigated showed signs of cracking, made visible by seepage of water and the associated leaching of mineral salts. The lining may separate from the original arch by shrinkage of the concrete or by further deterioration of the arch material at the interface, which would mean that it would not increase the load capacity as much as if it were fully attached. It was not possible to check this in the cases surveyed. Rusting of the reinforcement must be a serious concern and every effort should be made to exclude water from the structure.

Most processes rely on the nozzle operative to control the water content of the sprayed concrete which has led to variable quality. British Rail has carried out trials of a Hungarian system in which the water content is controlled at the mixing stage, and this has produced a more reliable product with reduced rebound.

5.5 Prefabricated liners

Arch ring thickness is increased by attaching a metal or glass reinforced cement lining (usually corrugated) to the soffit as permanent formwork, and filling the space between it and the arch ring with concrete or grout. As with sprayed concrete, it is quick to apply and involves no disruption to traffic or services, but it reduces the size of the arch opening and does not enhance the appearance of the bridge. Care needs to be taken to ensure that the space between the arch and the formwork is fully filled with concrete or grout.

In the cases studied, rusting corrugated steel and fixing bolts were found, and grout loss at sheet joints due to poor fit.

5.6 Underpinning

Underpinning involves excavating material from beneath the foundations and replacing with mass concrete. A sequence of work is followed to ensure that the stability of the existing structure is not compromised. The work is labour intensive. The cases studied appeared to have been successful.

5.7 Invert slabs

An invert slab (see figure 1) is a slab of concrete or masonry placed between the abutment walls or piers with its top surface at or below river bed level. It helps to prevent scour. If incorrectly installed however, there is a risk of scour beneath the slab, particularly at its downstream end, and this was found in one of the cases studied.

5.8 Tie bars

Tie bars are used to restrain further outward movement of spandrel walls. They consist of a bar passing through the full width of the bridge, with pattress plates at each end, generally secured by a nut and washer, to provide the restraint to the wall. If the arch ring requires strengthening at the same time a more common solution is to use a concrete saddle which will also relieve the spandrel wall of outward forces.

In one of the cases studied there appeared to have been further movement of a spandrel wall since installation of the tie bars. Rusting of the exposed parts, in one case severe, was also found. Damage due to expansion of the rust may occur. Wenzel and Maus [5] suggest at least 20mm of grout surround the bar. The use of stainless steel bars could also be considered, or the application of cathodic protection.

5.9 Replacing some or all of the spandrel fill with concrete

This method is used to stabilise outward movement of spandrel walls. When the whole of the fill is replaced, the method is akin to saddling and is likely to be used to deal with arch and wall problems at the same time. The work is invisible once completed. Traffic and services are likely to be disrupted during installation. Few defects were seen in the cases studied, except for the appearance of leachate.

6. RELATIVE COST OF REPAIR METHODS

From the cost data collected during the survey it was possible to estimate the cost of various methods of repairing or strengthening an arch ring. As an example a bridge with a semicircular arch of span 4.5m, headroom 3.5m (rise of arch plus height of abutment), total length at road surface 15m, and width 6m was examined. The costs (at 1990 prices) are given in table 2. The cost/m^2 of grouting was significantly different for the two examples examined in the survey, so two costings are included in the table.

Table 2 Cost of various repair methods for example bridge

Repair method	Cost (£)
Repoint arch ring	4000
Sprayed concrete (t=150mm)	10800
Grout (£269/m^2)	15300
Concrete saddle	23300
Grout (£433/m^2)	24700
Corrugated steel lining	57000

It should be noted particularly that the cost of repointing is modest compared with the other techniques which serves to re-emphasise the point made earlier that routine and timely maintenance is vital and cost-effective.

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8. ACKNOWLEDGEMENTS

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Historic American Bridges

Ponts anciens aux Etats-Unis

Historische amerikanische Brücken

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SUMMARY

Of the approximately 575'000 bridges in the United States, about 1'500 have been classed as historic. Increasingly, the public recognizes the historic and technological value of interesting old bridges, and they want them preserved and rehabilitated. The state of bridge preservation in America will be presented as will techniques that have been successfully applied to keep historic bridges in service. Bridges dating from the 1760s through to the 1950s will be discussed.

RÉSUMÉ

Il y a approximativement 575'000 ponts aux Etats-Unis, dont environ 1'500 sont considérés comme historiques. De plus en plus le public reconnaît la valeur et l'intérêt historique et technologique d'anciens ponts et désire les voir remis en état et conservés. La situation de la conservation des ponts en Amérique est présentée, de même que les techniques qui ont été appliquées avec succès pour maintenir en service des ponts historiques. L'article traite de ponts qui datent de 1760 à 1950.

ZUSAMMENFASSUNG

Von den ungefähr 575'000 Brücken in den Vereinigten Staaten sind 1'500 als historisch wertvoll erklärt worden. In zunehmendem Masse erkennt die Bevölkerung den historischen und technologischen Wert interessanter alter Brücken und verlangt, dass diese geschützt und wiederhergestellt werden. Der Beitrag schildert den Stand der Brückenerhaltung in Amerika und einige erfolgreich eingesetzte Techniken, um sie im Dienst zu halten. Es werden Brücken aus den Jahren 1760 bis 1950 behandelt.

INTRODUCTION

In the development of transportation modes, the United State of America is a relatively young country when compared to Europe, Africa, and Asia. Stone bridges survive on roadways built by the Romans many years before Christ, and Egypt and Israel still have remnants of aqueducts more than 2,000 years old. The oldest bridges in the United States, like the country itself, date only to the late-18th century. Yet America's infrastructure is receiving a great deal of attention and study by the public who have become increasingly interested in having its significance acknowledged and preserved.

The United States has approximately 575,000 highway bridges and 100,000 railroad bridges of which easily a third were built before 1940. Many of the road highway bridges date to the period of great internal improvement of the 1920s and 1930s when states and counties were building the network of roads that still carry most of the vehicular traffic in the country today. Of this inventory of old bridges still in service in America, increasingly many are acknowledged as being historic, and efforts are being exerted by all three levels of government (local, state, and federal) to preserve them.

DEFINITION OF HISTORIC BRIDGE

What is considered a historic bridge in America is a question with many answers. It could be based on age or technological significance, like Roebling's important suspension bridges at Cincinnati and Brooklyn. It can also be viewed as historic even though it is an example of a common type that has done little more than survive in relatively unaltered condition.

Procedures have been established to designate a bridge as historic. The National Register of Historic Places is the official list of those sites, objects, buildings, structures, like bridges, and districts that are considered worthy of preservation. It was established by the National Historic Preservation Act of 1966 and is a joint federal and state program that has evolved as the single most significant means of enhancing the country's attitude and actions toward appreciation and preservation of interesting old bridges.

Listing of historic structures in the National Register is handled on the state level by the State Historic Preservation Officer (SHPO). Each state has one, and their responsibility is to assist local, state, and federal groups and agencies with ensuring that National Register eligible properties are not destroyed. The SHPO and their supporting staff keep an eye on historic bridges and other resources and do what they can to encourage their preservation.

Guidelines for listing in the National Register were established, and they purposely were kept broad to be applicable to all types of resources ranging from Wright airplanes to early 20th century parks and residential neighborhoods. The criteria specify that a structure should be at least 50 years old and have integrity of location, design, setting, materials, workmanship, feeling, and association. If those qualifications are met, then a structure is investigated to see if it has some distinguishing features that makes it notable and worthy of preservation. Some of those features could be special technology, patented details, association with a prominent designer, being the oldest example of a type, or located in a historic setting.

Although being evaluated eligible for the National Register does not automatically mean that a structure will be preserved or that it cannot be destroyed, it does highlight its significance and certainly makes it more difficult to demolish without a very compelling reason. National Register status is particularly effective if federal funds or permits are involved with a project. Legislation from the Department of the Interior, Environmental Protection Agency, and Federal Highway Administration, and others, afford various interests the opportunity to get involved with planning bridge projects so that ingenious contributor who will help produce the solution that can be supported by good engineering judgement and preservation practice.

As illustrated by the examples listed below, there are many historic bridges that have been rehabilitated and returned to service. Solutions vary from one type of bridge to another, but all illustrate that sensitive rehabilitations are an economical alternative to new construction.

The once common wood truss covered bridge is a particularly popular American icon. Vulnerable to fire, rot, and high water, the timber bridges that remain in America are prized for their scenic and historic qualities. Many, like the 1887 Partridge "scissor" truss bridge in Ohio (Figure 1), have been stabilized and preserved in situ as part of a park. Now dedicated to pedestrian use, the truss was strengthened by the addition of steel stringers below the timber floor beams, and the truss members were conserved. These are commonly used procedures for rehabilitation wood truss bridges.

Another means of preserving covered bridges is to relocate them. An 1870 Col. Long-type timber truss span threatened with demolition as the result the building of a reservoir was moved to a golf course. It was rehabilitated and now carries golf carts and maintenance equipment.

Pony, or low, metal truss bridges often require specialized solutions to strengthen them to be able to carry light traffic without adversely effecting its historic fabric. In Bergen County, New Jersey, a patented Phoenix-section wrought iron pony truss span of insufficient load capacity was reinforced by the unobtrusive addition of beams adjacent to and below the truss lines. The new members work in concert with the trusses to carry live loads. A riveted Warren pony truss bridge fabricated in Scotland in 1890 and shipped to Hawaii illustrates that good maintenance can keep a historic bridge in service. The bridge was moved to its present location in 1919 where it has carried local traffic ever since the move.

Not many 18th- and early 19th-century stone arch bridges have survived because of floods and deterioration. Their useful life can be prolonged indefinitely with good maintenance. The two-span Choate Bridge in Ipswich, Massachusetts built in 1765 and widened in kind in 1820 has been in continuous service carrying heavy traffic on the town's main street. The recent sensitive repointing and resetting of loose or missing stones will ensure its continued service for years to come. A similar but later stone arch bridge in New York state was rehabilitated in a manner that preserves its original appearance, but live loads are supported on prestressed concrete beams hidden within the spandrel walls (Figure 2).

Reinforced concrete deck and through arch bridges are particularly susceptible to damage from salt and moisture penetration. In many instances replacing the deteriorated material in kind can restore the bridge to sufficient capacity. An important ca. 1930 patented Marsh through (rainbow) arch at Wichita, Kansas, is being rehabilitated primarily by reconnecting the encased hangers to the arch and floor beams (Figure 3).

Wrought iron and steel through truss bridges built from the late 1870s through the 1930s were abundant in America, but they are becoming increasingly scarce. Consequently, through truss bridges should be preserved whenever possible. The 1891 pin-connected multi-span Parker through truss at Chattanooga, Tennessee is being rehabilitated to provide a crossing for light trains and pedestrians. Closed to traffic for many years, the bridge was thoroughly inspected and evaluated. It could be returned to service by installing cables to provide a redundant stress path. The cables are concealed within the historic trusses.

A 1912 Pratt through truss bridge over the Hanalai River on Kauai Island in Hawaii was strengthened with the addition of Warren pony trusses set outside the original truss lines in the 1960s. The state wanted to replace the altered but historic span with a highrise multi span bridge. In the face of considerable opposition and years of negotiations, the historic trusses were preserved by transferring live-load stresses to the existing pony trusses that were strengthened.

Figure 1. Rehabilitated 1887 wood truss bridge at Canal Winchester, Ohio.

all perspectives are considered. If a bridge cannot be saved, it should be photographed and its plans and history documented and made part of the national archives in Washington, D.C.

Since the inception of National Register in 1966, the number of acknowledged historic bridges has increased dramatically and an estimated 1,500 bridges have been listed or determined eligible for the Register. Most states have done a survey to identify and evaluate their historic bridges, and the survey findings are being used when proposing new projects. Conflicts and delays are being avoided because the historicity of a bridge has been addressed at the beginning of the planning process.

PROBLEMS AND SOLUTIONS

The problems associated with preservation of historic bridges are numerous. Historic bridges are often functionally obsolete or structurally inadequate because they were designed for lighter and smaller loads. The roadways may not be wide enough to accommodate modern traffic requirements, and the portals of many of the through trusses are not high enough for passage by trucks. Additionally old bridges have frequently suffered from lack of maintenance. Modern transportation and traffic needs versus the desire to retain and rehabilitate historic bridges is a continual conflict that keeps bridge owners, historians, and engineers occupied finding solutions on a site-specific basis.

The easy way out is to remove the historic bridge and build a new structure. But the easy way is often not the best route. Many construction methods and techniques are available to retrofit and rehabilitate the historic bridge and keep it in some transportation mode without damaging the historic fabric or intent of the span. As a result of the available technology and know-how, the attitude of the bridge owner should be as follows

> First, let me try to study ways to upgrade the existing bridge to make it safe for the traveling public. if I cannot achieve this within reasonable economic limits, then, and only then, will I consider replacement.

Too often the planning process is reversed and the preservation community has to "fight" the bridge owner, who has already decided beforehand that the old bridge must go. The role of the engineer in this dilemma is to become an impartial and

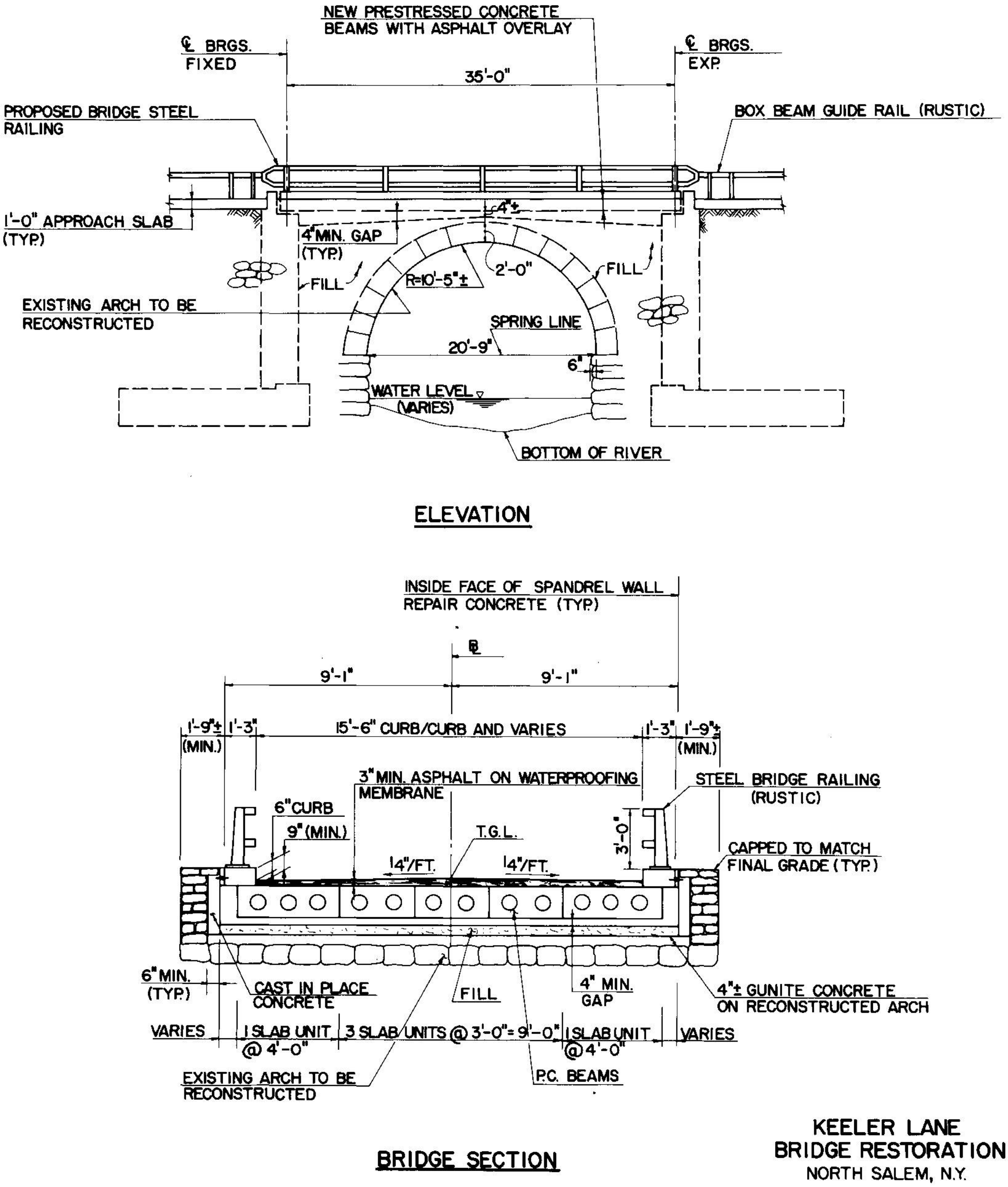

Figure 2. Rehabilitation plans for ca. 1840 Keeler Lane stone arch bridge at North Salem, NY.

Figure 3. Circa 1930 patented Marsh encased steel through arch span at Wichita, Kansas.

Some historic suspension bridges were reconstructed to improve their load carrying capacity without the need to replace the main suspension cables. A most interesting 600'-long canal aqueduct in Lackawaxen, Pennsylvania was converted to a highway bridge after the canal was abandoned early in this century. The historic 1849 span was designed by John Augustus Roebling, and it is the oldest extant Roebling suspension bridge in service. While the superstructure underwent many modifications, the cables, composed of special wrought iron wire dipped in hot linseed oil, have survived in excellent condition and will continue to do so as long as maintained properly.

Not every historic bridge can be saved. An eye bar suspension bridge at Lordville could not be saved (Figure 4). The main concern with the span was the settlement of one tower leg that caused one chain to be lower than the other thus distorting the eye bar and hanger connections beyond economical repair. Because the bridge had been determined historic, it was photographed and documented in accordance with federal requirements before it was demolished.

The ca. 1860 patented King bowstring pony truss bridge in New Jersey was the only example of the historic truss type in the state, but it was too narrow and too deteriorated to warrant rehabilitation (Figure 5). Instead of being demolished, the rare old bridge was dismantled and stored until a suitable, protected site can be identified. While its original setting will be lost, the technologically important artifact will be recaptured.

An important bridge that was lost was the ca. 1950 steel through arch bridge at Bellows Falls, Vermont. Designed by J.R. Worcester, it was closed to traffic because it was determined to be structurally unsafe. Two days and five dynamite blasts later, it finally succumbed. With some encouragement from the bridge owner, the arch could have been retained and upgraded as part of a larger project.

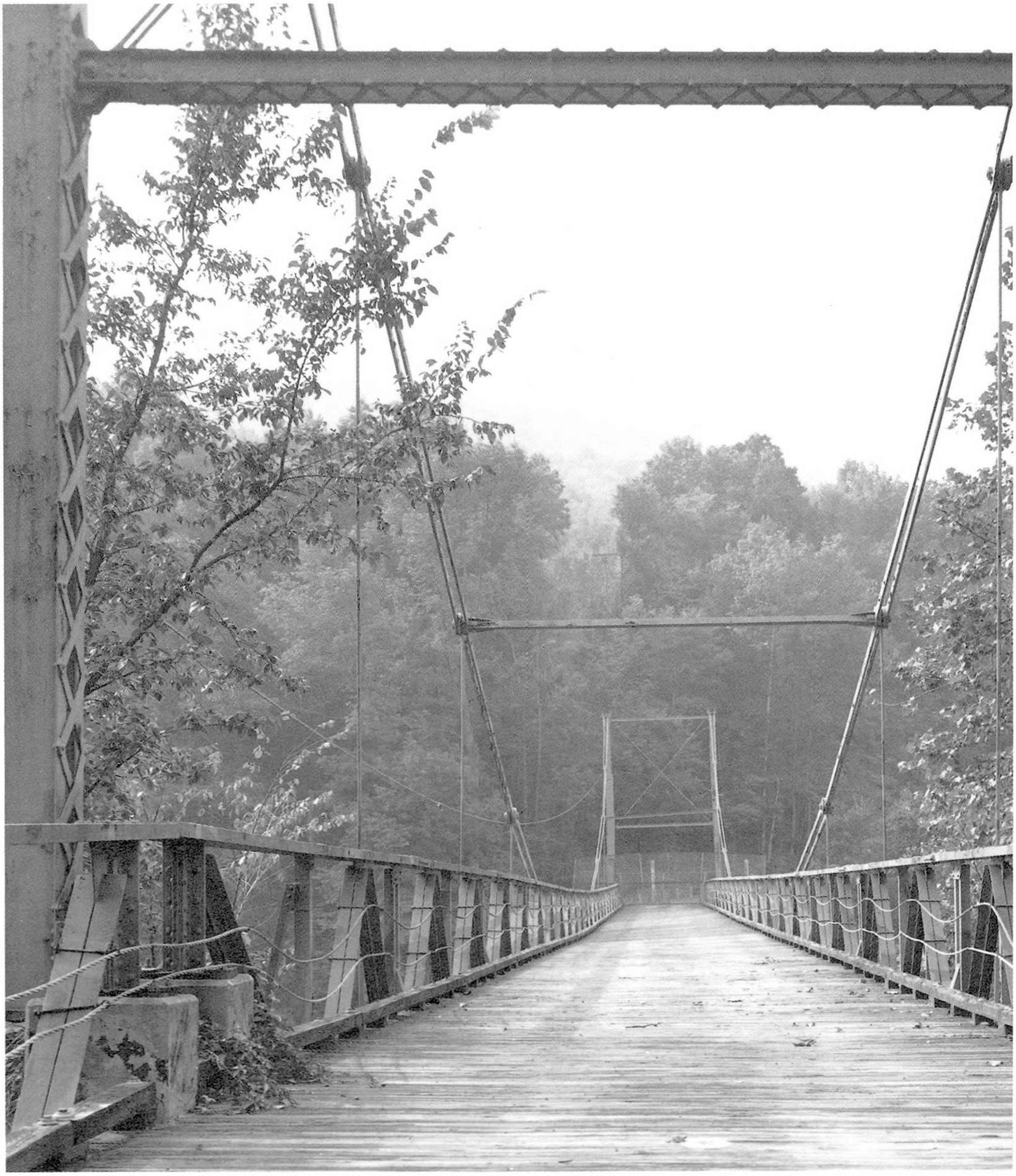

Figure 4. Ca. 1902 eyebar suspension bridge at Lordville, NY that failed and was recorded and demolished in 1986-1987.

Figure 5. Ca. 1860 King bowstring truss bridge at Allenwood, NJ.

CONCLUSION

The preservation of historic bridges in America will continue to be an uncertain proposition which will only be solved on a project by project basis. The publicity surrounding the disposition of historic bridges is improving, and both the public and bridge owners are becoming increasingly aware that historic bridges are worth preserving. The technology and engineering expertise are available to upgrade and reuse historic bridges, and bridge engineers are being encouraged to avail themselves of this know-how. It is hoped that no deserving historic bridge will be demolished without a public hearing and discussion. It may be tedious and time consuming, but it guarantees that historic bridges will not disappear, at least not quietly. This would be intolerable.

Preservation of the Venetian Bridges

Conservation des ponts de Venise

Erhaltung der Brücken Venedigs

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SUMMARY

The paper deals with a research investigation started with the aim of assessing the present state of the Venetian bridges, providing at the same time historical and structural information. The research is divided into three main steps: historical investigation through bibliographical and recorded sources; listing of bridges by type and location; graphical and photographic filing of public bridges, classified by their district.

RÉSUMÉ

Cette recherche a pour but d'étudier la situation actuelle des ponts de Venise, en fournissant des renseignements concernant l'histoire et la structure des ouvrages d'art. La recherche est partagée en trois parties: enquête historique avec renseignements bibliographiques et d'archive; liste des ponts selon modèle de construction et lieu; archive graphique et photographique des ponts publics, classés selon leur arrondissement.

ZUSAMMENFASSUNG

Es handelt sich um ein Forschungsprojekt mit dem Ziel, den aktuellen Zustand der Brücken Venedigs zu untersuchen, damit aber auch gleichzeitig geschichtliche und konstruktive Informationen über diese Brücken zu erhalten. Die Forschungsarbeit wird in drei wichtige Gruppen unterteilt: Historische Untersuchung mittels bibliographischer und archivarischer Quellen; Katalogisierung der Brücken nach Ort und Bauart; Graphische und photographische Bestandsaufnahme der öffentlichen Brücken, unterteilt nach ihrer Lage in den verschiedenen Stadtteilen.

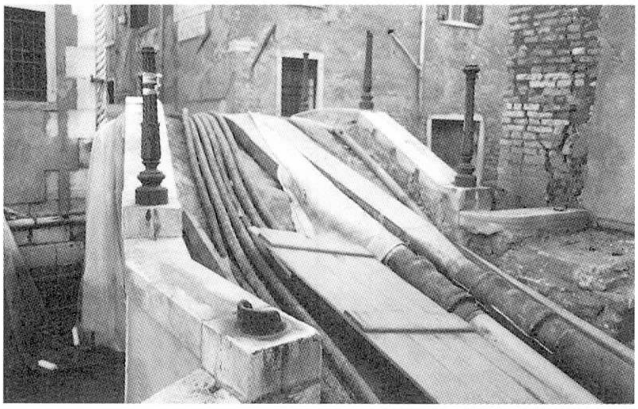
1. INTRODUCTION

In Venice continuity of pathways is ensured by 343 bridges. Big bridges and small bridges, mainly the work of unknown masters. In recent years, the urgency to restore many of these structures has led to the discussion on how to ensure their future conservation on the whole.

Venetian bridges have many peculiarities, due to the following reasons: the characteristics of the venetian environment, which often led to foundations on unstable ground; the instability of the soil, liable in the centuries to a slow but continuous lowering; structures are often built in already saturated urban site; salient humidity combined with chemical action of salt; mechanical action of water in the canals, worsened by the constant increase in wave motion due to the modern means of transportation.

Early venetian bridges were built in wood, erected on flat piles, low arched and with no steps so that they could be ridden by horses. In the 16th century the town began to thoroughly modify its appearance and the obsolete wooden bridges gave way to the more lasting arched brick bridges. Notwithstanding iron structures, greatly fashionable in the last century, brick bridges represent at present the majority (263). Most of them are in an evident state of physical degrade and lacking adequate and up-to-date census of their real condition. Recent visual inspections showed up a distressing situation, in which the bridges – that still bear on their structural body many pipe-lines (gas, electricity, telephone ...) – present diffused cracks, tiring out of haunches, worn out ties, micro-organisms and saline efflorescences which cause detachment of plaster (Figs 1-3).

At the Istituto Universitario di Architettura di Venezia a research has been undertaken, with the aim to illustrate the present state of the town bridges. The investigation was carried out in three phases: a) historical investigation; b) listing of bridges; c) annotated graphic and photographic filing. The work is still in progress, in collaboration with the venetian municipality, within a general restoration programme.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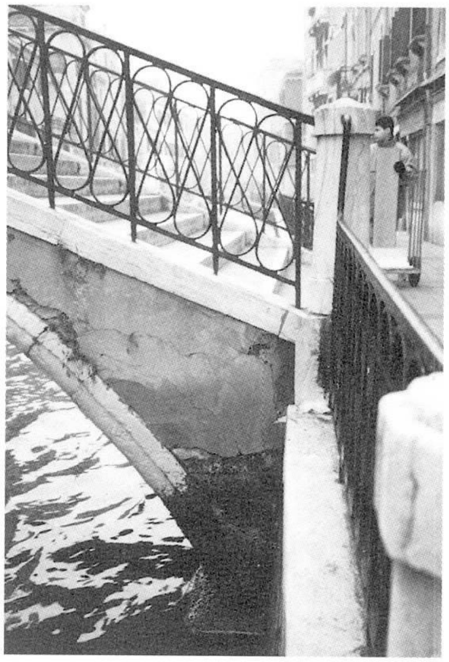
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Fig. 1 Services on a bridge.
Fig. 2 Detachment of plaster.
Fig. 3 Saline efflorescence.

2. VENETIAN ENVIRONMENT AND BUILDING PROCEDURE

The area on which Venice is built was a lagoon region with a lot of silt, sand banks, marshes, meandering canals, with neither potable water, freestone, lumber, nor wood for domestic use. The low bearing capacity of the ground soil, and the difficulty in finding building materials, added to high environmental wilderness, affected perhaps more than anywhere else the way in which the town was built and renewed, from the single element to the whole urban configuration. This has led to an architecture which exhibits a strict correlation between statical behaviour and functional and distributive organization, together with high interdependence between building materials and formal style.

The soil is made of non homogeneous strata of silt, clay and sand. In particular, under a more or less recent sediment of soft sitly clay, 1 to 6 m thick, one can find either a 2 to 4 m layer of compact clay named "caranto" or alternate layers of medium silty clay with dense sand.

The common building procedure provides a stone foundation 2 to 3 m below sea level based on a simple or double layer of timber joists, the so called "madieri" placed directly on "caranto", whenever possible, or conversely on elder, elm, oak or larch piles 2 or 3 m long.The soil strata loaded this way, slowly consolidate, due to partial expulsion of interstitial water present among soil particles, this implies the progressive improvement of their loading capacities but also a great reduction in volume and therefore a subsidence directly proportional to put loads. To insure regular subsidence of the structure, various kinds of pile, slab and raft foundations were used for the same built unit.

In rebuilding, which was most frequent for bridges, whenever possible old under-sea foundations of demolished buildings were used, regardless of distribution and regularity of shape, having the advantage of being founded on already consolidated ground soil.

The statical behaviour of venetian buildings is based on disconnections and continuities, which allow the highest degree of relative movement among the various components, while assuring stability on the whole structure.

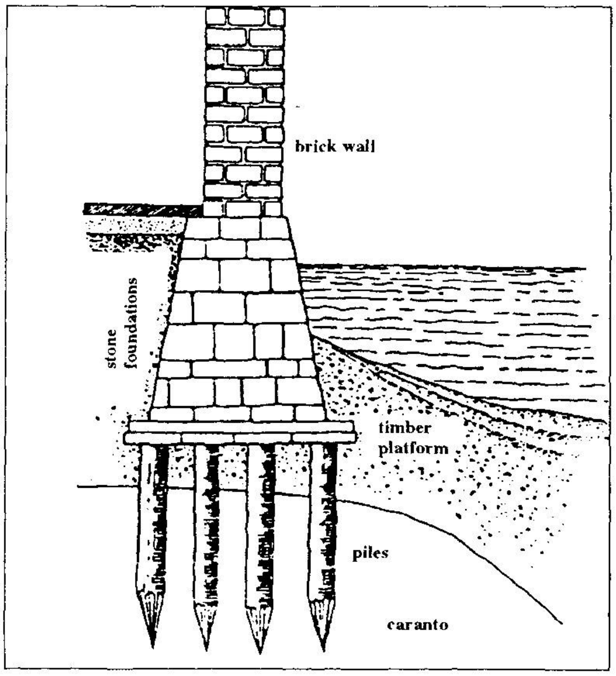

Fig. 4. Venetian building procedure.

Bridges play a very delicate role in this equilibrium, due to the need to match their own statical requirements with those of the buildings joined to them.

3. HISTORICAL INVESTIGATION

Historical investigation is carried out through the analysis of the urban site and of the street system, following a chronological order, based on the historical town maps by Jacopo De' Barbari, Ludovico Ughi, Bernardo and Gaetano Combatti and on maps and papers registered in the venetian archives.

Stone bridges, which date back to the 12th century, were very few, and wooden bridges – being built on piles which rotted easily owing to dampness – required frequent maintenance and were soon replaced by arched brick bridges. In times past, they were constantly maintained in good state by the owners and by the government of the Serenissima Repubblica, as stated by innumerable decrees, provisions and warnings. The pathologies once suffered by these structures are not much different from the present ones, the essential difference being that while, formerly, a suitable restoration followed shortly the damage, nowadays a fair knowledge of the bridge condition simply does not exist.

4. TYPOLOGICAL STUDY

4.1 The plans

The bridges issued from the "calli" (pathways) and the "calli" are the spontaneous offspring of buildings, regardless of future need of their connection, as both the structure and the shape of many bridges prove. The aim of linking the buildings without altering them has given birth to some utterly unexpected forms: there are twisted bridges that link up "calli" that are not one in front of the other; drawbridges like the one of the Arsenale, which had to leave ships pass; private bridges to enter houses and palaces. Moreover, we can have bridges linking two opposite "fondamente" (pathways running along a canal) or two sides of the same "fondamenta" interrupted by a canal, which can defined as *regular*, while others are more or less *irregular*, or even "storti" (twisted) as they are named both in the official and popular toponymy. The latter are side bent over the canal or skew to the canal axis because they have to link up two streets leading to the water but not facing each other. Their shape is defined by pre-existing structures, with the scope to meet the needs of contiguous houses.

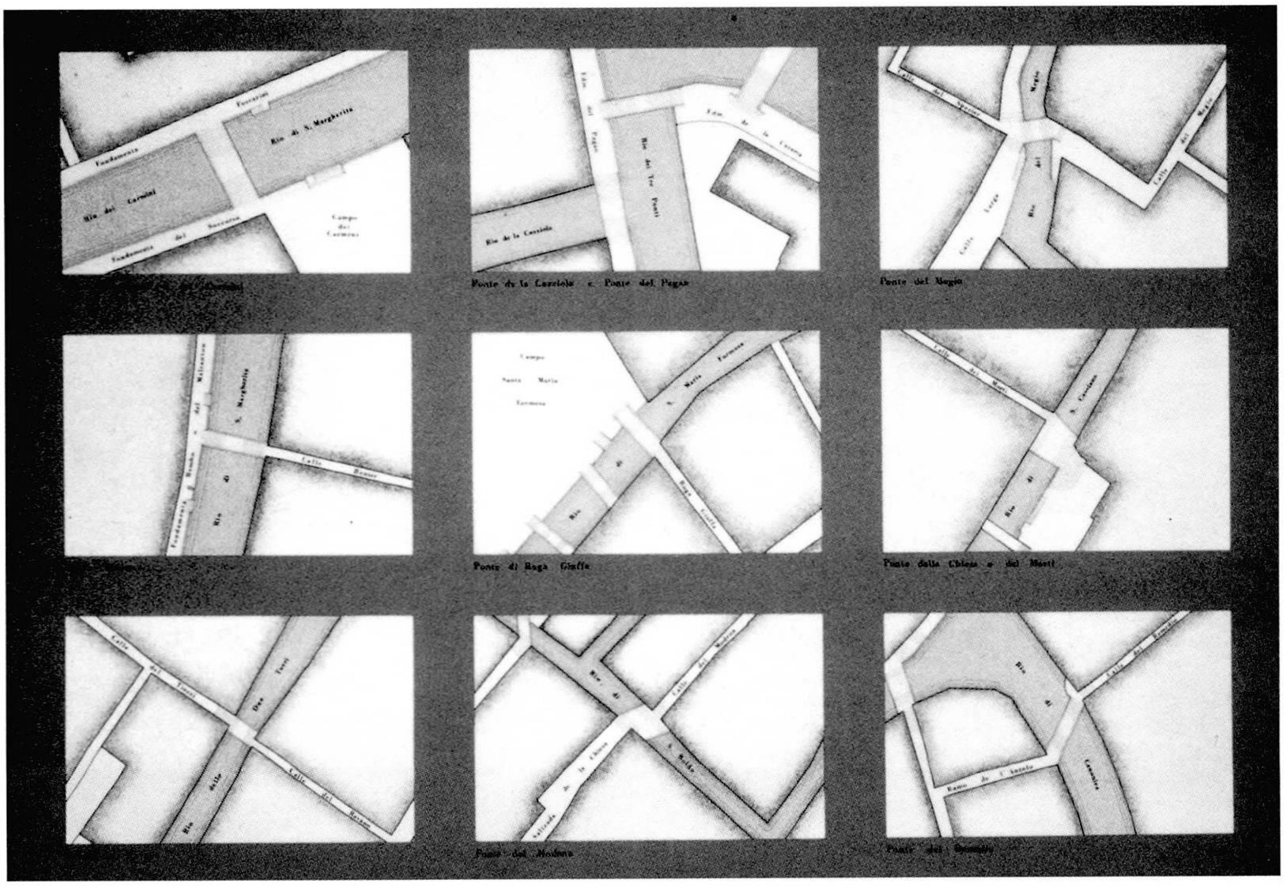

Fig. 5 Example of plan typologies of venetian bridges.

These bridges have ramps branching out in several directions, widening tops to allow the opening of entrances of houses and shops; little pensile paths are not rare: four, five or even twenty metres long, they were thrown to enter houses which otherwise could be reached only by water.

Some bridges are double, *i.e.* they cross with separate arches, which can be parallel or perpendicular to one another, the same canal or two different canals, sharing one or even both ramps. Although each bridge has its peculiar shape, some standardization has been studied based on the type of connection they allow: between "fondamenta" and "fondamenta"; between "calle" and "fondamenta"; between "calle" and "calle", as shown in fig. 5.

4.2 The arch

Our study classifies nine different arch typologies (see Fig. 6 and Tab. 1).

The arch of a bridge has to be high enough to let the boats, loaded with goods, pass; with the exception of the three-arched bridge, called ponte Tre Archi on Cannaregio canal, and the mid-arched one at S. Maria del Giglio, all venetian bridges have only one arch, to meet the needs of water traffic. Bridges crossing very narrow canals often have a round arch; in two cases they have an ogive or gothic arch, a shape not easily found in bridge construction.

The half-elliptic and the double girder bridge appear only once (ponte della Feltrina and ponte dell'Arsenale). The arch shape is related to the canal width: in the narrowest canals the use of round, parabolic or ogive arches is preferred, while in the widest canals the depressed arch is more usual (Tab. 1).

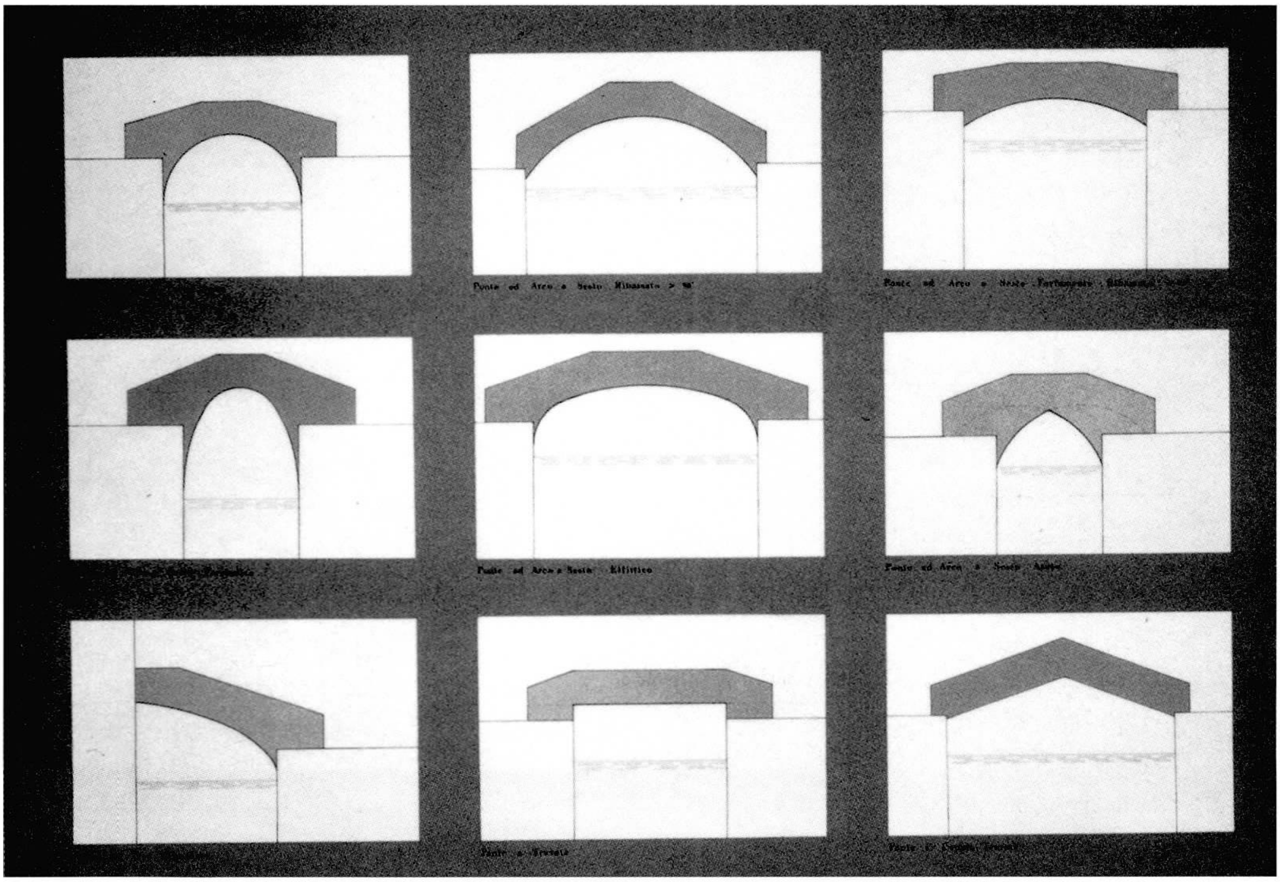

Fig. 6 Schematization of the nine arch typologies of venetian bridges.

Table 1 Venetian bridges arch typologies.

Arch typology	*Quantity*
1. depressed	156
2. very depressed	55
3. elliptic	36
4. parabolic	9
5. round	37
6. ogive	2
7. half arch	1
8. girder	46
9. double girder	1
Total	343

The elliptic arch appears mostly in 19th century rebuilding, when Giuseppe Salvadori was chief engineer at the Municipal Technical Office – *i.e.* the rebuilding in 1827 of the ponte Donà, with elliptic vault, at Fondamente Nuove –.

In the *very* depressed arch category there are bridges with peripheral arch angle less than 90°; bridges with an angle over 90° are instead simply depressed.

The bridges with girder arch are made of wood and of iron, not of masonry, with the exception of the ponte Santa Sofia at Cannaregio.

5. THE FILING SYSTEM

Our bridge filing system gives quick information on quantity, type of structure and location. To this purpose we have made use of several instruments, namely: six maps (one for each "sestiere", as venetian districts are called) (1:2000 scale), synthetical and analytical tables.

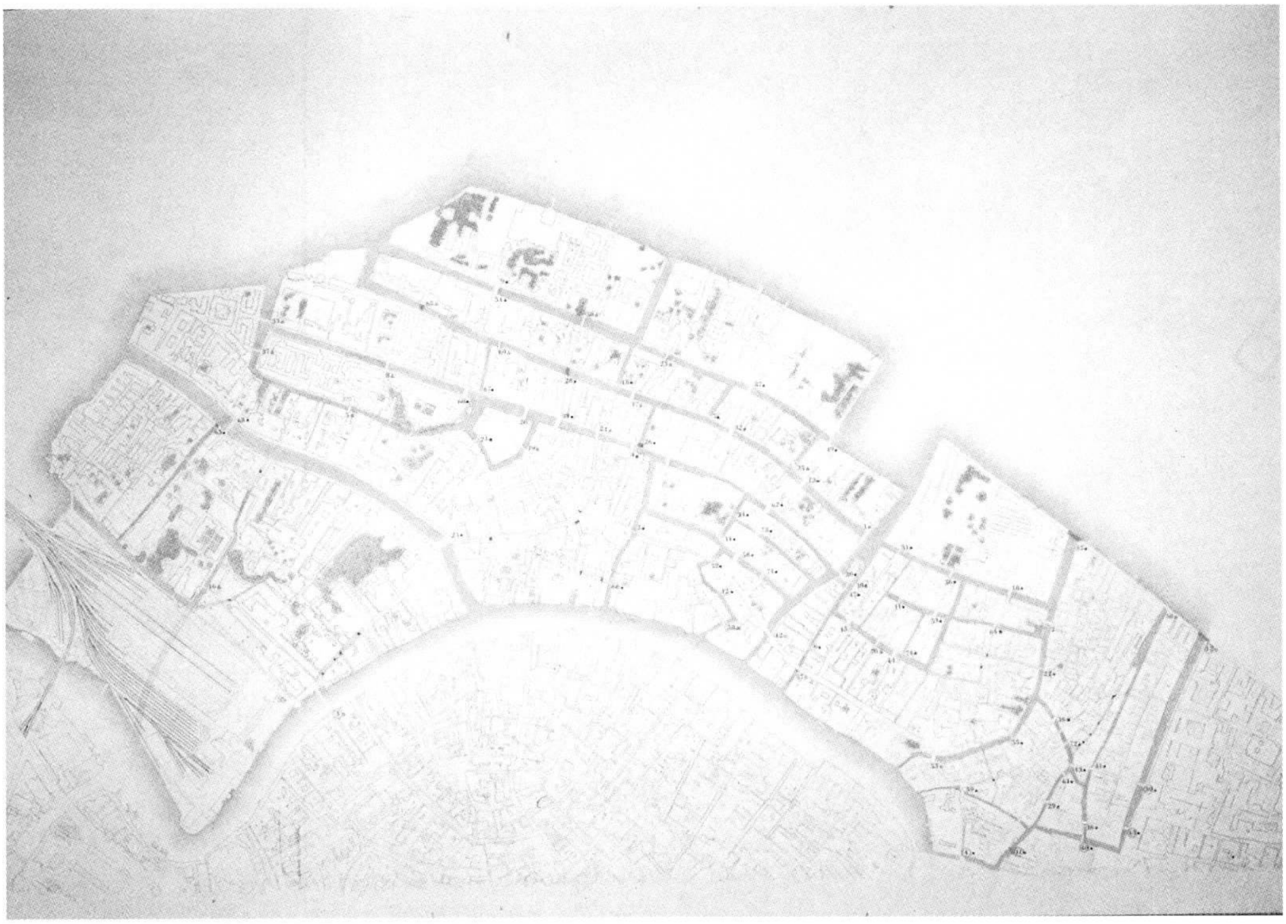

Fig. 7 Example of district map.

5.1 District maps (1:2000 scale)

The analysis develops through six district maps which clearly locate all the bridges pertaining to the "sestiere". A number is assigned to each bridge, circled if the bridge connects two different "sestieri". With this number every bridge will be referred to, both in synthetical and in analytical tables. The symbol and the colour next to the number indicate the building material and the type of arch, respectively (Fig. 7).

5.2 Synthesis tables

Synthesis tables report various information: number, name, canal, place, arch type, skew backs, on each bridge of the district. Beside the progressive numbering and the bridge name we indicate the canal crossed (except for the "cavana" bridges, which do not cross a canal but link two "fondamenta") and the most important nearby sites; technical data on bridge structure, *i.e.* its building material, type of arch and skew backs, are also reported.

5.3 Analytical tables

These tables (Fig. 8) allow the visualisation of each structure thanks to the graphical support given by a planimetric frame (Atlas of Venice), a plan and a front view (1:200 scale), and a picture. Each file includes some notations to allow exact identification of the bridge and gives information on place, canal, date of construction (if available), structure (arch typology and building material) and details such as coats of arms, inscriptions, sculptural decorations.

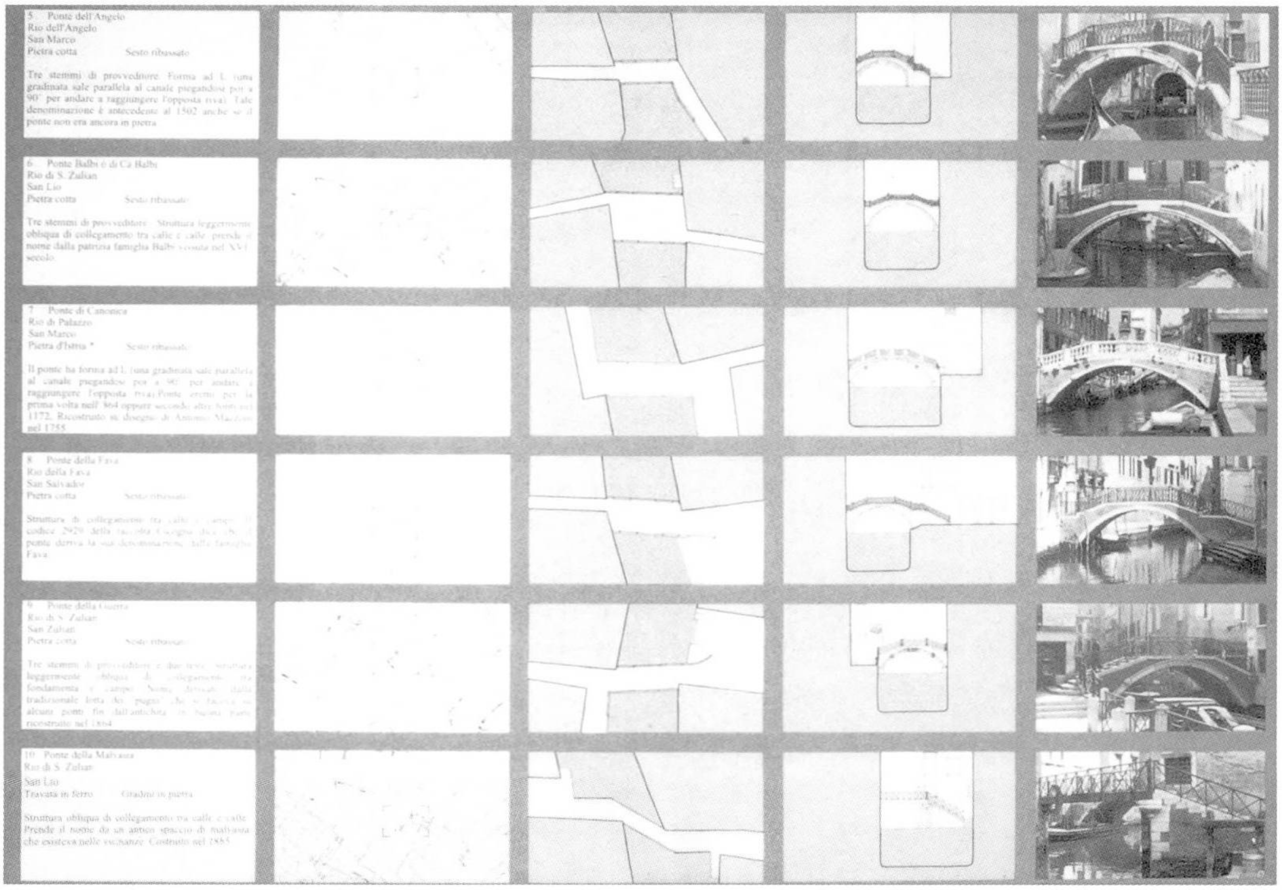

Fig. 8 Example of analytical table.

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Repairs to English Monuments: Some Case Studies

Réparation de monuments anglais: études de cas

Einige Reparaturbeispiele an englischen Baudenkmälern

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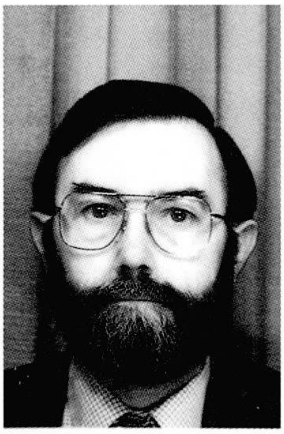

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SUMMARY

The English Heritage approach to the conservation and repair of historic buildings and structures in England is a minimalist approach. The main aim is to conserve the building as found and to carry out the minimum repairs necessary to ensure its safety and long life and to ensure that it will cause no danger either to its occupants or to passers-by. The aim of this paper will be to illustrate this philosophy by means of a series of brief case studies of structural repairs carried out under the direction of the author.

RÉSUMÉ

L'approche anglaise à la conservation et la réparation de bâtiments et structures historiques est une approche minimaliste. L'objectif principal est de conserver le bâtiment dans son état actuel et de réaliser les réparations minimales, afin d'en garantir la sécurité et une longue durée de vie et d'assurer qu'il ne représentera aucun danger pour ses occupants ou ses utilisateurs. Le but de cet article est d'illustrer cette philosophie au moyen de quelques études de cas de réparations structurales réalisées sous la direction de l'auteur.

ZUSAMMENFASSUNG

Die English Heritage-Stiftung geht mit der Einstellung eines Minimalisten an die Erhaltung und Reparatur historischer Gebäude und Ingenieurbauten in England heran. Das Hauptziel besteht darin, den gegenwärtigen Zustand zu erhalten und nur die notwendigsten Unterhaltsarbeiten vorzunehmen, damit die Sicherheit und das Fortbestehen der Gebäude sichergestellt und weder Bewohner noch Passanten gefährdet sind. Der Artikel schildert diese Philosophie anhand einiger Fallbeispiele von Reparaturarbeiten, die unter der Leitung des Autors durchgeführt wurden.

1. INTRODUCTION.

1.1.1 The English Heritage philosophy for the conservation and repair of ancient monuments, historic buildings and other historic structures in England is one of carrying out as little work as possible to the fabric in order to preserve it. It is a minimalist approach.

1.1.2 The main aim is generally to conserve the building as found rather than to restore it to some previous state however scholarly that restoration might be. Only the minimum repairs necessary to ensure its safety and long life and to ensure that it will cause no danger either to its occupants or to passers-by are carried out.

1.1.3 The aim of this paper is to illustrate this philosophy by means of a series of brief case studies of structural repairs carried out under the direction of the author.

2. THE SCHEDULING AND LISTING OF HISTORIC BUILDINGS.

2.1.1 There are a number of grades of importance of historic fabric in the United Kingdom. Major ruined structures are often "scheduled" as ancient monuments; they are generally uninhabitable and medieval and earlier although later buildings can also be scheduled.

2.1.2 The most important structures are "listed" at grade I and others can be "listed" as grade II* and grade II. Important townscapes can become "conservation areas". All of these grades of structure are subjected to the same philosophy although the application of the rules can vary according to circumstance and importance.

2.1.3 A building or other structure is not scheduled or listed for one particular feature but as a whole, all parts being considered important. Clearly however some parts will in fact be less important than others. All demolition, complete or partial and all alterations need consent prior to the commencement of work starting on site.

2.1.4 The hidden part of a structure is often considered to be a very important part of the building and is dealt with accordingly. Alterations to the interior of a building or indeed its hidden fabric are scrutinized closely to ensure that wherever possible no important detail is lost during alterations.

3. CHANGES TO SCHEDULED AND LISTED BUILDINGS.

3.1.1 Most people understand that the exterior appearance is of historic and aesthetic value even if they do not subscribe to that view themselves. Discussions often arise about interior details and about details that are hidden such as the carcassing of the floor structure. This may contain unusual details but may equally be a very common form of construction. Either way the removal or alteration of such hidden structure is not allowed lightly by English Heritage but decisions will vary from case to case.

3.1.2 While a building with the original external shell but with a completely new internal structure may look the same externally to passers-by, the building is not considered, according to this philosophy, to be historically correct. English Heritage generally objects to facadism work. It also objects frequently to major alterations to the structure of floors and the like, particularly when this involves the removal of details such as joist to beam joints. It would much prefer to see material added to a structure to strengthen it if weak areas are a problem.

3.1.3 The following case studies show how this minimalist approach is applied.

4. CASTLE BOLTON, NORTH YORKSHIRE.

4.1.1 Castle Bolton is a massive and very well preserved medieval castle built for Richard de Scrope who was granted a licence to build a castle by King Richard II in 1379. Castle Bolton was not only built as a castle, but considerable efforts were put in to make domestically comfortable. In 1568-69 Castle Bolton served as a prison for Mary Queen of Scots and it was slighted by Parliament in the English Civil War in the 1640's. Since being partially deliberately destroyed, the castle has been largely unoccupied and remained very much a ruin. A small part of the castle currently functions as a restaurant, together with shop and some exhibition space. No serious attempt at restoration has ever been made.

4.1.2 Some relatively low-key works were carried out early this century. In recent years the owner of Castle Bolton has instituted a major programme of conservation works with financial and professional assistance from English Heritage. The work being carried out has been the minimum necessary to ensure the continued stability of the castle in its current state. During these works no attempt has been made to rebuild anything and the only additions made are in locations necessary to support dangerously overhanging masonry.

4.1.3 The major problems with Castle Bolton were due to the ingress of dampness into the masonry and the subsequent growth of weeds, plants and trees. In some cases some quite substantial trees were growing in the masonry in inaccessible positions.

4.1.4 Before any work could commence thorough photogrammetric surveys were made of the building, both internally and externally and a substantial amount of scaffolding was erected. Archaeologists carried out very careful surveys of the wall structure and made alterations and additions to the photogrammetric surveys where close inspections showed errors.

4.1.5 The work carried out on the structure of Bolton Castle falls into a number of categories:

- Pointing. The major amount of work done at Castle Bolton is very careful pointing of the masonry work. Old and decayed pointing is raked out and replaced with new lime mortar pointing.

- Weatherproofing of wall tops. Many of the walls at Castle Bolton are in excess of 1m wide and therefore provide very good places for plant growth. The tops of the walls were dismantled where the masonry was loose and then rebuilt. Great efforts have been made to ensure that the stones on the face are replaced in their original position to ensure that no historic detail is lost.

- Additional supporting works. The additional supporting works thought necessary at Castle Bolton, apart from numbers of small stainless steel dowels to fix loose masonry back locally, consists of some vertical square tube supports in stainless steel. These have been installed to support major overhangs of potentially unstable masonry and are intended as a clear statement of 20th century minimal intervention.

Figure 1.
Castle Bolton.

Major overhangs of masonry now supported by stainless steel posts.

5. LEIGH COURT BARN, WORCESTERSHIRE.

5.1.1 Leigh Court Barn is reputed to be the largest cruck barn in the United Kingdom and measures some 43m by 11m by 11m high. It has been in use as a barn since its construction in the medieval times but recently, due to changes in farming practice, it has become more of an occasional store than a barn. Being such a large structure its repair was beyond the financial means of its owner and therefore English Heritage made substantial grant aid financially and provided all the professional services necessary for the restoration work.

5.1.2 The feet of the crucks sit on masonry walls approximately 1m above ground level. The ground conditions are often fairly wet. The thrusts from the feet of the crucks had pushed out these masonry walls to varying degrees and the barn had taken on a distinct lean lengthways. These quite considerable movements had caused some damage to bracing members in the timber frame.

5.1.3 It was decided in this instance to partially dismantle the barn in order that the main cruck arch-braces could be pulled vertical, it being felt that the lean which had taken place was causing undue stress on the timber framework.

5.1.4 Additionally, due to the poor state of the supporting walls, new foundations were installed. The foundations consisted of large concrete pads supporting steel columns, which in turn were connected to the bottom of the cruck arches. The concrete foundations being below ground were of course completely buried and the steel supporting posts have been totally surrounded in the original masonry, which was replaced in its original locations. The original timber work was replaced almost 100% with very few members being renewed. Some minor repairs were made to severely decayed members. In order to be certain of the structural adequacy of some of the more decayed rafters simple load tests were carried out to prove that these were strong enough for their purpose.

5.1.5 The whole barn was re-assembled with no signs of modern intervention whatever and few new timbers being used.

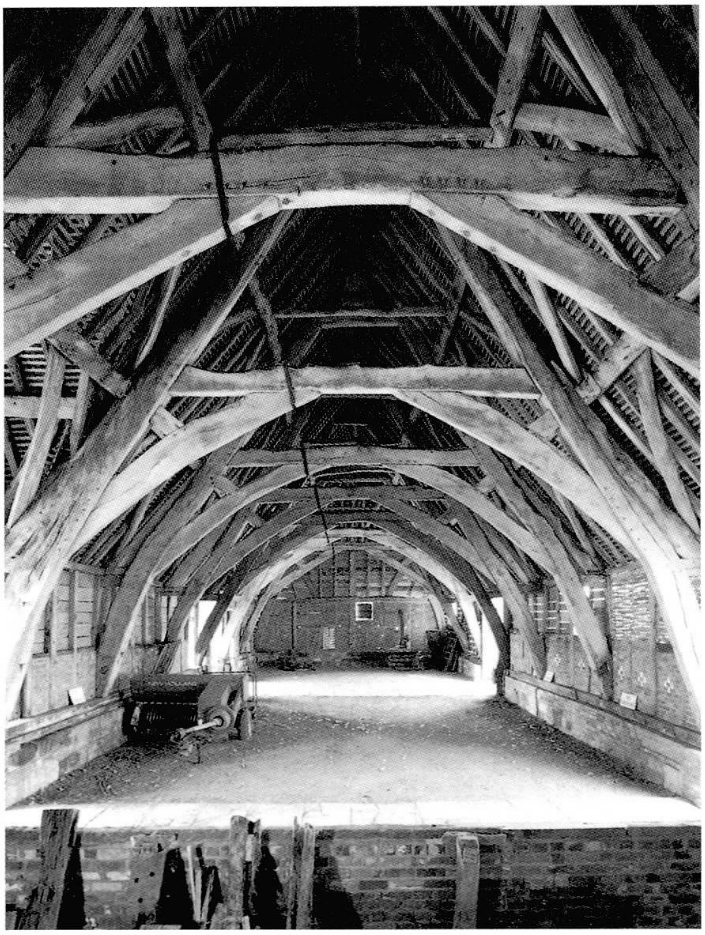

Figure 2.

Leigh Court Barn.

6. ST ANDREW'S CHURCH, GREENSTED, ESSEX.

6.1.1 St Andrew's Church is the only surviving example of a Saxon timber framed church in England and has been dated to AD 845, making it the oldest wooden church in the world. The only remaining Saxon part of the church are the vertical oak logs that form the walls of the nave. The roof of the nave is Victorian, whilst the chancel and porch are Tudor.

6.1.2 It was noticed in early 1990 that one roof truss had broken Further investigations showed that a second roof truss was also damaged and that the nave was beginning to lean to the north. A scheme of accurate structural monitoring was immediately installed and when this monitoring proved continuing movement, shoring was installed to the north wall to restrain any further leaning. Investigation of the construction of the building showed that it is, structurally, an exceedingly complex and much altered building. Its complicated construction and important history restricted the possible repair methods available.

6.1.3 Although the roof covering was in good condition it was decided to strip this to expose the rafters to insert steel T-bars into the upper surface of the principal rafters and to introduce steel bracing to the upper faces of the secondary rafters.

6.1.4 The intention of this was to repair the fractures in the principal rafters rather than to replace them and to strengthen unbroken principal rafters. The bracing created a strong diaphragm on each slope of the nave roof such that sideways forces were transmitted to the substantial brick built chancel arch and the west gable of the nave. When the building was re-roofed all of the bracing work was hidden and the only visible signs of the repair works are some small steel brackets forming end plates to the principal rafter repairs hidden under the overhanging eaves of the nave.

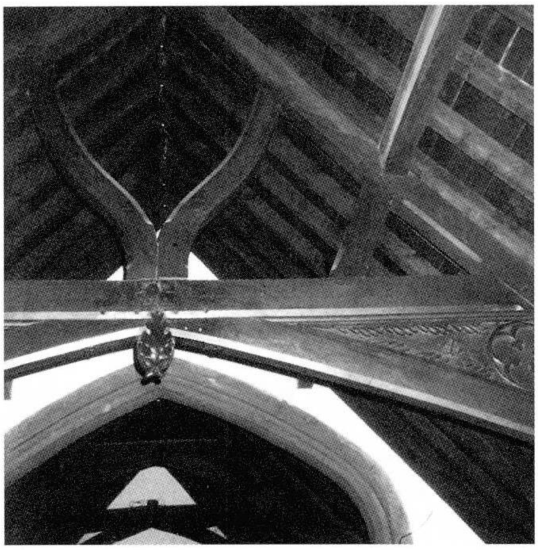

Figure 3.

Greensted Church. The roof truss joints had pulled apart and the principal rafter had broken at the purlin.

7. BURTON CONSTABLE HALL, HUMBERSIDE, ENGLAND.

7.1.1 Burton Constable Hall is a very large brick built Tudor house, mainly built about AD 1570. Problems were noted when cracks appeared in the very elaborate ceiling of the Long Gallery which is on an upper floor.

7.1.2 Investigations showed that the beam supporting a substantial gable wall over a bay window at the end of the Long Gallery had decayed due to insect attack. This decay was allowing the timber beam to deflect and to crack the ceiling below. Damage was also being caused to the substantial brickwork wall above the beam.

7.1.3 For various reasons, not least the importance of the ceiling, and the massive amount of brickwork which it carried, removal of the decayed beam was not practical. It was therefore decided to insert some steelwork into the roof space at a slightly higher level than the decayed timber beam and to ensure that this new steelwork relieved the decayed timber beam of its load. A steel channel was inserted behind the gable wall in the roof space close to the gable wall. This steel channel was brought into the roof in 2 sections and bolted together using high strength friction grip bolts. Two further beams were inserted at 90° to this steel channel and buried into the wall over the decayed timber beam. Padstones were inserted on top of the cross beams and when the concrete padstones had matured sufficiently the bolts connecting the cross beams and the main channel beam were tightened, thus transferring load from the existing decayed timber beam to the new steel channel.

7.1.4 This work was carried out without any temporary works and with only minimal damage to the existing building being caused by two small holes for the cross beams and some removal of roof coverings for access.

8. ST JOHN'S ABBEY GATEWAY, COLCHESTER, ESSEX.

8.1.1 This gateway, although small, is a fine 15th century example of flintwork construction common to this area of England. The structure generally was in good order, but the walls to the spiral staircase leading from ground level to the first floor and the roof were beginning to seriously deteriorate and form numerous cracks. Investigations showed that the walls to this spiral staircase were in many instances no more than 200mm in thickness. In some places the thickness was reduced even from this minimal amount. The thinness of the walls precluded the installation of ring beams into the thickness of the walls and the narrow access stairway meant that anything fitted internally had to be of minimum dimensions.

8.1.2 Various schemes for inserting steel rings were investigated, but the final solution decided on was to attach stainless steel expanded metal to the inside face of the turret walls. Stainless steel bolts had their heads partially ground down to enable them to be fitted into joints between the flintwork and these were grouted in position with a lime mortar. These bolts were inserted at close centres. The stainless steel mesh sheets were attached to these bolts and curved to follow the curvature of the inside face of the walls. Stainless steel mesh was bolted to the wall using washers and half-nuts. This then provided a good restraint to the inside of the masonry with minimal thickness. At this stage the inside face of the turret stair was covered with expanded metal bolted to the wall tying the entire inside face of the wall together. The expanded metal was rendered with a lime mortar render more to prevent serious injury to people using the stairs rather than for any structural reason.

8.1.3 Once again, the original construction of the gateway was not altered or interfered with in any way and the minimalist approach was adhered to.

9. THE IRON BRIDGE, IRONBRIDGE, SHROPSHIRE.

9.1.1 The Iron Bridge in Shropshire, as will be well known, is the world's first cast iron bridge, being built in 1779 and opened for traffic in 1781. It spans the river Severn at Coalbrookdale with a span of some 32m. In 1981 a scaffold was erected to enable access to be gained to all parts of the bridge. Subsequent survey work showed that there were 83 cracks to various members of the bridge structure. It was decided after considerable thought and a certain amount of computer analysis of the structure, that no action need be taken to repair any of these fractures and therefore work was restricted to thoroughly repainting the bridge and some cosmetic repairs to items such as railings.

Figure 4.

Ironbridge.
A typical fractured member.

9.1.2 The reasons for taking no action were:

- Some years previously a reinforced concrete strut had been put across the bed of the river to restrict movement of the bridge abutments.

- The arch crown connecting detail which connects the main arches together was a rigidly fixed detail and this had in no instance cracked or become deformed and this clearly showed that no major movements had taken place.

- There was concern that repairing any fracture could, if new movements occurred, create a further fracture rather than allowing the bridge to move on an old fracture point.

- No work was done on the understanding that regular inspections of the bridge were possible and that any further movement would be detected quickly.

10. CONCLUSION.

It is hoped that the foregoing case studies show that the philosophy of minimum intervention and of conserving the structure in its "as found" state whilst ensuring its stability is a structurally satisfactory alternative to a full restoration scheme.

New Concept for Foundations of Saint Michael Compound in Rome

Nouveau concept pour les fondations de Saint Michel à Rome

Neues Konzept für die Gründung des alten St-Michael-Komplexes in Rom

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P. Marchetti, born 1950, graduated in Archit. at the Univ. of Rome in 1975 and in 1977 joined the Superintendence of Architecture of Veneto. Since 1980, with the Superintendence of Archit. of Lazio, she has been involved with design and direction of restoration works in Rome.

SUMMARY

The ancient Saint Michael compound needed restoration. High compressibility of soils and varying water table levels, linked to the hydrological pattern of the Tiber, have damaged the masonry structure throughout the centuries. In order to compensate for future settlements, the new foundations are not connected to the old, and transmission of loads is regulated through hydraulic jacks placed between old and new structures. A monitoring program checks the behaviour of structure and foundations, the effectiveness of the remedial measures and the validity of the initial hypothesis of the project.

RÉSUMÉ

L'ancien bâtiment de Saint Michel a été restauré. La haute compressibilité des sols combinée avec le niveau variable de la nappe phréatique dépendant du profil hydrologique du Tibre ont causé de nombreux dommages au bâtiment en maçonnerie au cours des siècles. Afin de compenser les tassements futurs, les nouvelles fondations ne sont pas liées aux anciennes et la transmission des charges est réglée par des vérins hydrauliques placés entre la maçonnerie et la tête des pieux. Un programme de surveillance du comportement de la structure et des fondations a été réalisé afin de contrôler l'efficacité de l'intervention et la validité de l'hypothèse initiale du projet.

ZUSAMMENFASSUNG

Der Sankt-Michael-Gebäudekomplex ist kürzlich einer Restauration unterzogen worden. Die hohe Zusammendrückbarkeit des Baugrunds und die durch den Tiber beeinflussten Schwankungen des Grundwasserspiegels verursachten am Mauerwerk über Jahrhunderte grosse Schäden. Um künftigen Absenkungen vorzubeugen, wurden die neuen Fundamente nicht mit den alten verbunden. Für die Lastübertragung wurden hydraulische Pressen zwischen Mauerwerk und Pfahlköpfen eingebaut. Ein Computerprogramm überwacht das Verhalten der alten und neuen Bauteile, die Wirksamkeit der Massnahmen und überprüft die Gültigkeit der dem Projekt zugrunde liegenden Hypothese.

1. INTRODUCTION

This project is an example of functional restoration carried out with intergrated conservation criteria, with the aim of saving a building that was abandoned for incidental motives, but which retained its validity as part of the historical urban fabric of the city by adjusting its use to the different events that occurred. The St. Michael Complex, will become a "cultural zone" with the purpose of entertaining people within an accessible area for exhibitions and cultural displays. The project is the result of an interdisciplinary operation between the Sovrintendenza of Rome and the consulting engineering of Prof. Giorgio Croci, with the cooperation of eng. Valter Maria Santoro.

2.HISTORICAL SURVEY

The St. Michael Hospice (photo 1) was founded under a papal initiative to solve the problem of beggary and homelessness in Rome at the end of the seventeenth century, works continued for the following 150 years. Thus the St. Michael complex is architectonically fragmented in contrast to the formal and unified elevation along the Tiber river.

photo 1

The initial nucleus, contructed between 1686 and 1689 by Carlo Fontana and Mattia di Rossi consists of the buildings surrounding the "Cortile dei Ragazzi" (Courtyard of the Boys).

In successive years the building was enlarged with the construction of the enclosing wall along the Via San Michele, a wool factory (1693), a men's prison (1701), all designed by the architect Fontana.

The part of the complex facing Porta Portese (1706-1709) was used for small shops and rented rooms, and following this, it became Customs Officers barracks; the building then became a shelter for the elderly (1708), the church of Our Lord Jesus Christ (1710) and the small church of Our Lady of the Good Voyage (1712).

In 1734, Pope Clement XII commisioned Ferdinando Fuga to design a women's prison next to the barracks. In 1796 the complex was further enlarged with the addition of the conservatory for unmarried women and between 1831 and 1834 with the construction of low buildings used as artisans' workshops.

The St. Michael complex retained it's proper functions, that of a place of rehabilitation and above all a site of important artisan activities from its creation at the end of the 1870's. Following the Unification of Italy and the loss of papal patronage it suffered a rapid decline culminating with the events of World Wars when it was used for the temporary housing of evacuees, which accelerated the natural process of degradation. It was in this precarius structural condition that the building was put up for sale in 1968.

The state, exercising the right of pre-emption, bought the building and converted it into the Ministry of Culture.

The complicated operations of restoration and strengthening, carried out by the Superintendent of Architecture for Rome, began in 1973.

The new function of the buiding meant an increase in the live loads which made the design of strengthening to the superstructure and substructure necessary.

The works on St. Michael's are continuing: the areas of the ex-men's and women's prisons as well as the barracks are being finished as they required an in-depth study and complex interventions to the foundations, carried on by the ICLA Enterprise.

3. GEOTECHNICAL CHARACTERISTICS OF THE SOIL

The geotechnical characteristics of the soil have rendered the interventions on the foundations very delicate.

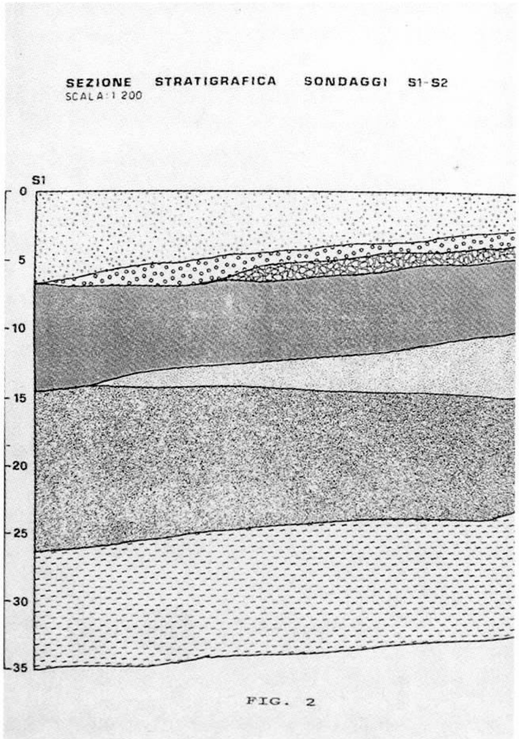


FIG. 2

photo 2

The various buildings bear onto a stratum of alluvial sediments from the nearby Tiber river, consisting of a succession of highly compressible strata which have responded and continue to respond to the periodic oscillations of the level of the river. In fact from the geognostic tests carried out (in the course of which 4 "Casagrande"-type piezometers were installed) it was found that the hydraulic pattern of the area is constantly evolving and is directly connected to the variation in level of the river. In the case of the last few years, long periods of intense rain and longer periods of drought have occurred. All this together with the heavy weight of the complex has produced, throughout the history of the building, a series of static failures such as local fractures, distorsions and differential settlements, of which cracking is the most highly visible signal.

The series of geotechnical tests have, between them, identified and characterized the principal lythotypes of the soil which may be summed up in the following manner (photo 2):

a) SOFT SILTY CLAY: (immediately below the surface fill of variable thickness) is around 10m thick and is defined as brown [undrained shear strength $c_{u\ max}$ = 25 KPa];

b)SAND: from -13m to -23m below ground level: condensed medium-fine brown with thin weak silty layers [angle of friction $\phi' = 33°-43°$; drained cohesion c' 0-9 KPa];

c) CLAYEY SILT: blue-grey with thickness exceeding 10m [angle of friction $\phi' = 27°-34°$; drained cohesion c' = 2-15 KPa].

4. INTERVENTION CRITERIA RELATING TO SOIL DEFORMATION

The whole St. Michael complex has been the object of various reinforcement interventions during the last decades; these concerned the strengthening of the superstructure (masonry, vaults, slabs) and foundations. The latter have mainly been reinforced by widening the contact surface of the soil in order to reduce the average pressure under the foundations.

The lengths of the individual buildings (upto 150m) and the mutual interconnections related to the essential geotechnical problems, resulted in an overall approach which invisaged the segmentation of the buildings into several parts, each one capable of resisting the potential settlement caused by the compressibility of the soils (rather than subsidence phenomena). Thus a project which involves the creation of a joint between the Restoration Institute wing and the ex-Men's Prison is now in progress, as well as one between the Barracks and the building along the Tiber.

The sensistivity of the surface soils to the pore pressure linked to the hydrological situation of the Tiber river, lead to the design of partial underpinning of the building in order to isolate the building from the more compressible layers

Because of the change of use and the increase in load to the foundations, a new system of foundations "*parallel*" which could carry the new loads without effecting the upper strata was realized.

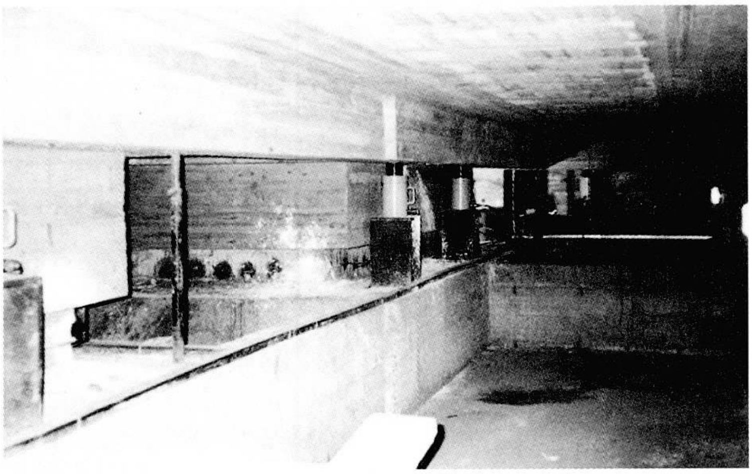


photo 3

This system (photo 3) was carried out using micropiles of diameter 200mm, placed next to the the perimeter load-bearing wall and connected by means of a strong ring beam (0.50 m x 0.80 m) and was loaded by oleodynamic jacks (photos 4-5) which lifted the building, reducing the load to the existing foundations which were also reinforced with a layer of concrete and finally some prestressed Dywidag bars.

Where the reaction of the jacks could have produced highly eccentric forces with consequent tilting moments, a horizontal bracket connected to the

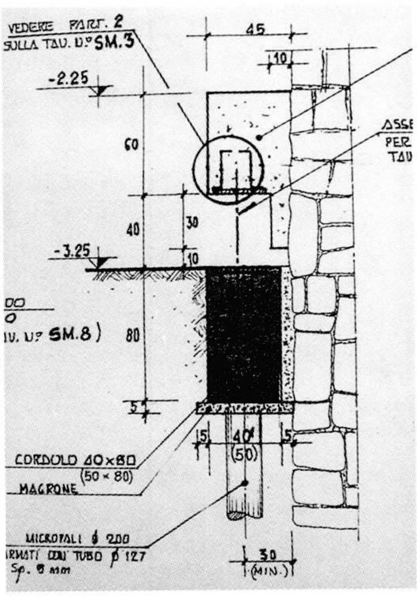


photo 4

head of the micropiles, was placed in niches made in the wall, so that the load could be received close to the baricentre of the wall.

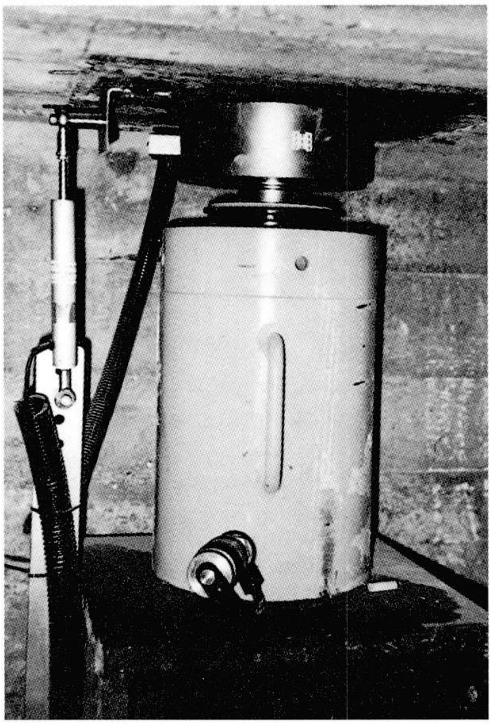


photo 5

In such way only some loads will apply to the micropalesinstead of total trasmission through passive underpinning that would have acquired more extensive intervention with much more micropales.

In the course of the works, very strong vertical load tests were carried out on the micropiles, much higher than the true working loads to be carried by the piles, in order to guarantee the resistance.

The gradual transfer of loads to the new foundations was carried out during the evolution of the interventions to the super-structure.

The new foundations were loaded in phases. Some phases coincided with the increase in load due to the restructuring works at the various floor levels and therefore with the different phases of the super structure works. Initially, the removal of the fill from the vaults and some floors partially unloaded the existing foundation (picture 1 a,b) , then, as the repair works were carried out, the new foundations began to function by carrying all the new loads themselves (picture 1 c,d).

During the adaptation of the old foundations, also in the course of various loading phases of the foundations, a control of the structure was preprepared by means of a monitor of various physical quantities the knowledge of which was necessary for the evaluation the phenomena of differential settlements between the various zones of the foundations or to interpret the effect of a variation in loading transmitted over time. The monitoring system was carried out by Tecnocontrolli s.r.l., of Rome.

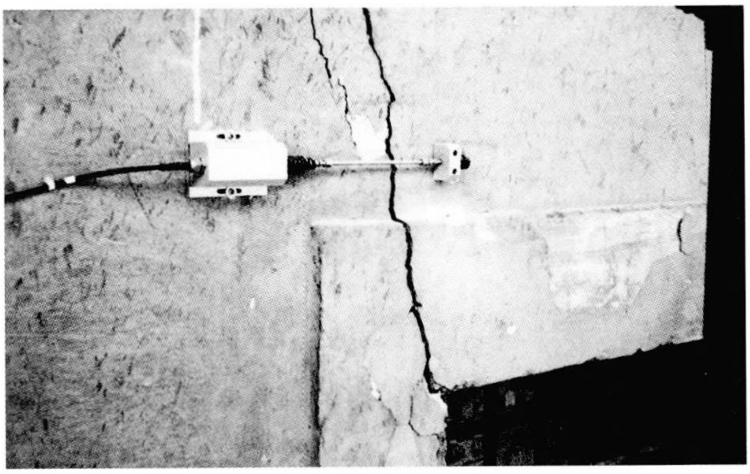
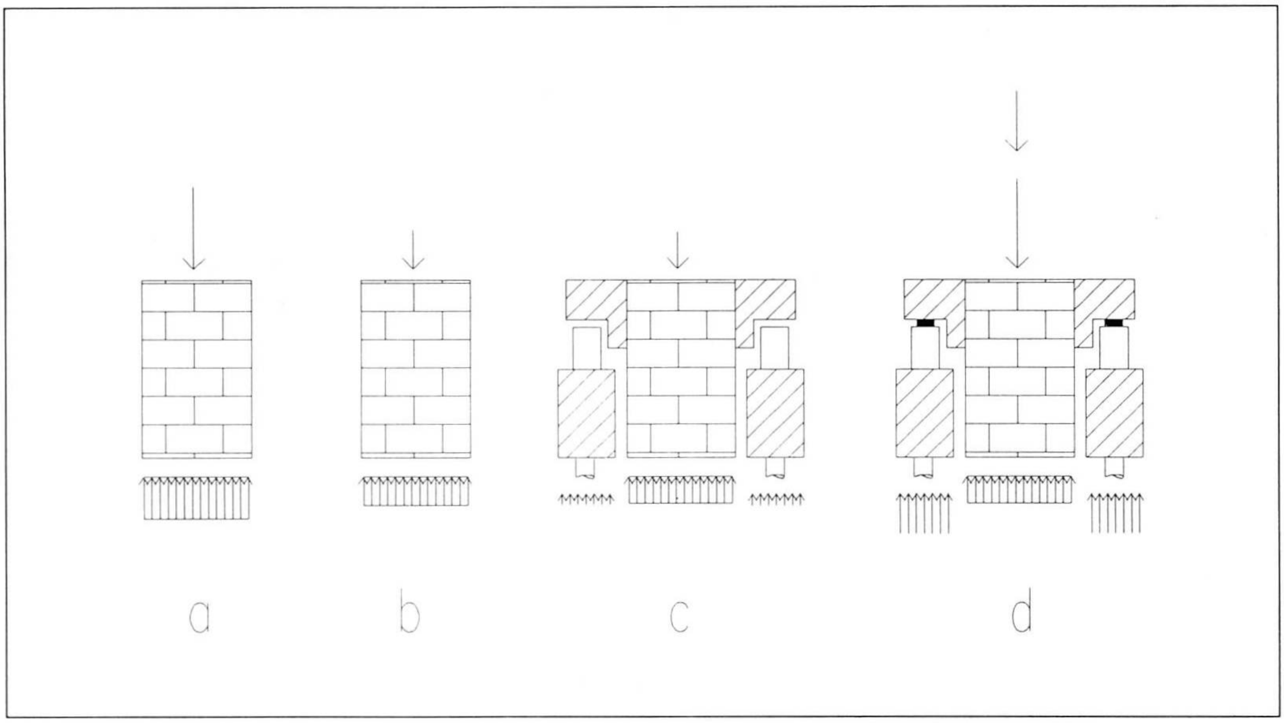


photo 6

In particular, 12 "load sensors" were placed (photo 5) between the piston of the jack and the steel plate set inside the bracket of the old foundation (they gave an indication of the load transmitted by the jacks), 12 displacement sensors capables of indicating the relative displacement between the new and old foundations and 10 deformometers (photo 6) placed on the larger cracks present in the upper levels were used to verify that unforseen detrimental effects would not occur during the loading phases.



picture 1

A manual control was carried out on a larger scale alongside the automatic control, with high precision optic instruments and read every 20-30 days, in order to indicate any absolute movements and any relative displacement between the measuring points (90 in number) placed on the façade and in the courtyard.

The examination of the curves obtained from the data provided by the monitoring system and by the reading of the manual instruments confermed the validity of the initial hypotheses and thus gave an indication that the structure was behaving correctly.

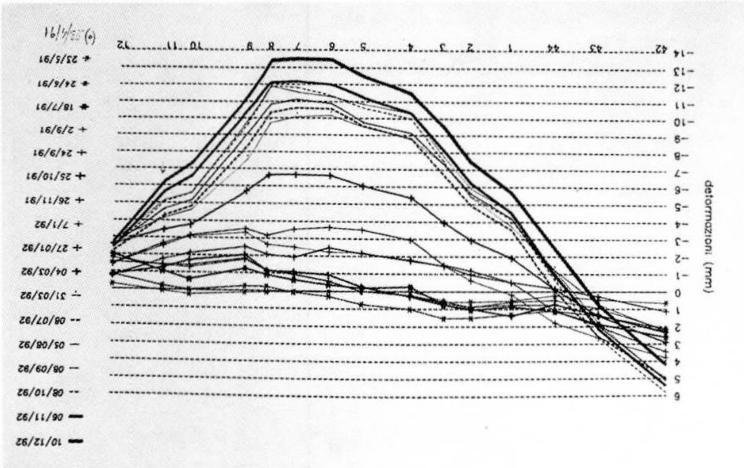


photo 7

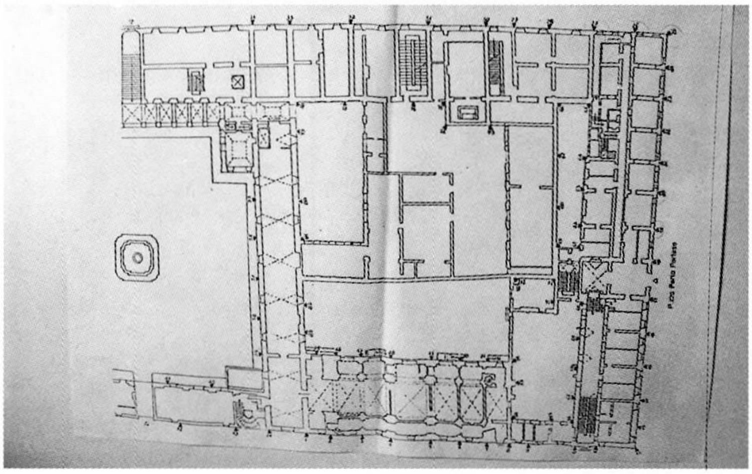


photo 8

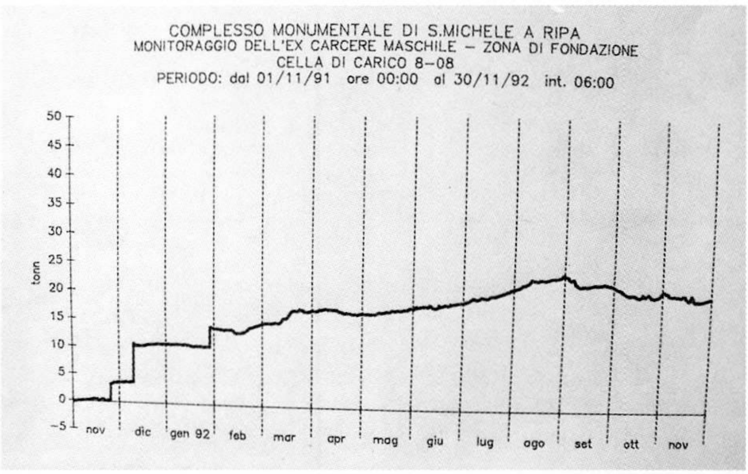
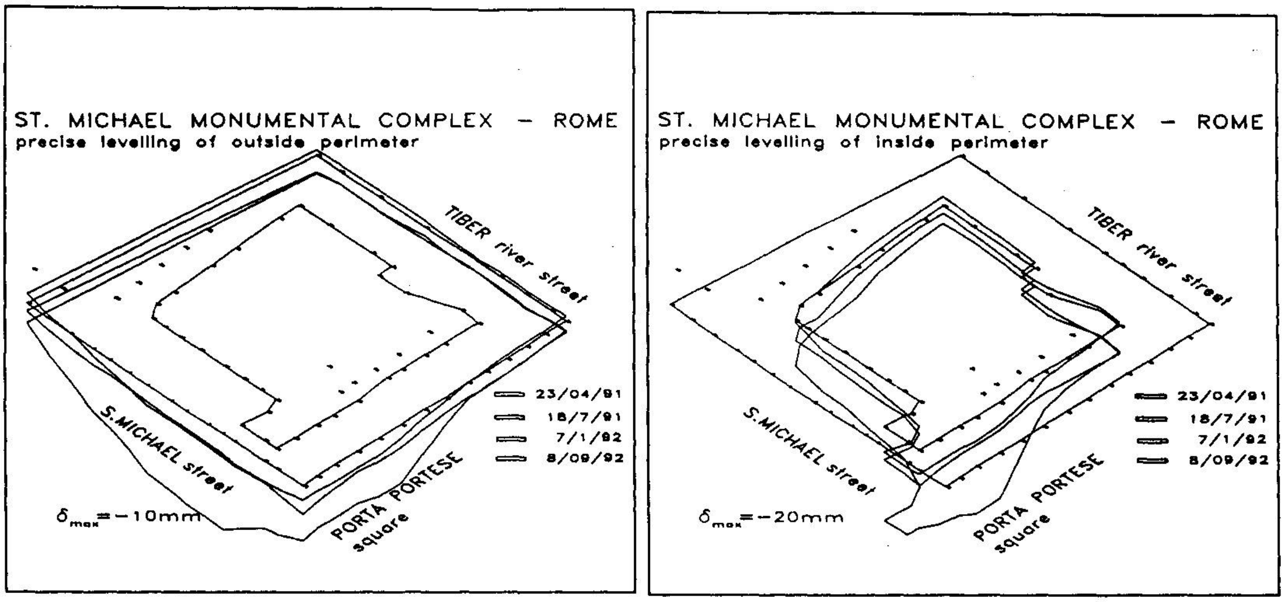


photo 9

The results of the topographical measurements are represented in the diagram (photo 7); they were precisely measured at key positions along the perimeter wall (photo 8) (picture 2), with reference to the façade of the ex-men's prison on Via San Michele. These graphs were made after the first phase of the transfer of loads to the micropiles. In the diagram the development of the settlements in correspondance with the key positions along the façade (each curve represents a measurement), they become closer and therefore the increments in the settlements tend towards zero .This trend clearly indicates that the transmission of the loads to the parallel foundations is very effective. In fact we can observe a partecipation of the micropiles to the bearing capacity of the existing foundations although they are not called to support the total load.

This result is confirmed by the diagram of the load sensor which indicates the value of the loads on the foundations in correspondance to the jacks (photo 9). In this diagram the different phases of the load transfer, through the jacks, are evident from the variations of the forces. During the monitoring period it is possible to detect a gradual increase of the pressure in the jacks, corresponding to the trend observed in the adjustment of the settlements of the structures which, together with the sudden variations verified during the loading phases, appear to have a positive drift: the load on the foundation structure increases over time.



picture 2

5.CONCLUSION

The intervention described represents a criterion for the strengthening of foundations using a new concept that has provided excellent results. In fact, notwithstanding the difficulties connected with the inferior characteristics of the soil and the precarious state of the super-structure, the result was so satisfactory that we are proposing its use for the Poli Palace: the building just behind the famous Trevi Fountain. The success of the project was ensured by the use of the monitoring system, above all for the verification of the correspondance between the theoretical scheme and the real results, and the possibility to "adjust" the loads in the various phases of the works.

ACKNOWLEDGEMENTS

The authors would like to thank Eng. Fabio Macrì and Lesley Goldfinger B.Eng. for their help in preparing this report.

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Leveling and Protection at the Metropolitan Cathedral in Mexico City

Compensation des tassements et sauvegarde de la Cathédrale de Mexico

Hebung und Schutzmassnahmen an der Kathedrale von Mexiko-Stadt

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Social Development Office
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Sergio Zalidvar Guerra, graduate of the National Univ. of Mexico and postgraduate of the Univ. of Rome, was honoured by the Mexican Government before the sixteenth General Conference of UNESCO, from which the Convention Concerning the Protection of the World, Cultural and Natural Heritage emerged. For 25 years, he has dealt with the cultural heritage of his country.

SUMMARY

The Metropolitan Cathedral in Mexico City, one of the most important architectural monuments in the Americas, is built on extremely soft lacustrine clays, over many remains of the ancient City of Tenochtitlan. From the earliest stages of its construction, the cathedral has been affected by ongoing differential settlements due to a heterogenous process of consolidation within the clay deposits. Remedial actions are all intended to counteract the effects of deformations induced in the cathedral as a consequence of this consolidation process.

RÉSUMÉ

La Cathédrale de Mexico, un des plus importants monuments d'architecture dans les Amériques, fut construite sur des argiles lacustres extrêmement tendres, au-dessus des restes de l'ancienne Ville de Tenochtitlan. Dès le début de sa construction, la Cathédrale a été affectée par les problèmes de tassements différentiels, dûs aux processus de consolidation hétérogène des dépôts d'argile.

ZUSAMMENFASSUNG

Die Kathedrale von Mexiko-Stadt, eines der bedeutendsten Baudenkmäler Amerikas, wurde auf extrem weichen, tonigen Seeablagerungen gebaut, auf den Resten der antiken Stadt Tenochtitlan. Seit Baubeginn führte der ungleiche Konsolidierungsprozess im Ton zu Setzungsunterschieden. Die Sanierungsmassnahmen konzentrieren sich auf den Ausgleich der sich an der Kathedrale zeigenden Folgen der Deformationen.

INTRODUCTION

Construction of the Metropolitan Cathedral started during the second third of the XVIth century, from 1573 and was concluded at 1815, over some of the remains of ancient constructions of the prehispanic city of Tenochtitlan. The cathedral, one of the most important architectural monuments in the Americas, was built over a masonry platform about 1.2 m thick which was itself founded on a mass of wood short stakes separated some 60 cm, having a diameter of 20 cm and lengths of 2.5 to 3.5 m. This foundation system was borrowed from existing prehispanic technology. The total weight of the structure is about 127 344 t, and the pressure transmitted to the subsoil is in the order of 12.2 t/m^2. The building is made of volcanic masonry rock; its plan dimensions are 66.36 x 122.26 m it has five naves, a central dome and two bell towers, 60 m high.

The Metropolitan Sagrario parish church, adjacent to the Cathedral, was built between 1749 and 1768. It is also founded on a volcanic rock masonry plataform wich is partly superimposed on the Cathedral's, in its western side. The remainder of the plataform rests on 30 cm thick mortar bed made out of lime and sand wich was placed upon at thin layer of charcoal; on this layer, a mass of wooden stakes, 8 to 12 cm diameter and 2.5 m in length and separated about 1.8 to 2 m each, is set. The weight of this church is 22,500 t and pressure transmitted to the subsoil is about 10 t/m^2. Its footprint is 47.20 m wide and 47.77 m long.

Aztec buildings were also constructed over artificial fills, which consolidated the underlying upper clay strata (fig. 1).

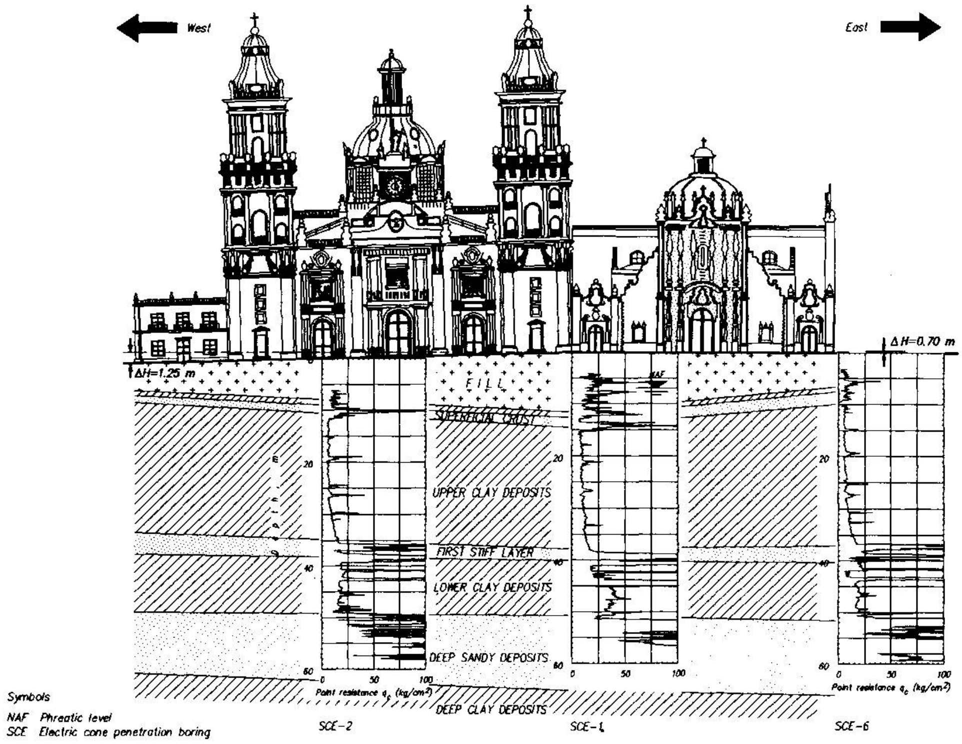


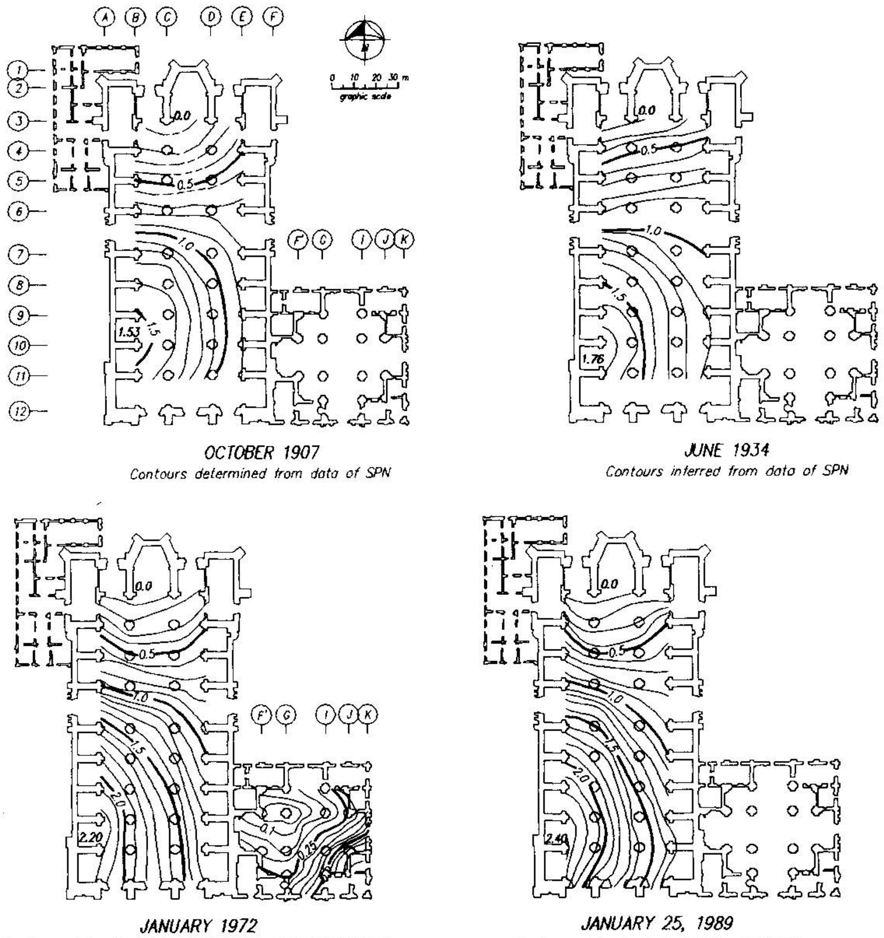
FIG 1 STRATIGRAPHY AT THE SITE AND DIFFERENTIAL SETTLEMENTS AT GROUND LEVEL

Considerable variations appear in the platform thickness, apparently caused by differential settlements that started to occur from the very beginning of the construction. The Cathedral has continued to be affected by ongoing differential settlements. This condition can be verified by architectural adjustments and corrections performed in some of its parts; among these are columns of different higths and wedged masonry layers, such as those found between the first and second windows of the west side tower, which were used to compensate for differential settlements. The continuous differential settlements are well documented since the end of last century.

This paper describes the actual conditions of the Cathedral and the measures adopted to save it, based upon the results of a comprehensive field investigation program. The job is being carried out by a group of the most outstanding mexican profesionals, which I have the honor to lead; they are Messrs. Enrique Taméz, Enrique Santoyo, Fernando López Carmona and Roberto Meli.

THE PROBLEM

In April 1989, severe cracking along the building southeast-northwest direction, revealed that alarmingly large settlements were taking place in the Cathedral. A review of all topographic surveys, as well as of all previous actions taken to mitigate the effects of differential settlements followed; the effects of the construction of a subway and a circular drainage tunnel built in front of the Cathedral were also considered. Surveys performed in 1907 revealed the existence of a differential settlement of more than 1.5 m between the apse and the west tower; in 1972, this settlement reached 2.2 m (fig. 2) and exceeded 2.4 m in 1990. Between the two towers, differential settlements are presently 1.25 m. The Sagrario church has tilted easterly and settled more than 90 cm differentialy with respect to the Cathedral.



OCTOBER 1907
Contours determined from data of SPN

JUNE 1934
Contours inferred from data of SPN

JANUARY 1972
Contours determined from data of Ingenieria Experimental

JANUARY 25, 1989
Contours determined from data of PICOSA

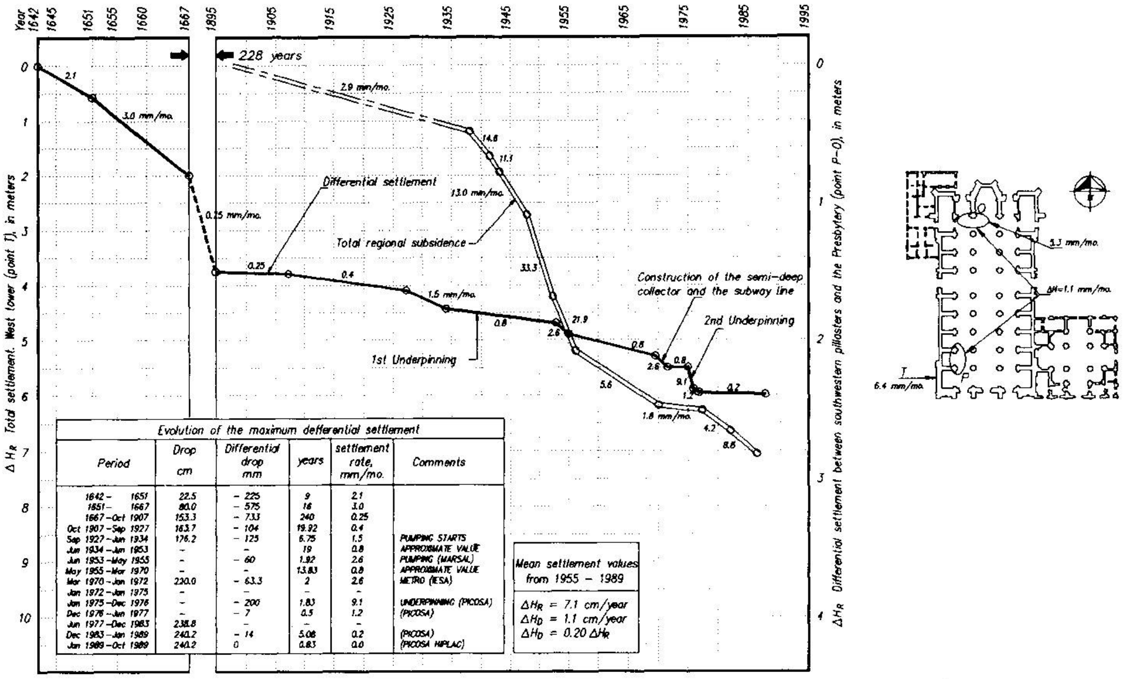
FIG 2 EVOLUTION OF THE DIFFERENTIAL SETTLEMENTS

History of the Settlements

Regional subsidence is a consequence of the consolidation of the soft clay layers brought about by the over-exploitation of deep aquifers. Between 1900 and 1930 the regional subsidence rate averaged 2.9 mm/month; it ranged 11 to 14 mm/month during the fourties and reached 33 mm/month in the mid-fifties.

Strict measures banning the exploitation of existing wells and the construction of new ones were rigidly enforced, after city authorities were prompted to do so by geotechnical engineers. These measures paid off: by the end of the sixties, the subsidence rate was 5.8 mm/month and 1.8 towards 1975. These favorable tendencies were unfortunately reversed by 1978, when the subsidence rate started increasing again, from 4.2 mm/month in that year, to 8.8 in 1983. Presently, it is 5.9 mm/month.

Differential settlements in the Cathedral due to non-homogeneous distribution of compressibilities within the underlying soft clay deposits, accounts for 20 percent of the total differential settlements between the apse and the west tower (fig. 3).



Evolution of the maximum defferential settlement

Period	Drop cm	Differential drop mm	years	settlement rate, mm/mo.	Comments
1642 – 1651	22.5	– 225	9	2.1	
1651 – 1667	80.0	– 575	16	3.0	
1667 – Oct 1907	153.3	– 733	240	0.25	
Oct 1907 – Sep 1927	163.7	– 104	19.92	0.4	
Sep 1927 – Jun 1934	176.2	– 125	6.75	1.5	PUMPING STARTS
Jun 1934 – Jun 1953	–	–	19	0.8	APPROXIMATE VALUE
Jun 1953 – May 1955	–	– 60	1.92	2.6	PUMPING (MARSAL)
May 1955 – Mar 1970	–	–	13.83	0.8	APPROXIMATE VALUE
Mar 1970 – Jan 1972	220.0	– 63.3	2	2.6	METRO (IESA)
Jan 1972 – Jan 1975	–	–	–	–	
Jan 1975 – Dec 1976	–	– 200	1.83	9.1	UNDERPINNING (PICOSA)
Dec 1976 – Jun 1977	–	– 7	0.5	1.2	(PICOSA)
Jun 1977 – Dec 1983	238.8	–	–	–	
Dec 1983 – Jan 1989	240.2	– 14	5.08	0.2	(PICOSA)
Jan 1989 – Oct 1989	240.2	0	0.83	0.0	(PICOSA HIPLAC)

FIG 3 MAXIMUM REGIONAL AND DIFFERENTIAL SETTLEMENTS

As the subsoil under the Cathedral undergoes different subsidence rates, the resulting pattern of differential settlements is quite complex. These facts had not been properly understood during previous attempts to underpin the Cathedral.

PREVIOUS UNDERPINNING ATTEMPTS

Between 1930 and 1940, fill materials were removed from the foundation cells, defined by the grid of masonry beams, and these beams were latteraly confined with reinforced concrete also a concrete raft was incorporated into the foundation of each cell. As a consequence of these modifications, pressures transmitted to the soil were reduced by roughly 3 t/m^2; the behavior of the Cathedral improved over a short period. A new attempt to reduce or eliminate differential settlements began in 1972; this time the Cathedral was underpinned with control piles which not behave completely as expected. Pile driving was very problematic and was carried out with great difficulty.

These piles are provided with a device to control the magnitude of loads at their heads can be applied. This feature, according to suppositions made on the project, could have allowed the effective correction of the whole building and eventually suppressed differential settlements.

A number of factors prevented the success of this system. For instance, some piles were defectively driven and others were not properly socketed into the hard layer, others turned out to be short and became floating piles. Positive frictional forces in excess of 120 t would allow the transmission of projected loads to the pile heads by assuming that this would be added to the point bearing capacity. However, compression of the clay strata due to regional subsidence generated downward drag forces on the piles, i.e. negative skin friction, rather than the expected positive frictional forces. Thus, the overal capacity of the piles was drastically reduced; assuming no defective piles, the individual capacity is only 100 t. If the working suppositions for control piles had been verified, the system could have suppresed sudden settlements, but it would not have avoided differential settlements because the foundation raft unavoidably follows surficial movements; otherwise, an apparent emergence of the building would ensue. In that case the totality of loads, about 130,000 t, would have to be borne by the piles, that have an overall capacity some four times smaller.

By adding about 1,500 control piles, the weight of the Cathedral could be completely transferred to deeper strata. This alternative was carefully examined and set aside when it was realized that there is virtually no space for driving more piles under the Cathedral as it stands; new piles could be accommodated but only if substantial and exceedingly costly structural modifications were made at the basement level.

The influence of the subway and the deep drainage tunnel were studied along with a survey of piezometric conditions. The subway became a drain because of the procedure with which it was built; nowadays, the volume of water flowing into it is no longer large and its influence on the behavior of the Cathedral has consequently diminished. Water flowing into the drainage tunnel seems to be significant, but neither of these two factors is as important as the pumping of water from deep wells, which is extensively and massively occurring throughout the Valley of Mexico. Piezometric levels are frankly worrying; the loss of hydraulic heads at depths ranging between 25 and 50 m is about 18 t/m^2. It means that the phreatic level which is actually located at a depth of 7.4 m and at 3.5 m in 1972 will slowly descend to a depth of 25 m, assuming no changes in the present pumping conditions. It may be possible by installing a slurry trench 65 m deep and a system of infiltration wells, to reduce the consolidation process only 18 to 25 percent.

Natural water content in Mexico City clay can be as high as 500 percent. Once extracted by pumping, restituting water by injection as it has been several times suggested is extremely difficult, only partially effective and very expensive. The expansion of the urban zone has reduced the area of infiltration for the replenishment of the aquifers in the Valley of Mexico, and many problems have risen as a result.

Average total regional settlement in the lake zone is 7.50 m since the turn of this century, which will roughly double in the next 75 years, according to analyses made on the basis of compressibilities determined from undisturbed samples. Future regional subsidence will give rise to more differential settlements in the Cathedral. Predictions of settlement over the periods of 1990 to 2010 and 1990 to 2065 have been clearly plotted (fig. 4).

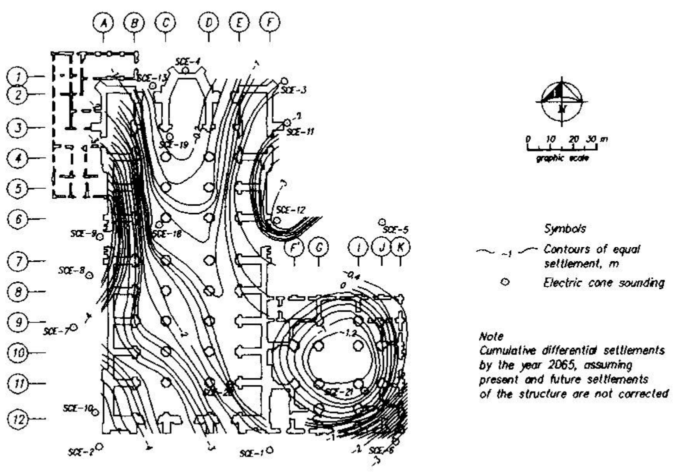


FIG 4 ESTIMATED CUMULATIVE DIFFERENTIAL SETTLEMENTS BY THE YEAR 2065

Towards 2010, differential settlements between the apse and the west tower will be 3 m; between the apse and the east tower, 1.6 m and between the central line and the corners of El Sagrario, 0.7 m. In 2065, settlements in the same points will increase to 4.4 m, 2.0 m and 4.2 m respectively. Clearly, these buildings would collapse between 2010 and 2030, if remedial actions are not taken to improve the behavior of their foundations. These structures have satisfactorily resisted many earthquakes through the centuries, but this situation might change as progressive settlements and distortions will implacably increase their seismic vulnerability.

PROPOSAL FOR SOLUTION

Four procedures for facing the problem have been analyzed. The first one is the addition of 1,500 piles, each with a minimum individual capacity of 100 t. The construction reasons for rejecting this procedure were carefully considered.

The second one involves the use of 240 large diameter (2.4 m) piers supported by resistant strata located at 60 m depth. This method would require the construction of very a grid of sturdy beams, in order to transfer the total weight of the structure to the piers heads, which would be provided with sophisticated control devices. Economic considerations, as well as the forseeable construction difficulties have led to the rejection of this method.

The construction of an impervious slurry trench along with water injection wells, in order to reestablish piezometric levels and reduce the rate of regional subsidence has been analyzed. Adopting this solution would imply the continuous operation and maintenance of the injection points. However, stoppages in operation of injection pumps would reactivate subsidence ipso facto. Rather than an integral solution, this method is a palliative, or a complementary measure to be used in conjunction with other methods. The volume of water required for the injections without the slurry trench is roughly equal to the daily needs of 15,000 people and certainly an unaffordable luxury in Mexico City where water is so scarce.

The fourth proposed procedure is a method of underexcavation which was deemed to be the most feasible solution, technically and economically. In this method, the magnitude of differential settlements is reduced by excavating soil from the more deformable strata, in order to achieve a uniform subsidence rate over the structure. The following considerations were taken into account:

a. The basic geometry of the Cathedral gives it the capacity to resist strong seismic demands, as the differential subsidence has reduced this basic capacity. If conditions existing before 1935, when regional subsidence began increasing at an alarming rate, can be restored the earthquake-resistant capacity of the Cathedral would also be restored.

b. The method allows for geometrical adjustments in the suprestructure, which can reverse the effects of deformations developed over the last few decades. For instance, inclinations of some of the pillars and columns already amount to more than 3 percent of their height; tilting of the lateral naves has increased the chord in the arches of the main nave by more than 40 cm, cracking and fissuring support points as well as the vaults.

Briefly stated, the underexcavation method consists of removing soil by means of horizontal borings drilled from a large diameter vertical shaft; excavation is carried out below the foundation level, preferably within the plastic clays. The volume of soil excavated gives rise to controlled settlements whose velocity can be adjusted at will. Selective excavation allows the correction of distortions in the platform and the superstructure. Analytical tools for predicting the effects of underexcavation derive from plasticity based methods, commonly used in tunneling. Perforation of a hole within the soil mass produces displacements; if the hole collapses and a new one is drilled, surface settlements will increase. Successive drilling and underexcavation will eventually cause the required amount of corrective settlement.

A hydraulic machine for drilling horizontal borings is placed at the bottom of the access shafts; the borings, 7.5 cm in diameter and 6 m long, are horizontally drilled; upon their removal a small inute tunnel is left, which gradually closes and eventually collapses in about 20 hours; successive radial perforations performed at predetermined time intervals are used to precisely control the magnitude and rate of the ensuing settlements.

This procedure has been implemented in the church of San Antonio Abad, located about 1 km south of the Cathedral. The geometry of the vault of San Antonio Abad is similar to the main vault of the Cathedral; soil and subsoil characteristics are roughly the same in both locations.

The work at San Antonio Abad served as an experimental model for the thechnique. Tools and mechanical instruments were perfected and times needed for closure of horizontal drillings were established. This experimental model allowed for the development of a good control and monitoring system for the procedure.

Results of the experiment developed in San Antonio Abad were highly encouraging and the decision to implement this method in the Metropolitan Cathedral was, to a large extent, based upon that success.

The general deformation pattern in the Cathedral shows a hump towards the northern part of the central nave and generalized subsidences towards the south and southwest. Hence, the purpose of underexcavation would be to level off this hump and by so doing, restore in part the verticality of the columns. The periphery of the superstructure has been propped in order to make the lateral naves rigid, while the central one was left free. Induced tilts will tend to close the central arch, maybe causing some fracturing in the vault, which will have to be repaired afterwards.

If conditions existing in the year 1934 can be reestablished, a comprehensive refurbishment of the Cathedral will follow. It is likely that a new phase of underexcavations will be necessary in about 20 to 25 years. Over this period, the Cathedral will undergo settlements caused by regional subsidence that will induce small distortions in its superstructure.

This procedure started to be implemented in 1991 and some of its results can now be seen, as the construction of shafts (fig. 5) and the effect of continuously pumping water have modified deformation trends by reducing the magnitude of differential settlements and stopping the tilting of lateral naves and their external walls. Future work will result in reversing the movements of naves and walls; columns and walls will also rotate until conditions existing in 1934 can be restored. The project does not intend to straighten the building completely.

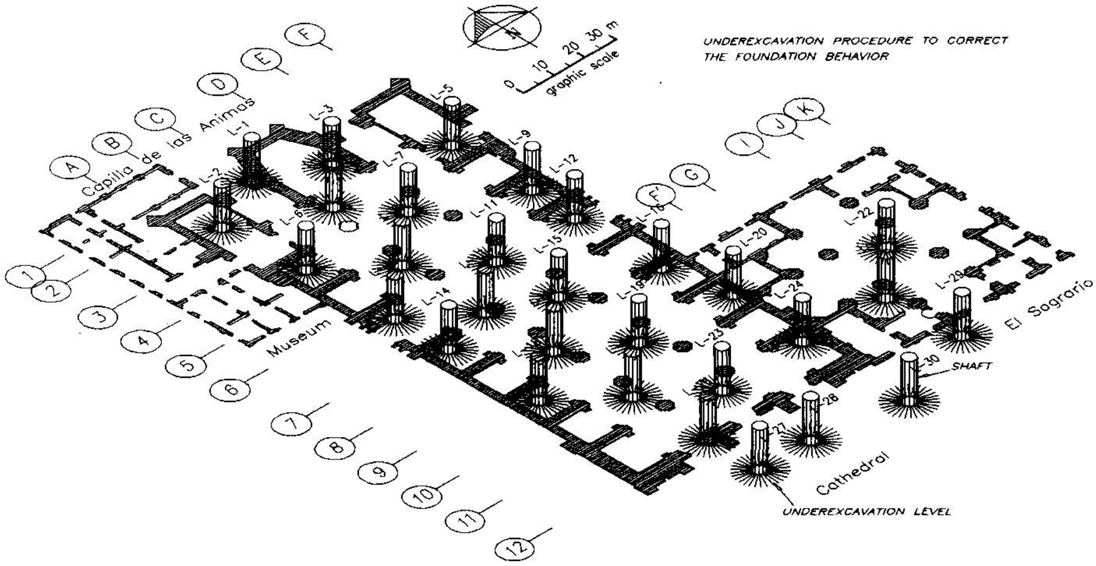


FIG 5 SHAFT AND RADIAL UNDEREXCAVATION BORINGS LOCATION

A complex monitoring system, including leveling surveys and convergence measurements inside the Cathedral, is being used to control this method. Likewise, careful records of the volumes of the excavated soil will be continuously carried out during the underexcavation process. The first stage of underexcavation work will start in 1993 and is expected to span over six to eight years.

El Sagrario will be subjected to a similar process, which will first stop and then invert present trends, rotating the structure towards the Cathedral and correcting differential settlements as required. The procedure may be repeated in twenty to thirty years.

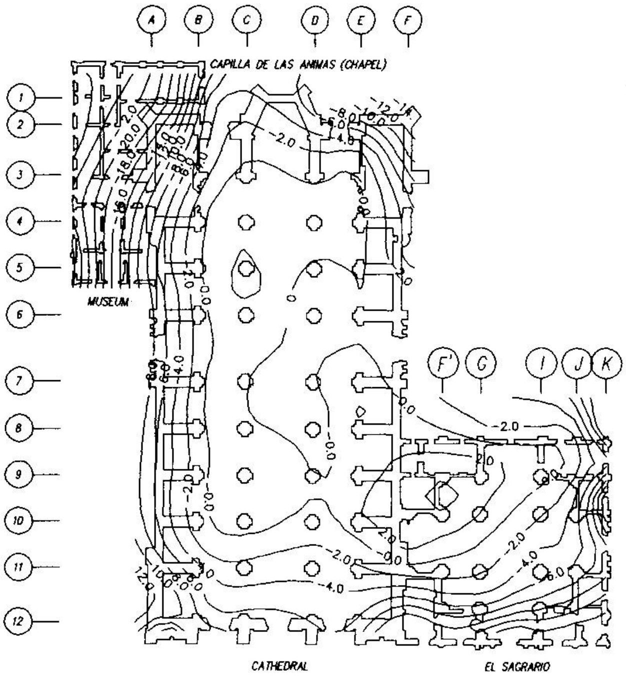


FIG 6A CONTOURS OF EQUAL SETTLEMENT RATE, IN mm/year; MEASUREMENTS FROM JAN 7, 1991 (N20) TO SEPT 2, 1991 (N28)

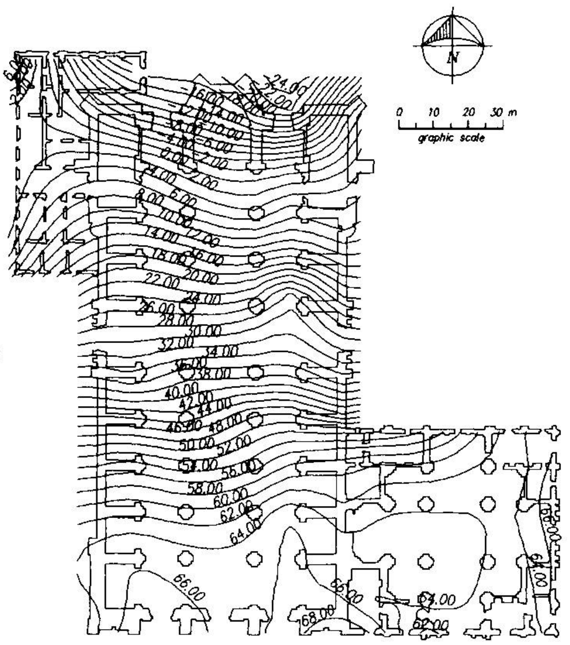


FIG 6B CONTOURS OF EQUAL SETTLEMENT RATE, IN mm/year; MEASUREMENTS FROM OCT 25, 1991 (N-1a) TO OCT 26, 1992 (N-25a)

CONCLUSIONS

Figures 6A and B shows annual rate of settlement contours for the Cathedral and the Sagrario before and after the construction of the shafts. Clearly, the effect has been definitely beneficial for the condition of both structures in terms of the floor level configuration. Also, no significative damage to the structures ensured as a curequence of these movements. Up today the main driving factor for achieveng this is, the selective operation of dewatering system for the excavations of the shafts.

Much finer and precise control of settlements can definitely be achieved by means of the underexcavation technique already developed. Hence, the results shown in fig 6 are both encouraging and they also demostrate the structure ability to withstand the induced settlements.

Restauration d'une chapelle à Uccle, Bruxelles

Instandsetzung einer Kapelle in Uccle, Brüssel

Rehabilitation of a Chapel in Uccle, Brussels

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J.-L. Hilde, né en 1948, obtient son diplôme d'ingénieur civil des constructions en 1971 à l'Université de Liège et rejoint le bureau SECO où il s'occupe principalement du contrôle de la construction de ponts et de grands ouvrages hydrauliques, comme l'ascenseur de Strépy-Thieu.

A. Janiaux, né en 1949, obtient son diplôme d'ingénieur civil des constructions à l'Université Libre de Bruxelles, en 1972. Il s'occupe des problèmes de constructions souterraines impliquant principalement des procédés spéciaux d'exécution. Actuellement, il est consulté pour d'importants projets d'assainissement.

RÉSUMÉ

La chappelle Notre-Dame du Bon-Secours est un petit édifice du XIIème siècle ayant subi de nombreux désordres liés à la mauvaise qualité de sa fondation superficielle. L'article détaille surtout la conception et la construction des nouvelles fondations profondes réalisées malgré l'exiguïté des lieux et les faibles dimensions des pièces de la structure. Il décrit également les solutions adoptées pour le transfert des charges vers la nouvelle infrastructure.

ZUSAMMENFASSUNG

Die Kapelle ist ein Bauwerk aus dem XII. Jahrhundert, welches durch die schlechte Qualität seiner Flachgründung zahlreiche Schäden erlitten hat. Dieser Artikel beschreibt die Planung und die Ausführung der neuen tieferen Fundamente, welche trotz des geringen zur Verfügung stehenden Raumes und der kleinen Abmessungen der Bauteile ausgeführt worden sind. Der Artikel beschreibt ebenfalls die gewählten Lösungen, um die Lasten auf die neuen Bauteile abzutragen.

SUMMARY

The chapel is a XII century small building affected by important defects due to the poor quality of the foundations. The paper details the conception and building operations of the new deep foundations in spite of the cramped conditions and the small dimensions of the structural elements. The chosen solutions to transfer the loads onto the new substructure are described.

1. L'EDIFICE

Le chapelle est un édifice de style roman bâti sur un plan rectangulaire de 17 m x 12 m.
Sa construction daterait du XII ième siècle mais le choeur a été reconstruit en 1412.
La première restauration remonte à 1693, la seconde date de 1932.
Le monument est classé depuis 1938.
Le corps principal du bâtiment se compose de trois nefs parallèles de 11,65 m de long séparées par deux rangées de quatre colonnes distantes de 3,50 m.
Les portées intérieures sont respectivement de 1,93 m, 4,20 m et 1,93 m.
La nef centrale est prolongée sur une longueur de 5,60m pour le choeur que jouxte la sacristie carrée de 4,50 m de côté.
La structure de la chapelle fondée directement sur le sol se compose des murs périphériques en maçonnerie de pierres blanches de 0,45 m d'épaisseur et de deux files de colonnes en pierre bleue de 0,45 m de diamètre.
Les colonnes supportent des arcs de forme elliptique également en pierre blanche surmontés par les maçonneries de briques de terre cuite sur lesquelles s'appuient les toitures.
Les plafonds sont constitués d'un revêtement de plâtre apposé sur un lattis accroché à la charpente en bois de la toiture.
La toiture de la nef centrale présente deux versants inclinés à 60°. Elle est prolongée par un clocheton octogonal au-dessus du choeur.
Les toitures des nefs latérales présentent un seul versant à 45°.
Toutes les toitures sont constituées de charpentes en bois recouvertes d'ardoises.
La figure 1 montre l'ensemble de l'édifice.

2. LES DESORDRES

La chapelle est située à front de la rue de Stalle à Uccle. Cette importante voie de pénétration qui relie la sortie de l'autoroute Paris-Bruxelles au centre de la ville, est empruntée chaque jour par des dizaines de milliers de voitures et de camions.
Cette circulation intense et divers travaux de voirie et d'égouttage exécutés à proximité immédiate du bâtiment ont largement contribué à l'aggravation des désordres qui affectent l'édifice depuis longtemps.
Avant la restauration de 1990-1991, des crevasses importantes affectaient les structures en pierre et en brique. Elles trouvent leur origine dans les tassements différentiels importants subis par les éléments structuraux fondés sur un sol de piètre qualité et gorgé d'eau.
Les restaurations précédentes n'ont donné aucun résultat durable car elles n'ont jamais remédié au défaut de fondation.
Un vaste projet de modification des voiries avoisinantes est à l'origine de la réhabilitation complète de la chapelle en commençant par la création d'une nouvelle fondation.

3. LE SOL

Une campagne d'essais géotechniques préliminaires a permis de cerner les causes des désordres et d'orienter le choix du mode de renforcement de l'infrastructure.
Deux essais de pénétration statique (CPT - 200 kN) ont mis en évidence un sol de très mauvaise qualité sur les 7 premiers mètres sous les massifs de fondation existants (résistance à la pointe $C_{kd} < 2$ MPa) et la présence sur les 5 m suivants d'une couche d'argile dont le C_{kd} dépasse un peu 3 MPa et dont le frottement s'accroît rapidement.

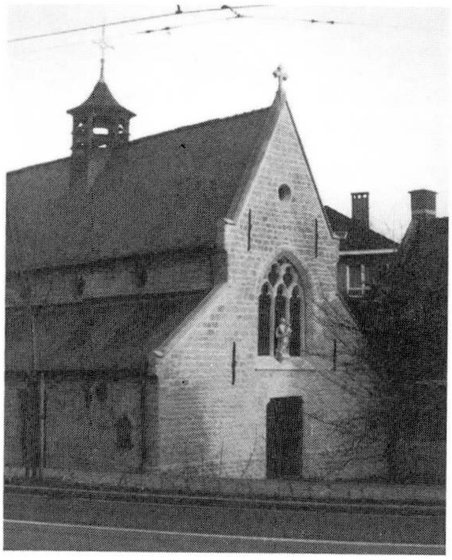

Fig. 1 - Vue de la Chapelle

LES INTERVENANTS

Maître de l'ouvrage	: Administration des Routes de Bruxelles Capitale
Ingénieur-Conseil	: SIETCO S.A.
Entrepreneur Général	: WEGEBO S.A.
Sous-traitants	: FRT S.A SMET BORING S.A. LABOREX BVBA
Contrôle technique	: Bureau SECO S.C.

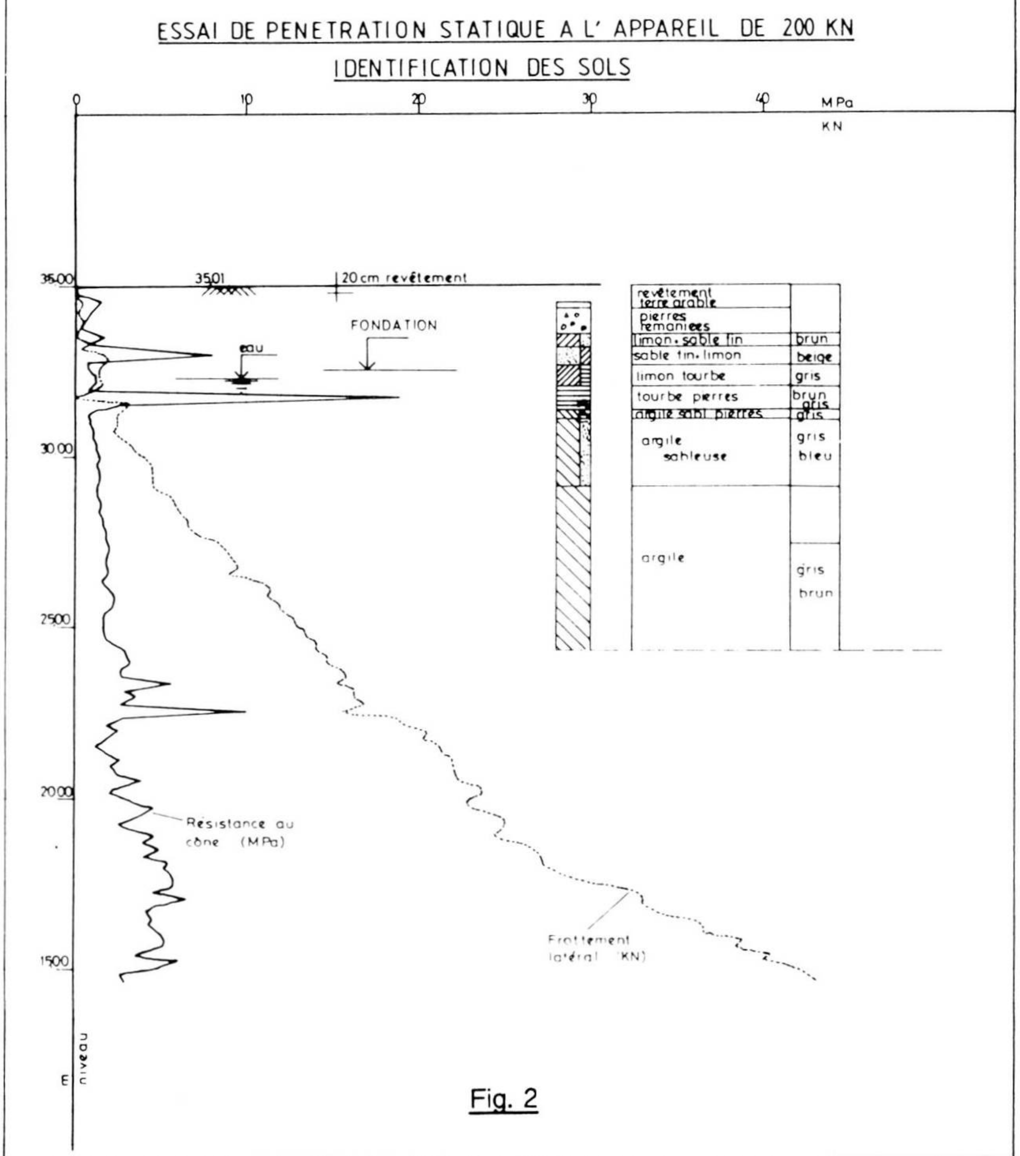

Fig. 2

Ces résultats ont été confirmés par deux forages carottés avec prise d'échantillons remaniés et non remaniés sur lesquels ont été effectuées les analyses de laboratoire habituelles (granulométrie, limites d'Atterberg, poids volumique, teneur en eau, indice des vides, degré de saturation) ainsi que 34 essais oedométriques pour déterminer les courbes contraintes-déformations et le coefficient de perméabilité à différents niveaux du sous-sol.
L'examen visuel des échantillons a également révélé la présence de passes tourbeuses dans les 3 premiers mètres sous la base du bâtiment.
Le niveau de la nappe aquifère se situe à environ 2 m de profondeur soit immédiatement sous la base des murs.
La figure 2 reprend les résultats d'un essai de pénétration ainsi que la succession des premières couches de sol identifiées lors du forage voisin.

4. LA NOUVELLE FONDATION

Les résultats des essais géotechniques ont clairement montré l'inadéquation de la fondation directe et la nécessité de créer une nouvelle infrastructure pour le bâtiment en reportant les charges sur une couche de sol convenable au moyen de fondations profondes.
La réalisation de celles-ci ne devait cependant entraîner aucune aggravation des désordres existants, ce qui interdisait l'utilisation de machines lourdes et génératrices de vibrations.
En outre une partie des pieux devait être exécutée à l'intérieur du bâtiment dont les faibles dimensions des baies et l'exiguïté des espaces intérieurs interdisaient l'accès à des engins encombrants.
Plusieurs solutions pouvaient répondre à ces sujétions.
Les solutions de reprises en sous-oeuvre ont rapidement été abandonnées vu le niveau élevé de la nappe phréatique et les risques d'accentuer les désordres par un terrassement sous les fondations existantes.
En fait le choix s'est porté sur la réalisation de micropieux.
Habituellement les micropieux sont forés directement au travers des massifs de fondations à stabiliser avec une légère inclinaison sur la verticale et ce de façon alternée pour équilibrer les efforts horizontaux. C'est le contact pieux-structure qui permet la transmission des efforts.
Dans le cas de la chapelle, les dimensions des massifs de fondation n'étaient pas suffisantes pour permettre le passage convenable des efforts et plus particulièrement les composantes horizontales ne pouvaient être correctement équilibrées sans la construction de poutres de chaînage de part et d'autre des murs.
En outre le forage en biais vis-à-vis des lits de maçonnerie sur toute la hauteur du massif d'appui risquait d'accroître les désordres.
Ces considérations ont conduit au choix de la réalisation de micropieux verticaux avec créations de structures en béton armé pour permettre le transfert des charges.
Cette solution présentait en outre l'avantage de ne devoir travailler à la structure qu'après la réalisation complète des pieux c'est-à-dire après amélioration du sol à proximité des fondations existantes

5. LES MICROPIEUX

En fonction de la localisation et de l'intensité des charges à reprendre, l'ingénieur-conseil a prévu des pieux de 50 et 90 kN de capacité portante.
Les pieux présentaient un diamètre de 0,15 m et une barre d'armature centrale de type GEWI.

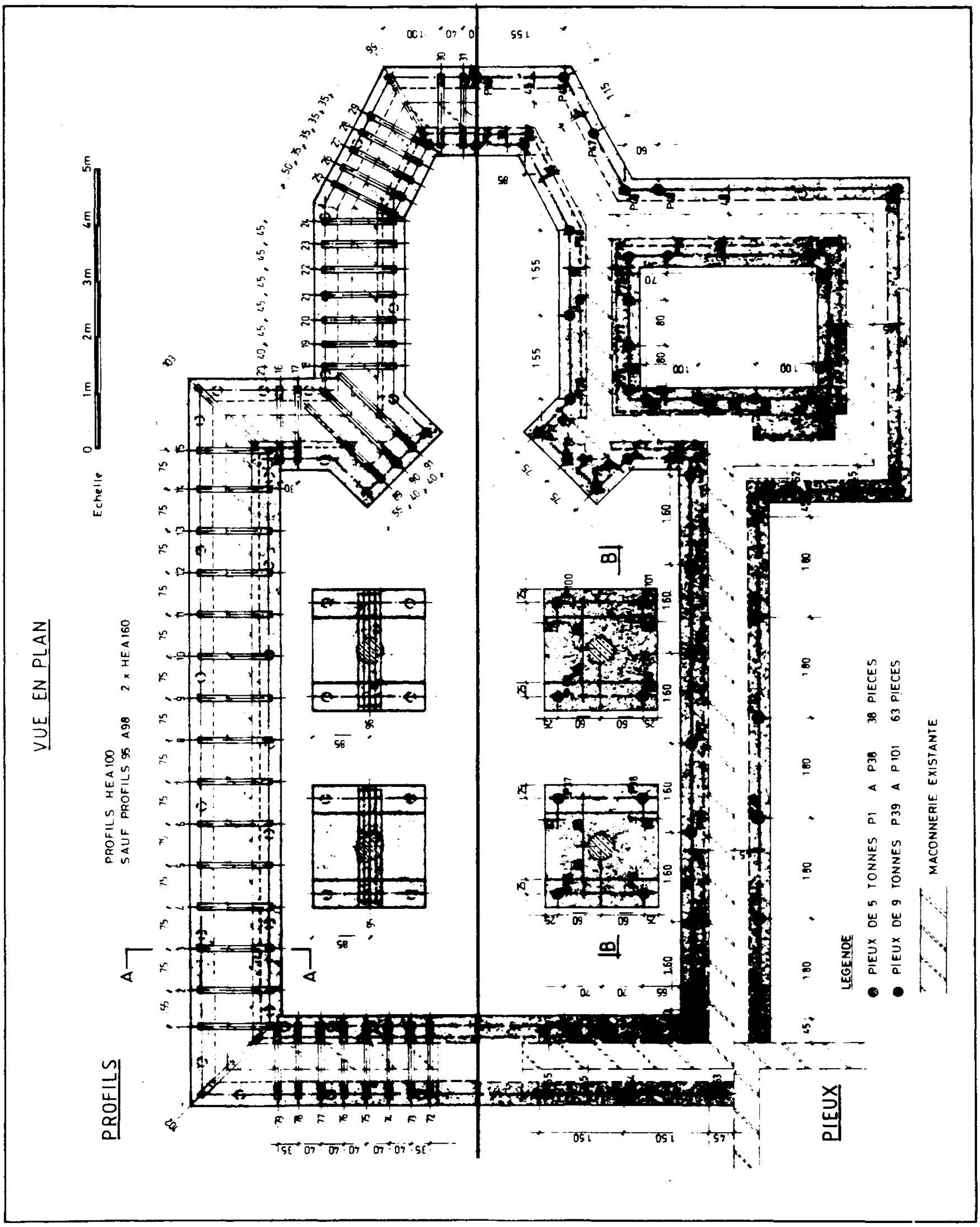
VUE EN PLAN
Echelle
0 1m 2m 3m 4m 5m
PROFILS HEA100
SAUF PROFILS 95 A 98 2 x HEA160
PROFILS
PIEUX
LEGENDE
PIEUX DE 5 TONNES P1 A P38 38 PIECES
PIEUX DE 9 TONNES P39 A P101 63 PIECES
MACONNERIE EXISTANTE

Fig. 3

Ils se distinguaient par la longueur et le diamètre de l'armature ; ainsi pour
50 kN une longueur de 13 m et une barre de 22 mm de diamètre et pour 90 kN une longueur de 18 m et une barre de 25 mm.
Le mode de réalisation prévu, basé sur les résultats des essais de sol, comportait plusieurs phases.
Le forage d'abord à sec sur une profondeur de 3,50 m pour placer un fourreau métallique destiné à protéger les couches contenant des passes tourbeuses était poursuivi par lançage.
Le bétonnage au moyen d'un microbeton mis en place par un tube plongeur suivait le placement de la barre d'armature composée de tronçons de 3 m de long assemblés par des manchons.
Afin de contrôler la capacité portante et l'enfoncement des pieux sous charge de service, un essai préalable de mise en charge a été effectué sur quatre pieux d'essais réalisés dans le terrain voisin à 5 m de la chapelle.
Les deux premiers pieux de 18 m (1) et de 13 m (2) ont été bétonnés gravitairement, les deux autres de 18 m (3) et 13 m (4) ont été bétonnés sous pression.
La charge d'essais des pieux de 18 m a été poussée jusqu'à 360 kN, celle des pieux de 13 m jusqu'à 325 kN.
La figure 4 montre les résultats de ces essais.
La comparaison des diagrammes effort-enfoncement mesurés a surpris en montrant que les pieux de 18 m se comportaient moins bien que les pieux de 13 m et que le bétonnage sous pression n'apportait aucune amélioration sensible des résultats.
Comme le pieu d'essai (2) présentait un enfoncement de 1,7 mm pour une charge de
90 kN et une sécurité vis-à-vis de la rupture supérieure à 3, le pieu de 13 m de long bétonné gravitairement a été retenu pour l'ensemble de la fondation.

6. LE TRANSFERT DES CHARGES

Après réalisation des pieux verticaux des deux côtés des murs périphériques et des murs intérieurs, des poutrelles métalliques HEA 100 ont été scellées au moyen de mortier sans retrait dans des trous forés parallèlement aux lits de maçonnerie au moyen d'outils à tête diamantée pour éviter les vibrations.
La figure 3 reprend les micropieux et les profils métalliques.
Des deux côtés des murs, des poutres en béton armé coiffant les têtes de pieux et enrobant les extrémités dépassantes des poutrelles ont permis le transfert des charges vers la nouvelle fondation (figure 5).
Le problème posé par les quatre colonnes centrales était plus délicat car leurs massifs de fondation étaient trop petits pour appliquer la même méthode.
Outre les pieux, il convenait de construire une nouvelle semelle et de transférer les charges des colonnes aux pieux sans provoquer de nouveaux désordres.
Le mode de réalisation retenu comportait plusieurs étapes.
Pour chaque colonne, le forage de quatre pieux a été suivi par l'exécution de deux poutres parallèles en béton armé coiffant chacune deux pieux.
Suivant les deux axes de séparation entre les nefs, des cintres métalliques destinés à reprendre les charges transmises aux colonnes par les arcs en pierre ont été montés en prenant appui sur les poutres en béton au droit des pieux.
La liaison entre les pièces métalliques et les arcs en pierre, a été réalisée par l'intermédiaire de vérins sous des plateaux horizontaux supportant les maçonneries de brique avec bourrage supérieur pour obtenir un contact continu.

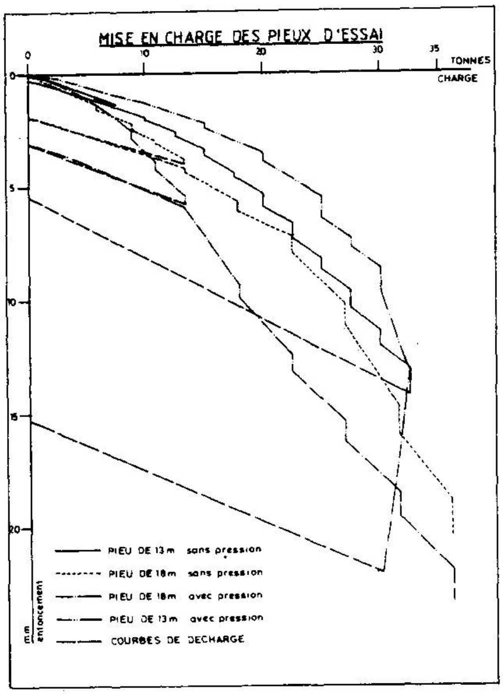

Fig. 4

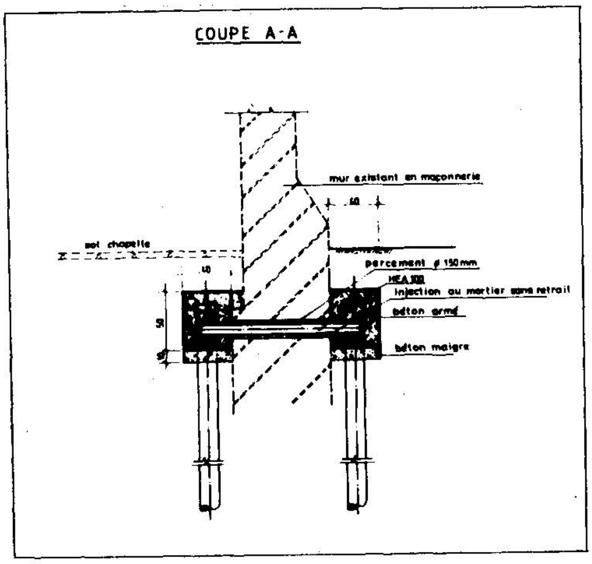

Fig. 5

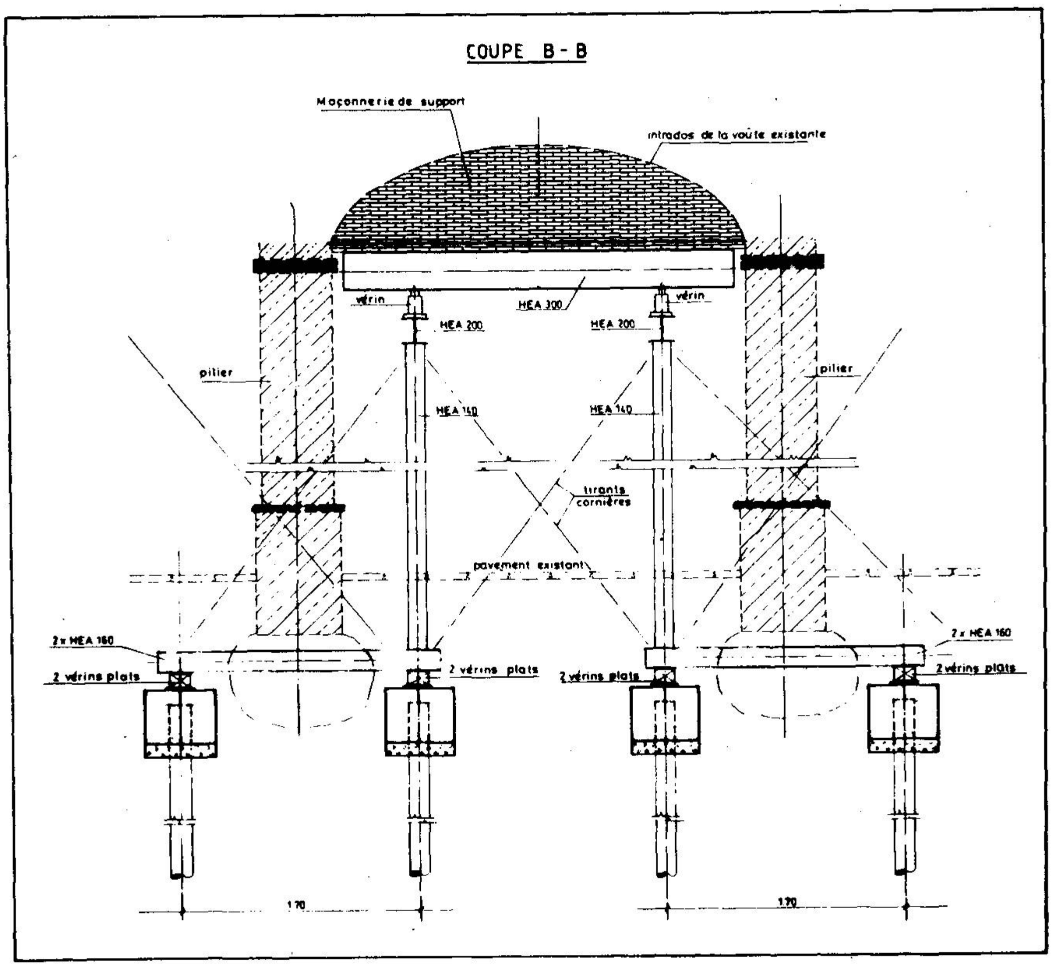

Fig. 6

La mise en pression des vérins correspondant à la charge théorique dans les colonnes a permis de décharger celles-ci et de contrebalancer les déformations élastiques de l'échafaudage et des pieux ainsi que les raccourcissements dus au fluage du sol sous ceux-ci.
Des carcans disposés autour des pieds des colonnes devaient empêcher toute dérive horizontale.
La figure 6 reprend l'ensemble des pièces de support provisoire.
La démolition délicate de la moitié du massif de fondation de chaque colonne a été directement suivie par la pose d'une poutrelle HEA 160 appuyée sur les poutres en béton par l'intermédiaire de deux vérins plats et bloquée contre la base de la colonne.
Une légère mise en pression des vérins a permis de reprendre les déformations élastiques du système d'appui et d'éviter le tassement ultérieur de la colonne lors de l'exécution des mêmes opérations pour l'autre moitié de sa fondation.
A ce moment toute la structure centrale de la chapelle reposait sur les cintres, les efforts introduits aux pieds des colonnes devaient seulement maintenir les pierres assemblées.
Le transfert des charges des cintres vers les colonnes a été effectué séparément pour chaque file de cintres par mise en charge simultanée au moyen d'une pompe manuelle des huit vérins disposés sous les poutrelles d'appuis des deux colonnes voisines.
Le vérinage effectué par palier avec vérification des mouvements des colonnes et des cintres au moyen des micromètres, a été poursuivi jusqu'au décollement entre les arcs et les maçonneries de remplissage.
L' échafaudage a été maintenu en place jusqu'à ce que les vérifications quotidiennes de la conservation de la pression dans les vérins aient montré que le fluage des nouvelles fondations était devenu négligeable.
A ce moment, les vérins ont été injectés au mortier, les cintres démontés et les poutrelles d'appui ainsi que les vérins ont été bétonnés dans une dalle de liaison entre les poutres en béton.

7. LA RESTAURATION

Après la création de la nouvelle fondation, les travaux de remise en état des structures ont commencé par un nettoyage complet des maçonneries en pierre et en brique. Toutes les crevasses ont été traitées d'abord en scellant des barres en acier inoxydable de part et d'autre des lèvres des fissures puis en injectant du mortier sans retrait pour boucher les ouvertures.
A l'extérieur, les joints des maçonneries en pierre ont été vidés et rejointoyés à neuf avant le siliconage des façades.
A l'intérieur toutes les maçonneries ont été restaurées avant la remise en état de la décoration.

8. LES CONCLUSIONS

La création de nouvelles fondations pour un petit édifice présente plusieurs problèmes particuliers liés aux faibles dimensions tant des pièces de structure que des aires disponibles pour l'exécution.
La conception et la réalisation des travaux doivent donc être menées avec toute la délicatesse voulue pour ne pas conduire à des dégâts irrémédiables.
Le recours à des moyens simples et facilement contrôlables est sans nul doute un gage de réussite.

An Engineering View of the Statue of Liberty Restoration

Restauration de la Statue de la Liberté du point de vue de l'ingénieur

Die Restaurierung der Freiheitsstatue aus der Sicht des Ingenieurs

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SUMMARY

The restoration of the Statue of Liberty was completed in 1986, in time for its centennial celebration. This paper describes the original structural system, its history, the materials and its analysis and design. Also described are the studies, surveys, investigations and analyses conducted to determine the conditions existing and the repairs and modifications necessary for the restoration of the structure. Finally, the solutions considered for the long term preservation of the Statue are discussed, the decision-making process is described and the solutions implemented are presented.

RÉSUMÉ

La restauration de la Statue de la Liberté a été complétée en 1986, à temps pour la célébration de son centenaire. Ce document décrit la structure originale, l'histoire, les matériaux et l'analyse du projet. Sont aussi décrites les études, les mesures, les recherches et les analyses entreprises pour déterminer les conditions existantes et les réparations et modifications nécessaires à la restauration. Pour terminer, les solutions considérées pour la conservation à long terme de la Statue sont discutées, la démarche pour la prise de décision est décrite et les solutions retenues sont présentées.

ZUSAMMENFASSUNG

Die Freiheitsstatue wurde 1986 rechtzeitig zum hundertjährigen Jubiläum restauriert. Der Bericht beschreibt sowohl Originalkonstruktion, Geschichte und Baustoffe wie Analyse und Gestaltung. Ausserdem werden die Studien, Vermessungen und Berechnungen dargelegt, um den aktuellen Zustand und die notwendigen Reparaturarbeiten und Anpassungen zu bestimmen. Zum Schluss werden die für eine langfristige Erhaltung der Statue in Betracht gezogenen Vorschläge, der Entscheidungsprozess und die angewandten Lösungen geschildert.

1. ORIGINAL STRUCTURAL SYSTEM [8]

1.1 Structure of the Statue [1] [2] [5] [6] [7]

Earlier monumental works in copper repoussè (the technique of hammering copper sheets in relief) supported on an iron armature was stabilized by mass masonry on the interior. An example of this is the colossal statue of St. Charles Borromeo constructed near Aroma, Italy, in 1697.

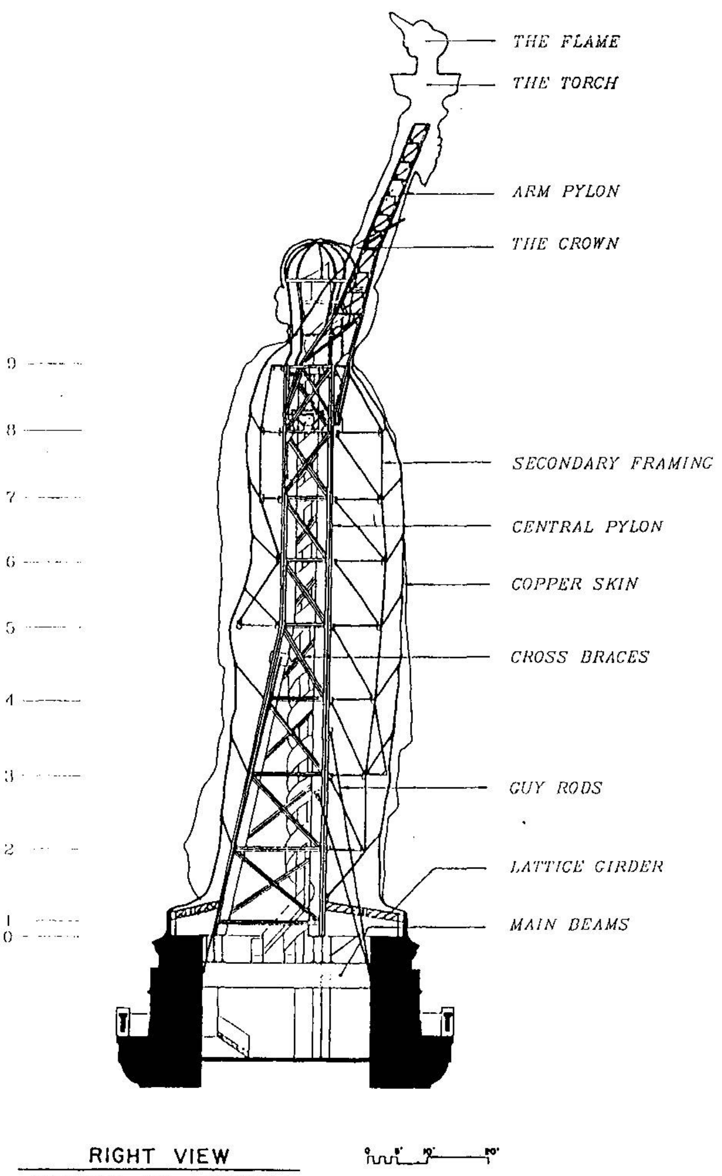

Fig. 1 The Structure of the Statue

Eugenè Viollet-le-Duc, the first architect-engineer to work on Auguste Bartholdi's statue, Liberty Enlightening the World, conceived its structure as a series of sand-filled interior coffers. Viollet-le-Duc died in 1879 after only the torch, arm and head were built. Gustave Eiffel, the famous French bridge engineer, was called upon to complete the structure.

Eiffel's design for the structure was innovative for his time (Fig. 1). This unique system consists of a trussed central pylon and secondary framing to support the copper skin and iron armature through an assembly of sliding and articulated elements which permitted "breathing" of the Statue under thermal and wind loadings. The most ingenious aspect of his design is the concept of flat bars to connect the skin support system to the secondary framework. The steeply inclined bars are installed in compression and provide the resilience to allow adjustment to changes due to temperature and wind pressures. This design was also suited to the planned prefabrication in Paris and subsequent dismantling and re-erection in New York.

As innovative as Eiffel's concept for the structure was, his selection of the material for the structure, puddled wrought iron, had been used for centuries.

For the analysis of the pylon structure the force polygon graphical technique was brought to Eiffel by Maurice Koechlin, a student of Karl Culmann who developed the method at the Federal Polytechnic Institute in Zurich. Only the structure of the central pylon was analyzed for the static lateral loads used by Eiffel in the design of railway bridges.

1.2 Anchorage and Pedestal Structure

The pedestal (Fig. 2) is constructed of concrete of various mix designs. Above ground, the pedestal is faced with granite. At the time of its construction, the pedestal was the largest mass concrete structure ever built.

The anchorage system for the Statue was produced in the United States and consisted of two (2) levels of dunnage beams, the main beams at the top and the anchor beams located about 60 feet below. The pylon is anchored to the main beams. Transfer of the overturning moment from the pylon is made from the main beams to the anchorage beams through tie rods. The anchorage beams are embedded into the concrete walls of the pedestal structure at each of their ends, engaging sufficient concrete to stabilize the structure against overturning.

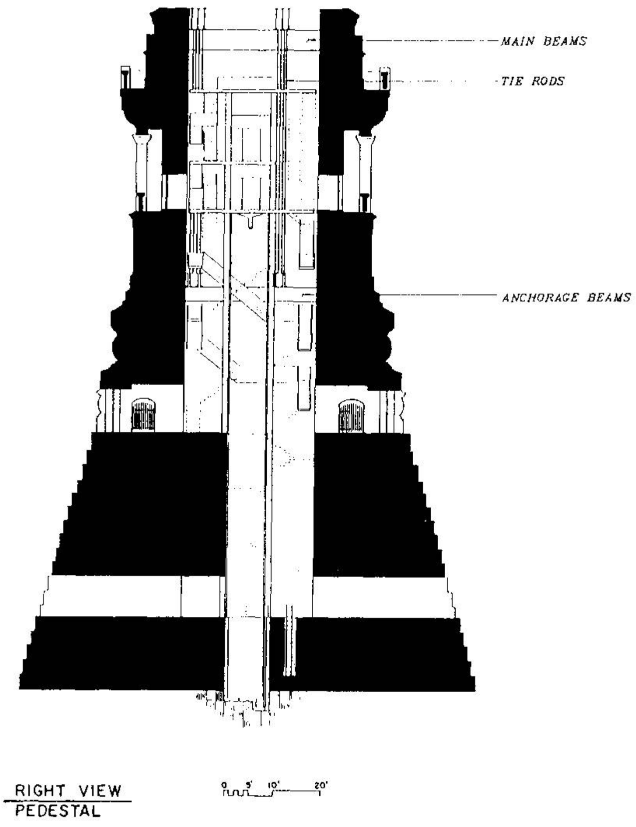

Fig. 2 Foundation, Pedestal and Anchorage

2. FIELD AND LABORATORY SURVEYS AND STUDIES

2.1. Surveys and Studies

The first phase of the restoration plan required a comprehensive survey and investigative program to identify the problems. This necessitated detailed field measurements to document the structure; field and laboratory studies including measurements of wind effects; measurements of stresses, displacements and frequencies and accelerations of the structural elements; and metallurgical studies to determine the properties and condition of the existing materials, including fatigue properties.

2.1.1. Field Tests

Some concern existed regarding the durability and fatigue exposure of the puddled-iron framework of the Statue, especially since it was noted that, by its nature, the material contained numerous inclusions, resembling cracks, which could act as "stress raisers." A program of inspecting the structure for fatigue problems was implemented. If fatigue is a problem in structures of this type, it initially manifests itself in cracking around rivet holes in areas of high stress. Therefore, rivets were removed in the high stress areas of the shoulder structure and the members were inspected visually with the aid of dye penetrants and magnetic particles. In addition, radiographic (X-Ray) inspections of select rivets, without removal of rivets, were made to determine if they were loose and to provide additional information on possible fatigue cracking around the holes.

2.1.2. Structural Instrumentation

To identify some of the hidden structural problems, the Statue was studied with the aid of strain gauge measurements[2] and accelerometers. The strain gauge measurements helped to identify two potential problems: weakness in the right shoulder and looseness of the anchorage of the Statue. The accelerometer measurements were taken to verify the results of the dynamic analyses in terms of the dynamic response of the Statue to wind, i.e., frequencies, accelerations and displacements. These measurements were correlated with wind measurements taken from an anemometer mounted on the torch platform.

2.1.3. Laboratory Studies - Material Properties

The puddled iron used in the framework of the Statue has a fibrous structure (highly directional) and contains many imperfections and slag inclusions. It was natural, therefore, that there was concern about the ability of the material to survive another 100 years, particularly from the viewpoint of fatigue. As a result, under the direction of Ammann & Whitney, an extensive program of testing was carried out by both Lehigh University and CETIM in Paris to determine the metallurgical and physical properties of the material. The tests undertaken by Lehigh University included tensile tests, Charpy V-notch tests (CVN), and fatigue crack propagation (FCP) tests on the angle and plate material removed from the Statue[3]. Tests undertaken by CETIM in France included fatigue tests in terms of stress range versus number of cycles to failure (SN).

3. STRUCTURAL ANALYSES [4]

3.1 General

Because of the historical importance of the Statue of Liberty, the preservation of evidence regarding its structural evolution was a high priority. A balance between structural adequacy and preservation needs, therefore, became an essential requirement of Liberty's rehabilitation. To meet this objective, the Statue was subjected to rigorous structural analyses including finite element analyses of its static and dynamic behavior, field measurements of its dynamical behavior, fatigue analyses of its remaining life, and the field and laboratory studies as previously described in Section 2.

3.2 Finite Element Model (Fig. 3)

The main pylon of the Statue, including the shoulder and the arm framing, was modelled as a three dimensional space truss. The geometric properties, including member sizes and makeup, were obtained largely from field reconnaissance surveys. Any stiffness contributed by the copper skin was conservatively omitted from the model; however, the influence of the "secondary" framework, which transfers the reactions from the skin to the main pylon, was included. The results of the dynamic analysis show that the two lowest modes, having frequencies of 1.66 hz and 1.90 hz, contribute over 95 percent to the response of the Statue under wind loads.

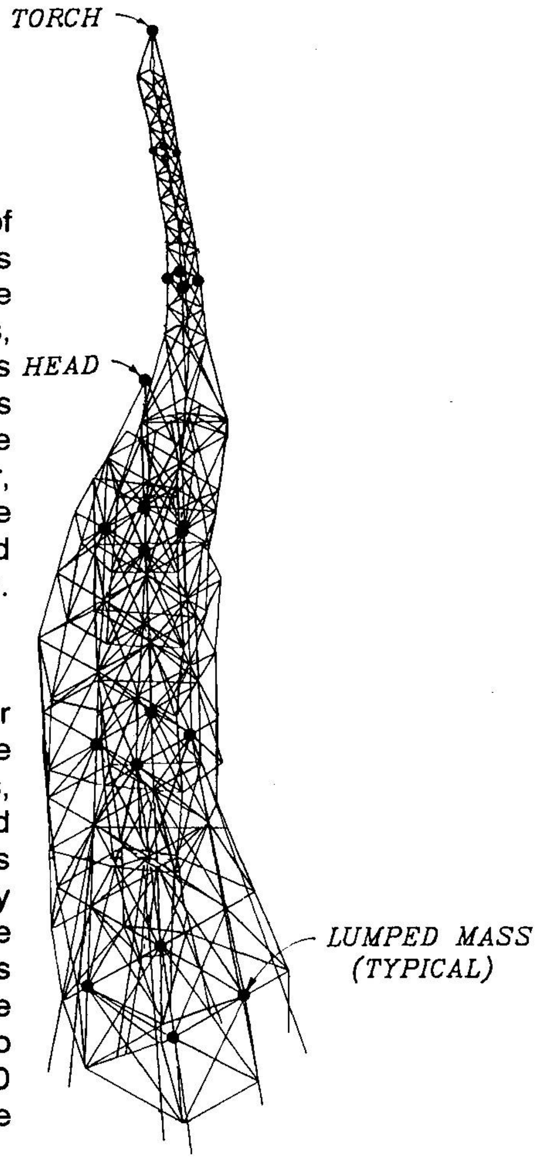

Fig. 3 Finite Element Model

3.3 Probabilistic Analysis

The significant environmental loads on the Statue are due to wind. Inasmuch as wind loads vary randomly in both time and space, it would have been inaccurate to apply conventional concepts in the analysis for wind loads. While such concepts usually result in safe and economical designs, the need for a realistic and reliable assessment of the fatigue life of the shoulder and main pylon prompted the application of probabilistic concepts in the calculations of the dynamic response of the Statue, and the attendant stresses, under the action of wind loading.

The maximum or extreme wind speed used for a survival analysis was based on a 100-year storm for the New York City Area. The turbulence characteristics of the wind, given in terms of the power spectrum of the longitudinal velocity fluctuations, was based upon observed data for exposures similar to those existing in the New York Harbor - mainly coastal areas.

Because the dynamic response of a structure is critically dependent upon its damping ratio, the damping in the statue was established by field measurement. The measured damping turned out to be favorably high, two percent of critical. This relatively high percentage of damping, compared to buildings and structures in general, is attributed to the unique manner in which the copper skin of the statue is attached to the main pylon. This attachment permits the skin to sway somewhat independently of the main pylon and thereby increases the overall damping through frictional hysteresis.

Aerodynamic damping, which is brought about by the wind induced forces relative to the vibration of a structure set into motion by the wind itself, amounted to about 0.5 percent of critical bringing the total damping ratio up to 2.5 percent.

The static response of the Statue to the mean wind loading component was obtained via standard static analysis methods using the finite element model. The additional dynamic response to wind gusts and vortex shedding was obtained from standard normal mode theory in conjunction with power spectral techniques. From this dynamic analysis, the ratio between the peak and mean response was found to equal 2.0. This results in an equivalent static wind speed of 128 miles per hour at the feet of the Statue and 141 miles per hour at the torch.

By comparison, Eiffel's loads equate to a static wind speed of 148.0 miles per hour, uniform with height. Although Eiffel's load is higher, the tributary area of the Statue in Eiffel's analysis is somewhat less than the as-built area, and the net effect is that the results computed by Eiffel for the main pylon are substantially similar to those produced by a 100-year storm used in the Ammann & Whitney analysis.

3.4 Fatigue Analysis

The analysis for fatigue effects consisted essentially of determining the number of cycles of different stress levels in various critical joints in the structure and combining these to determine the amount of fatigue life consumed in the past 100 years and projecting the life remaining when exposed to repeated cycles of wind. The distribution of the wind speeds used is based on Weather Bureau measurements for the New York City. In effect, then, the maximum stress levels were determined for a series of wind speeds each having a relative frequency of occurrence and duration.

It was found that winds in the range from about 30 to 50 miles per hour had the greatest influence on the fatigue life and not the greater stresses from higher wind velocities nor the more frequent stresses at lower wind velocities.

Apart from the laboratory tests for cyclic fatigue strength and resistance of the puddled iron to crack propagation, two factors in the fatigue analysis were of paramount importance in assessing the fatigue life of the Statue. The first was the stress concentration factor which is representative of the actual peak stress at the rivet holes. The second was the effect of the random exposure in determination of the equivalent number of cycles to failure for comparison with standard fatigue life curves based on traditional constant amplitude input. The stress concentration factor, based on traditional constant experimental data, was taken equal to 3.0.

Combining the exposure of the structure to the cyclic loadings with the results of the fatigue tests performed in the laboratory by CETIM and Lehigh University determined the projected life of the structure of the Statue. From a fatigue viewpoint, the structure has a projected life greater than 2,000 years.

4. STRUCTURAL REPAIRS

4.1 General

As determined from the investigations, studies and analyses, the problems that most necessitated the repairs and restoration of the structure of the Statue of Liberty were the weakness in the right shoulder and the head arches of the structure and galvanic corrosion of the armature.

4.2 Shoulder and Head Arches

The shoulder and head arch problems have existed since the Statue was constructed, a result of the misalignment of the head and shoulders from the originally designed positions, which created flexibility and overstresses in these areas. In the shoulder, the main deficiency was lack of torsional resistance which resulted in dangerous levels of axial stresses in critical members.

Several attempts to repair the weak shoulder were made in the past, most apparent of these is the unsuccessful attempt made by the U.S. Army in 1932. During the current project, two alternative solutions were proposed. The first alternative, termed the replacement scheme, consisted of removal of several members of the existing deficient shoulder and reconstructing the shoulder with a new truss, to achieve structural continuity to the main pylon. The second alternative, the repair scheme, adds several members to the existing structure to achieve the required rigidity and factor of safety. The history of the problem and proposed repairs of the shoulder are illustrated in Fig. 4.

Because the existing structure had historical significance, being as initially erected in Paris, it was decided to implement the repair scheme.

The repair of the head arches consisted of the addition of structural members to eliminate the eccentricities and restore the structural continuity as originally intended.

4.3 Armature

The problem of galvanic action between copper and iron was understood at the time of the Statue's original construction. An attempt was made to insulate the copper from the iron framework using asbestos cloth soaked in shellac. Although the insulation was a novel solution and worked for a period, with time the material deteriorated, became saturated with moisture and salts due to condensation and leakage in the marine environment of Liberty Island and, in the presence of this electrolyte, galvanic action took place.

Extensive studies were conducted to determine the most suitable material to replace the iron armature of the Statue. This meant determining a material that not only would be compatible electrically with the copper skin but would also have the same mass and

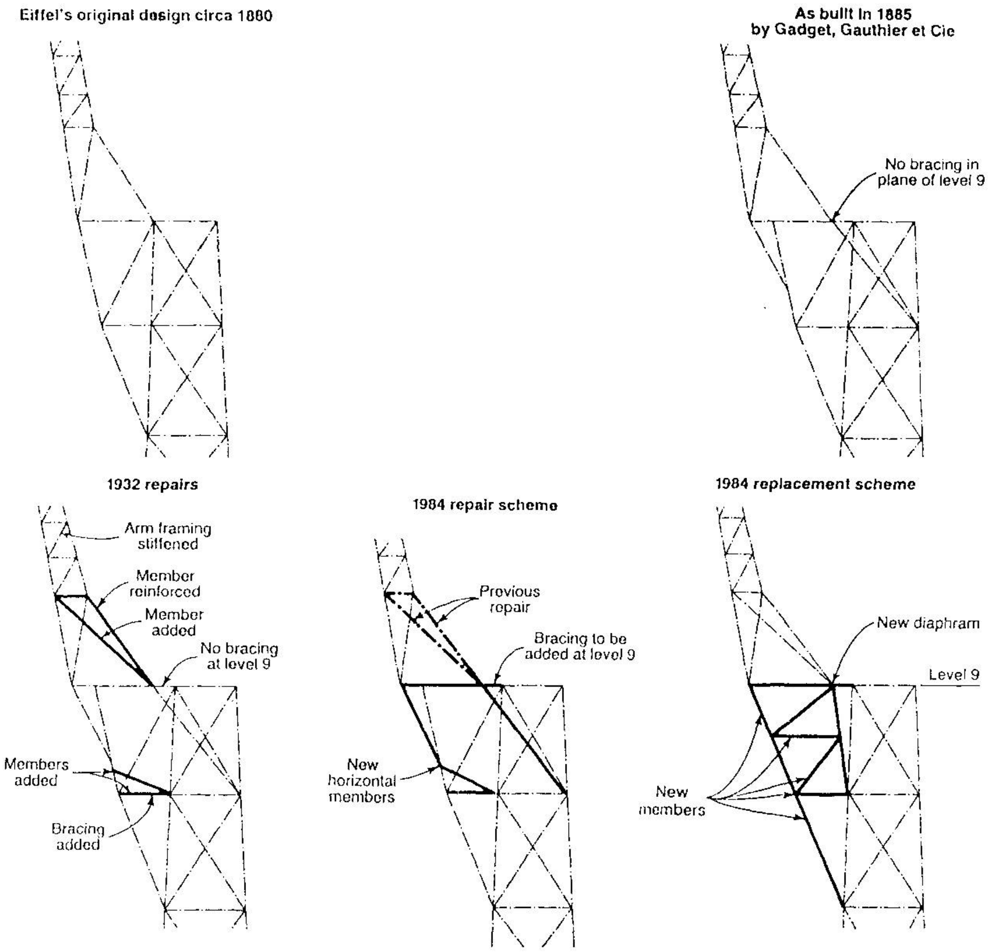

Fig. 4 History and Proposed Repairs of the Right Shoulder

stiffness as the iron armature it was to replace. The latter objective was intended to insure that the static and dynamic response would not be altered.

To achieve these goals Ammann & Whitney recommended a ductile ferrous material either protected or inert. The National Park Service, in consultation with metallurgists, conducted extensive electrolysis tests and weathering tests on the several alternative materials including mild steel, copper alloys and various types of stainless steel.

Type 316L (UNS S31603) stainless steel became the material of choice for the armature replacement, since it had minimal galvanic reaction with the copper and duplicated the stiffness characteristics without a change in the dimensions or weight of the armature.

The material is difficult to fabricate without changing its mechanical properties and its corrosion resistant characteristics. This necessitated the development of special fabrication procedures, including forming, annealing and passivating the material[5].

As an added measure of protection, skived Teflon® was used between copper elements and the stainless steel armature bars. The Teflon® tape also acts as a lubricant to insure free movement of the copper skin and the structural support system.

5. SCAFFOLD [5]

To gain complete access to the entire structure, interior and exterior scaffolding was designed and erected by Universal Builders Supply, Inc., under the structural supervision of Ammann & Whitney. Because the Statue is unique and irreplaceable, the exterior scaffold was designed for a 100-year wind-recurrence velocity. In addition, a minimum distance of 1.5 feet was required between the scaffold and the copper skin of the Statue to allow for relative movement in severe wind storms. The resulting aluminum scaffold was an innovative, efficient and spectacular structure. The scaffolding and rigging greatly facilitated the execution of the construction operations and was a major factor in insuring the timely completion of the project.

The 81 feet square by 240 feet high structure became the tallest free-standing scaffold ever constructed.

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Strengthening of a 400 Year Old Ottoman Minaret

Renforcement d'un minaret ottoman de 400 ans

Verstärkung eines 400 Jahre alten ottomanischen Minaretts

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SUMMARY

The Minaret of Rotunda in Thessaloniki was built in 1590. The structure has a total height of 36m and is made of stone and brick masonry. The monument has been damaged by several strong earthquakes. During the earthquake of June 1978 the damage increased dangerously. This paper presents the procedure followed for the assessment of the bearing capacity of the structure, including in situ measurements, laboratory tests and geotechnical-seismological studies of the area. Finally, alternative strengthening solutions are analyzed and evaluated.

RÉSUMÉ

Le Minaret de la Rotonde à Thessalonique a été construit en 1590. La structure d'une hauteur de 36m est construite en maçonnerie de pierres et de briques. Le monument a subi des dégâts à cause de séismes puissants. Pendant le séisme de juin 1978, les dégâts ont augmentés dangereusement. Cet article présente la méthode d'estimation de la capacité portante de la structure et contient des informations sur la structure et l'état du monument, des mesures sur place, des essais en laboratoire et des essais géotechniques et sismiques sur place. Enfin, des solutions alternatives de renforcement de la structure sont analysées et évaluées.

ZUSAMMENFASSUNG

Das Minarett von Rotunda in Thessaloniki wurde 1590 gebaut. Es ist 36m hoch und besteht aus Stein- und Ziegelmauerwerk. Mehrere Erdbeben, besonders aber das vom Juni 1978 haben dem Monument bedrohlichen Schaden zugefügt. Der Bericht schildert die Methode, nach welcher die Tragfähigkeit des Minaretts durch Messungen am Objekt, Laborversuche und geotechnisch-seismologische Untersuchungen bestimmt wurde. Alternative Sanierungsmassnahmen werden durchdacht und bewertet.

1. INTRODUCTION - HISTORICAL BACKGROUND

The Minaret of Rotunda of Thessaloniki was built in 1590 in the courtyard of the Roman Rotunda (300 A.D.), when the building was converted from Christian church to Muslim mosque during the long period of Ottoman occupation of the city. The monument, erected in an area of relatively high seismicity, has been damaged due to several strong earthquake excitations as well as to weathering effects. During the earthquake of June 20, 1978 the damages were intensified. The 9th Ephorate of Byzantine Antiquities, officially in charge of the monument, carried out the necessary emergency works and entrusted the investigation of the structural condition and the design of repair and strengthening interventions of the monument to the Lab. of R/C Structures, Univ.of Thessaloniki, Greece. In the following chapters the phases of the research project are presented in short.

2. IN SITU AND LABORATORY RESEARCH AND MEASUREMENTS

The 9th Ephorate provided to the research team a complete series of architectural drawings, based on detailed surveying of the monument, the results of 64 hammer tests of bricks and mortar and a preliminary report [1]. In order to enrich the available data, a research project including constructional pathology surveys, in situ and laboratory measurements and tests, in situ measurements of the fundamental period of the structure and geotechnical and seismological studies of the territory was carried out.

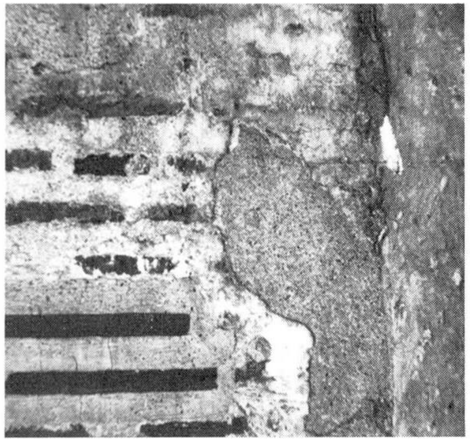
2.1 Geometry of the Structure

The Minaret is located at a distance of 3.50m western of Rotunda (see Fig.1a). It consists of a 4.50x4.50m square cross-sectioned base, 7.50m in height and is founded on a thick stiff red clay layer approximately 6.00m below the ground level. The base is supporting the trunk of the Minaret consisting of two successive twenty-sided trunkated conical parts of 22.00m total height crowned by the muezzin's balcony and the highest fifteen-sided cylindrical part 6.00m in height (see Fig.2). The external diameter of the trunk decreases gradually from 3.25m to 2.10m and finally to 1.75m at the highest cylindrical part. The narrow entrance (0.85x1.95m) is located at the northern face of the base. The internal helical staircase with gradually decreasing diameter (D=1.80-1.45m) leads up to the balcony at a height of 29.50m. The small muezzin's doorway (0.50x1.50m) is located at the southern face of the cylindrical trunk penetrated till the top by the narrow staircase (D=1.35m). Five narrow loopholes are located along the height of the southern side of the trunk.

The base of the Minaret is consisted of successive stone and brick masonry layers while the masonry of the trunk is pure brickwork built by flattened bricks (30x40x4cm) with thick mortar joints (4cm),(see Fig.1b). The balcony consists also of brickwork supported by successive horizontal wooden beams tangentially projecting from the cylindrical trunk. The staircase is masoned together with the surrounding trunk. At the edge of every step a wooden beam is located diametrically and is embedded in the masonry of the trunk, while along the axis of the staircase a masonry column (D=0.25m) is constructed (see Fig.1c).

2.2 Pathology - Emergency interventions

As it can be easily seen in Fig.2, the base of the Minaret is unplastered. The plaster covering has been collapsed at the major part of the trunk, while at the rest of the surface, as well as at the upper cylindrical trunk, it is heavily cracked. Limited superficial deterioration of masonry, especially at the northern face of the trunk has been detected and confirmed by the hammer tests. The wooden beams at the highest steps of the staircase have been heavily corroded due to rain water passing through the open top, as the wooden acute conical roof of the Minaret fall down, probably during an earthquake, at the begining of the



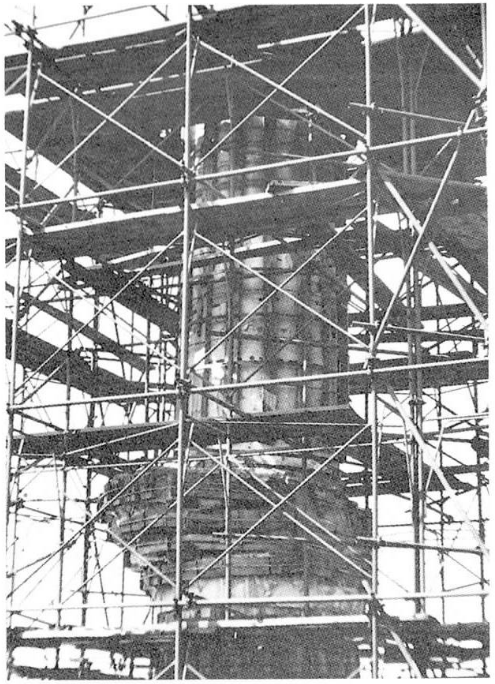


Fig.1

(a)Rotunda-Minaret: Southern view.(b)Masonry close-up: Merely visible horizontal crack.(c)Internal staircase. (d)Balcony and upper cylindrical part

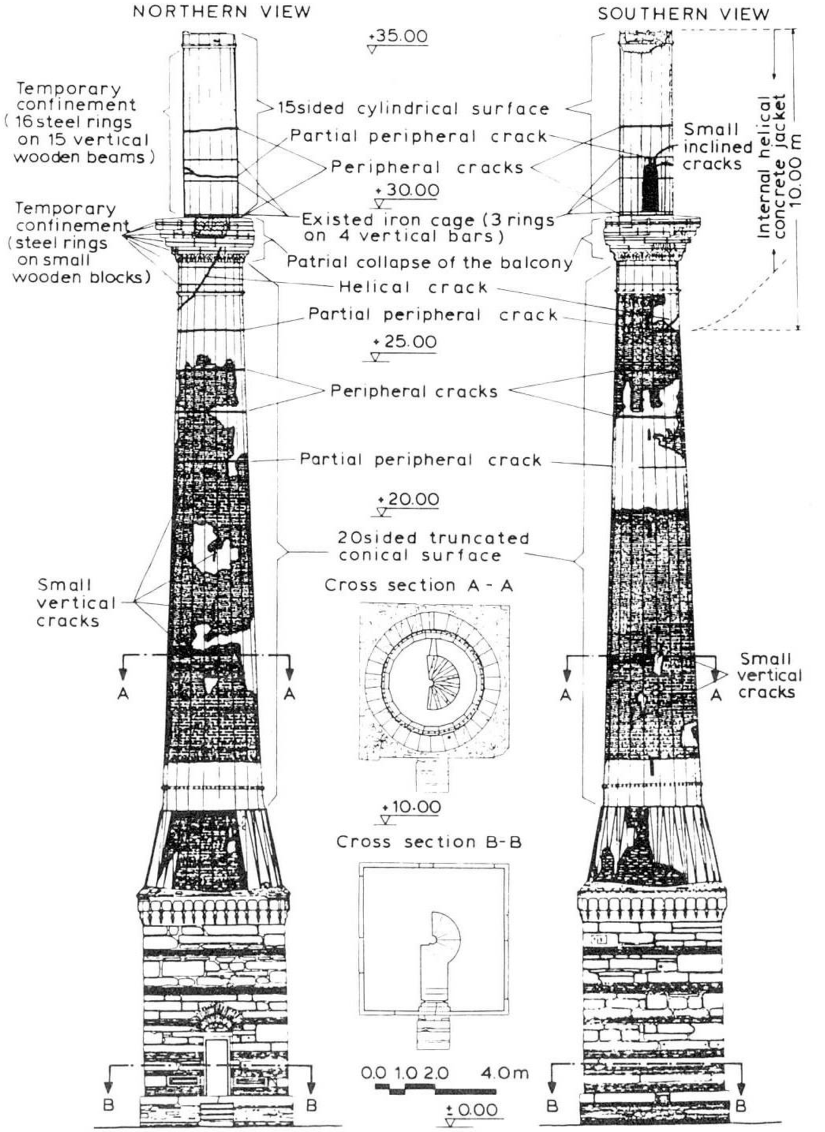


Fig.2 Northern and Southern elevations of the Minaret. Cracking pattern, external emergency interventions

present century. Successive horizontal peripheral cracks have been ascertained at the upper two fifths of the Minaret and have been drawn in Fig.2. There is one helical crack starting from the highest loophole, following the ascenting steps and cutting the balcony. All the cracks, being under vertical compression, are just visible (see Fig.1b). During the earthquake of June 20,1978 (epicenter 30km from Thessaloniki, magnitude 6.5 grades in Richter scale) a part of the balcony collapsed and the upper half of the cylindrical trunk was slightly translocated (see Fig.1d,2). It must be pointed out that, despite of the damages, no permanent horizontal drift of the structure has been ascertained and no foundation problem has been detected.

Emergency works were undertaken just after the 1978 earthquake including general external scaffolding (see Fig.1a) and temporary external confinement by steel rings and wooden beams as well as internal helical concrete jacket at the upper 10m of the Minaret (see Fig.1d,2).

2.3 Mechanical characteristics of bricks, mortar amd masonry

Non destructive tests in combination with a limited number of laboratory destructive tests were applied to make an estimation of mechanical characteristics of bricks, mortar and masonry. To this direction a new series of hammer and ultrasonic tests on bricks and mortar, beyond those performed by the 9th Ephorate, was carried out. Three masonry "triplets" were extracted from the base and the trunk of the Minaret (see Fig. 1b) and were used for the laboratory tests.The results are shown in Table 1.

Material	Compressive strength	Flex.tensile strength	Dynamic Elast.Modulus
Bricks	f_{bc} = 17.56MPa	f_{bt} = 3.52MPa(1)	E_{bd} = 12300MPa(3)
Mortar	f_{mc} = 1.28MPa	f_{mt} = 0.36MPa(1)	E_{md} = 1150MPa(3)
Masonry(5)	f_{wc} = 2.61MPa	f_{wt} = 0.11MPa(2)	E_{wd} = 2600MPa(4)

(1) Wide scattering, (2)Direct unstucking strength of bed joints, (3)Estimated from the ultrasonic tests, (4)Calculated by using a specific formula, (5)Pure shear strength of joints : f_{wso}= 0.19MPa

Table 1 Mechanical characteristics of bricks, mortar and masonry

Significant difference in the composition of mortars between the highest cylindrical part and the rest of the Minaret below the balcony has been found out, although the compressive strengths of the mortars were almost equal. Consequently, either the Minaret had been erected in two phases, or the part over the balcony had probably been collapsed and rebuilt. In Fig.3 the idealized masonry stress-strain curve adopted for the analyses and the stress-strain curves of concrete and steel for the strengthening interventions are shown.

2.4 Fundamental Period of the Minaret

To determine the in situ fundamental period of the Minaret, the ambient vibration method was used, based on the local traffic effect, with the aid of a sensitive portable system. Two measurements have been carried out, the first one before the emergency interventions (1979) and the second twelve years later (1991). The fundamental periods were found equal to 0.80 and 0.97 sec respectively. Under the reasonable assumption that the metal scaffolds, loosened by the time, have no influence on the response of the structure, the increase of the fundamental period must be attributed to the added mass of the internal concrete jacket on the top of the Minaret.

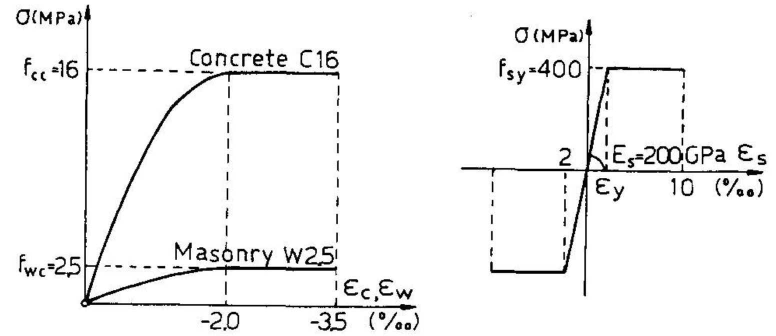


Fig.3 Idealized stress-strain curves for masonry and intervention materials (Concrete, Steel)

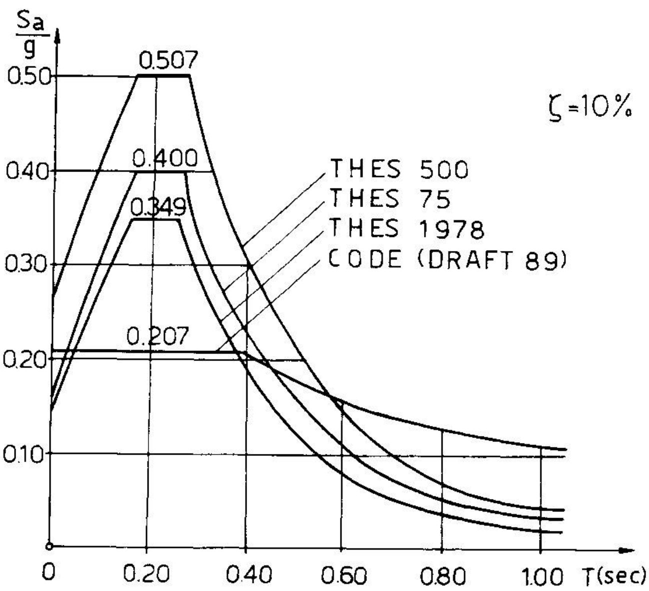


Fig.4 Design seismic accelelation spectrums

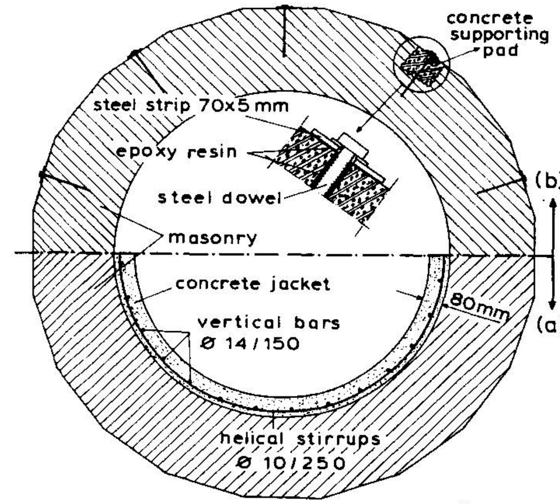


Fig.6 Semisections of Minaret trunk: (a)Internal concrete jacket, (b)External steel strips

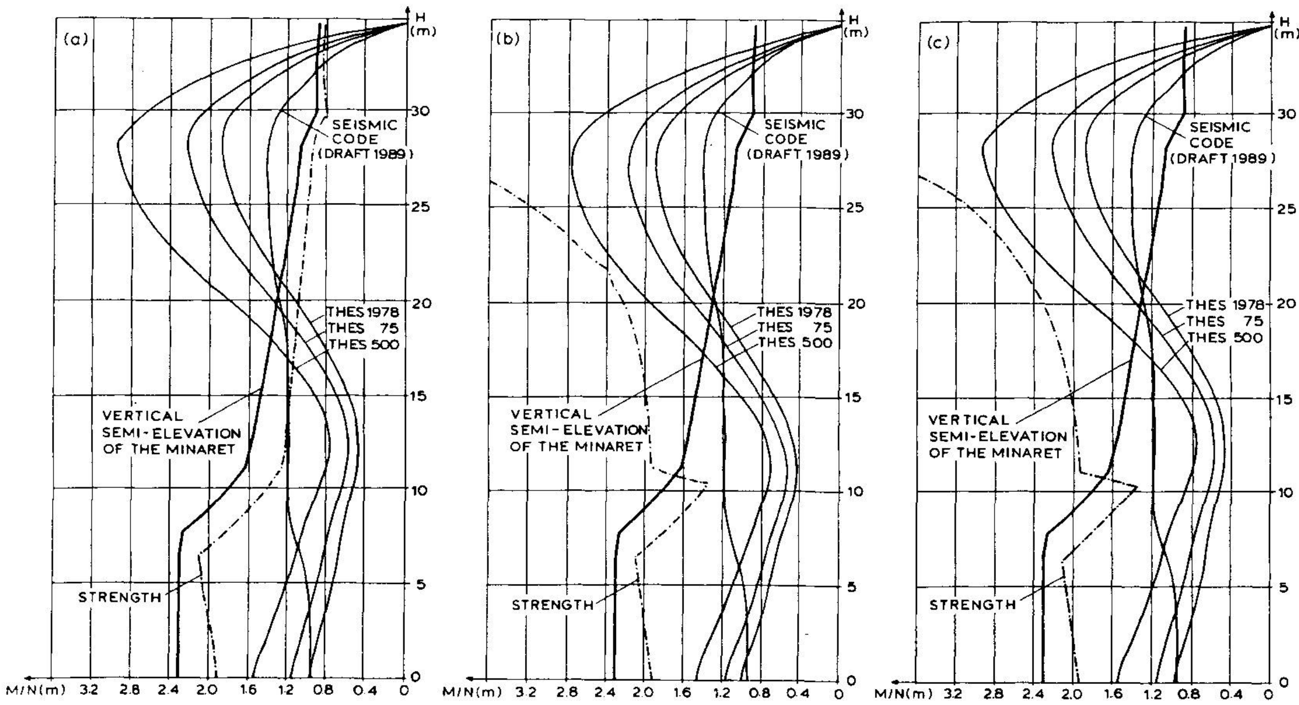


Fig.5 Eccentricity curves under seismic spectrums and strength eccentricity curve:(a)Virginal state,(b)Concrete jacket,(c)External steel strips

2.5 Geotechnical and Seismological study of the territory

To determine the soil shear modulus (G) throughout the foundation depth, two bore holes were drilled very close to the Minaret and the Cross Hole technic was carried out. Resonant Column Tests on foundation red clay samples were also performed. Based on the obtained values of G, the deformational characteristics of appropriate soil springs, necessary for a reliable analytical model, were estimated. The analytical procedure was performed using four elastic acceleration response spectrums. Three of them were based on the seismic excitation of 1978 at the region of Thessaloniki. The motion at bedrock of this earthquake was multiplied by the factors 1.000, 1.267, 1.756 and then, using the SHAKE program [2], the ground motions at the site of Minaret were determined. The first motion (THES 1978, α_{max}=0.172g) corresponds to the earthquake of 1978 since the other two correspond to similar earthquakes for return periods of 75 (THES 75, α_{max}= =0.218g) and 500 years (THES 500, α_{max}=0.302). For the estimation of the latter maximum ground accelerations a statistical evaluation of the strong ground motions at the territory from 1500 till 1990 was performed. The adoption of such a long return period (500 years) was necessary because of the monumental character of the structure and the need of a long-life intervention. The fourth response spectrum corresponds to the new Greek Seismic Code (draft 1989). These response spectrums, after being appropriately smoothed and normalised, were multiplied by the foundation factor θ=0.8 and were devided by the behaviour factor q=1.5 to take design spectrums (see Fig.4).

3. BEARING CAPACITY AT VIRGINAL STATE. ALTERNATIVE STRENTHENING PROPOSALS

3.1 Analytical Model and Verification

The analytical model at virginal state was a vertical cantilever, consisted of 33 overground and 6 underground beam elements. The mass was assumed to be concentrated at the 6-DOF nodes of the model. The stiffness of the foundation was represented by horizontal, vertical nad rotational linear elastic springs. To simulate the mass and stiffness imposed to the structure due to concrete jacket intervention, additional mass at appropriate nodes and new beam elements connected to the same nodes were taken into consideration.

The type of analysis adopted was a linear dynamic spectral analysis, as it is recommended for tall slender type structures [3]. Throughout the analytical procedure, the SAP80/86 Structural Analysis Programs were used [4]. From pilot analyses it was proved that, for the adopted values of E_{wd} and stiffness of the soil springs corresponding to 0.8 G_{max}, the calculated fundamental periods were in very good agreement with those measured in situ.

3.2 Bearing Capacity at Virginal State

Since the shape of the structure practically does not permit the application of significant live loads, the only considerable vertical load is the self-weight of the structure. On the other hand, the Minaret is all around well protected against strong winds. Consequently, the only load combination to be considered is the seismic action and the dead load.

The structure at its virginal state was analyzed under the four design spectrums of Fig.4. The corresponding results are presented in Fig.5a by means of eccentricities of the dead load axial forces due to seismic bending moments (e=M/N). It must be pointed out that these diagrams are more informative than the conventional bending moments and axial forces diagrams since they include their combined action on the structure. From Fig.5a it is obvious that over the height of about 17m, where the eccentricity curves surround the semi-elevation of the structure, the masonry, having an almost zero tensile strength, is unadequate to carry the pair of (N,M) since the axial forces act outside of the cross section.

In order to determine more precisely the region of the structure which needs to be strengthened, the strength curve shown in Fig.5a was determined with the aid of a specific computer program which can calculate the ultimate bending moment, under a given axial force, of a polygonal section composed of various materials (masonry, concrete, steel) and given stress-strain relations of the materials (see Fig.3). It can be seen that under the height of 14m, the structure is adequate to carry out the combination of dead and seismic loads. In order to secure a reasonable safety factor, slightly greater than one, it was decided that the region to be strengthened extends from 12m height to the top of the structure.

It must be pointed out that these analytically obtained findings, concerning the region of inadequasy, match very well with the pathology of the structure (cracking pattern, probable collapse and reconstruction of the upper part).

3.3 Alternative Strengthening Proposals

The following alternative strengthening intervention proposals have been considered and evaluated:
- Internal vertical prestressing tendors
- Internal 3D steel truss
- External inclined lightly prestressed cables
- Internal concrete jacket
- External vertical steel strips nailed to the masonry.

The presence of the internal staircase arises difficulties in applicating anyone of the internal interventions, the 3D steel truss being the most inconvenient since a lot wide drillings are needed. The vertical prestression was proved to be impracticable since the prestressing force needed to adequately decrease the seismic bending moment was too high for the low-strength masonry, especially at the top of the structure. Concerning the application of external inclined lightly prestressed cables, it must be pointed out that there is not enough space around the structure to give to the cables the appropriate inclination. On the other hand, the aesthetics of the monument would be rather inacceptable.

The existance of the internal concrete jacket at the top of the Minaret, which is very difficult to be removed, is a positive argument for the proposal of concrete jacketing to be chosen. Anyhow, it must be pointed out that the vertical reinforcing bars of the jacket have to penetrate the staircase helix to secure the function of the jacket as a tube. Moreover, to secure the unified action of the composite section of masonry and concrete, the bond on their interface must be increased by deepening the mortar joints. From the above mentioned, it is clear that concrete jacketing is an almost irreversible intervention.

The external reinforcing by means of steel strips nailed to the masonry, although it is not a conventional strengthening method, especially for relatively low strength materials, it is a method of rather low inconvenience and almost fully reversible.

After these prelimirary considerations and the rejection of the fisrt three strengthening proposals, detailed analytical investigation of the latter two proposals was performed and is presented in the following. As it was mentioned before, the region to be strengthened extends from 12m height to the top.

3.3.1 Internal concrete jacket

The helical concrete jacket, constructed after the 1978 earthquake at the upper 10m of the structure, had a thickness of 80mm and a longitudinal reinforcement Φ14/150, S400. It was decided that the already constructed jacket must be extended, having the same geometrical, mechanical and constructional characteristics (see Fig.6a), by welding the existing longitudinal reinforcements to the new ones. Using the modified analytical model mentioned in chapter 3.1, it was proved that the jacketing had a limited influence on the dynamic characteristics of the structure. In Fig.5b, the eccentricities of the axial forces due to seismic bending moments under the design spectrums of Fig.4 are shown. It is obvious that, in comparison to the relative curves of Fig.5a, the differences are small. The calculation of the ultimate bending moment of the cross sections under the acting axial force was based on the specific computer program mentioned in chapter 3.2. From the strength curve, shown in Fig.5b, it can be concluded that the strengthening proposal is adequate.

A calculation of the shear stress acting on the interface between masonry and concrete gives a maximum value of 0.06MPa, which can be easily sustained by the bond strength of the masonry to concrete interface (see also footnote (5) of Table 1), especially after deepening of the mortar joints before concreting. To secure the durability of the intervention, the use of austenitic stainless steel for the reinforcing bars is recommended.

3.3.2 External steel strips nailed to the masonry

After some pilot calculations, the use of ten, grade S400, austenitic stainless steel strips having an effective cross section of 50x5mm was adopted (see Fig.6b). Because of the relatively small increase in stiffness and the practically zero increase in mass due to steel strips, the response of the structure, under seismic loads was assumed to be equal to the one of the virginal state. Using the same computer program, the strength curve shown in Fig.5c was calculated and it can be seen that it is almost identical with that of Fig.5b, which means that the selected strengthening proposals are equivalent in strength.

The maximum shear force per unit length, which has to be transferred from each strip to the masonry, was found to be equal to 29KN/m. It is obvious that the greater the number of the dowels the easier the transfer of the shear force but, on the other hand, the greater the number of intervention points. Finally it was decided to nail the dowels every 0.50m along the strips. To prevent local failure of masonry under the dowel force, small concrete supporting pads will be embedded in the masonry. On the other hand, to avoid buckling under compression, the strips will be stuck on the masonry surface using epoxy resin.

It must be pointed out that the strengthening proposals mentioned above stand under the approval of the appropriate authorities (9th Ephorate of Byzantine Antiquities and Supreme Archaeological Council, Ministry of Culture).

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Stabilité du Choeur de la cathédrale de Quimper

Stabilisierung des Chors der Kathedrale von Quimper

Stabilization of the Choir of the Quimper Cathedral

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B. Mouton, né en 1948, obtient son diplôme d'Architecte DPLG à l'École Supérieure des Beaux-Arts en 1972. Architecte en Chef des Monuments Historiques depuis 1980, il est actuellement en charge de la restauration de la Cathédrale de Quimper pour le compte du Ministère de la Culture.

RÉSUMÉ

Le choeur de la cathédrale de Quimper, reconstruit en style gothique dès 1230, révéla peu à peu une erreur de conception dans les culées externes des arcs boutants qui n'assurent pas le contrebutement nécessaire. La solution adoptée consiste à employer la post contrainte à l'aide de tirants en acier inoxydable, introduits par forages dans les maçonneries des culées et des arcs boutants, afin de rétablir l'équilibre par l'introduction des pressions nécessaires.

ZUSAMMENFASSUNG

Der Chor der Kathedrale von Quimper, im gotischen Stil ab 1230 wiederaufgebaut, zeigte nach und nach einen Konzeptionsfehler in den äusseren Widerlagern der Bogenendstücke, die nicht das notwendige Gegengewicht sicherstellten. Die angewandte Lösung besteht in der Verwendung der Vorspannung. Mittels Bohrung wurden rostfreie Ankerbolzen in die gemauerten Widerlager und Bogenendstücke eingelassen, so dass unter Anwendung der notwendigen Spannung das Gleichgewicht wiederhergestellt werden konnte.

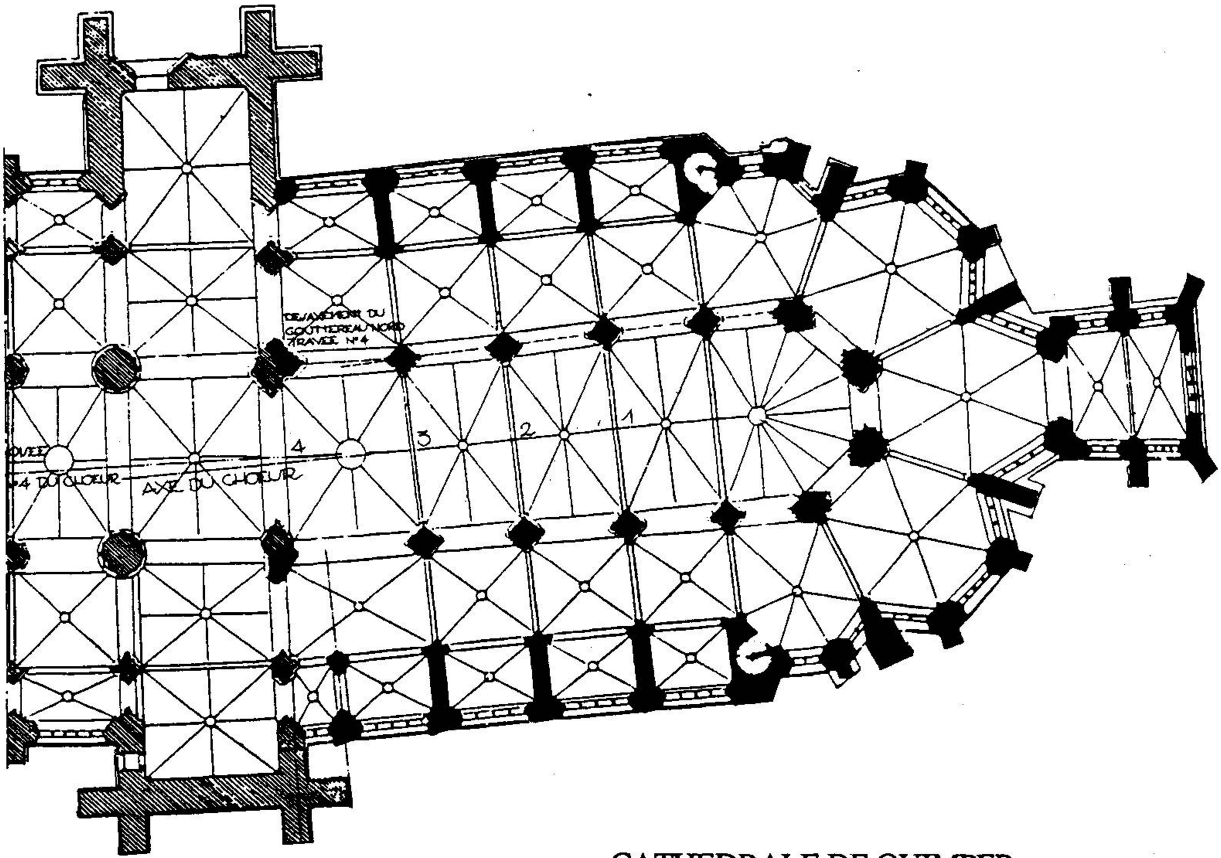
SUMMARY

The choir of the Quimper Cathedral, rebuilt in gothic style from 1230 onwards, revealed little by litte a conceptional error with regard to the external abutment of the flying buttress which did not provide the necessary counter weight The solution consisted in applying a post tensioning method. Stainless steel tension springs have been inserted by drilling into the masonry abutment and flying buttress, thus the balance could be regained by applying the necessary tension.

1. HISTORIQUE

Le chœur de la Cathédrale Saint Corentin fut reconstruit en style gothique à partir de 1230. Les travaux durèrent jusqu'en 1335, date à laquelle le culte put à nouveau y être excercé.
L'ensemble était alors clos (charpente-couverture) et les arcs boutants très probablement déjà en place. Les voûtes ne furent construites que de 1408 à 1415 suivant le système de croisées d'ogives quadripartites recouvertes d'un badigeon polychrome en 1417.
Peu de temps après, dès 1424, la nef fut reconstruite suivant un parti délibérement homogène avec le chœur, se poursuivant ensuite par le transept dont le bras Nord achevé en 1486, consacra l'aboutissement. C'est dans son état que l'édifice, traversa ensuite les siècles.

En 1862, la restauration de l'édifice fut entreprise. L'architecte diocésain Joseph BIGOT précisait en particulier que les tirants métalliques ancrés dans les murs gouttereaux du chœur et fixés aux entraits de charpente, étaient "pourris" et qu'il était nécessaire de les remplacer. Il s'agissait d'un renfort de stabilité mis en place au XVIIIe siècle (voire même avant), et dont la présence révèle que la stabilité du chœur causait depuis longtemps de très anciennes inquiétudes.
A la même époque, les passages exitants aux pied des culées externes furent murés au Nord dans un but de consolidation.



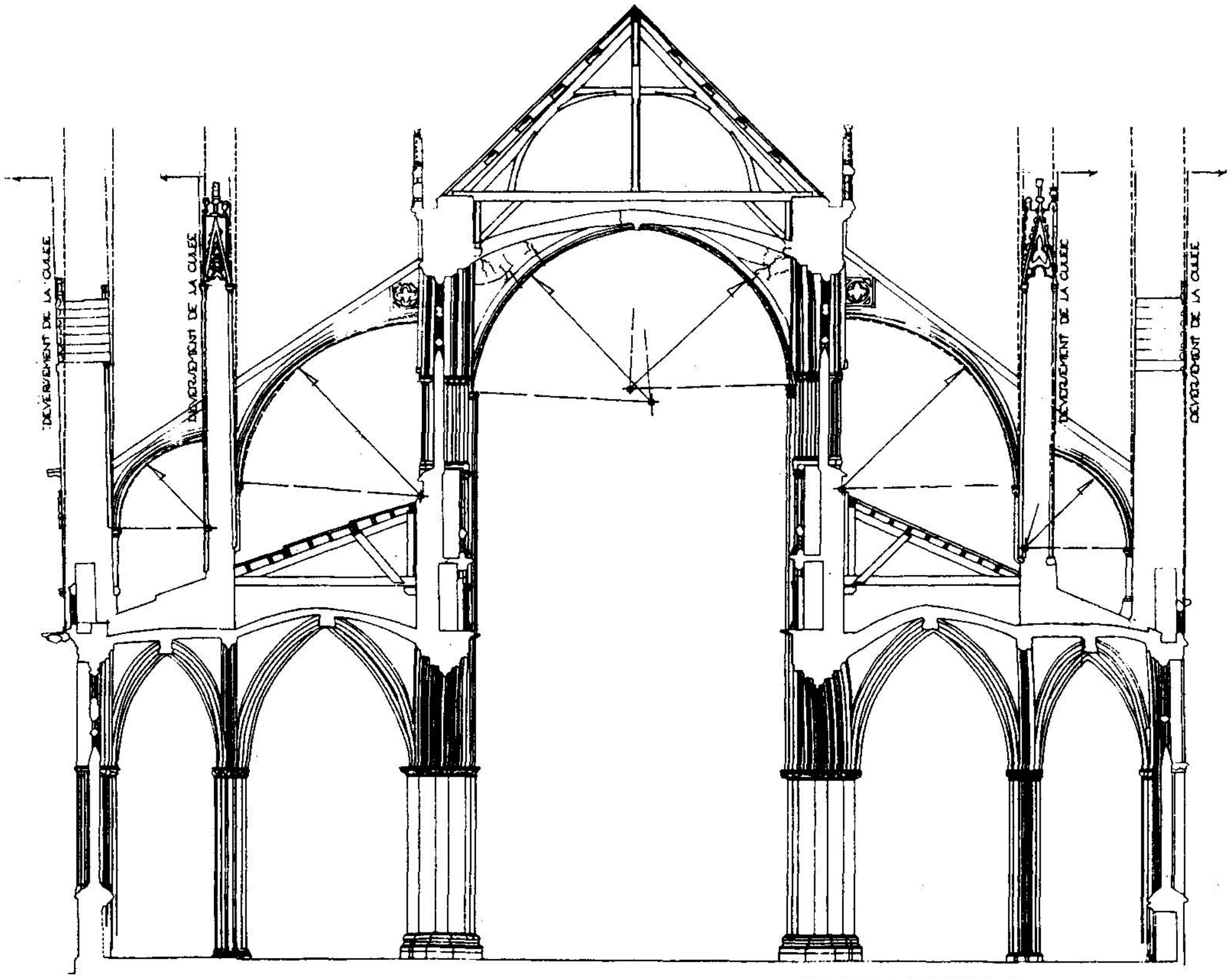
CATHEDRALE DE QUIMPER
PLAN DU CHŒUR
D'APRES CHAUSSEPIED, CONGRES ARCHEOLOGIQUE 1914

2. DESORDRES

2.1. Surveillance

En 1982, devant les nombreuses fissures qui étaient apparues sur l'enduit du XIXe siècle, une campagne de surveillance fut entreprise ; des témoins de plâtre placés sur l'extrados des voûtes, au droit des fissures se brisèrent en quelques semaines, révélant un mouvement actif.
Les relevés stéréophotogrammétriques entrepris peu après (1985) montrèrent des déformations, qui pouvaient être interprétées de deux façons :
- déformations des maçonneries au moment du décintrage des arcs et de la prise des mortiers. Ce serait le cas très probable des voûtes du haut chœur par exemple, dont les déformations ne s'expliquent par aucun mouvement spécifique ;
- déformations dues à des affaissements et des défaillances : arcs déprimés, appuis déversés : c'est le cas en particulier des arcs boutants et des culées qui les épaulent. Ces désordres peuvent être actifs.

En particulier, on a pu noter un important déversement de la culée externe des arcs boutants, alors que le mur gouttereau des chapelles situé en dessous, avait gardé sa verticalité.

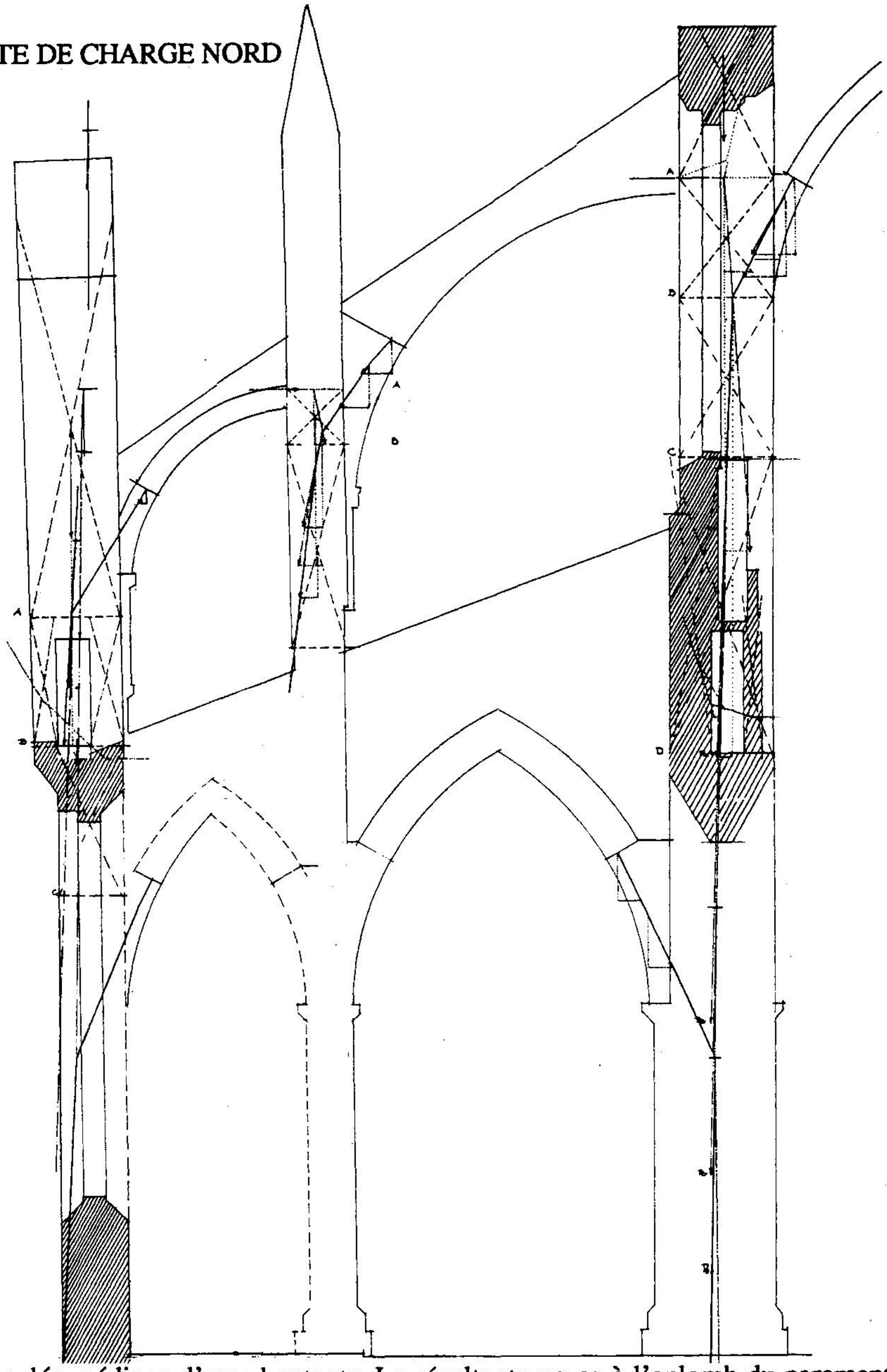


ETAT DES DEFORMATIONS

2.2. Analyses

Les études de stabilité commencèrent d'abord par une reconnaissance du sous-sol et mirent que le sol est suffisant pour soutenir l'ouvrage.
L'analyse du complexe voûtes à croisées d'ogives + arcs boutants, fut entreprise avec une méthode dérivée de la statique graphique (dite méthode de Merry), et révéla les résultats suivants :
- Stabilité excellente des voûtes du chœur contrebutées par les arcs boutants. La résultante passe par le tiers central en pied de pile.

SYNTHESE DESCENTE DE CHARGE NORD



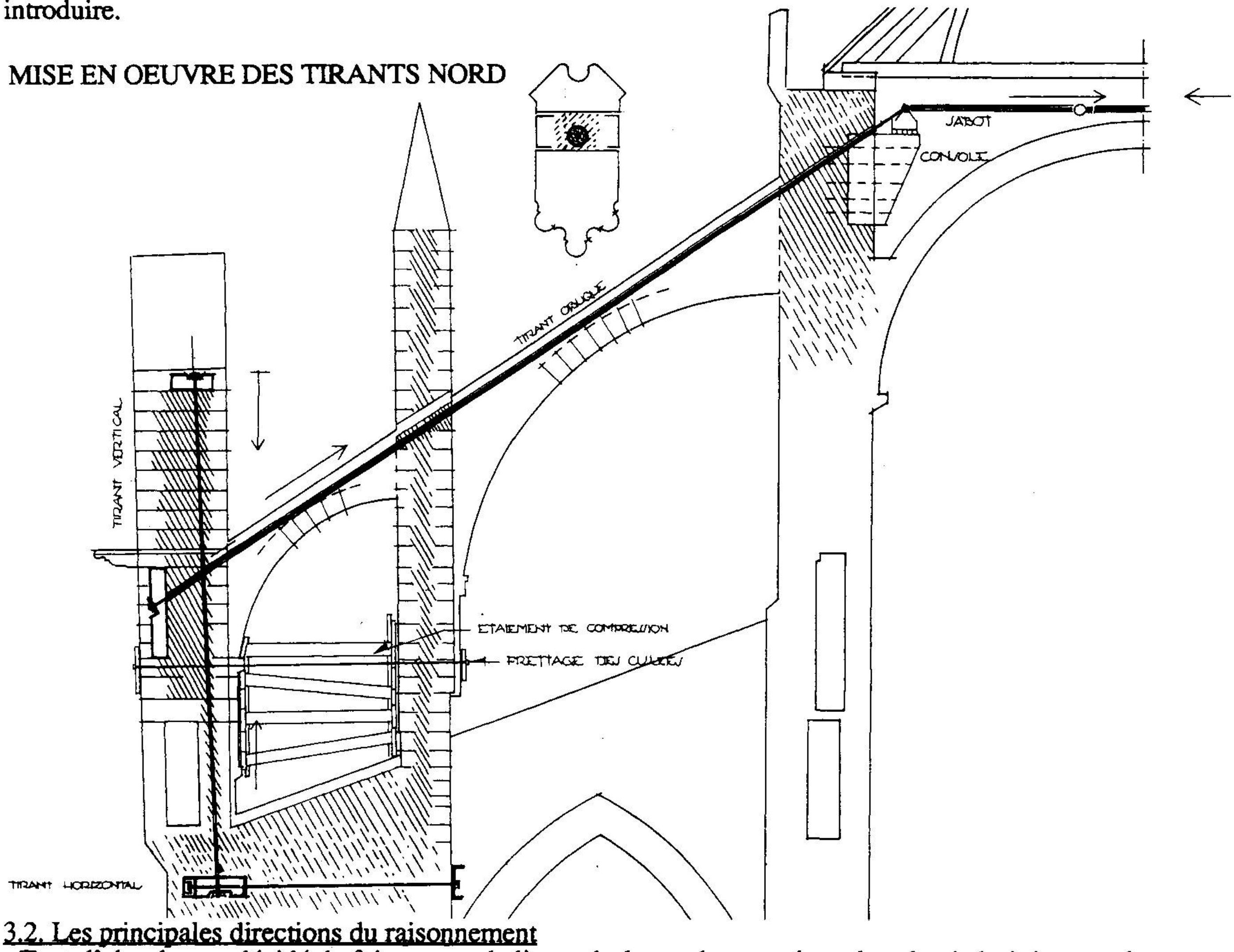
- Stabilité limite de la culée médiane d'arcs boutants. La résultante passe à l'aplomb du parement externe, au pied de la culée. Mais le mur diaphragme de séparation des chapelles qui monte jusqu'au comble de bas côté, peut assurer un épaulement satisfaisant de la culée.
- Stabilité défaillante de la culée externe, avec sortie de résultante au niveau du chéneau de toiture du bas côté, ce qui explique l'articulation observée dans les alignements des aplombs extérieurs, (révélée par la stéréophotogrammétrie). Ce phénomène est accentué par le fait que, contrairement à la manière couramment adoptée dans l'architecture gothique, (ce qui fut le cas à Quimper dans la nef), les culées externes du chœur ne sont pas prolongées vers le bas par un contrefort, mais s'appuient sur une corniche qui est en porte à faux et aggrave l'effort au déversement. Par effet en chaîne, des désordres sont apparus naturellement sur les arcs boutants et la voûte du chœur. Mais les valeurs quantitatives sont de faible importance, ce qui a permis à l'ouvrage d'absorber ces excédents.

3. PROJET

3.1 Recherche du parti de consolidation
Les recherches s'étaient fixées deux objectifs :
- action sur les causes et non sur les effets ;
- intervention par la mise en œuvre de tensions complémentaires compatibles avec l'équilibre dynamique existant. Dans ce cas, abandon délibéré des techniques par prothèses artificielles lourdes du type pinces, chainages, équerres en Béton Armé, qui introduisaient à la fois des points durs, et ensuite un paramètre statique passif totalement en contradiction avec la statique dynamique de l'architecture gothique.

L'état "d'équilibre idéal" auquel on voulait arriver fut obtenu par calculs en partant du point de résultante souhaité, et en remontant le funiculaire, notant à chaque point névralgique les actions à introduire.

MISE EN OEUVRE DES TIRANTS NORD



3.2. Les principales directions du raisonnement
- Tout d'abord, on a décidé de faire passer la ligne résultante des pressions dans le piedroit interne du passage de circulation situé au pied des culées d'arc boutant, (muré par M. BIGOT et dont la réouverture devait servir de test de résultat).
- Accentuation de l'effet pinacle de la culée externe.
- Reprise des pressions excédentaires de la volée inférieure de l'arc boutant.

3.2.1. "L'effet pinacle", destiné à verticaliser les pressions est le poids des maçonneries 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 renversement s'en trouve donc amélioré, et la technique adoptée est celle de la post-contrainte par tirants verticaux forés.

3.2.2. La similitude des dispositions Nord/Sud favorise des solutions symétriques. Il a été envisagé de créer une pression sur les culées externes Nord et Sud, agissant en sens inverse des pressions excédentaires par l'intermédiaire de tirants, ancrés à chaque extrémité dans les culées externes, cheminant sur le dessus des arcs boutants et réunis deux à deux en comble par tirants transversaux, l'ensemble étant articulé suivant un schéma en "trapèze". La valeur de la tension imprimée aux tirants est fonction de la composante horizontale nécessaire, calculée au niveau de la culée.

3.2.3. La présence de tirant situé en partie haute des volées d'arcs boutants peut agir en fibre tendue et contribuer à s'opposer aux mouvements de flexion pouvant résulter d'éventuels effets de compression excédentaires. Cette disposition permet donc d'assurer en même temps un rôle de sécurité.

3.2.4. Cette disposition pemet d'intervenir en comble sans aucune incidence sur la charpente existante qui peut être conservée en totalité avec toutes ses modifications. L'ensemble est totalement réversible.

3.3. Valeurs calculées

La difficulté était sans privilégier une action par rapport à une autre, de recréer un nouvel équilibre de tension, en cohérence et en harmonie avec l' équilibre gothique initial.

Malgré la conception symétrique Nord/Sud du chœur, la partie Sud accuse des dimensions moins développées, ce qui signifie des pressions moindres dans l'ensemble du contrebutement. C'est la valeur excédentaire de pression des arcs boutants Sud qui a servie à définir l'intensité des efforts destinés à s'opposer au déversement (1,2T).

L'effet pinacle nécessaire dans les deux culées externes Nord et Sud est de 4,5T, appliqué suivant une direction légèrement oblique, pour permettre à la fois le passage dans le piedroit interne de la culée, et à l'inverse, une position aussi centrale que possible en tête pour assurer une meilleure répartition de la compression sur la culée.

Dans la partie tournante du chœur, on a constaté que :

- les arcs boutants supérieurs ont un développement comparable à ceux des travées courantes ;
- les arcs boutants inférieurs sont beaucoup plus courts, et provoquent une pression réduite sur les culées externes ;
- les culées externes sont prolongées vers le bas par des contreforts.

En conséquence, il n'était pas nécessaire d'entreprendre l'introduction d'une pression s'opposant au déversement, l'action ou l'effet pinacle étant largement suffisant, et seul l'effet pinacle s'est révélé suffisant.

4. CHANTIER

La restauration du chœur fut inscrite dans la loi de programme sur les Monuments Historiques et les travaux ont commencé en Janvier 1990 pour une durée de quatre ans.

4.1. Action sur les maçonneries anciennes.

La restauration de 1868-1870 n'avait assuré que la réfection des joints, sans intervenir sur le cœur des structures. Aussi a-t'il été nécessaire d'entreprendre au préalable la restauration des maçonneries, par refichage, rejointoiement et injection (par coulis gravitaire ou à faible pression) de chaux hydraulique naturelle blanche, définie comme la plus proche et la plus compatible avec les mortiers d'origine encore en place.

On a constaté à cette occasion une bonne cohésion des maçonneries anciennes, avec de rares faiblesses localisées.

4.2. Tirants subverticaux des culées externes.

- Restauration des culées et des arcs boutants (cf. ci-dessus).
- Etaiement de la volée externe, par mise en place de frettage réunissant les deux culées (médiane et externe) pour les assimiler à un "mur plein". La volée supérieure d'arc boutant continuant naturellement à jouer son rôle dans l'équilibre de la voûte du chœur.
- Dépose du couronnement du pinacle de la culée externe.

Etablissement sur l'arase de la culée, dans la hauteur dans la dernière assise, d'une "platine" Béton Armé ferraillée en périphérie (épaisseur 20cm) ; et d'une autre en pied de culée, exécutée en sous œuvre en deux parties successives alterne/interne.

- Forage : diamètre 7cm maximum autorisé, sur une hauteur de 7m.
Le tracé n'était pas vertical mais oblique, défini visuellement par cordeaux, puis transmis à la machine par réglage angulaire. La tolérance pour assurer le passage dans le piedroit de la culée (piedroit côté toiture) était telle qu'une dérive de 1% maximum était acceptable par rapport à la cible prévue.
Résultats obtenus :
2cm soit 0,3% dans le cas le plus défavorable.
5mm soit 0,08% dans le cas le plus favorable.
- Mise en place d'une tige d'acier inoxydable filetée, assemblée en trois parties par manchons. Ancrage sur la platine basse par platine métal, écrou et clavette ; ancrage sur la platine haute et la même façon avec écrou et contre écrou.
- Mise en tension à 4,5T et serrage régulier chaque jour (pour absorber l'effet de relâchement). La stabilisation de la tension a été obtenue au bout d'une semaine.
- Injection de coulis de ciment en enrobage du tirant d'inox afin d'assurer à chaque assise une intime liaison à chaque assise avec les maçonneries traversées.
- Repose de la tête de pinacle ; dépose de l'étaiement de la volée externe.

4.3. Tirants obliques
- Dépose des chaperons d'arcs boutants formant goulottes d'évacuation des eaux pluviales ; création dans la partie neutre, (remplissage situé au dessus de la partie clavée des arcs), de saignées longitudinales pour recevoir le tirant.
- Forage oblique dans culée médiane et culée externe ; création d'un passage dans le gouttereau haut.
- Mise en place d'une platine BA dans culée externe, après dépose avec soin du parement extérieur .
- Mise en place d'une console en béton armé dans le comble.
- Mise en place de tirants en acier inoxydable lisses, logés dans une gaine PVC qui les isole de la maçonnerie pendant tout leur trajet, l'essentiel de l'effort devant se concentrer exclusivement au point d'appui sur la culée externe et non le long de l'arc.
L'intérieur de la gaine PVC a été garni de mousse de polyuréthane pour isolation et maintien du tirant dans l'axe de la gaine.
Après coup, repose des parements de pierre et des chaperons d'arcs boutants.

FORAGE DES TIRANTS SUBVERTICAUX

4.4. Tirants transversaux et accrochage du trapèze
En comble, sur les consoles BA encastrées dans les murs gouttereaux, mise en place des articulations destinées à réunir les tirants obliques avec le tirant traversant le comble.
- Cette articulation fait naître une résultante vers le bas qui améliore encore, si cela est nécessaire, la stabilité des appuis de voûtes du haut chœur.
- Elle ne doit pas être bloquée, mais rester souple, afin de suivre tous les éventuels mouvements pouvant affecter séparemment ou non le flanc Nord ou Sud de l'édifice (vent, pluie, température, etc ...). La solution initiale de rotule sur galets fut remplacée par un bras oscillant plus simple à réaliser et offrant un résultat comparable.

- Sur chaque tirant oblique à son arrivée en comble, et sur le tirant horizontal de liaison, a été placé un extensomètre (soit 3 par trapèze), de sorte à établir clairement la valeur des tensions de chaque tirant, et éventuellement localiser un accident pouvant intervenir dans le système de consolidation.
- Le tirant de liaison était prévu en acier inoxydable avec enrobage d'un isolant thermique. Mais du fait du cœfficient de dilatation très élevé de l'acier inoxydable, seule une isolation par manchon chauffant semblait pouvoir donner une stabilité satisfaisante. Cette formule a été écartée pour des raisons évidentes de maintenance, et la solution a pu être trouvée dans l'emploi de matières composites dérivées de la fibre de carbone (thermiquement stable, incombustible, non magnétique, etc...). Les résultats ont immédiatement marqué une stabilisation de la tension à plus ou moins 6%, de la tension nécessaire.
- L'ensemble de l'installation est mise en surveillance durant un cycle climatique complet (1 an) de sorte à en suivre les effets, et en apprécier l'équilibre. Les passages de circulation au travers des culées externes ont été reouverts ; les voûtes ont été réenduites. Aucun désordre n'est apparu depuis trois ans et semble confirmer l'efficacité des mesures adoptées.

ACCROCHAGE DE TRAPEZE EN COMBLE

5. CONCLUSION

Cette opération de consolidation quasiment achevée est actuellement et s'intègre à un programme beaucoup plus vaste de restauration complète du chœur de la cathédrale de Quimper, portant également sur les toitures, les maçonneries, les vitraux, le décor intérieur et le mobilier.

Elle est l'illustration des nouvelles tendances de recherche en matière de consolidation de structures, (appliquées en France et également à l'étranger par les missions françaises), et qui consiste à proposer, face aux techniques de consolidation par prothèses de blocage en béton armé (pinces, équerres, chainages), dont l'expérience a montré après plus de quarante ans, la très difficile assimilation par les maçonneries anciennes, une alternative qui se définie ainsi :

- Ce sont des interventions conçues comme mesure d'appoint venant compléter les consolidations par régénération des structures dans leur état aussi proche que possible de celui d'origine.

- Elles consistent à agir le plus près possible des lignes d'action des pressions et s'y intégrer avec les amplitudes les plus faibles.

- Ces techniques mettent en œuvre des matériaux contemporains mais ne nécessitent pas en principe une technologie particulièrement lourde ; elles visent un respect absolu de la réversibilité.

Cathédrale de Beauvais: de l'incertitude à la décision

Die Kathedrale von Beauvais: Von der Ungewissheit zum Entschluss

Cathedral of Beauvais: From uncertainty to decision

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J.-L. Taupin, né en 1931, s'est attaché à la consolidation de grandes charpentes anciennes et de voûtes d'églises et de palais. Il est actuellement en charge des cathédrales de Beauvais et Meaux. Il enseigne au Centre d'Histoire et de Conservation des Monuments Anciens de Chaillot.

RÉSUMÉ

La cathédrale de Beauvais, défi d'élancement et de précision, reste inquiétante. La Nef absente, la Tour-Lanterne de 1569 (153m) tombée en 1573 sont facteurs de déstabilisation. La résonance des structures de grand allongement est l'objet d'une attention croissante. Une faiblesse de fondation des parties ouest non analysée est probable. Les déformations depuis 1966 sont attestées: éclats d'arcs, voûtes fissurées, mesures récentes sur cibles. La collecte de données exactes et un diagnostic accepté par tous les partenaires sont indispensables à un programme rationnel de conservation.

ZUSAMMENFASSUNG

Diese geometrisch herausfordernde Kathedrale bleibt ein Sorgenkind. Ein 153m hoher zentraler, 1566 gebauter, jedoch 1573 eingestürzter Turm und das fehlende Schiff stören heute noch wegen fehlenden Balken das Gleichgewicht. Windeinflüsse erzeugen Eigenschwingungen. Im westlichen Fundament gibt es anscheinend Probleme. Seit 1966 ist der Alterungsvorgang offensichtlich (Bogenbrüche, Gewölberisse, geodäsische Verschiebungen). Genaue Daten sowie eine durch alle Beteiligten angenommene Beurteilung des Baudenkmals sind die für ein Restaurationsprogramm notwendigen Voraussetzungen.

SUMMARY

The Cathedral of Beauvais, a structural and geometrical challenge, remains a matter of worry. A 153m central tower (erected in 1566, collapsed in 1573) and the missing nave still bring unbalance today, due to a missing buttress. Wind generated oscillations raise a growing concern. Problems are suspected at the western foundation. Since 1966 alteration process is obvious (split arches, vaulting cracks, recent geodesic displacements). Diagnosis acceptable by any partner and adequate preservation plan depend upon exact data.

La cathédrale de Beauvais, conçue comme un défi (élancement, hauteur, précision) reste de siècle en siècle un sujet d'inquiétude. L'absence des parties occidentales (Nef) et les séquelles de l'écroulement de la Tour-Lanterne 4 ans après sa construction (153 m de haut, à la Croisée du Transept sur l'avant dernière travée construite) sont des facteurs hautement inducteurs de déstabilisation.
L'incidence des vents, principalement par le mécanisme de mise en résonance des structures de grand allongement vertical (culées rayonnantes par exemple) est devenue l'objet d'une attention croissante.
Des phénomènes impliquant les fondations des parties occidentales (les dernières construites) sont probables, mais l'état de la connaissance du site ne permet pas encore de les analyser .
La progression des déformations sur la période 1966/1993 au moins est attestée (éclats d'intrados de doubleaux, fissuration de voûtes à 48 m de hauteur, déplacements de cibles géodésiques).
La collecte de données exactes est une source indispensable à la préparation d'un programme rationnel de conservation.
Un préalable obligé est l'établissement d'une description de l'état actuel de l'édifice susceptible d'être acceptée par tous les partenaires concernés.

<u>Architecture</u> :

St-Pierre de Beauvais est né du dessein d'ériger une structure plus haute et plus aérienne que ce qui avait pu être réalisé auparavant. Un schéma géométrique absolu, simple, riche se déploie à partir d'un axe vertical de révolution planté au centre d'une étoile régulière à 13 branches, à partir de -15 mètres sans doute (fondations) jusqu'à +50 mètres (corniches). L'architecte a voulu dans un seul geste d'excellence réunir des catégories que nous dispersons aujourd'hui : la géométrie, la théologie, l'architecture, la maîtrise des structures. La nappe des voûtes et la voilure du haut comble exposée aux vents jusqu'à 60 mètres de hauteur reposent sur 60 supports : ceux-ci, au niveau des grandes fenêtres, dans la partie XIII/XIV° s., couvrent à peine 7,15% de la superficie incluse dans le périmètre des culées d'arcs-boutants.

<u>Des risques depuis le XIII° siècle</u> :

* Des murs gouttereaux minces et largement échancrés et des voûtes de 25 cm environ étendues sur des travées de 15 m par 9 m, étaient portés par des "quilles" de 50 mètres épaisses de 1,20 mètre ou peu s'en faut. Une dizaine d'années après l'achèvement du Chœur, des parties de voûtes s'effondrent mais sans ruiner les structures ni du Rond-Point ni les supports du Chœur.

* Vers 1300 dans un respect exemplaire de la pensée créatrice du monument on épaissit le mur gouttereau au-dessus des grandes fenêtres dédoublées, on intercale 6 piliers nouveaux pour porter cette masse accrue, on remodèle la double courbure des voûtes hautes (quadripartite transformées en sexpartite) qui ne demandent que de minces contreforts entre les fortes culées originelles. Le rythme ample et égal des immenses culées verticales qui scandent le pourtour de l'édifice n'est pas défiguré.

* fin du XV°s. on décide de reprendre le chantier pour la construction d'un Transept jugé nécessaire à la stabilité "iceluy cueur est sans croisée ne nef, au moyen de quoy est en danger de totale ruyne et trébuchement, s'il n'est contreboutté par le secours desdictes croisée et nef..".(1518)

* Sur les 4 piles de 52 mètres de la croisée du Transept à peine achevées, on construit en 1564 la Tour-Lanterne.. En 1569 une flèche de charpente culminant à 153 mètres est portée par une lanterne de maçonnerie à 2 immenses étages ajourés dont la voûte flotte à plus de 100 mètres du dallage. En 1572 des experts sont appelés à y examiner d'inquiétantes dégradations. La Tour s'effondre 1 an plus tard.

* Après reconstruction des 4 travées de voûtes contiguës à la Croisée on se résigne à construire contre les piliers occidentaux des contreforts particulièrement massifs : suspension d'activité pour des temps indéterminés. Les piles+massifs extrêmes, pesantes, insuffisamment fondées, déversent de siècle en siècle. Au XIX°s. on doit éliminer la voûte de Nef déchirée. Au Nord du Chœur, on doit aussi reprendre plusieurs piliers entre les 2 collatéraux brisés par déversement.

* Dans les 50 dernières années ont été réparés des dégâts de guerre apparents (5 bombes ont frappé la cathédrale en Juin 1940), et on a entrepris de supprimer des tirants métalliques extérieurs (Sud-Ouest du Transept et Nord du Chœur). Depuis, l'attention est périodiquement attirée sur le fait que des mouvements non-équilibrés continuent selon toute évidence à se développer :

1/ 1966 : chute de débris de pierre due à des brisures à l'intrados d'arcs doubleaux du Croisillon Sud ([1]),

2/ à partir de 1982 on constate que les culées du Chevet privées de leurs tirants de fer anciens sont mises en mouvement par le vent ([2]),

4/ réouverture de fissurations dans les voûtes du Transept rejointoyées entre 1972 et 1974,

5/ en 1992 mesure des vitesses de déplacement de cibles géodésiques posées en 1985 sur les grands piliers,

La nature de l'incertitude. Quel instrument pour transposer rationnellement l'information ? :

L'incertitude qu'inspire cette structure immense, inachevée, sinistrée, travaillée de dérives et de fractures, est perçue depuis des années. Un paradoxal contraste oppose la gravité d'événements redoutés et la difficulté d'en évaluer l'imminence. Le gigantisme entrave la perception rationnelle des poids et des efforts. Les voûtes que la hauteur 6 fois répétée de la chaire n'atteindrait pas encore, paraissent irréelles.
Un problème crucial est donc de trouver des moyens **d'observation, de preuve et de présentation de preuve** : méthodes de mesure, méthodes de raisonnement, méthodes de communication.

Un mode de raisonnement :
* Il faut susciter la conscience de ce que des mouvements amorcés depuis des siècles ne peuvent pas indéfiniment se poursuivre, et de ce qu'ils ne créditent pas l'édifice d'une mythique invulnérabilité : un instant d'équilibre ultime et irréversible doit survenir. Il faut faire voir qu'une résolution catastrophique n'est pas obligatoirement réservée à la 3ème ou à la 7ème génération qui nous suivra, et que même s'il devait en être ainsi, nous n'aurions pas le droit de laisser se préparer ce funeste cadeau à nos Successeurs. Des besognes préliminaires conventionnelles, l'aspiration à d'apaisants consensus œcuméniques, retardent le moment d'accepter, de décider, d'agir utilement.
* Il n'est guère possible de raisonner par analogie : nous avons peu de chance de connaître des événements du type de ceux qu'on appréhende. Peu de structures comparables existent ou ont existé, peu de processus analogues ont été commentés.
* S'en remettre aux démonstrations théoriques ? Les formules conventionnelles sont simples mais approximatives, les formules sophistiquées sont prometteuses mais contestées. Féconde interdisciplinarité : d'essentielles questions peuvent être soulevées par les remarques d'un visiteur attentif et fasciné, des perspectives fondamentales ouvertes par les avis de tels Experts de l'aéroélasticité, de l'archéométallurgie, ou des matériaux pierreux.
* À ce stade, c'est **dans le bâtiment lui-même que se trouvera l'instrument démonstratif des dérives.** Saura-t-on **montrer** les mouvements, la corrélation spatiale des mouvements, l'accélération des mouvements ?

Quels modes de mesures seraient plus aptes à observer, inspecter, surveiller ?

La mise en œuvre des campagnes de mesures présente diverses difficultés.

a/ Elle sont souvent coûteuses et délicates et la diversité des techniques fait craindre de mauvais choix,

b/ Les résultats produits, quoique précieux, sont rarement d'une évidence péremptoire, et il faut beaucoup de persévérance pour tirer toute l'information de la fastidieuse abondance des résultats des mesures,

c/ Une mesure a souvent pour "mérite" de montrer qu'une autre mesure serait d'un plus grand intérêt, suggestion peu compatible avec la tentation d'exposer des preuves d'efficacité,

d/ La probabilité d'acquérir une information significative dépend directement de la durée des mesures et de l'étendue du domaine couvert. L'enveloppe des coûts nous enferme dans une relation d'incertitude : brèves mesures en continu, ou mesures discontinues sur une longue période ?

e/ La technologie de la mesure évolue et rend elle-même obsolètes ses propres pratiques. Comment poursuivre avec de nouvelles techniques plus performantes, alors que le suivi des anciennes méthodes était déjà difficile à financer ?

Évolution des techniques d'observation dans le chantier de Beauvais :

1890 Dans les années 1880/1910 la reprise en sous-œuvre de piliers des collatéraux Nord, a-t-elle été décidée sur le seul constat visuel de destructions locales du matériau ou sur des vérifications de l'intégrité géométrique de la structure, voire sur des mesures de dérives ? Il est possible que des présences humaines dans l'édifice plus permanentes qu'aujourd'hui, aient permis une détection et une interprétation plus directe des évolutions nocives. Maintenant le retard pris dans les interventions et la discontinuité des inspections dans le temps comme dans l'espace, cumulent leurs effets et établissent une ambiance d'incertitude. D'où l'obligation de recourir à des méthodes et à des instruments pourvoyeurs de mesures objectives.

1960 La période 1960/70 est marquée par des initiatives prometteuses. À la suite des brisures de 2 arcs doubleaux des hautes voûtes, une quarantaine d'extensomètres à cordes vibrantes consultables à distance ont été posés dans un secteur du Croisillon Sud, de 40 à 48 m. de hauteur. Le suivi n'a pas été maintenu. Une campagne ambitieuse de relevés par restitution photogrammétrique a été amorcée, première étape pour remédier à un énorme manque de représentation graphique de l'édifice. En même temps l'architecte J.P. Paquet et l'Institut Géographique National ont esquissé une méthode de diagnostic de stabilité des voûtes, en associant une représentation photogrammétrique des voûtes à des épures-test de stabilité théorique via l'informatique de l'époque. Cette tentative a été abandonnée à la mort de l'architecte. Elle aurait été dépassée aujourd'hui par les progrès survenus dans l'application de l'analyse aux éléments finis à l'espace tridimensionnel ([3]), mais elle révélait la nécessité d'un instrument adapté pour travailler sérieusement sur un tel monstre.

1980 Des systèmes d'observation introduits dans les années 1980 par l'architecte Yves Boiret ont été une autre étape d'un progrès d'objectivité dans la connaissance des évolutions en cours :
- restitution photogrammétriques IGN pour quelques aspects : coupes des batteries d'arcs-boutants et coupes horizontales générales,
- premier réseau de cibles géodésiques créé dans l'édifice (campagne "zéro" en 1985),
- à la suite du constat visuel en Octobre 1982 du balancement de culées du Chevet et de la brisure consécutive de plusieurs arcs-boutants rayonnants, suivi extensométrique sur quelques mois pour l'étude statique de la flexion de l'une des culées rayonnantes du Chevet,
- première phase d'évaluation en soufflerie des contraintes statiques imposées par le vent à diverses faces de l'édifice,
- mesure des caractéristiques mécaniques de carottes extraites d'une culée ([4]) et observation des efforts statiques sur une maquette simplifiée en maçonnerie de cette culée.

1992 Les préoccupations prioritaires sont fidèles à plusieurs principes ([5]) :
- condenser le plus complètement possible les observations antérieurement faites, établir la figuration visuelle des listings de chiffres déjà acquis, produire par leur croisement systématique de nouvelles conclusions, et bénéficiant d'un peu plus de recul, les résumer et repérer les compléments nécessaires,
- constituer le tableau de la situation en éclairant les données par des informations de nature quantitative ou/et qualitative aussi variées que possible. Vérifier dans quelle mesure ces informations se confirment les unes les autres, exploiter les complémentarités et les convergences.
- discuter la rentabilité de divers types de mesures : coûts, valeur de persuasion, sujétions de maintenance, longévité, facilité de lecture, exactitude, et développer l'appareillage de mesures.

<u>Résultats acquis sur les 30 dernières années</u> :

Études des effets du vent

L'essai en soufflerie a fourni des données sur l'intensité relative des poussées ([6]) en 217 points sélectionnés sur une maquette de l'édifice par le CEBTP, mais les mesures, traitées en moyennes, par économie, ne saisissent par l'instabilité des impacts aérodynamiques ni la genèse des phénomènes de résonance.
L'observation du fléchissement d'une culée rayonnante du Chevet a été conduite au moyen de 23 extensomètres à base longue. L'expérience menée au laboratoire du CEBTP sur un modèle réduit de maçonnerie de l'une de ces culées, par paliers jusqu'à destruction, parallèlement à l'étude des coefficients d'élasticité d'éprouvettes extraites du monument, a fourni des données utiles sur les caractéristiques des matériaux.
Les démarches orientées sur l'aspect statique des efforts, n'atteignent pas les processus de mise en oscillation produits sur les grands éléments longilignes du monument par l'échappement de l'air dans un milieu géométriquement complexe. Récemment des observations (photo et vidéo) faites à l'improviste par temps de rafales ont donné des vues directes de phénomènes d'oscillation, et permis une estimation d'amplitudes (3 à 6 cm) et de fréquences (de l'ordre de la seconde). Des simulations préliminaires de mises en résonance de ces éléments ont été commencées sur modèle informatique ([7]). Mais à ce jour aucune auscultation par accéléromètre sur le modèle de celles faites sur 13 cathédrales anglaises et 2 églises allemandes en 1966 ([8]) n'a pu être effectuée. Ces analyses sont indispensables à la compréhension du mode de fonctionnement des tirants de fer en légère flexion (supprimés vers 1960 dans le contexte d'un développement encore insuffisant de l'histoire des techniques) qui établissaient une solidarité entre les culées rayonnantes par un effet d'amortissement inertiel.

Mesures géodésiques de surveillance de la posture de différentes parties de l'édifice :

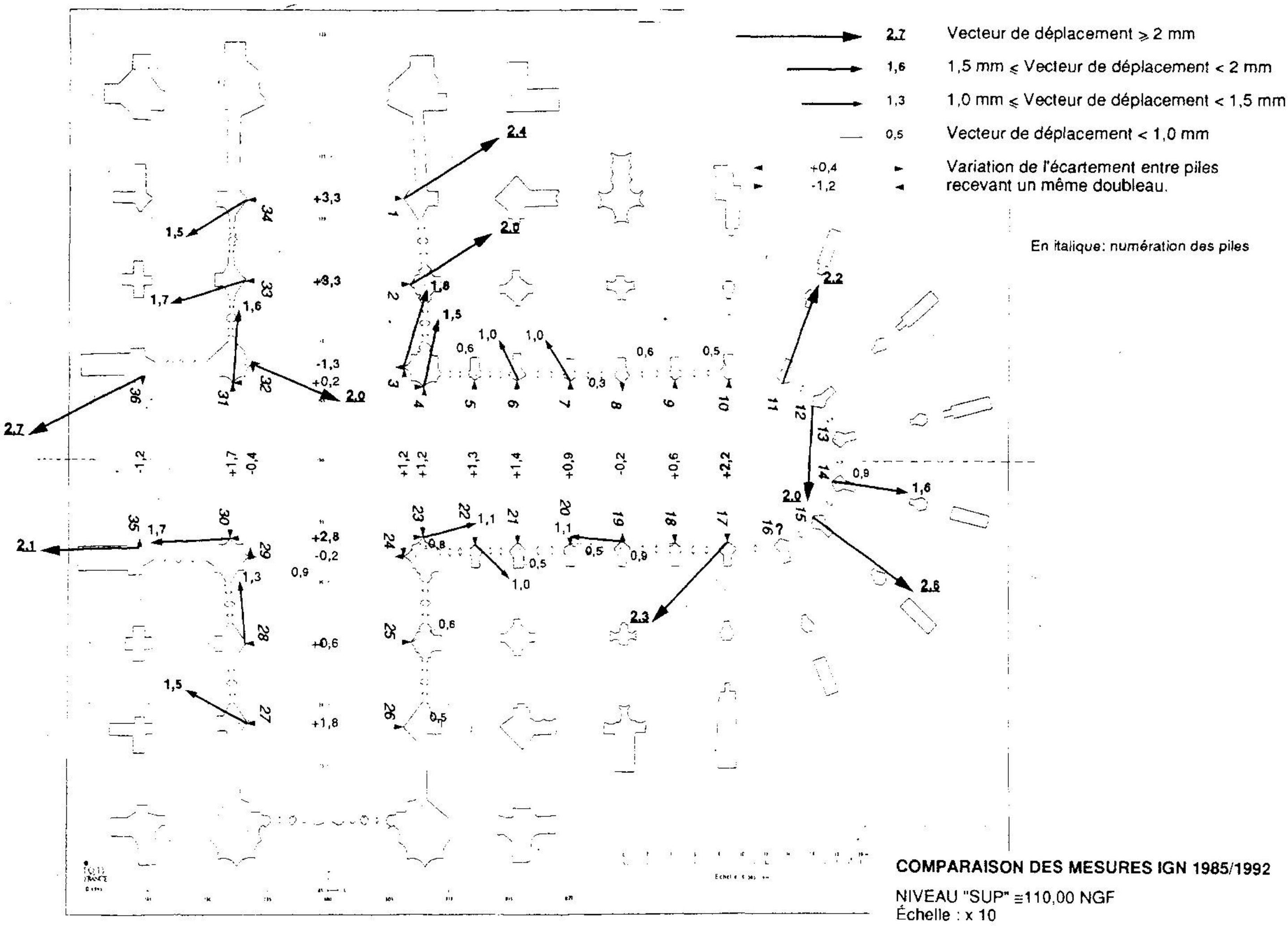
L'IGN a été chargé en 1985 par l'architecte Y. Boiret et l'ingénieur M. Bancon de mettre en place une surveillance géodésique destinée à évaluer la vitesse de progression des déformations à l'œuvre dans les 36 grands piliers qui forment l'ossature interne de l'édifice (108 cibles réparties sur les piliers en 3 étages).

La 1ère confrontation à "l'état zéro" de 1985 a été faite en Mai-Juin 1992, et la réexploitation améliorée du résultat brut des "mesures zéro" par application rétroactive d'un traitement tridimensionnel, a été effectuée en Juillet-Septembre 1992. le niveau de précision de la "campagne zéro" a été monté vers le niveau de précision que la campagne 1992 doit à l'instrumentation et aux logiciels actuels et à la création de points de visée supplémentaires à mi-hauteur de l'édifice ([9]).
La mesure des vecteurs de déplacements a bénéficié d'une amélioration de précision de facteur 5 en moyenne. L'IGN a pu assurer une approximation de l'ordre de 1 mm des résultats obtenus en planimétrie. Si la méthodologie n'avait pas pu être améliorée a posteriori, les marges de confiance inhérentes à la technique initiale de traitement des données n'auraient pas permis de certifier l'existence de mouvements.

Les seuils de précision étant déclarés, il est établi que sur la durée de 7 années 1985/1992, des mouvements se sont développés dans plusieurs organes fondamentaux de la cathédrale :

pour 11% des cibles (6 cibles sup.+ 2 cibles méd.)	: mesures comprises entre 1 et 1,5 mm,
pour **15%** des cibles (4 Rond-Point + 7 Transept)	: mesures comprises entre **1,5 et 2 mm**,
pour 12% env. des cibles (4 Rond-Point, 5 Transept)	**: mesures de 2 mm et plus.**

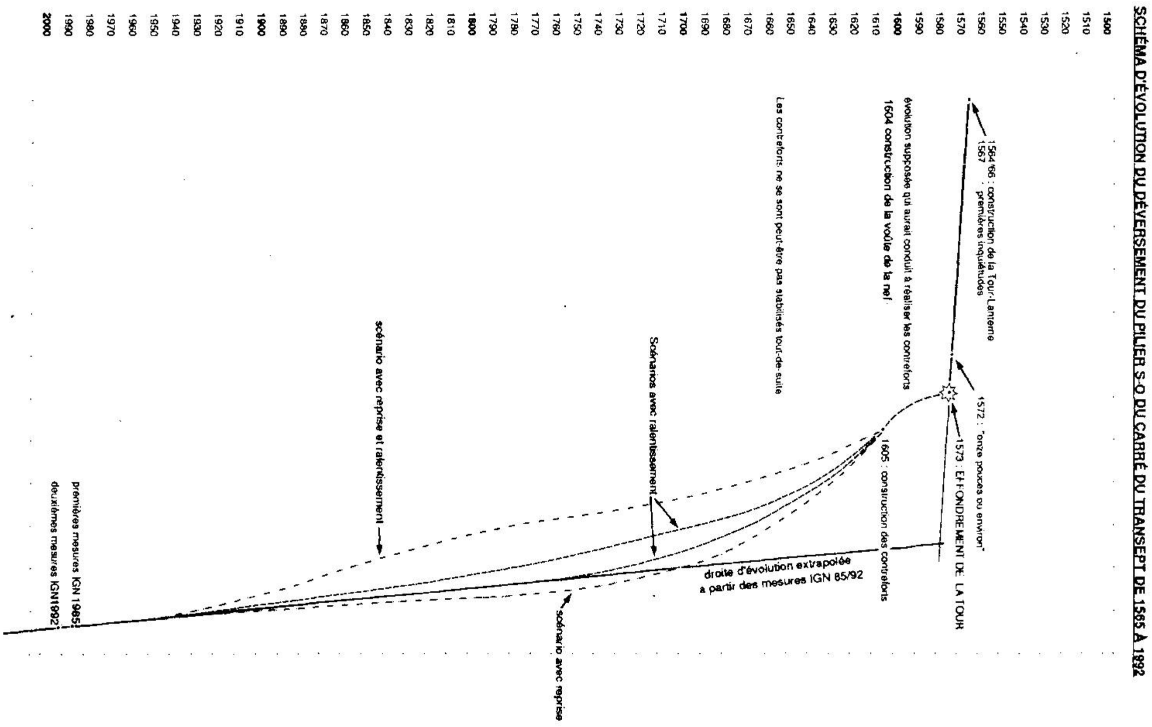
Au total **1 pilier sur 4 se serait déplacé en tête de plus de 2 mm** (9 piliers sur 36).



COMPARAISON DES MESURES IGN 1985/1992
NIVEAU "SUP" ≡110,00 NGF
Échelle : x 10

Quatre principaux phénomènes apparaissent ([10]) :

- tendance d'ouverture radiale ou tangentielle affectant la majorité des piliers du Rond-Point,
- poursuite du déversement des 2 piliers de Nef adossés aux gros contreforts Ouest,
- tendance des supports de voûtes de Transept à s'écarter transversalement,
- tendance de certains supports du Transept à se déverser vers le Nord,



Comparateurs, mesures de convergence

Les programmes de mesures proposés s'orientent vers 2 objectifs : dégager une représentation générale des mouvements et de leurs corrélations, permettre une surveillance de points critiques où on pourrait chercher, avec les réserves qui s'imposent, des indicateurs d'alerte. Une meilleure connaissance des antécédents de l'édifice (alertes et confortations survenues depuis 100 ans au moins) devrait faire progresser la compréhension des altérations géométriques. Parmi les divers types d'instruments disponibles, les comparateurs, appareils simples et sensibles, semblent devoir donner des indications utiles sans pourtant imposer les frais et les sujétions de maintenance d'un système de saisie en continu, pourvu que la routine de lecture soit maintenue de façon sérieuse et ininterrompue.
La surveillance géodésique ignore ce qui se passe entre ses cibles, et ce qui est plus grave, au-dessus de celles-ci : au-dessus des voûtes, dans la tranche d'espace où on pressent que se produisent des renversements sous l'effet des contraintes aléatoires issues des charpentes plus ou moins détériorées. Des relevés de déformées de supports et d'arcs sont nécessaires. Il est possible d'associer à cette procédure des **mesures de convergence** (positions relatives des cibles repères), précises et que leur facilité d'exécution permet de répéter plus souvent.

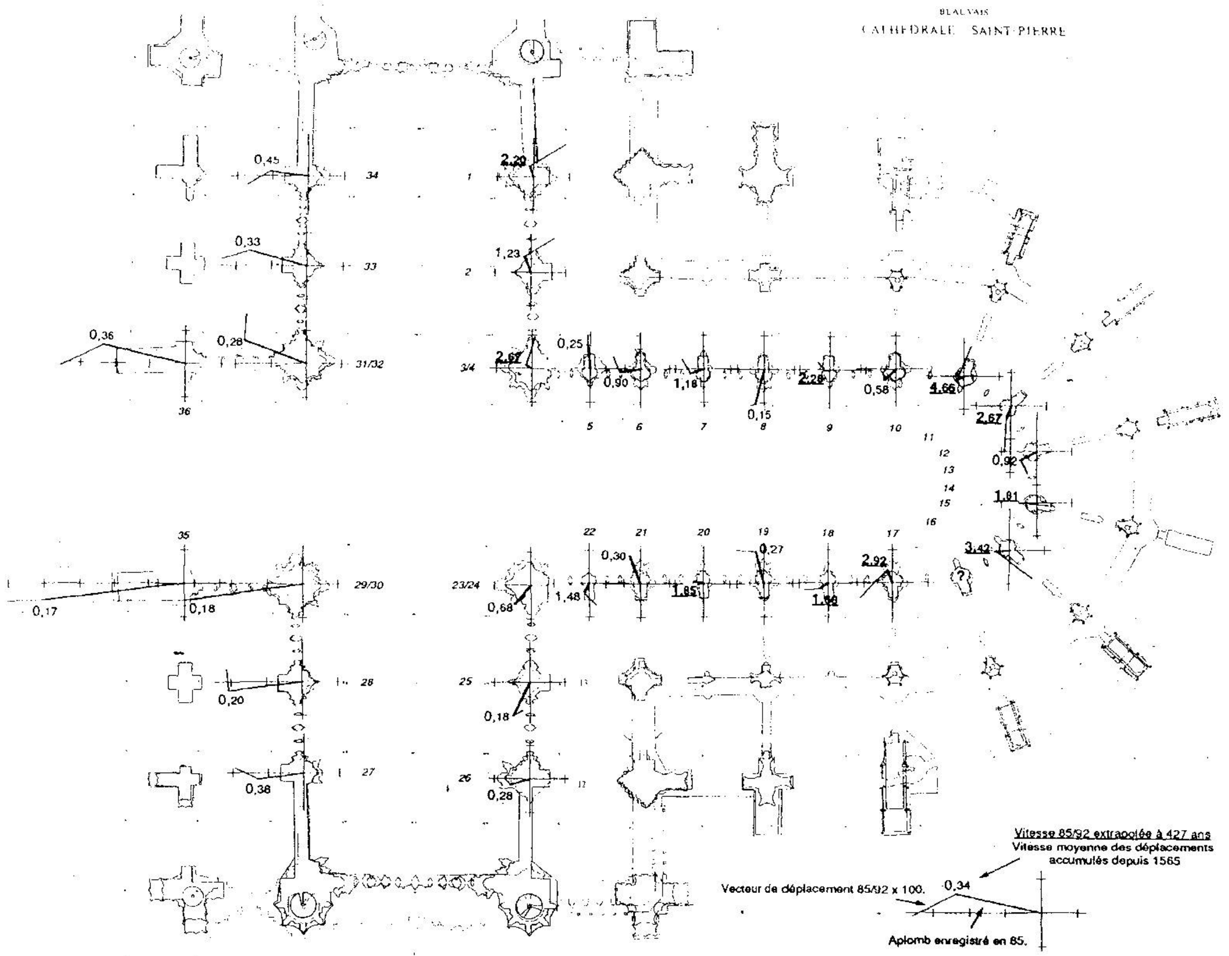
Modélisation informatique

L'intuition et les observations conduisent à considérer l'hypothèse que certaines parties de la cathédrale de Beauvais sont déjà engagées dans un domaine proche de l'instabilité élastique. Toute intervention physique sur le bâtiment (réparation de charpente, déchargement par démontage d'une partie de la couverture de plomb, fondation d'échafaudage sur piliers, murs gouttereaux ou arcs diaphragmes, ancrages de câbles d'étaiement, etc.) peuvent introduire **un léger et fatal surcroît de contraintes**.
Les mesures sont utiles à condition d'être replacées dans le contexte de toutes les données disponibles. Le problème posé à Beauvais est d'une très grande complexité. Il faut se féliciter que la diffusion de logiciels adaptés à la modélisation des contraintes au sein de grandes structures permette aujourd'hui **des simulations à blanc préalablement à toute intervention sur le site**. Cette méthode doit être alimentée par des données descriptives fiables : géométrie, caractéristiques de matériaux, saisie des traumatismes tels que les fissures, informations (quasi inexistantes à l'heure actuelle) sur le sol porteur et les fondations. Certaines opérations peuvent être simultanées, et cela d'autant plus que la conception de la modélisation peut s'appuyer sur l'approche modulaire de sous-ensembles progressivement mieux définis. Il ne semble pas y avoir d'autre méthode fiable à mettre en concurrence. Les méthodes plus anciennes subissent de plein fouet, et aggravées par d'autres griefs, les objections que certains croient devoir faire à l'analyse aux éléments finis.

Estimation de la vitesse des dérives

Prenant pour hypothèse que les structures ont été élevées sur des aplombs corrects ([11]), le diagramme ci-après expose pour chaque pilier la valeur du rapport de "**la vitesse 1985/1992**" à la "**vitesse moyenne déduite des déplacements cumulés des origines à 1985**" (par ex. pour les 2 piles occidentales de la croisée du Transept, à partir du début du chantier de la Tour-Lanterne : 1992-1565=427 ans).

Les valeurs de ce rapport, inférieures à **1** dans 11 cas sur 14, indiquent qu'il s'est trouvé dans des points particuliers, des phases de plus rapide déformation dans le passé. Cela est attesté pour les 4 Piliers du Carré, dont l'histoire est décomposée en 3 périodes contrastées : 1565/1573, 1573/1605 et 1605/1992. L'énergie dissipée dans le sol du Carré par la chute de la Tour-Lanterne a produit au pied des 4 gros Piliers une compression du terrain dont les effets ont agi un certain temps sur les massifs supports de ces piliers. En revanche on note une accélération : zone d'enlèvement des tirants du Chevet.



13 piliers existent depuis 750 ans : Piliers 6, 8, 10, 11, 12, 13, 14, 15, 16, 17, 19, 21, 23/24,
5 piliers existent depuis 700 ans : Piliers7, 9, 18, 20, 22,
12 piliers existent depuis 460 ans : Piliers 1, 2, 25, 26, 27, 28, 29/30, 31/32, 33, 34, 35, 36
2 piliers existent depuis 420 ans : Piliers 3/4, 5,
2 piliers existent depuis 390 : 2 contreforts Ouest

COMPARAISON DES VITESSES DE DÉPLACEMENT EN TÊTE DES PILIERS : DES ORIGINES À 1985 ET DE 1985 À 1992

CONCLUSIONS :

Il est sage d'investir dans les mesures et dans leur interprétation puisqu'elles sont de façon évidente l'outil qui déterminera le destin d'un monument si toutefois les fruits d'une préparation rationnelle sont mis à profit avant des blessures irréversibles. Toute technique de mesure donne fatalement à observer un édifice comme à travers quelques trous percés dans un écran opaque. Comment répartir une instrumentation forcément limitée ? De maîtres d'Œuvre en maîtres d'Œuvre se fortifiant par degré à mesure de l'enrichissement de la culture ambiante de chaque décennie, se développe un processus de compréhension cumulatif qui prépare les choix à opérer.

Au maître d'Œuvre, intermédiaire responsable entre les Producteurs de mesures et le Récepteur de conclusions documentées qu'est le Maître d'ouvrage, incombe la tâche difficile de faire émerger et faire accepter un jugement solide sur la hiérarchie des priorités nécessaires pour organiser la conservation. Puissent les quelques remarques produites ici, susceptibles peut-être de quelques généralisations dans le domaine de la commande de l'activité mentale, être utiles à d'autres personnes impliquées dans quelque situation analogue.

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1 Les joints des naissances des arcs ont été vidés ou garnis de plaques de Néoprène formant couche de répartition de contraintes en 1972/74. Les travaux alors ont été limités en pleine connaissance du fait que la consolidation restait entièrement à entreprendre.

2 En fait les culées sont mises en résonance par l'impulsions des échappements aérodynamiques produits sur les culées voisines.

3 K.D. Alexander & R. Mark, W.J. Beranek de Université de Delft, G. Croci, Chiaruggi, R. Barthel de l'Université de Karlsruhe, École d'Architecture et École nationale supérieure d'Hydraulique et de Mécanique de Grenoble, et sur le sujet même de la cathédrale de Beauvais, G. Lamboley en 1992

4 "Détermination de la qualité de la maçonnerie d'un contrefort, et vérification de la stabilité d'une maquette de contrefort sous l'effet du vent", CEBTP, Janvier 1988.

5 Travail présenté par l'A.C.M.H. J.L. Taupin (Rapports de Janv 92 et de Sept 92).

6 Valeur, en chaque point, du "coefficient adimensionnel de pression moyenne statique locale". "Étude aérodynamique sur modèle rigide dans la Soufflerie à couche limite turbulente (soufflerie SACLIT mise au point par J. Wianecki) du CEBTP à St-Rémy-lès-Chevreuse", Mai 1987.

7 Application du logiciel SYSTUS - Bureau d'Études de Génie Civil G. Lamboley, 1992.

8 "Cathedral vibration and the sonic bang", Aug. 1966, Université de Southampton, collabor. de E. Szechenyi.

9 L'accès à des cellules du grand Triforium a été ouvert pour créer des bases de théodolite supplémentaires.

10 La précision locale dépendante du mode d'appréhension de chaque point étudié, est explicitée par l'ellipsoïde d'erreur propre à chaque point ("90% de chances que le point réel se trouve à l'intérieur de l'ellipsoïde décrit", ou "risque que sur n points, les coordonnées annoncées pour n' d'entre eux soient inexactes").
L'IGN rend compte de déplacements mesurés avec une précision du demi-millimètre en planimétrie et en altimétrie dans les surveillances géométriques d'ouvrages d'art (Viaduc de Passy, Viaduc du TGV sur la Marne, Parc des Princes). Les améliorations méthodologiques permettront d'anticiper efficacement la date du prochain contrôle qui, au rythme quinquennal, devrait être 1997.

11 Ce qui laisse peu de doute si on songe au risque encouru, à la facilité des vérifications par fil à plomb et au luxe de précautions prises en liant les piliers au montage par plusieurs nappes superposées d'entretoises métalliques.

Structural Message of the Tower of Babel

Message structural de la Tour de Babel

Konstruktive Botschaft des Turms von Babel

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SUMMARY

The most famous Ziggurat, in its historical and structural identity, is chosen to exemplify the achievements in massive masonry construction of ancient Mesopotamia. The postulated disposition for its reconstruction is met, in the form of a pilot research study by generating few original technical parameters - in terms of mechanics and strength of materials. On this basis, some constructional essentials are specified.

RÉSUMÉ

Le plus fameux ziggurat, de par son identité historique et structurale, a été choisi comme exemple des structures massives en maçonnerie dans la Mésopotamie ancienne. Suivant certaines tendances visant à la reconstruction du monument, des recherches pilotes ont été faites, lequelles ont mené à la découverte de paramètres techniques originaux en ce qui concerne la mécanique et la résistance des matériaux. Sur cette base, certains problèmes majeurs de construction sont soulevés.

ZUSAMMENFASSUNG

Das durch seine historische und bauwerkliche Identität berümteste Zikkurat dient als Musterbeispiel vollendeter Massivbaukunst im alten Mesopotamien. Im Sinne einiger Tendenzen, die auf eine Rekonstruktion hinzielen, wurden Pilotuntersuchungen vorgenommen. Dabei wurden einige der urspünglichen technischen Parameter betreffend Mechanik und Werkstoffestigkeit entdeckt. Auf dieser Basis werden gewisse grundlegende Tragwerkseigenschaften dargelegt.

1. INTRODUCTION

In archaeology, it is the fate of all excavated ancient findings that, when not properly protected in good time against various effects of uncovering, they suffer accelerated destruction. This rule is fully confirmed in Babylon. Since the excavations of R. Koldewey, started in 1899 and carried on for ca. twenty years, the exposed remains were later, either widely touched by atmospheric influences (erosion and deflation) or human invasion (capture of bricks for present needs), resulting in major vanishing of the relics under sand and rubble.

This state lasted until the fiftieth, when Iraq became independent; the protection of ancient monuments has become official policy of the authorities. Concerning Babylon, the publication /1/ states: "...Babylon belongs to all peoples and all nations. Visitors from all over the world are anxious that something should be done to further the restoration and reconstruction of Babylon's principal buildings, so that the city's former grandeur may be better appreciated."

In 1978 a broad plan for the protection of the most impressive ancient monuments, concerning the total country, was launched in Iraq /2/. As regards Babylon, this plan anticipated three types of activities:
- surveys of individual objects as basis for a competent future synthesis;
- development of some techniques (lowering of water-table, drainage and insulation of masonry constructions, conservation of clay bricks - dried and baked, production of bricks on site from salty clays, etc.);
- enlargement of old and construction of new museums on site.

Simultaneously, excavation works were carried out, as a return to or continuation of those of Koldewey.

Concerning the revival of the Tower of Babel, there were large differences in opinions and, therefore, this problem remained still open. The eventual rebuilding of the Tower was found to be preceded by careful "philosophic" analysis and advanced technical study, as well as - accumulation of large funds.

All those plans must have suffered considerably due to the Gulf War but the promotion of a policy friendly towards the restoration of Babylon, the Tower of Babel in particular, will always stay alive. This report is thought as a response to that challange. It concentrates on the Tower of Babel /3/, discussing especially some of its structural aspects, in terms of mechanics and strength of materials. May be it could contribute to the reconstruction of the Tower and of Babylon as a whole (Fig. 1).

2. BACKGROUND

It is commonly agreed that history /4/ and engineering /5/ were born in Sumer. Human civilization gained much through the emergence of the Sumerian city-states - at ca. 4500 B.C. Stage-towers, or ziggurats, were their most spectacular structures - later spread all over ancient Mesopotamia; the famous Tower of Babel is one of over thirty ziggurats discovered there until now.

Typical stage-tower was a gigantic, stepwise shaped, pedestal base carrying a temple at top. Together with a lower temple, it was for the Mesopotamians, most likely, the house of god and the gate of heaven. That idea is confirmed by all the ziggurats' names. In particular, E-temen-an-ki, standing for the Tower of Babel, means "House of the Foundations of Heaven and Earth."

"House" or "Foundation" are words articulating the engineering traditions of the Sumerians. Construction is deeply rooted in their past, as results from the Epic

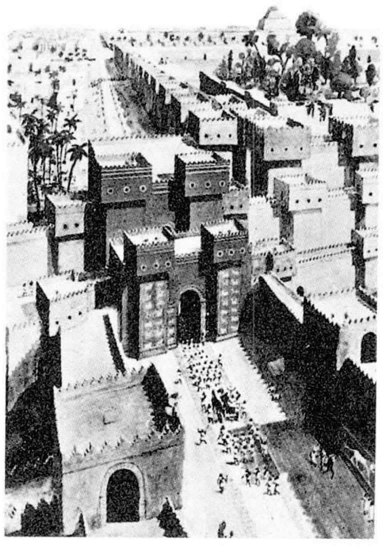

Fig. 1 Image of Babylon /1/

of Gilgamesh /6/ (Fig. 2). Ziggurats were built out of adobe and brick, since natural stone was scarce. The elements were simply placed and connected by clay or asphalt. Reed mats were applied at fixed levels of the ziggurat inner shafts - in order to achieve a better distribution of loading. Wooden anchors had to increase the stability of the ziggurat mantle walls. General impressions of the stage-towers, as seen originally by their builders, are given in Fig. 3 and Fig. 4.

"Examine the substructure,
view the brickwork.
Is the brickwork not of brick?
Did the Seven Sages
not lay the foundation?"

Fig. 2 Quotation from the Epic of Gilgamesh /6/

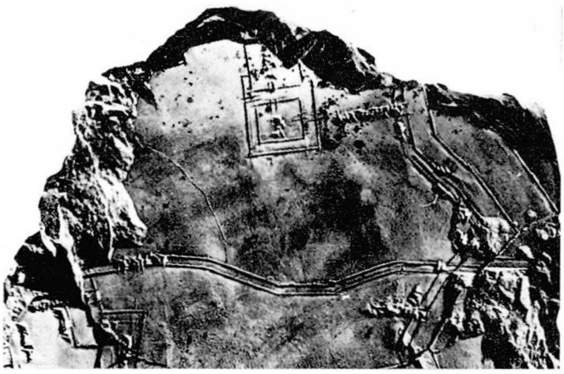

Fig. 3 Clay tablet with a stage-tower plan /7/

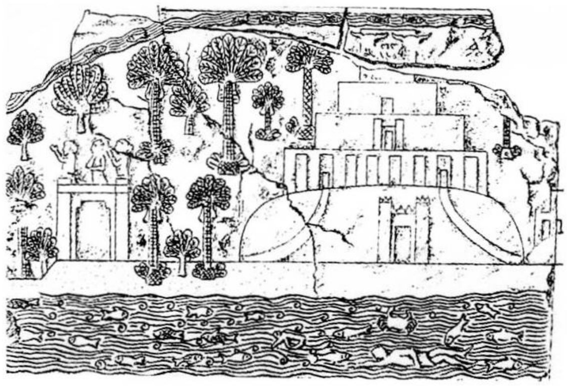

Fig. 4 Relief from Nineveh showing a stage-tower /2/

3. HISTORY

Biblical tradition caused that the Tower and the City of Babylon form certain entirety: "...as people moved eastwards they found a valley in the land of Shinar where they settled. They said to one another, 'Come, let us make bricks and bake them in the fire.' For stone they used bricks, and for mortar they used bitumen. 'Come,' they said, 'let us build ourselves a city and a tower with its top reaching heaven'" /8/.

Babylon or "bab-ilu" means "gate of god" /9/ (engineering implication of "gate" is evident). There are no data of archaeology on its genesis: high ground water level of to-day obstructs the penetration below the stratum of the 14th century B.C. Earliest cuneiform reference to Babylon as a secondary locality comes from the 21st century B.C. It appears that only during the Old Babylonian Period (ca. 1900-1600 B.C.) the city, with its god Marduk, became a weighty center of domination. But the construction of E-temen-an-ki was launched, probably, much later, when Nebuchadnezzar I (1123-1101 B.C.) was king of Babylon; when it was completed, is not certain /2/.

Babylon's conquest in 689 B.C. by the Assyrians caused that "the temple tower out of brick and earth", as mentioned by their king Sennacherib, was seriously destroyed /2/. But, for political reasons, already his son Esarhaddon and grandson Ashurbanipal tried to restore the Tower; first of them mentioned: "I ordered to rebuild the temple tower E-temen-an-ki on the old site 180 ells long and 180 ells wide" /2/. But the work must have not been much advanced at that time.

Only after the fall of Nineveh in 612 B.C. and the decline of Assyria, the victorious ruler of Babylon, Nabopolassar (625-605 B.C.) was able to speed up the reconstruction of the Tower. He wrote: "Concerning E-temen-an-ki, the stage-tower of Babylon, that ... declined and laid in debris, Marduk, the lord, ordered me to consolidate its foundations within the bossom of underworld, and to make its top similar to heaven." He added, "I ordered baked bricks to be made. Since one had to consider abundant rains from heaven ... I commanded rivers of tar to be brought over the Arakhtu canal." His son, Nebuchadnezzar II (604-562 B.C.) finalized this work, recording: "All peoples of numerous nations I forced to work on the construction of E-temen-an-ki ... The high seat of Marduk, my lord, I prepared on its top ... I raised the top of E-temen-an-ki using baked bricks with bright blue glaze" /9/. Without question, it was the culmination of the Tower's splendour. Unfortunately, it ceased shortly thereafter, when in 539 B.C. Babylon was seized by the Persians under Cyrus II (559-530 B.C.).

During the Persian rule Babylon greatly deteriorated. Major destruction of the city was done by Darius I (522-485 B.C.) and Xerxes I (486-465 B.C.) /2/ - but the Tower of Babel continued to exist. By 460 B.C. it was seen by Herodotus, who reported: "In the center therin a powerful tower, one stadion in length and width, is built, and on that a second, thereon another and so altogether eight towers, always one above the other. Stairs are arranged on the outer side around all the tower" /9/.

Nevertheless, within years E-temen-an-ki must have suffered serious degradation, so that Alexander the Great, after taking Babylon in 331 B.C., decided to repair it. The related reference of Strabon (60 B.C. - 20 A.D.) is the following: "There was also the tomb of Bel, that, as said, was demolished by Xerxes. It was a foursided pyramid of baked bricks, being itself one stadion high, with each of the sides one stadion long. Alexander wanted to restore it, but the undertaking was enormous and required much time, since the removal of the ruins alone required the work of two months for 10,000 men, so that he was not able to complete the action started." Archaeological excavations have confirmed that report, uncovering a deposit with 300,000 cubic meters of Tower rubble, which corresponded to the 600,000 days of work mentioned by Strabon /2/.

The rule of the Seleucids (311-126 B.C.) introduced Seleucia as their capital, with which Babylon became secondary. In 140 B.C. it was captured by the Parths, what resulted in a serious depopulation of the city. By 199 A.D. Septimus Severus found it completely deserted /10/.

To-day there is only the trace of the Tower of Babel (Fig. 5), a very imperfect echo of the past glory (Fig. 6).

Fig. 5 Archaeological traces of the Tower of Babel /1/

Fig. 6 Reconstruction of the Sacred Area (Temenos) in Babylon /1/

4. STRUCTURE

The typical feature of all the previously cited original notes on Babylon and its Tower was that they contained many references to engineering. Some of them concerned E-temen-an-ki's structure, especially the s.c. E-sag-ila Tablet /2/ - here until now left without a detailed analysis. Those descriptions are very important since, presently, only poor traces of the Tower could be found (Fig.5). Different scholars used these notes to develop their Tower images /3/ (Fig. 7).

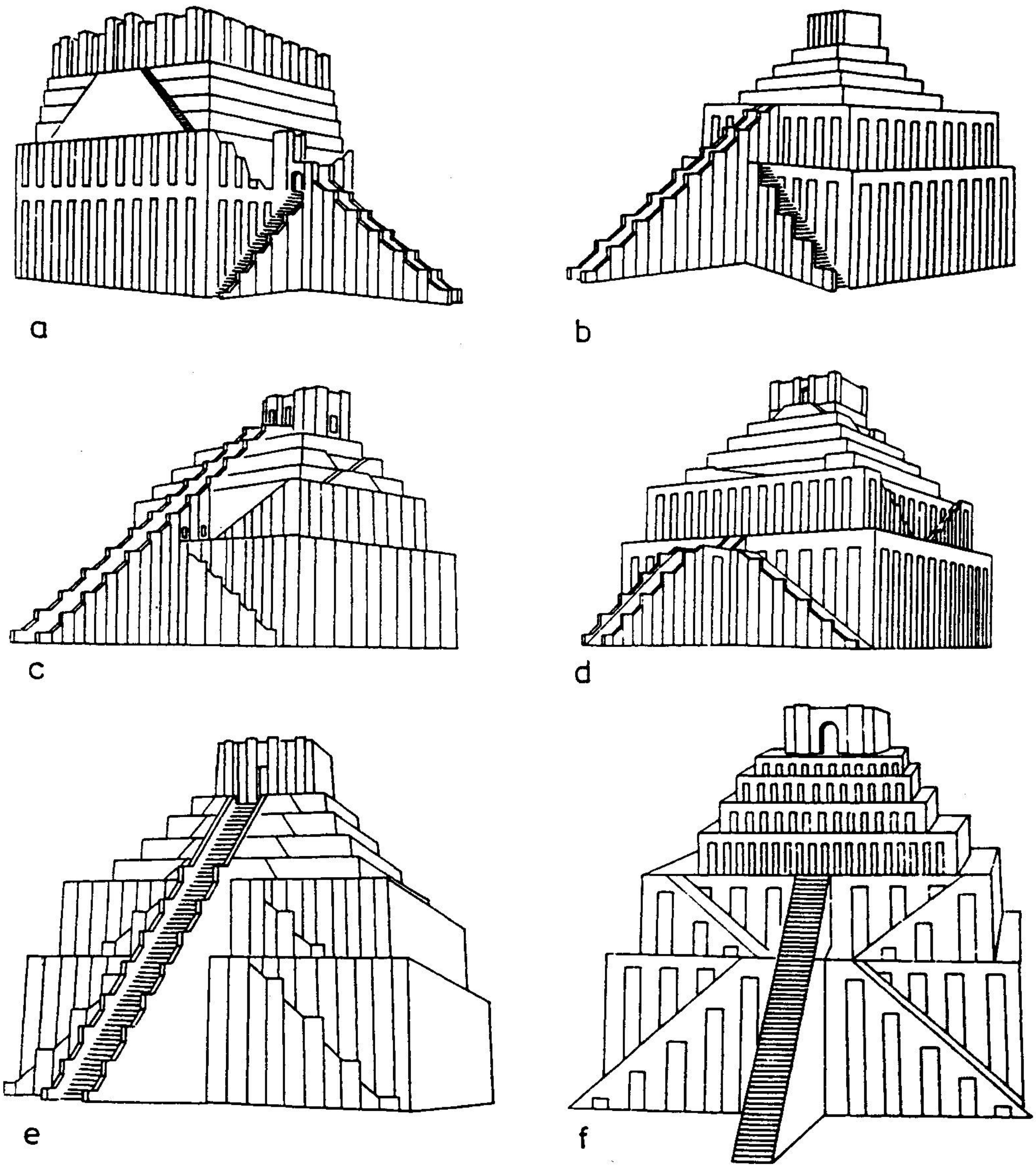

Fig. 7 Reconstructions of the Tower of Babel - by
a - Koldewey /2/ b - Dombart /2/
c - Andrae /2/ d - Busink /6/
e - Moberg /2/ f - Unger /2/

Using the original data of the E-sag-ila Tablet and applying the conversion of ell-dimensions into the metric ones, the most probable configuration of the Tower can be developed, as seen in Fig. 8 /3/.

The E-sag-ila Tablet notice about seven stories of the Tower is not conform with that of Herodotus, who reported eight (he must have regarded the foundation or one part of the double level top temple as separate stories), but both notices agree that E-temen-an-ki's length, width and height dimensions were equal. However, Herodotus, and later Strabon too, reported those dimensions to match one stadion (192.27 m - Olympic or 177.35 m - Delphic), what was just twice as much as in the Tablet. Since the Tablet data harmonize with those of Esarhaddon (180 ells = 91.55 m), and because they are conform with the findings of archaeology, author accepted the Tower dimensions as shown in Fig. 8 /3/.

The structural volume of the established shape was also point of research. According to /2/, the mature construction of E-temen-an-ki was that given by the vertical section of Fig. 8 a. Author of this report postulates the Tower structure to represent a co-operating composite of two main parts (Fig. 8 b):

- the square container like enveloping wall, constructed out of baked bricks connected by asphalt - being, according to the sexagesimal counting system of the Babylonians, in thickness one sixth of each story length, and
- the cased inner shaft made out of adobe and debris of former constructions, together with local sandy clay soil - everything with layers of reed inside.

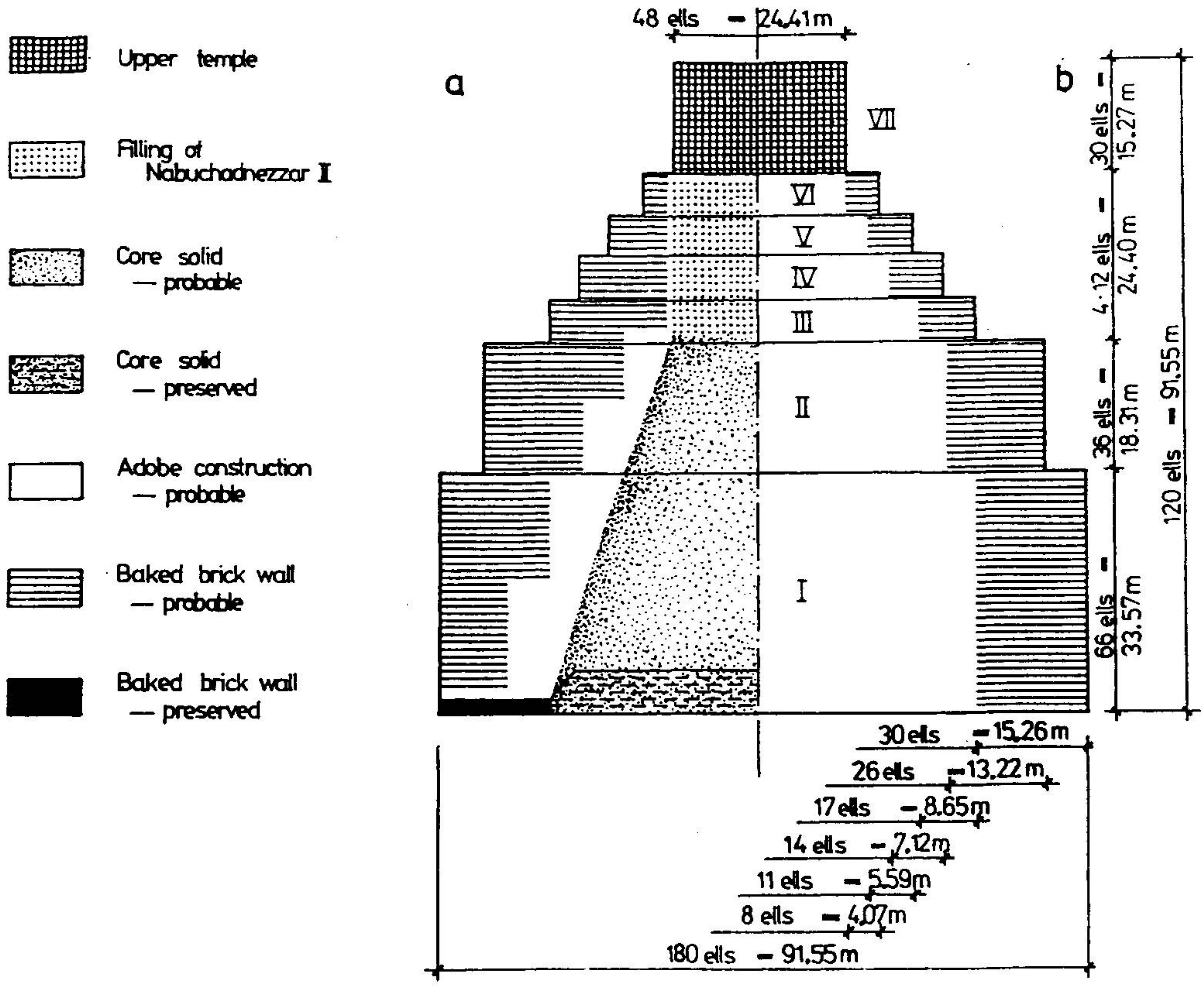

Fig. 8 Structural characteristics of the Tower of Babel - by a - /2/ b - author

5. ANALYSIS

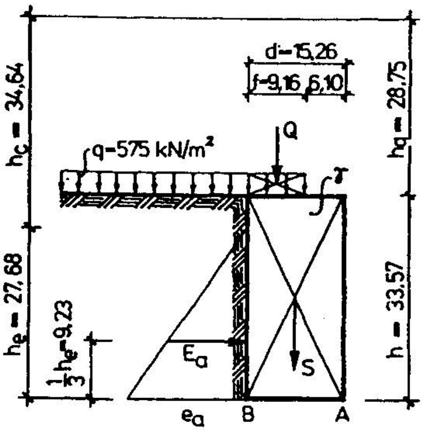

Fig. 9 Tower of Babel mantle wall analysis /3/

In order to approach the eventual reconstruction of the Tower of Babel, some basic technical parameters of its original construction should be determined. The main specification concerns the normal stresses (vertical pressures) σ at the bottom levels of the particular stories I-VII (Fig. 8 b) of the Tower. Assuming the volume specific gravity of its structural material as $\gamma=20$ kN/m^3, the stresses σ, supposed to be uniformly distributed, are the following /3/: $\sigma_I=305$ kN/m^2, $\sigma_{II}=273$ kN/m^2, $\sigma_{III}=291$ kN/m^2, $\sigma_{IV}=319$ kN/m^2, $\sigma_V=353$ kN/m^2, $\sigma_{VI}=575$ kN/m^2, $\sigma_{VII}=1103$ kN/m^2.

Another check should be referred to the ziggurat mantle wall (Fig. 9) /3/ - its strength and stability. The normal stresses at the bottom of this important structural element can be developed using the conservative system of a retaining wall, taking into account the dead load in question and the horizontal pressure of the inner shaft material. The concentrated vertical loads are the following:

$$P=Q+S=(q\cdot f+\gamma\cdot h\cdot d)1.00=5267+10246=15513 \text{ kN}.$$

The assumed angle of internal friction $\phi=30^o$ yields the factor of active earth pressure /11/

$$K_a=\tan^2(45^o-\phi/2)=1/3.$$

The depths of the fictitious soil layers, representing the surface loading q and the taken cohesion c=200 kN/m /12/, are

$$h_q=q/\gamma=28.75 \text{ m}, \qquad h_c=2c/\gamma\sqrt{K_a}=34.65 \text{ m},$$

respectively. Accordingly

$$e_a=\gamma(h+h_q-h_c)K_a=185 \text{ kN/m}^2, \qquad E_a=0.5\cdot e_a\cdot h_e\cdot 1.00=2560 \text{ kN}.$$

Thus, the bending moment adequate for the rectangular unit bottom of the wall, is

$$M=E_a(h_e/3)-Q(d-f)/2=7565 \text{ kNm}.$$

The related cross-sectional area A and modulus of section Z are:

$$A=d\cdot 1.00=15.26 \text{ m}^2, \quad Z=1.00\cdot d^2/6=38.81 \text{ m}^2.$$

Correspondingly, the compressive stresses at the base edges A and B amount to

$$\sigma_A=(P/A)+(M/Z)=1212 \text{ kN/m}^2, \qquad \sigma_B=(P/A)-(M/Z)=822 \text{ kN/m}^2.$$

Besides that, the Tower wall stability, i.e. rotation about A and sliding along AB (Fig. 9), should be verified. Concerning rotation, the resisting and disturbing moments are, respectively:

$$M_r=S(d/2)+Q(d-f/2)=134429 \text{ kNm}, \qquad M_d=E_a(h_e/3)=23629 \text{ kNm},$$

and the related safety factor is

$$n_{rot}=M_r/M_d=5.69>\min.n_{rot}=1.25.$$

With the coefficient of friction $\mu=0.3$, the safety factor of sliding is

$$n_{sld}=\mu P/E_a=1.82>\min.n_{sld}=1.10.$$

Special interest should be paid to the settlements of the Tower, referred to the bottom of its lowest story /3/. Applying the central-point-method of /13/, with $\gamma=20$ kN/m^3, a=b=91.55 m, q=1103 kN/m^2 and the original oedometric modulus $M_o=50$ MPa, the maximum (uniform) settlement amounts to $\delta=1.034$ m, when neglecting the soil consolidation due to the Tower's repeated de- and construction in the past.

6. CONCLUSIONS

The performed engineering analysis of the supposed original structure of the Tower of Babel results in the following conclusions:

- The ratios of the stepwise shape did properly meet the option of equal own weight pressures only at the bottoms of stories VII to III; these pressures of ca. 300 kN/m^2 matched the safe bearing pressure for stiff clays /11/.
- The maximum own weight pressure of 1103 kN/m^2, found at the bottom of story I, considerably exceeds the safe bearing pressure of the local subsoil. Therefore, within ages, the calculated settlements of ca. 1 m must have occured. Thus, the customary consolidation of the ziggurat core structures, by reinforcing reed mats preventing structural disintegration, is understandable.
- The pressures at the foot edges A and B of the enveloping wall, amounting to 1212 and 822 kN/m^2 respectively, are also far above the safe bearing pressure of the local subsoil. Accordingly, in addition to the usual deflection associated with that of the core structure, the walls had a tendency of an outward rotation. Thus, the customary anchorage of the ziggurat walls, by wooden ties inside the trunk of the core structure, is understandable.
- The stability requirements regarding possible rotation and shift of the enveloping wall, in the conditions of rigid subsoil, were fairly satisfied. Thus, statically considered, the structural composition of the ziggurat core (adobe) and enveloping wall (baked bricks joined by asphalt), was safe.
- As stated, the settlements must have been very large and they could be met only by safeguarding the coherence of the ziggurat structure as a whole.

Concerning the anticipated rebuilding of the Tower of Babel, author suggests:

- a deep foundation to be built as a solid plate resting on piles, shafts or caissons, chosen according to the results of a reliable subsoil investigation;
- the properly shaped ziggurat structure to be made in box-type reinforced concrete, lined up by brickwork of high quality, well integrated with the former.

Such technically mature idea would, simultaneously, allow to utilize the closed space for the living needs of to-day.

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Restoration Process of the Fronteira Palace

Restauration du Palais Fronteira

Instandsetzung des Fronteira-Palasts

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SUMMARY

The 'Palácio Fronteira', located in Lisbon, Portugal, is one of the best examples of late XVII century Portuguese architecture. The magnificent ensemble of buildings and gardens were designed under the influence of Italian mannerism. The north loggia of the main building showed extensive deterioration due in part to damaged roof tiles. The rehabilitation procedure included the strengthening of the exterior facade, masonry walls and the installation of a steel roofing structural system.

RÉSUMÉ

Le 'Palácio Fronteira' situé à Lisbonne, Portugal, est un des plus remarquables exemples de l'architecture portugaise de la fin du XVIIème siècle. L'ensemble architectural grandiose des bâtiments et des jardins a été conçu sous l'influence du style maniériste italien. La loggia nord du bâtiment principal présentait des signes généralisés de détérioration, dûs à l'infiltration des eaux de pluie par la toiture. La technique de restauration utilisée pour la façade extérieure a été étendue aux murs porteurs en maçonnerie de pierre et comprenait l'installation d'une nouvelle toiture en structure métallique.

ZUSAMMENFASSUNG

Der 'Palácio Fronteira' in Lissabon, Portugal, ist eines der bemerkenswertesten Beispiele portugisischer Architektur des ausgehenden XVII. Jahrhunderts. Das grossartige architektonische Gesamtwerk aus Gebäuden und Gärten wurde unter dem Einfluss des italienischen Manirismus konzipiert. Die nördliche Loggia des Hauptgebäudes wies starke Schäden auf, weil durch das undichte Dach Regenwasser eingedrungen war. Die Instandsetzungsarbeiten umfassten eine Verstärkung der Aussenfassade der Mauerwerkswände sowie die Verstärkung des Daches durch eine Stahlkonstruktion.

1. THE HISTORICAL MONUMENT

1.1 The Origins

The initial hunting lodge located at the Monsanto park foothills in Benfica was far from the bustling XVI century city of Lisbon. When in 1667 the Field Marshal D. João de Mascarenhas (1632-1681), Count of Tôrre, asked his friend the Regent Prince D. Pedro to wait for a "little while" invitation to a hunting party, so that the Field Marshal could improve the hunting lodge building conditions at his Domain of "Morgado Novo" at São Domingos de Benfica, no one expected that such a magnificent architectural example would arise.

The earliest date found in the buildings - 1584 - engraved over one of the Chapel door lintel may correspond to some early construtction works undertaken. Tradition says that St. Francis Xavier celebrated his last mass here before departing for India in April 1541. The construction lasted probably from 1667 (or 1668) to 1675 and one of the earliest reports on the building construction by the Marquis Filippo Corsini stated that the Italien Prince Cosme de Medicis visited the site on February 7, 1669, - "a magnificient house is under construction in Lisbon outskirts" (*"si va al presente fabbricando..."*), (AZEVEDO [1]).

After the tremendous 1755 Lisbon earthquake, the Marquis of Fronteira's main palace in the city was completely destroyed and the Marquis D. Fernado de Mascarenhas sought shelter in his palace in Benfica, Fig. 1.

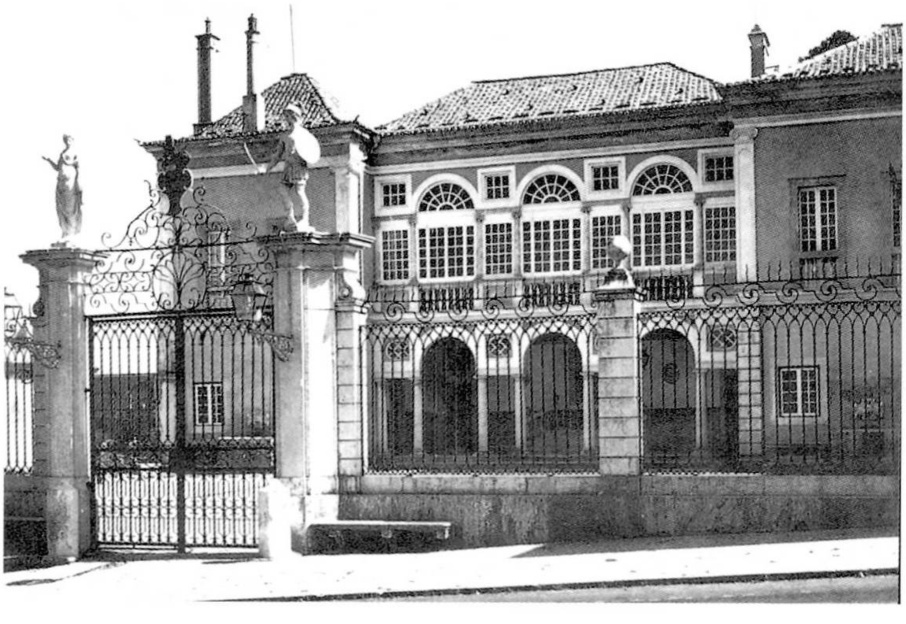


Fig. 1 - The Palácio de Fronteira in Benfica, (1988) Lisbon, Portugal. (AZEVEDO [1])

Important construction projects were completed in the following years in order to accomodate the family and servants. During the XVIII century, a new west wing adjoining the main north loggia facade and the entrance square yard were built. In the second half of the XXth century the city of Lisbon expanded and enveloped the Palace, leading to heavy traffic passing close to the Palace daily on roads never intended for such use. The vibrations caused by the traffic were very hard on the Palace and maintenance consisted only of superficial repairs such that the north loggia facade was near a state of collapse, when an extensive rehabilitation program was begun in the late 1980's.

1.2 The Architectural Compound

Although the Architect´s name is unknown, the main building facade design was probably inspired either by the engravings of Rubens "folio" of the Palaces in Genoa which shows the Villa Sauli by Galeazzo Alessi in 1555-1556, (KUBLER [2]), or on a well known drawing by the Italien architect Sebastiano Serlio, (AZEVEDO [1]). Clearly inspired by the Mannerist style, the north loggia main facade is slightly recessed from the two side towers. The ground floor triple arcade with separated columns supports a loggia with a similar first floor triple arcade supported by marble columns, Fig. 2.

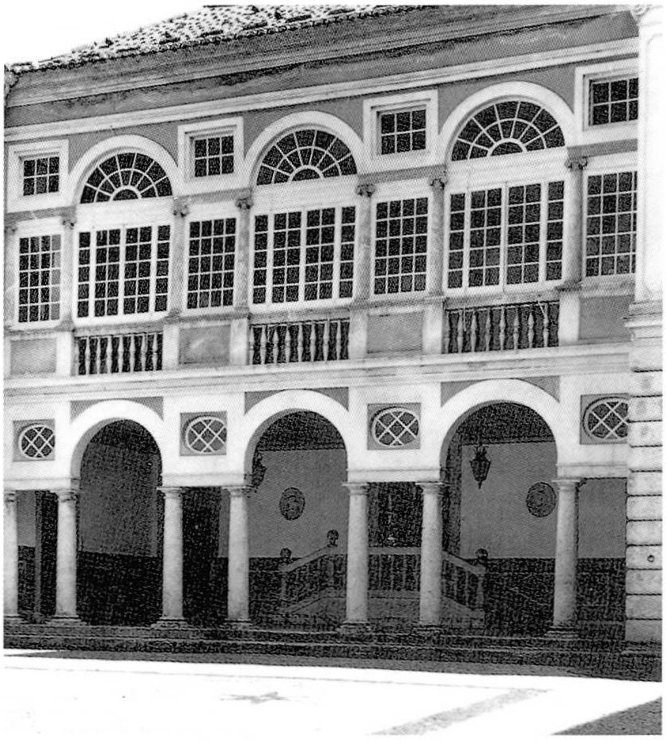


Fig. 2 - The North Loggia Main Facade. (AZEVEDO [1])

Although Alessi's building facade was richer than the prevailing Portuguese taste, the architect simplified the building's square - shaped layout by projecting simple tower-like elements from the four corners of the palace block. The whole building became a more intricate composition of volumes, (KUBLER [2]).

The rich decoration with its glazed ceramic tiles, the "azulejos", makes it one the most interesting palace in Portugal. Particularly interesting are the Battle Room "azulejos" which contain a remarkable series of battle scenes portraying the military history of the Restauration War (1640-1667), e.g., the battles of São Miguel (1658), Elvas (1659), Ameixial (1663) and Montes Claros (1665), Fig. 3. In these battles which guaranteed the Independence of Portugal from Spain, until the peace treaty signed in 1668, the First Marquis de Fronteira, D. João de Mascarenhas, is depicted in battle scenes where he distinguished himself with particular bravery.

a. "Azulejos" Panel

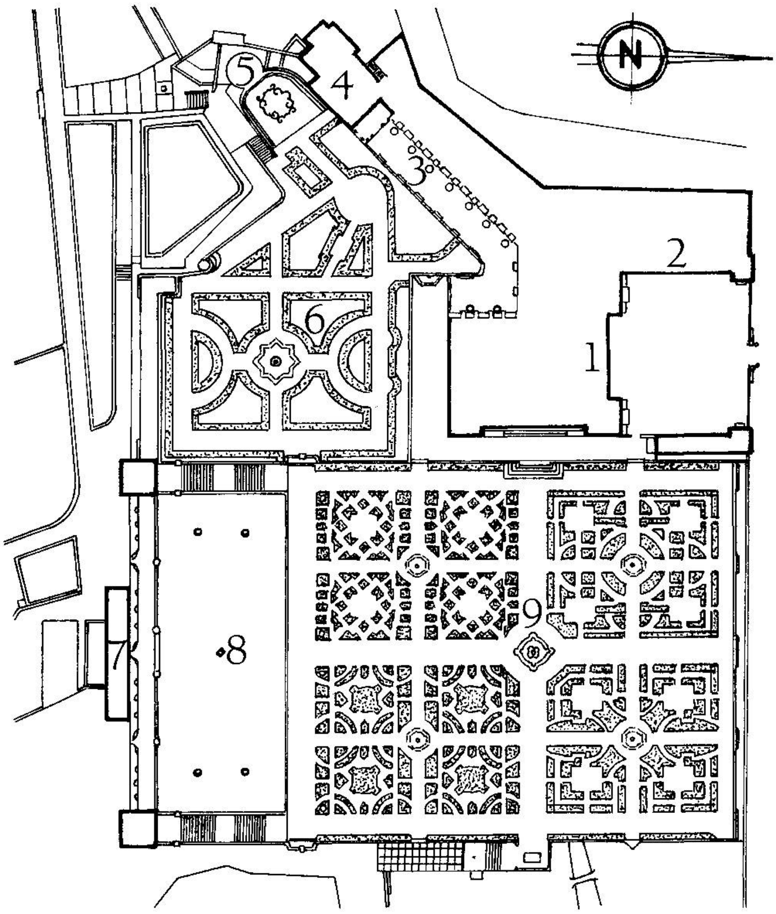
b. Detail

Fig. 3 - Battle Room "Azulejos" Panel.

The building's architectural design shows the clear link between the Portuguese Restoration taste and the North Italian models of Mannerist style.

1.3 The Gardens

The beauty of the gardens, strongly influenced by the Italian style, with the elegant building design creates a magnificent architectural compound built in an epoch in which noblemen took greater pleasure in wars than in arts, (CASSIANO NEVES [3]), see Fig. 4, (QUIGNARD and MECO [4]).



LEGEND:

1- Main building north loggia;
2- XVIIIth century west wing;
3- Statues verandah;
4- Chapel;
5- "Casa do Fresco"
6- Venus Gardens;
7- Kings Gallery;
8- Great Lake;
9- Classical Garden.

Fig. 4 - The Palácio Fronteira and its Gardens (GIL [5], CARITA and CARDOSO [6].)

The building's surrounding gardens and lakes are an elegant solution to a delicate problem, i.e., to create an enjoyable environment with enought water supply during the summer months. A series of descending water reservoirs (supplied by the nearby Monsanto Hills underground drains) starting at the "Casa do Fresco" (5), and continuing down to the Great Lake (8) are used to water the Venus Garden (6) and the Classical Garden (9), Fig. 4.

Adjacent to the Classical Garden is the Kings Gallery (7) with the remarkable marble statues (first floor) and the "azulejos" panels depicting twelve knights and two Marquis of Fronteira. The spatial effect created by the reflection in the Great Lake pond strongly enhances the garden, the Kings Gallery and the Palace building ensemble, Fig 5.

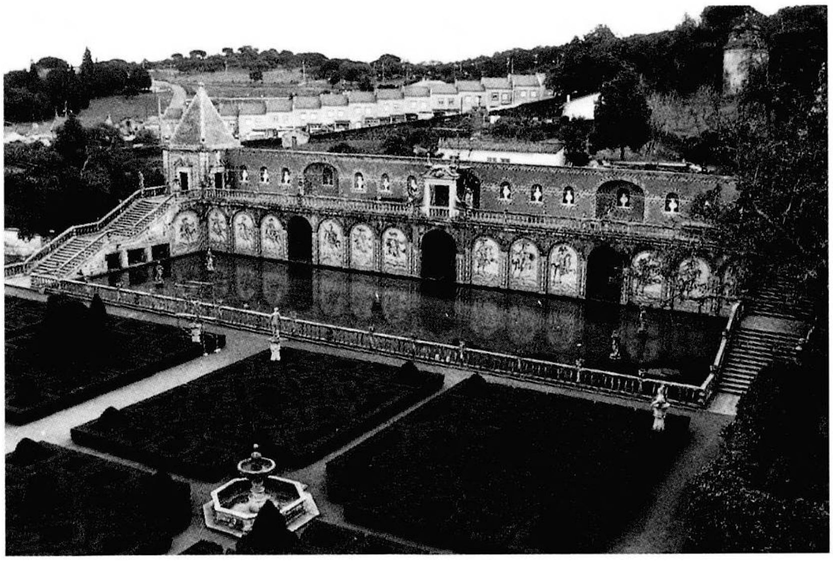
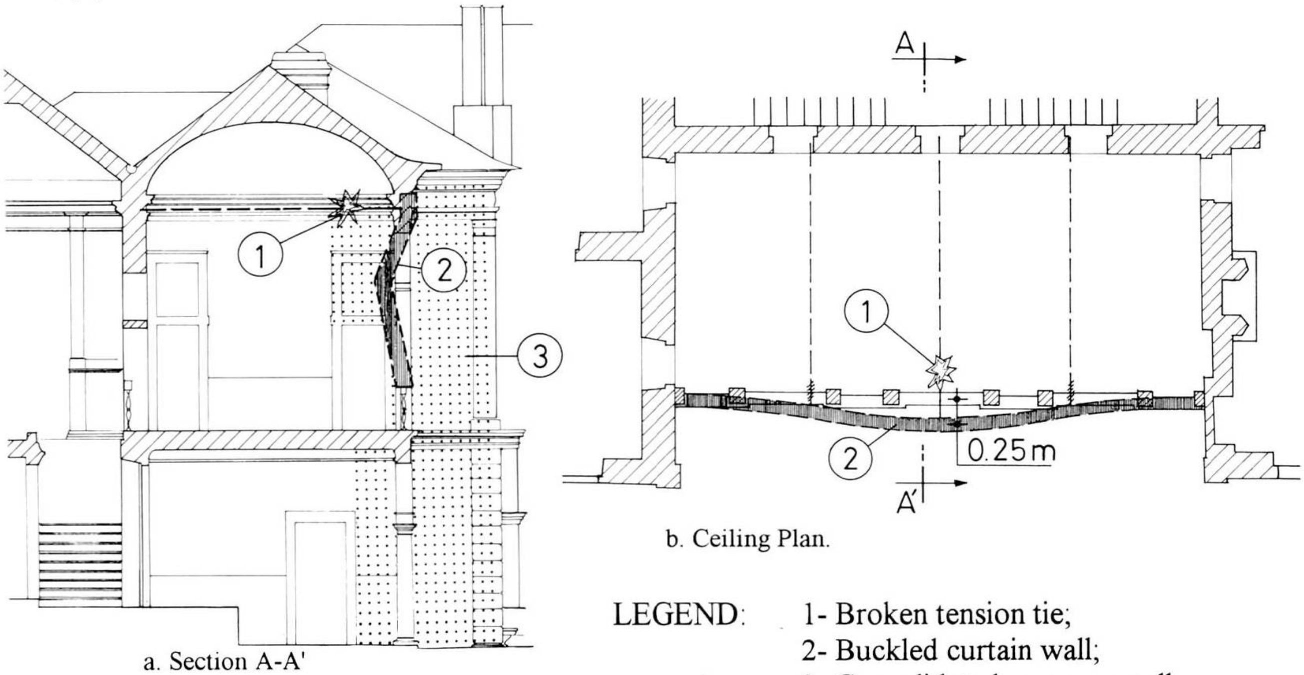


Fig. 5 - The Kings Gallery and the Great Lake

2. THE STRUCTURAL RESTORATION PROCESS

2.1 The Near State of Collapse Diagnosis

The site report of the Summer of 1988, showed that the North Loggia facade was on the verge of collapse. The first floor marble columns displayed a horizontal inward displacement which reached 0.25m at the facade midspan. The cornice had horizontal outward displacement of 0.25m, Fig. 6. The facade's generalized bent shape resulted from: (1) a midspan ruptured iron tension bar; (2) the continuous debris falling from the roof the inner plaster painted shell ceiling adjacent to the facade; and, (3) the roof truss deterioration in the wood beam connnections.



a. Section A-A'

b. Ceiling Plan.

LEGEND: 1- Broken tension tie;
2- Buckled curtain wall;
3- Consolidated masonry wall.

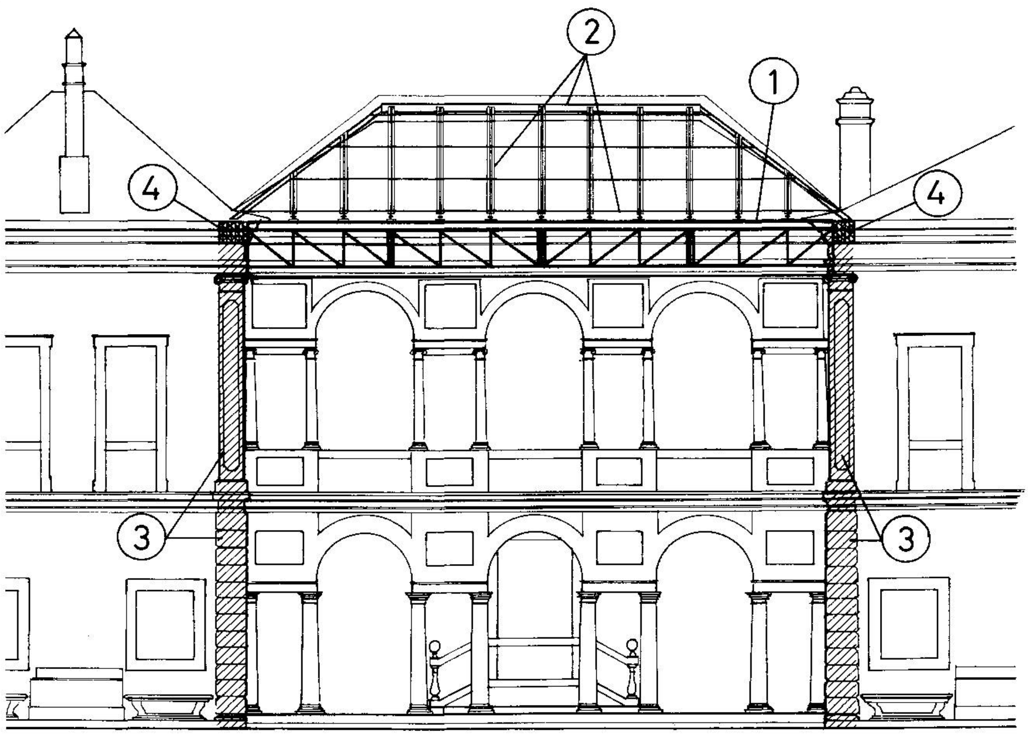
Fig. 6 - North Loggia Facade Collapse Mode.

The existence of heavy compression loads on the 11.50m long "curtain" open wall (4.0m height and 0.25m thick) support system created the ideal conditions for the 2.0m tall marble columns to buckle. The bracing provided by the iron bar ties with a 35mm x 55mm section disappeared when the rainwater, (infiltrating through cracked roof tiles and opened masonry joint), corroded the anchorage eyebar, Fig. 6. The wood roof beams also showed extensive deterioration mainly in the connections at the creast (broken roof tiles) and at the edge supports. Consequently, the bracing effect that once existed throught the roof trusses slowly vanished as rotten wood connections developed.

2.2 The Recommended Structural Solution

The site visit (Summer 1989) showed an extensively damaged shored roof area in which particular care was taken to preserve the inner ceiling's plaster shell paintings. The ovelying roof debris was removed and the masonry clay brick cornice was disassembled, after the moulds and contours were recorded. The recommended structural retrofitting solution consisted in creating a huge portal frame made of a 11.60m long longitudinal steel truss girder and two 0.60m x 3.00m x 10.0m masonry stone columns. The existing wood roof structure was suspended through a second roof steel system installed with hinged connections to the longitudinal beam to avoid roof bending moments.

The objectives with this recommended structural solution were: (1) to reduce the cornice dead load on the columns; (2) to tranfer the roof loads applied on the exterior thin wall to the thicker side walls via the longitudinal steel girder; (3) to provide a continuous spatial bracing between the U-shaped masonry wall envelope, the open exterior "curtain" wall and the roof system; and, (4) to increase the safety factor of the existing structure, Fig. 7.



LEGEND:

1- Longitudinal beam;
2- Roof truss system;
3- Reinforced masonry walls;
4- Reinforced concrete ring beam.

Fig. 7 - North Loggia Facade - Structural Solution

In order to guarantee the overall stability careful site inspection was conducted of the masonry walls Throught several 25mm holes drilled in the walls it was verified that the existing mortar filling the joint had either disintegrated after 300 years in service (probably washed away by infiltrated rainwatwer) or was in such poor condition that had to be strengthened. A limited "*reticolo cementato*" technique was used in a 3.00m wall portion. About six hundred 1.0m- long holes were drilled, filled with cement mortar and a 8mm steel rebar. In order to reduce the possibility of steel corrosion the exterior wall was protected with cement epoxy-based mortar.The steel longitudinal beam was coated in rich galvanized zinc to protect it from corrosion and in order to match the existing building cornice, a diamond-shaped zinc coated wiremesh covered with epoxy coated mortar was used, Fig 8.

The architectural detail was easily obtained through the use of these wiremesh panels. The beam top flange was composite with concrete and made continuous with the wall reinforced concrete (RC) ring beam. On the beam top flange stainless steel hinged supports were installed to connect the steel roof truss system. Before constructing the steel roof, the wood truss connections were thoroughly repaired, the inner plaster painted shell ceiling was inspected and a steel tension tie was installed to brace the longitudinal beam, Fig. 9.

Before laying the roof tiles, the wood roofing system was suspended from the steel system with zinc-coated steel rods. Then, moisture-resistant plywood panels covered with an undulated flexible waterproofing membrane were placed on top of the steel truss, so that the roof tiles could be fixed to the structural system.

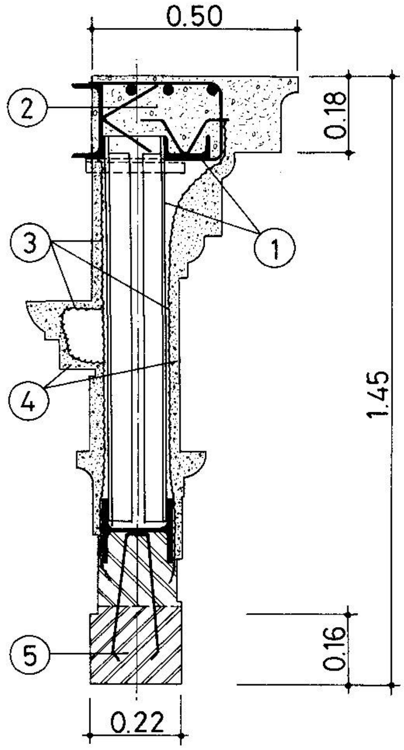


Fig. 8 - Longitudinal beam cross section.

Legend:

1- Steel members;
2- Cast in-situ concrete;
3- Steel wiremesh;
4- Epoxy coated morter;
5- Masonry arch keystone.

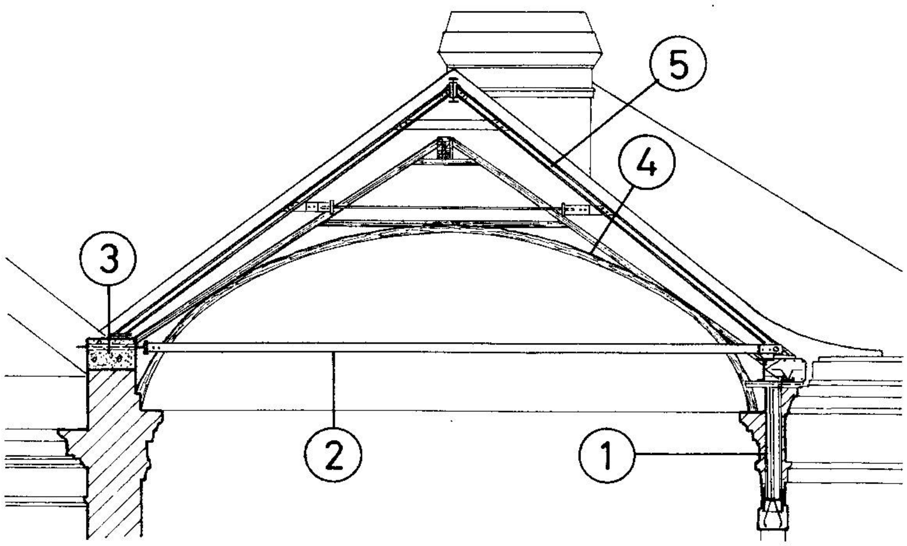


Fig. 9 - Palace North Loggia - Ceiling Cross Section.

Legend:

1- Longit. Steel Beam;
2- Tension Tie;
3- RC Concrete Ring;
4- Painted Plaster Dome;
5- Roof Steel Truss.

2.3 Project and Construction Limitations

Several important constraints related to this restoration project should be emphasized: (1) the use of a structural steel system was difficult because the need to fit the system into an existing building; (2) the difficulty of quantifying load actions, particularly of the material's dead load (e.g. the "moorish" roof tile system); (3) the need to use no-readily available materials (e.g. the facade steel wiremesh; epoxy-coating mortars) and to specify custom-made structural elements; (4) matching three hundred year old building techniques with the vast array of "high-tech" materials available nowadays; and, (5) defining a satisfactory factor of safety while ascertaining the existing safety conditions.

3. CONCLUSIONS

The "Palácio Fronteira" is one of the most well-known examples of the XVII century Portuguese domestic architecture with its buildings, gardens, lakes and the magnificent "azulejos".

In 1988, the main building's north loggia facade was near a state of collapse when a general restoration program was implemented. The strengthening of the existing stone masonry walls with the introduction of a structural steel system (longitudinal beam and roof trusses) was deemed necessary. Particular care was taken to maintain the existing building conditions and to make the new structural system compatible with that of the original Palace.

The use of a structural steel system (longitudinal beam and roof trusses) coupled to a RC ring beam was considered adequate to meet both architectural and engineering requirements. With this solution, the stability of the slender "curtain" wall in the building north loggia facade would be achieved.

4. AKNOWLEDGEMENTS

The author wishes to express his sincere thanks to Mrs. Lynne Archibald, Dr. Fernando de Mascarenhas (Marquis de Fronteira) and the Fundação das Casas de Fronteira e Alorna, The Faculty of Architecture-Tech. Univ. of Lisbon for the assistance provided in preparation/presentation of this work.

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Case Studies of Structural Preservation in Taiwan

Conservation de constructions à Taiwan

Fallstudie einer Tragwerkserhaltung in Taiwan

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SUMMARY

Owing to the traditional custom of dividing Taiwanese family property upon succession, it is very difficult to maintain the original old buildings for more than three generations. Even if not discarded, they would not be kept intact in the process of settling property ownership. Old houses are thus often left in disrepair. Due to the economic growth and urbanization of recent years, most ancient buildings have been destroyed. However, some have been removed and reconstructed. Thus it was possible to dismantle and inspect them. This paper presents some removal and reconstruction projects of old chinese style dwellings.

RÉSUMÉ

Dû à la tradition à Taiwan de répartir la propriété familiale lors d'une succession, il est difficile de maintenir d'anciens bâtiments dans l'état original pour plus de trois générations. Même s'ils ne sont pas détruits, ces bâtiments ne restent pas intacts lors de la répartition de la propriété. Souvent les vieilles maisons sont laissées dans un état délabré. Suite à la croissance économique et l'urbanisation récentes, maints anciens bâtiments ont été détruits. cependant, certains ont été déplacés et reconstruits. Ceci a permis de les démonter et de les analyser. L'article présente quelques projets de déplacement et de reconstruction d'anciennes habitations de style chinois.

ZUSAMMENFASSUNG

Weil gemäss der taiwanesischen Tradition der Familienbesitz unter den Nachkommen aufgeteilt wird, ist es schwierig, alte Gebäude über mehr als drei Generationen hinweg in ihrer ursprünglichen Form zu erhalten. Selbst wenn nicht zerstört, überstehen sie den Besitzerwechsel nicht unbeschadet. Alte Häuser werden deshalb oft nicht unterhalten. Aufgrund des raschen wirtschaftlichen Wachstums und der Überbauungen der letzten Jahre wurden viele altertümliche Gebäude unwiderruflich zerstört. Einige werden jedoch abgetragen und an einem anderen Ort neu aufgebaut. Dies ermöglichte es, sie zu zerlegen und zu analysieren. Dieser Artikel stellt Umzugs- und Wiederaufbauprojekte altchinesischer Wohnhäuser vor.

1 INTRODUCTION

Time has developed the craftmanship and style of the construction but also destroy their production. Of course, those famous palace and castle represents the development of culture and collect the way of the life. Thus are well preserved. The old homestead are mostly simple and crude. They are disappeared or being evolve to form a new hermitage. People do not consider as their cultural property need to be conserved. Most of the Chinese homestead are the multiple winged compound or courtyard building which we called it as "Ho Yuan" (Fig.1). This kind of courtyard compund survived more than 3000 years in China and house an entire extended family for the empire as well as for the clan. Besides the hardware and shape, traditional ho-yuan also extolled a natural, layered and shifting spacial notion. Could this be applied in a vital, living way to modern architecture?

Most rural large-scale ho-yuan have not been so fortunate. With changes in the social structure, young people are flowing out to work or study in the cities, and the population has been continually outmigrating. Except for two or three of the households, most of the rooms have only a steel lock to keep them company. If tiles fall off, or pillars corrode, both traditional building materials and craftsmen are rare and costly. With no one looking after it, the whole structure just falls apart, until there is no alternative but demolition.

Another problem is the arguments on the questions whether the homestead should be torn down or not, and if the answer is "yes" whether it should be reconstruct on a new site with the way that the original structure and appearance can be preserved almost to its perfection.

In this paper we take some examples of the removing and reconstruction of the homestead, most of them are wooden structures used for Chinese immigrant clan to Taiwan or the aboriginal cottages, for case study (Fig.1,2). The way of study are thus:

First: to study the face that family life is central in traditional Chinese way of living, that it is linked closely with cultural life, and that traditional Chinese architecture reflects this humanistic social structure. In the face of rapid social changes, however, we should hold a balanced view toward the measures of preserving our cultural heritage.

Second: to make a brief analysis of the historical background of the owners, so that we may gain a better knowledge of the development of the dwelling. To offer a basis for a comparative study of the Homestead.

Third: to introduces in detail the arrangement, special structure,

decoration, enqraving, etc., and summarizes our views about these features.
Fourth: to treat the operation of dismantling and removing.
Fifth: to describe the project for reconstructing the old Homestead.

2 GENERAL DESCRIPTION OF THE STRUCTURES

A. The wooden framework is fitted together with hidden tenons and mortises, and nails. The nail heads are completely covered; apparently most of the nails are made of bamboo. The round gilded pillars are drum-shaped. The square eaves pillars are united smoothly with the stone pillars. The cross beams and purlins are all built with superb craftsmanship. The wooden wedges and the nail heads are all well covered. The locks of the main door are elaborately made double, with one explicit and the other impplicit (Fig.3).

B. The stone carving part: The stone materials are all connected by using hidden dovetail keys, and the stone materials and the wood materials are also connected by the use of tenons and mortises. The same is true of the doorposts, wainscots, stone thresholds, and pillar bases. With ordinary houses, the stone hinges and the doorpost pedestals are two separate parts, but in the Homestead they are carved as an integrate piece. The same skill is applied to the stone hinges at the bottom. The stone steps, stone wall bases and the friezes are all well proportioned in their dimensions. The igneous stone used in paving the inner court, the open drain, and the front court, though not carved very exquisitely, are well selected, and their sizes are all very accurated. Owing to their old age, the stone slates are no longer very even; but the crevices were adjusted by using copper coins as tools, and ground leveling was secured by melting lead underneath. These all show the great care taken in its construction (Fig.4).

C. The bricks and tiles: The ridges of the old Homestead are in one single ridge style. The ventilating bricks in the middle of the ridge are double-sliced chute, in the wicker ventilating window style. The tiles on the roof are in the southern Fukien style with a pitch of three to ten. Beneath the tiles are wooden rafters for nailing. The ventilating holes at the midtop of the gables are tracery made of unglazed bricks. The stone place on top of the doorways in the side buildings also have latticed ornamental bricks to give vent to the air within. This not only is logical but also forms different vertical surfaces that add to the beauty of the

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Fig. 1 Old homestead (Fu Kien style)

Fig. 2 Bamboo house of an aboriginal

Fig. 3 The wooden Framework

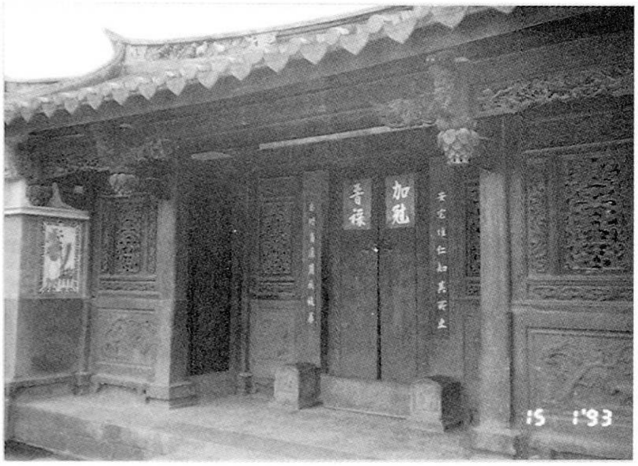

Fig. 4 The stone carving part

Homestead.

Three types of bricks laid horizontally in the facade of the lower building are in the southern Fukien style. Those on the rear part of the building are clay bricks. The face brick walls of the side buildings are engraved with tortoiseshell patterns. These probably resulted from a later addition or repair work. The stone slates below the stone waist piece are rather elaborately engraved in the pattern of tables. The entire facade of the dwelling looks neat and handsome. Although its altitude is relatively low, there is a sense of equilibrium (Fig.5).

D. The fine wood carving and the windows and doors: The main gate and the slide windows of the main hall are carved hollow. The ornamented waist panels design and engraving techniques are different from one another. The wainscots and waist panels of the doors and bar windows are all carved in one whole piece. The bar windows are of various designs, and since most of them are composed of many single pieces, they are fragile and not easy to preserve. Most of them have decayed and became loose which makes them rather difficult to repair after they are dismantled (Fig.6).

3 SOME REMARKS ON DISMANTLING AND REBUILDING THE STRUCTURES

As there is a relative lack of experience in preserving cultural property in our country, we maintained a very careful, faithful attitude when we actually engaged ourselves in dismantling project. We recorded down every bit of working experience as we went along, so that the record may be referred to constantly in our future preservation of the property. Before we removed it from the Old Homestead, we had measured, recorded, and numbered every piece of brick, stone, and wood; and then we drew the details to make blueprints for reconstruction of the Homestead at a later time. We also took a large amount of pictures to add to supplement the blueprints. After a minute investigation of the structure of the old Homestead, we reached the conclusion that, like other old buildings in general, it was constructed in the routine order of framework first, then the partitions, doors and windows, and finally the decorations. In dismantling, therefore, the reverse order had to be followed; that is, we had to remove the embellishments and accessories first, the partitions or screen-walls next, and then the dismantling was to continue from the roofs down to the foundations.

The most difficult part of the job in dismantling and reconstruction the house was locating the tenon and mortise joints. Here are some remarks on the dismantling and rebuilding of the structure:

1. The trussed girders must be taken off only after the hidden tenons and mortises have been disconnected, and the hidden nails removed. If force must be applied, it must be done lightly and under protection of padding boards. They must not be forcefully thrown down or hit by other objects.
2. The components that are to be taken off later and the parts too fragile to be taken off first are to be protected by coverings. Those objects that may fall apart are to be bound together beforehand.
3. In dismantling the wooden plugs and partitions, they must first be bound up, then a crane or windlass must be applied to remove the burden, and finally the hanging bar has to be put in horizontal position. Only after these precautions are taken, the actual dismantling can begin.
4. Before engraved portions of the doors and windows are taken off, they must be fixed with boards and other wooden materials to ensure that they will not be distorted or disorganized.
5. Before the clay, stone pieces, bricks, and tiles are taken off, they should be watered continuously for four hours to give them moisture, and then they are to be taken off piece by piece with a pitching chisel. They should never be pushed down to be chiseled off. After being taken off, they must be protected by being wrapped with cotton cloth and straw sacks.
6. Engraved stone piece should be protected by being wrapped with straw ropes and sacks and cotton cloth before they are taken off.
7. In order to make the reconstructed old Homestead endure, we have already mapped out plans for reinforcing its structure. Instead of spoiling the original style of the residence, they will preserve the original to a long future to come.
8. The original method and procedure of construction are to be followed so that the unique style of the original Homestead may be preserved. The procedures are rather complicated, but they are still worth trying in order to preserve the antique flavor. Besides, although residence was built nearly two hundred years ago, a careful examination reveals that these walls do not have the slightest signs of decay. This testifies to the high value of the engineering methods used in building this residence.

4 CONCLUSION

Architectural works of historical value are handiworks with stylistic characteristics. The art craftsmanship itself may be irrevocably lost if the architectural remains were wilfully destroyed. Special care must be taken in the process of reconstruction or repair.

The most difficult part is the roof trusses, the beams and columns. Most of the roof trusses and beams in the gables have decayed; they have to be replaced by those made from wood material of the same (or quite close) quality. As for the acquisition and replacement of other building materials there do not seem to have much difficulty.

In carrying out its reconstruction, we have to take care that the specifications and appearance of the materials are the same as the originals and that preventive measures be taken against humidity and termites. As the older generations of building technicians gradually retire from the scene, how do we bring new generations to succeed them--to continue and glorify their traditional building techniques?

Home is where the heart is. In this close-it-off add-it-on culture where there's always one room too few, all the semi-open spaces where one can see the sun of feel the rain - pations, rooftops, courtyards - are all being made into interior living space.

Is it that people have changed, or that architecture has? Has modern architecture really made people cold and mercenary? Or have people made buildings into places that cut off and isolate us from sentiment and reason? Whether or not the spirit of the traditional ho-yuan family compound can invigorate modern architecture is not just a problem of building materials and architects. We must survive it before they are vanished.

5 REMARK

Due to the misunderstanding of the materials used before and bad repairing the rebuilded homestead are being damped and even forming the crack on the corner of the door and window. Even the connection of the main and winged house are also cracked. The door are thus deformed. The rebuilding work are merely less than five years. The reason of those damage are mainly coursed by misuse of cement in stead of clay and gypsem. The foundation are used by concrete instead of original stone layer. These are the first presvation works in Taiwan. Experience gain from them will be good for future works (Fig.7-8).

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Fig. 5 The bricks and tiles

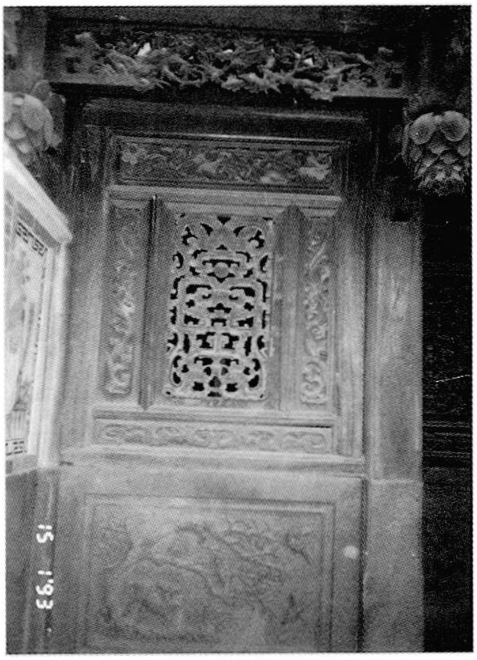

Fig. 6 The fine wood carving on window and door

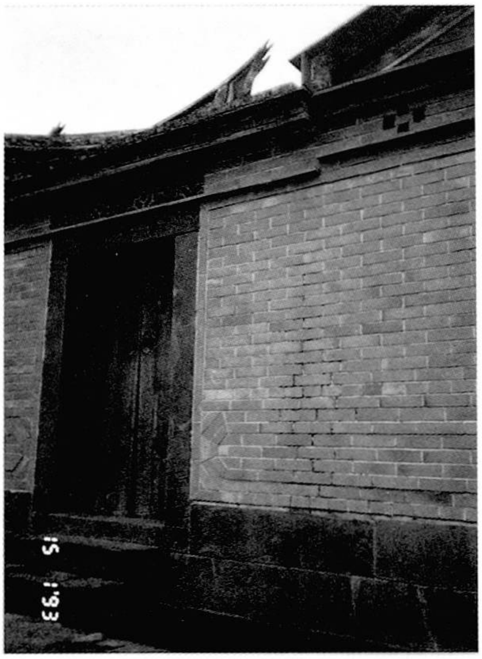

Fig. 7 Unequal settlement between main hall and wing house

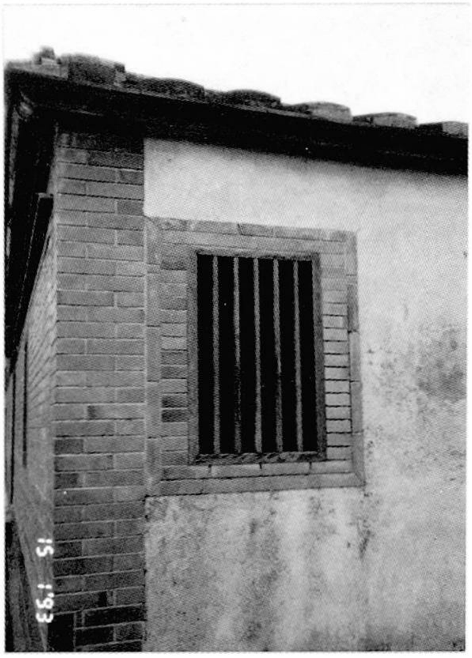

Fig. 8 The damped wall

The Study of an Old Palace Structure and its Repair

Etude de la structure d'un vieux palais et sa réparation

Studie eines alten Palastes und seine Reparatur

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SUMMARY

This is a comprehensive study carried out to diagnose and assess the reinforced concrete quality and structural safety of an old historic palace in Alexandria. The study covered different in-situ non-destructive tests carried out on concrete as well as mechanical tests carried out on steel bars to assess the materials quality and performance. Wooden trusses which represent the structural system on one of the main palace halls were structurally checked. Repair recommendations and techniques for the reinforced concrete elements and the wooden trusses have been brought up based on the corresponding study and analysis.

RÉSUMÉ

Cette étude vise à analyser et à évaluer la qualité du béton armé et la sécurité de la structure d'un palais historique à Alexandrie. Elle couvre différents essais, non destructifs, effectués en chantier sur le béton ainsi que des essais mécaniques sur les barres en acier afin d'en évaluer la qualité et la performance. Les poutres en bois, qui représentent le système structural d'une des principales salles du palais, furent examinées du point de vue structural. Les réparations et les techniques recommandées pour les éléments en béton armé ainsi que les poutres en bois, sont le résultat de cette étude.

ZUSAMMENFASSUNG

Der Artikel bewertete die Qualität armierter Betonelemente und die Tragfähigkeit eines historischen Palastes in Alexandria. Es wurden zerstörungsfreie In-situ-Versuche am Beton sowie mechanische Versuche an Bewehrungsstählen durchgeführt. Das Ziel war die Bewertung der Baumaterialien und ihrer mechanischen Beschaffenheit. Die Holzbalken, als wichtigste tragenden Elemente in einer der Haupthallen des Palastes, wurden strukturell untersucht. Aufgrund der gemachten Analysen und Versuche wurden Empfehlungen für die Reparatur des armierten Betons und der hölzernen Tragbalken abgegeben.

1. INTRODUCTION

In many countries engineers are confronted with problems of architectural heritage (such as historical buildings, monuments, bridges, etc.) needing structural repairs of retrofitting. The recent collapse of monumental buildings has shown that their vulnerability may not only be a risk of loss of a precious masterpiece but also a risk of human life. An important task for structural engineers is growing, and the corresponding knowledge and professional tools need to be developed. Extensive research work has been carried out in that field, Ref.[1-3]. The place under study is located in Alexandria and overlooks the Mediterranean Sea. It consists of two parts, the main building (the old structure) which was built more than one hundred and twenty years ago and the extension building (the new structure) which was built about fifty years ago.

2. DESCRIPTION OF PALACE STRUCTURE

2.1 The Main Building (the old structure)

This building was well maintained and in a good condition. Nevertheless, there was water leakage in one of its major halls. Figure (1) shows the layout of the hall under study. This hall is one of the most luxurious halls in the palace. The ceiling and walls contain precious masterpiece decorations. The main hall was 18 meters long and 11 meters wide. Connected to this hall from one side was an entrance hall (9 m x 7 m) and from the other side to a hall (9 m x 4.3 m) connecting it to the main throne hall.

The structural system consists of load-bearing masonry wall with a thickness of about 70 cm. The precious decorative false ceiling was hung by means of wooden trusses and beams resting on the bearing walls. The spacings between the trusses were 1.70 m, and between the trusses there was a wooden beam at the same level of the bottom chord of the trusses, as shown in Figure (1). Figure (4) shows the dimensions of a typical truss.

2.2 The Extension Building (the new structure)

The building consists of a basement, ground floor, first floor and a second floor. Figure (3) shows a general layout of the ground floor only. The area of each floor was 2000 m^2. The structural system consists of load-bearing walls and reinforced concrete slabs. Columns were sometimes used in lobbies and terraces but were not considered as a major supporting structural element.

3. VISUAL INSPECTION

From inspecting the hall false ceiling of the main building, sagging of the decorated false ceiling was noticed as well as traces of water leakage. But nevertheless, from inspecting the hall from the space between the false decorated ceiling and the roof, longitudinal cracks were quite visible in beams and trusses.

Figure (2) shows the extension building and a sample of the damaged parts. The building was abandoned for sometime and badly maintained. Traces of water leakage could be seen from water piping system at the basement, at the bathroom and at ceiling walls of the top floor (due to rain). Cracks and spalling of concrete could be seen in reinforced concrete elements, while some uncovered corroded steel reinforcement bars were detected.

4. EXPERIMENTAL TESTS CARRIED OUT ON THE REINFORCED CONCRETE

It was taken in to consideration that the tested specimen and testing locations were representative to the different degrees of damage. Figure (3) shows a sample of types and locations of such tests. Core specimens were extracted from various locations, their equivalent cube strength and chemical analysis (percentage of chlorides) are presented in Table (1). Schmidt Hammer test applications were carried out, a sample of these tests are presented in Table (2), and mechanical testing of steel bars for concrete reinforcement are presented in Table (3).

5. ANALYSIS OF DATA

5.1 The hall of the main building

The dimensions and features of the supporting trusses as well as beams were all measured from the site condition. Loads acting on the truss due to the top roof and decorated false ceiling were calculated. Stresses were checked for all critical sections and were within the permissible values.

5.2 The extension building

In general concrete is known to be a porous material. Therefore, due to the presence of harmful amounts of chlorides in concrete and with the help of moisture gained (i.e. absorbed due to water leakage, the humid environment, etc..) it would slowly penetrate the concrete. When the chlorides reaches the reinforcing steel, the natural corrosion protection barrier on the steel surface breaks down, and the corrosion process begins. Corrosion generated rust that increases the volume of the original steel. The corroding steel stresses the surrounding concrete, eventually forming cracks around the reinforcing bars. Therefore more salt and water enter as cracks reach the surface and the process feeds on itself. Tests carried out on concrete specimen summarized the following:

- Equivalent cube strength obtained by means of Schmidt Hammer rebound numbers revealed that the compressive strength was within the permissible values for such structural elements. The lower values of compressive strength at level two was mainly due to water leakage that affected the surface hardness of the concrete in that area.
- Compressive strength results based on core tests reveals a fluctuation of the concrete strength from a value of = 164 kg/cm^2 to a value of = 375 kg/cm^2. Although an increase of about 20% to such results could be made for the difference between in-site conditions and standard cube strength in the laboratory. Yet it reflects a lack of quality control applied to the concrete at the times as a whole.
- The chlorides content obtained for the different specimen representing the different floor ranged from the permissible values of = 0.205% to values of as high as 0.645% of cement weight which greatly exceeds the permissible values of 0.30% of cement weight. Hence steel bars embedded in such a concrete would be prone to corrosion attack.
- Concrete cover was removed in several locations to verify visually the effect of rust. Tests were carried out on samples of such bars. Table (3) shows some examples of poor results obtained by such tests on the reinforced bars. Reduction of steel

Core Test I.D.	Floor No.	Structural Element	Compressive Strength kg/cm^2	% of Chloride	Steel Bars Remarks
C	Basement	Cont. footing	298	0.226	
D	Basement	Cont. footing	282	0.368	
E	Basement	Slab	346	0.468	Pitting Corrosion
F	Basement	Slab	369	0.357	
G	Ground	Slab	236	0.645	
H	Ground	Slab	210	0.567	
I	Ist Floor	Slab	164	0.230	Corrosion of Steel Bars with a Visible Reduction of Diameter
J	Ist Floor	Slab	375	0.205	
A	2nd Floor	Slab	262	0.444	
B	2nd Floor	Slab	189	0.210	

Table (1) Results of Core Tests

Sample Identification	Basement Slab	Ist Floor Bathroom	2nd Floor Terrace	2nd Floor Lobby
Nominal Diameter mm	9.4	9	10.5	10.8
Measured Diameter mm	9.31	8.95	10.17	9.45
Yield Strength kg/cm^2	2653	3680	2542	2119
Tensile Strength kg/cm^2	3575	4246	3281	2796
Elongation %	9.5	14.5	10	29
Remarks	Minor Corrosion	Corrosion & Reduction in Area	Severe Corrosion	Minor Corrosion

Table (3) Mechanical testing of steel bars

Test No.	Floor No.	Structural Element	Average Rebound Number R.N.	Equivalent Cube Strength kg/cm^2
32	Basement	Slab	41.9	397
33	"	Beam	33.1	311
6	Ground	"	41.0	380
2	Ist Floor	"	36.9	299
9	2nd Floor	"	27.7	143

Table (2) A sample of the Schmidt Hammer (R.N.) tests diameters were quite visible especially in parts where pitting corrosion was observed. In general, pitting corrosion was obtained in bars embedded in concrete of the basement and ground floors where a large amount of Cl% was obtained by means of the chemical tests. While uniform corrosion on bars was detected in the first and second floors where leaking water was observed as could be seen from Table (1).

6. RECOMMENDATIONS AND METHODS OF REPAIR

6.1 The hall of the main building

Figure (4 and 5) illustrates the method used in repairing and strengthening the trusses of the hall. The following procedure was adopted:

- Collars were primarily loosely installed every about 50 cm to the truss members.
- The cracks were filled with a bonding material consisting of saw dust and an epoxy adhesive. Care was taken during filling to avoid any voids.
- The bolts of the collars were then tightened (during the initial drying period), thus forcing out any voids in the viscous epoxy and providing uniform bonding between the sides of the crack.
- Finally, the wooden structure was painted by an epoxy resin protecting it from humidity, decay and any environmental conditions.
- Leakage of water at the roof was fully checked and repair was carried out to ensure that no leakage would occur in the future.

6.2 The extension building

The following repair procedure was recommended:

- A full check and repair of the rain water drainage system, plumbing and maintenance of the building.
- Removal of all concrete cover for steel reinforcement where concrete has been affected and removal of all rust that might be found on the reinforcement bars by means of sand blasting.
- Areas where steel reinforcement was excessively corroded should be replaced by new bars.
- Steel bars should be covered by a protective layer against rust and bars should be covered by a bonding agent layer to ensure a good bonding between steel and newly placed concrete cover.
- Shot creeting with a dense mix ensuring that the mix fills in all the voids between steel bars and main concrete body.

7. CONCLUSION

1. The hall of the old building which is bearing wall with a false ceiling suspended by wooden trusses was proven safe. The deterioration of the ceiling was mainly due to water leakage while the cracks in the wooden truss members were mainly due to old age and lack of maintenance.
2. The method used in repairing the wooden trusses proved to be reliable on the grounds that the repair works has already been completed and no signs of cracks has appeared.

3. Deterioration of the extension building (reinforced concrete elements) as a whole was due to bad usage and poor maintenance. Corrosion of steel bars was mainly due to high chloride percent (>0.3%) and water penetration in concrete.
4. Old historical buildings based on old structural designs and methods of construction should be periodically checked and maintained as they are a symbol of national heritage and assets.

ACKNOWLEDGEMENTS

The author acknowledges the support of the Structural Engineering Department and its laboratories at Arab Consulting Engineers (Moharram-Bakoum).

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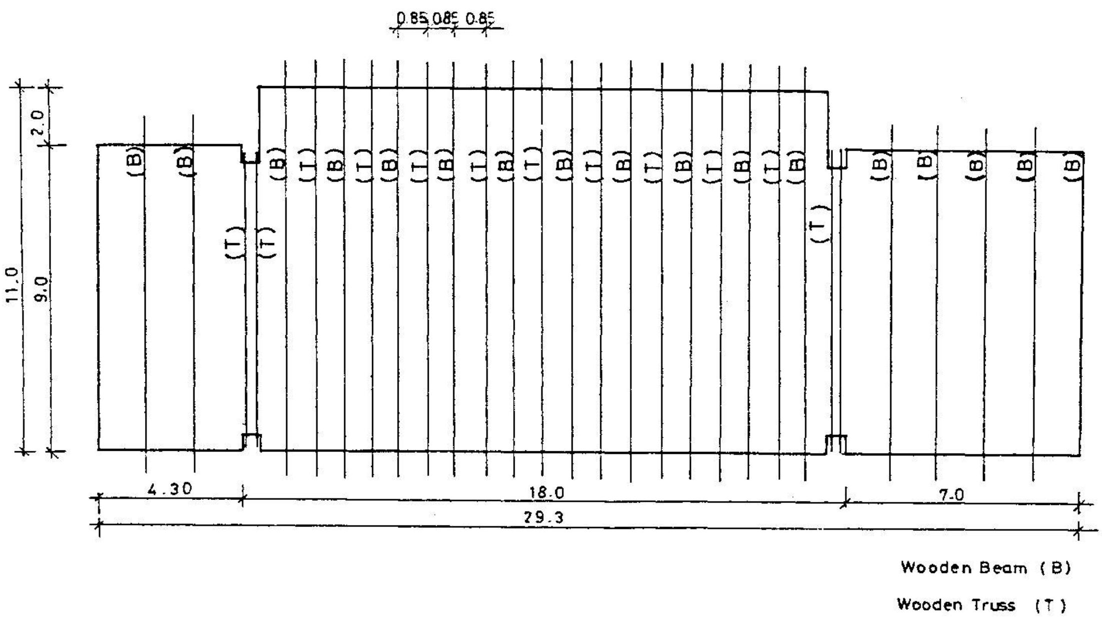
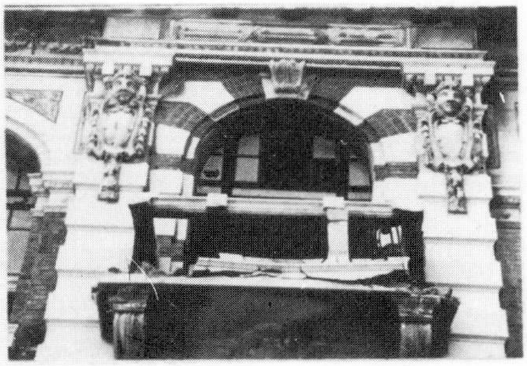
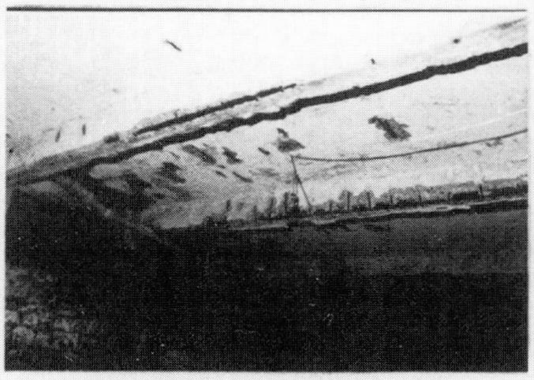


Fig. (1) Layout of the Hall (in the Old Building) showing Locations of Beams and Trusses

(a) Part of the Extension Building



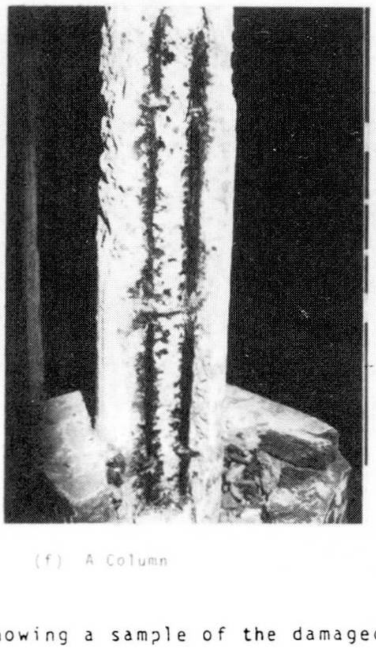
(b) A Balcony



(c) The Basement

(d) The Terrace

(e) A Slab



(f) A Column

Fig. (2) Photos taken to the Extension Building showing a sample of the damaged parts

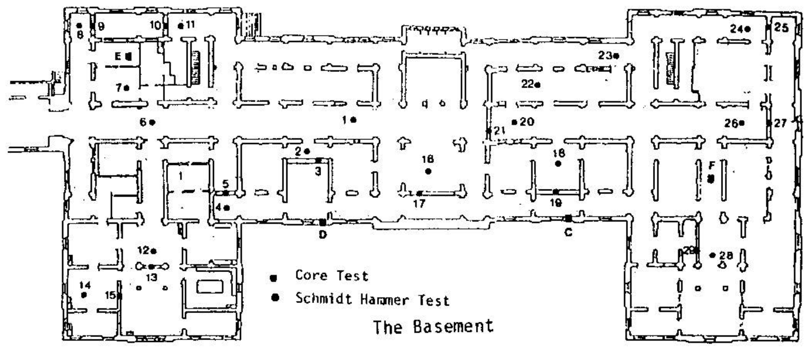


Fig. (3) Layout of the Extension Building (New Building) showing Locations of Tests

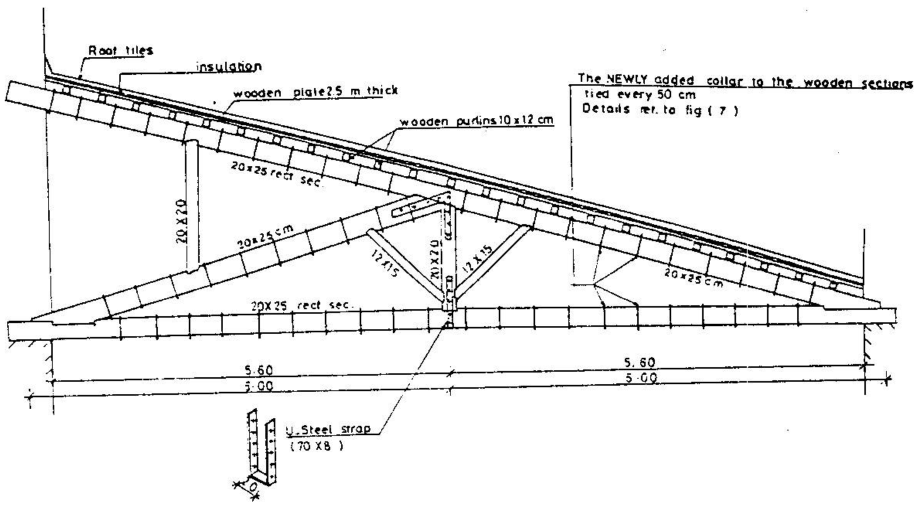


Fig. (4) A Typical Truss of the Old Building Hall (after repair).

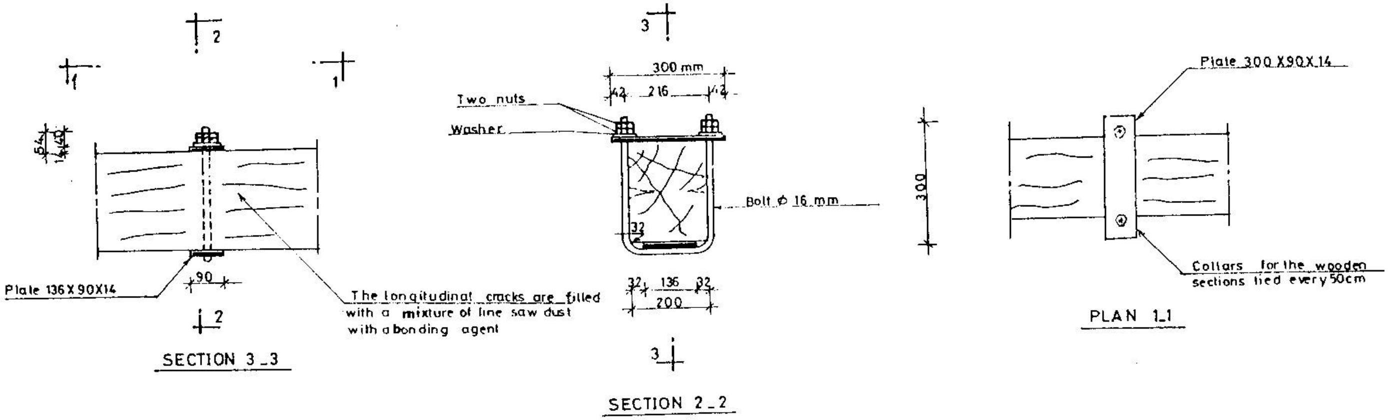


Fig. (5) Collar Details which were connected to the Truss Members

Rissbildung in historischen Kirchenbauten Analyse und Stabilisierungsmassnahmen

Development of Cracks in Historic Churches - Analysis and Stabilization

Fissuration d'églises anciennes - analyse et stabilisation

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ZUSAMMENFASSUNG

Die charakteristischen Rissbilder in historischen Kirchen mit zwei üblichen Gewölbeformen in Oesterreich werden im Zusammenhang mit deren Ursachen analysiert. Beispiele von Rissen, die vom charakteristischen Verlauf stark abweichen, werden durch besondere Lastwirkungen und geologische Bedingungen erklärt. Stabilisierungsmassnahmen durch Vorspannsysteme, die für Kirchenbesucher nicht sichtbar sein dürfen, werden beschrieben. Lage und Vorspannkraft von Spanngliedern wurden im Hinblick auf optimale Spannungsverhältnisse in einem Tonnengewölbe mit Stichkappen untersucht.

SUMMARY

Characteristic crack patterns of historic churches in Austria with two common kinds of vaultings are analyzed in association with causes due to residual stresses and loads. Examples of cracks deviating strongly from the regular pattern are explained by particular load effects. Stabilizing measures with prestressing systems are described. They should remain unseen for visitors to the churches. The situations of prestressing members and their magnitude were investigated with regard to optimal stress condition in a barrel vault with lunettes.

RÉSUMÉ

Le mode de fissuration caractéristique de la maçonnerie d'églises anciennes en Autriche, avec deux types classiques de voûte est analysée, ainsi que ses causes. Des exemples de fissures atypiques s'expliquent par l'effet de charge extraordinaire et les conditions géologiques. La stabilisation de la construction au moyen de systèmes de précontrainte, lesquels doivent rester invisibles pour les visiteurs de l'église, est décrite. La position et la force de la précontrainte sont étudiées en vue d'optimaliser les contraintes dans une voûte en berceau avec de lunettes.

1. ART UND URSACHE DER RISSE

Von reinen Putzrissen sei abgesehen. Nur Risse in bzw. durch das tragende Mauerwerk werden behandelt: Biege- und Trennrisse. Die sehr geringe Zugfestigkeit von Mauerwerk bedingt, daß Risse Trennbrüche des Materials zufolge von Zugspannungen darstellen und rechtwinkelig zur größten Hauptzugnormalspannung stehen. Sie geben daher einen deutlichen Hinweis auf den Spannungszustand bzw. Kraftfluß im Bauwerk zum Zeitpunkt der Rißentstehung und sind das Hauptindiz für die Ursachenanalyse [1] Hinsichtlich des Einflusses auf die Stand(Bruch-)sicherheit seien als Rißursachen unterschieden [2]:

a) Äußere statische und dynamische Kräfte (Lasten), die "Lastspannungen" bewirken,
b) Zwängungen infolge Temperatur- und Feuchtigkeitsunterschieden im Mauerwerk, sowie ungleiche Setzungen, die "Zwängungsspannungen" bewirken,
c) Eigenspannungen als schnittkraftfreie Spannungsanteile
d) Alterung als zeitabhängige Änderung von Materialeigenschaften.

2. BEWERTUNG VON RISSURSACHEN UND TYPISCHE RISSBILDER

Als wesentliche Grundlage der Planung von Reparaturmaßnahmen ist eine - zumindest qualitative - Bewertung dahingehend wichtig, in welchem Ausmaß die oben angeführten Ursachen zur Rißbildung beitragen. Man darf von der plausiblen Annahme ausgehen, daß das Bauwerk nach Fertigstellung rißfrei war, wenn auch nur kurzfristig. Es sind daher die Änderungen der Beanspruchungen im Bauwerk, die zu den ersten Rissen führen und dort auftreten werden, wo unter ständiger Last nur mehr geringe Druckspannungen oder gar Zugspannungen vorhanden sind.

Bei der Bewertung der Folgen der Rißbildung wird auch zu unterscheiden sein, ob die Zustandsänderungen progressiv zunehmend sind, z. B. Langzeitsetzungen, oder periodisch auftretend veränderlich sind, wie etwa infolge von Temperaturschwankungen und Schnee und Wind.

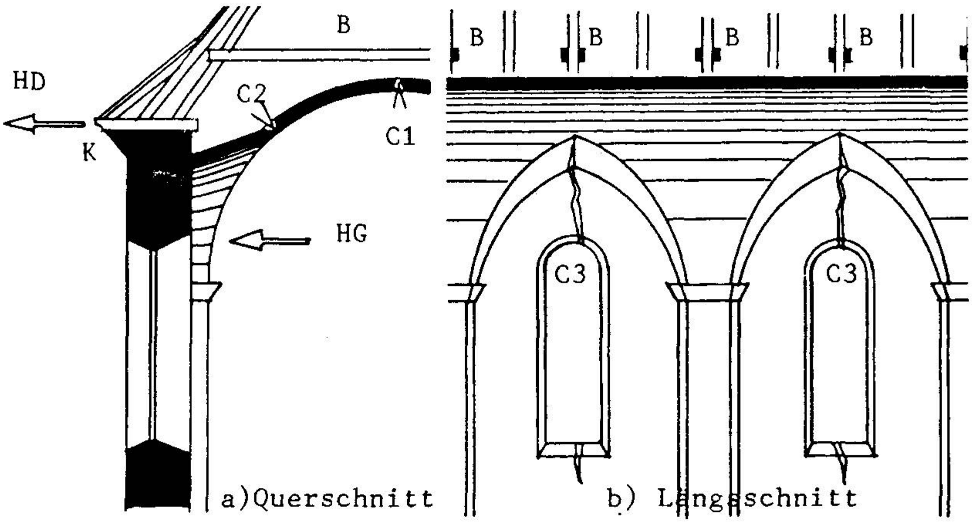
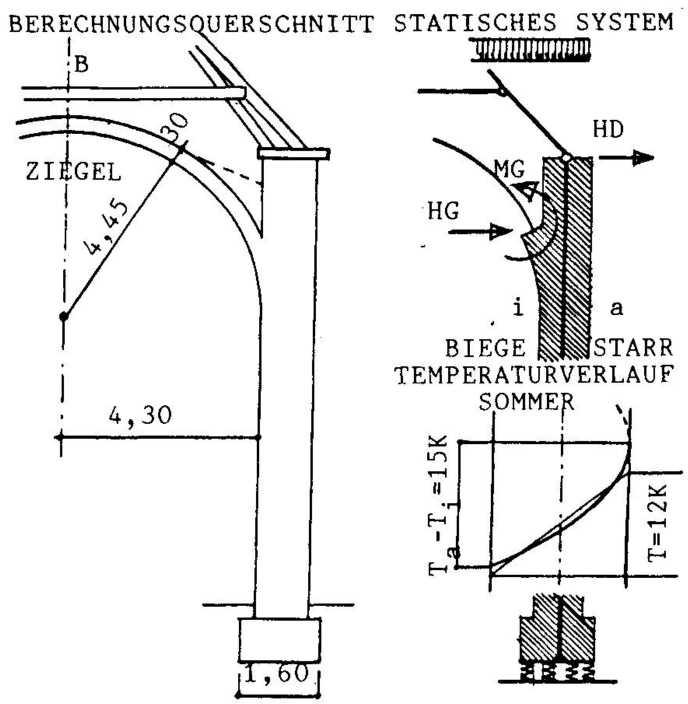


Fig. 1: Tonnengewölbe mit Stichkappen, Kehlbalken B

Die Bewertung der zu Rissen führenden kurzfristigen Zustandänderungen sei für die Bauweise nach Fig. 1 an einem Tonnengewölbe mit Stichkappen mit den fast immer im Scheitel an der Unterseite auftretenden Rissen C1 und den manchmal zu beobachtenden Rissen C2 an der Gewölbeoberseite in Kämpfernähe vorgenommen. Den einfachen Berechnungen wurde der Modellquerschnitt gemäß Fig. 2 zugrunde gelegt.



Der Steifigkeitseinfluß von Stichkappen ist in einer größeren Gewölbedicke erfaßt. Im Rechenmodell werden das Gewölbe als elastischer Kreisbogen, und die Wände vorerst als biegestarr angenommen, aber die drehelastische Einspannung im Baugrund für mittleren Boden mit dem Steifemodul von 70 N/mm^2 berücksichtigt. Angenommene Materialwerte des Ziegelgewölbes [1]: Elastizitätsmodul: $E_{min} = 3.10^3$ N/mm^2, Querdehnzahl 0,2, Temperaturdehnzahl $\alpha_T = 7.10^{-6}$/K. Der Horizontalschub des Gewölbes, HG, auf 1m Gebäudelänge beträgt hier rund 20 kN/m für das fest eingespannte Kreisgewölbe.

Fig. 2: Kirchenschiff mit Tonnengewölbe
Grundlage für Modellberechnung

Bei der Beurteilung der wichtigen Wirkung der Horizontalschübe der Konstruktion wird meist übersehen, daß Dachstühle dort, wo hochliegende Gewölbescheitel keine Träme zulassen, wie in Fig. 1 und Fig. 3 auch die Dachkonstruktion nicht unbeträchtliche Horizontalschübe, HD, auf die Außenwände ausüben; im Beispiel Fig. 2 als mittlerer Wert ca. 4 kN/m ständig.

Gemäß der Hypothese der Rissefreiheit im Erstzustand wird daher nur der Einfluß der möglichen Änderung von HG, z. B. einer in Österreich durchaus möglichen Schneelast von $s_0 = 3{,}0$ kN/m^2 - auf die Horizontale bezogen - als zu a) gehörende Ursache berücksichtigt. Es ergeben sich ΔHG-Werte um 5 kN/m und eine zugehörige Horizontalverschiebung der Gewölbekämpfer von mehr als 1,8 mm nach außen. Im Gewölbescheitel tritt an der Unterseite eine Zugspannung von mehr als 0,3 N/mm^2 an der Stelle C1 auf. Bei Berücksichtigung der Biegesteifigkeit der Wände wird der Lastanteil des Gewölbes größer. An der Einspannstelle erreicht die Zugspannung an der Gewölbeoberseite Werte über 0,6 N/mm^2.

Als Beispiel zu Zwängungswirkungen gem. b) sei der Fall eines Unterschiedes der Temperaturen an den Außen- und Innenflächen von 15 K im Sommer als realistische Rechenangabe gewählt und für den charakteristischen Temperaturverlauf quer durch das Mauerwerk, wird die krümmungwirksame lineare antimetrische Temperaturdifferenz $\Delta T = 12$ K. Für Punkt C1 folgt wieder eine Biegezugspannung von mehr als 0,3 N/mm^2. Identifiziert man die Zugfestigkeit des Gewölbes, hier quer zur Lagerfuge der Ziegel, mit der Haftfestigkeit zwischen Mörtel und Ziegel, so kann diese in den Grenzen 0,1 bis max. 0,3 N/mm^2 für "alten" Mörtel angenommen werden. Das heißt, daß die Spannungen, die sich den vorhandenen infolge ständiger Last (Primärzustand) überlagern, Initialrisse aus Biegung bewirken können.

Da extreme Schneelasten, wie die der Berechnung zugrunde gelegte, weitaus seltener sind als die angenommene Temperaturdifferenz, sind die Zwängungen als die häufigste Ursache der Initialrisse anzusehen.

Sowohl in Bauwerken gem. Fig. 1 und Fig. 3 sind vorwiegend lotrecht verlaufende Risse, C3, in den Längswänden der Kirchenschiffe zu beobachten. Wenn vorausgesetzt wird, daß diese Seitenwände der Gebäude im Sommer errichtet wurden und frei von Eigenspannungen sind, dann wird im Winter der obere Bereich deshalb Längszugspannungen erfahren, weil diese Zonen kälter als die unteren werden und durch die Fundierung als auch durch das eigene Gewicht gezwungen werden, gerade zu bleiben.

Allein ein plausibler $\Delta T = 15$ K-Wert zwischen Sommer- und Wintertemperatur des Mauerwerkes und einem $E = 5.10^3$ N/mm^2 für Bruchsteinmauerwerk, liefert bei vollständiger Dehnungsbehinderung und linearelastischer Rechnung eine Zugspannung ohne Kerbwirkung von 0,53 N/mm^2. Wenn die wirklichen Zugspannungen tatsächlich etwas geringer werden, so liegen sie doch erheblich über der zu erwartenden Zugfestigkeiten der Mauerwerke.

In der Bauweise gemäß Fig. 3 ist die Überwölbung in Joche unterteilt, die durch Gurtbögen getrennt sind. Das statische Verhalten der Gurtbögen und die anschließende und mitwirkende Zone der flachen Kuppeln ist in Querrichtung ähnlich dem Beispiel der Tonnengewölbe. Bei diesem - vor allem für das Barock typischen System - ragen die Kuppeln besonders hoch in den Dachstuhl hinein, sodaß die Horizontalschübe HD größer als in dem vorhergehenden Beispiel sind. Die Schalenbereiche zwischen den Gurtbögen sind in Kirchenquerrichtung wesentlich biegesteifer als die Gurtbögen bzw. als das Tonnengewölbe gem. Fig. 1. Bei allen Lastfällen, die zu einer horizontalen Verschiebung der Kämpferbereiche K nach außen führen, treten beim System gemäß Fig. 3 an den Stellen, vergleichbar C2 der Fig. 1, nicht nur Biegerisse auf, sondern auch eine völlige Durchtrennung der Gewölbeschale an den Stellen C4 der Fig. 3, wie sehr oft zu beobachten ist. Die horizontale Verschiebung der Mauerkrone K nach außen führt dazu, daß die Mauerrippen in Fig. 3b, die offenbar als horizontale Stützung der Gewölbeschalen gedacht waren, sich von der Mauerkrone ablösen und die dabei entstehenden bis zu 20 mm breiten Spalten C5 die Funktion dieser Rippen wirkungslos machen.

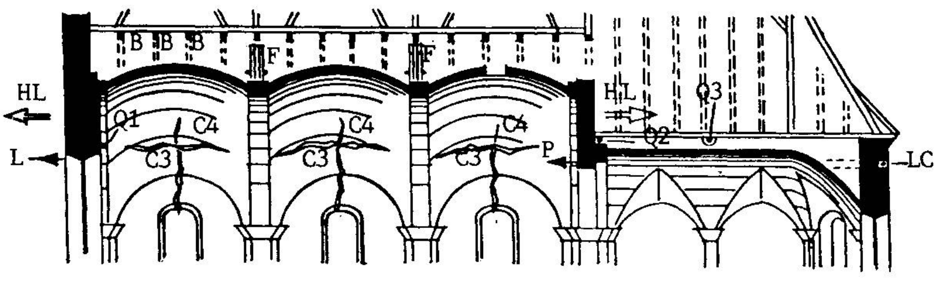
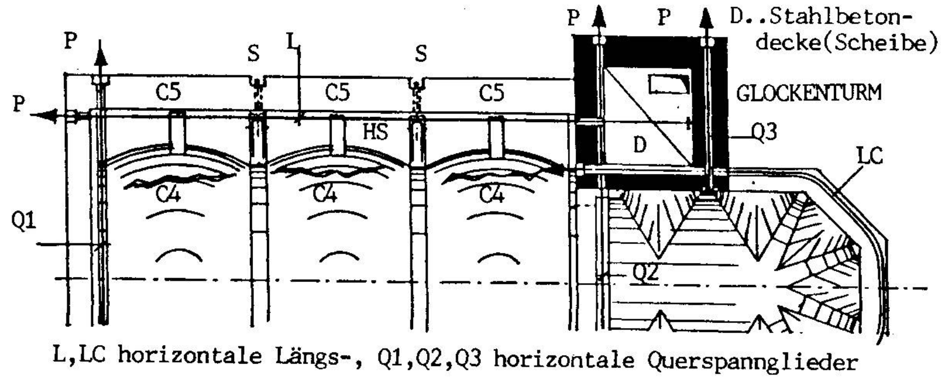


Fig. 3a: In Joche unterteilte Gewölbe, Längsschnitt



L,LC horizontale Längs-, Q1,Q2,Q3 horizontale Querspannglieder

Fig. 3b: In Joche unterteilte Gewölbe,Grundriß zu 3a im Dachgeschoß

Die flachen Kuppeln der Konstruktion gem. Fig. 3 üben als doppelt gekrümmte Schalen auch horizontale Reaktionen in der Längsrichtung des Gebäudes aus, die in Fig. 3a mit HL bezeichnet sind. Diese Horizontalkraft überträgt sich durch Schalenwirkung der Kuppeln auf die Längswände des Gebäudes und verstärkt auf diese Weise die Längszugspannungen in den Seitenwänden und trägt verstärkt zu C3 bei.

3. SONDERFÄLLE VON RISSBILDERN

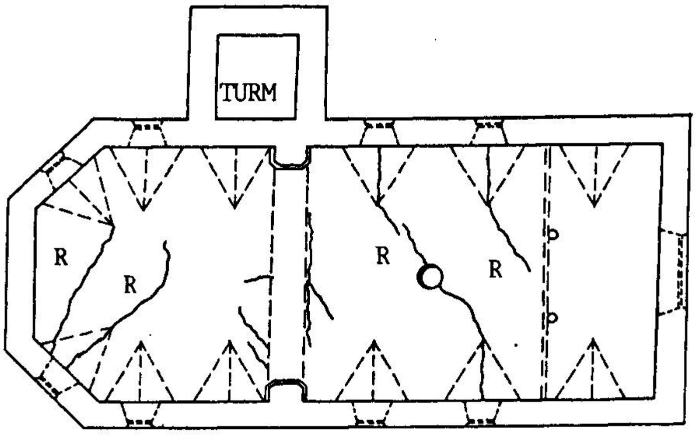


Fig. 4: Draufsicht auf Tonnengewölbe mit untypischem Rißbild R

Fig. 4 zeigt schräg verlaufende Risse im Tonnengewölbe einer kleinen Kirche auf gutem Baugrund, die mit oben dargelegten Überlegungen nicht in Einklang zu bringen waren. Die Ursache der Rißverläufe ließ sich erkennen als die Befragung der Bewohner ergab, daß während des II. Weltkrieges in kaum 70 m Entfernung von der Turmseite der Kirche ein kleiner Steinbruch errichtet wurde, der dzt. nicht mehr in Betrieb ist.

Deutlich sieht man, daß der Turm ablenkend auf die Richtung der Risse infolge der Erschütterungswellen wirkt.

In Fig. 5 sind im Grundriß Risse in einer Kuppel dargestellt, die in Tirol in Itter nach dem Erdbeben im Friaul 1976 verstärkt auftraten [2]. In dem Zentralbau hätte man nach statischen Gesichtspunkten Risse in Radialrichtung im unteren Teil der Kuppel erwartet. Die Orientierung der Risse zeigt aber deutlich ein Ausweichen der Nord- und Südwand nach außen, was auch zu der Trennung R der als horizontale Stützung der Kuppel gedachten Mauerrippen führte. Die Analyse der Risse und deren Lage ergibt:

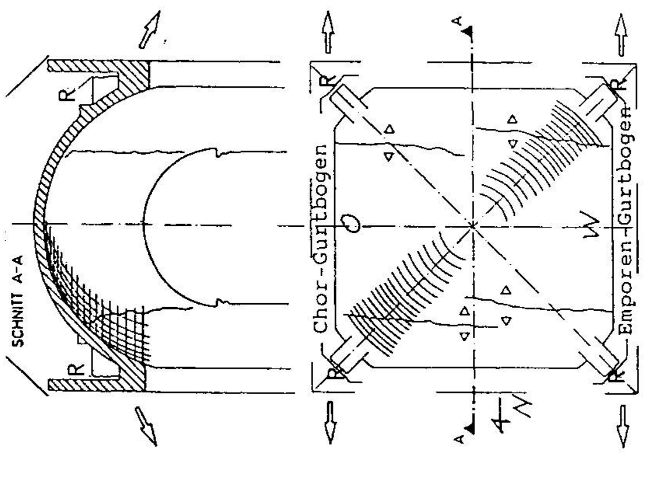


Fig. 5: Rißbild einer Hängekuppel, auf Gurt- und Schildbögen

Erstens liefert ein Temperaturuntschied von $\Delta T = 12$ K Zwängungsbiegespannungen von mind. $\pm 0{,}13$ N/mm², die aber proportional dem E-Modul des Mauerwerkes je nach dessen Güte auch bei $\pm 0{,}2$N/mm² liegen können. Diese Spannungen treten fast in der gesamten Kuppel auf.

Zweitens ergab der geologische Befund, daß die Flanken des schmalen Bergrükkens, auf dem das Gebäude steht, zu langzeitigen Hangbewegungen neigen und durch Erdbeben besonders dazu aktiviert werden können. Tatsächlich zeigte der alte Estrich des Kirchenfußbodens zwei Längsrisse als Anzeichen dafür, daß die in bezug auf die Kirchenlängsachse seitlichen Teile des Bodens der Hangbewegung gefolgt waren.

4. FOLGEN DER RISSE FÜR DAS BAUWERK

Bei Rissen infolge Zwängungen wird die Standsicherheit etwas, aber nicht entscheidend verringert. Diese Verringerung besteht bei den vorliegenden Bauwerken darin, daß im allgemeinen die Risse den Grad der statischen Unbestimmtheit stark reduzieren, z. B. die Wirkung des Tonnengewölbes oder der Gurtbögen vom voll eingespannten Bogen in Dreigelenksysteme verändern. Im Beispiel der Fig. 3 wird das statische Zusammenwirken der Jochsysteme und die Schalenwirkung der Kuppeln durch die Risse C3 in Richtung sich bildender Einzelsysteme verringert.

Auch wenn Bauwerke den Anforderungen hinsichtlich ausreichender Standsicherheit noch genügen, so können - besonders bei historischen Mauerwerks- und Steinbauten - spezielle Aspekte der Gebrauchstauglichkeit sehr wohl Maßnahmen erfordern, z. B. die zur Erfüllung der in Mustervorschriften genannten "Anforderungen aus Gründen des Aussehens", besonders in Zusammenhang mit der Rißbildung; dies besonders im Falle von historisch wertvollen Fresken. Aber auch die Tatsache, daß die erhöhte Beweglichkeit der gerissenen Konstruktion im Zusammenhang mit den Zwängungsspannungen zu Abplatzungen von Putzteilen führen kann, beeinflußt die Gebrauchstauglichkeit, weil von den Gewölben herabfallende Putz- oder sogar Ziegelteile Menschen gefährden können.

Die Standsicherheit wird dann stark beeinflußt, wenn die Rißbildung eine zeitlich progressive ist, wie z. B. durch zunehmende Neigung von Seitenwänden mit Pfeilern unter Wirkung von Seitenschüben der Bogen und Kuppeln nach außen. Die daraus folgenden Scheitelsetzungen flacher Bögen und Schalen können zu Brüchen bzw. Instabilität führen.

5. PRINZIPIEN FÜR DIE SANIERUNGSMASSNAHMEN [1], [2], [3]

Alleiniges Verfüllen und Verpressen von Rissen bei progressiven Vorgängen ist eine ungeeignete Maßnahme für langzeitigen Erfolg, denn sie beseitigen die Ursachen nicht - meist die oben genannten Horizontalbewegungen von Bogenwiderlagern mit den Ursachen in der Fundierung - und werden die fehlende Zugfestigkeit nicht oder nur lokal begrenzt bringen. Nachfundierungen können Erfolg haben und sind in manchen Fällen von c) sogar erforderlich - siehe Pisa. Sie sind aber meist teuer und kurzfristig schwer prüfbar. Gegen die Risse als Folge von Zwängen sind sie praktisch wirkungslos [4].

Die Maßnahmen sollten also nicht nur die Risse stabilisieren - wie z. B. durch Nachfundierung - sondern auch Rißbildung durch örtliche Zugspannungen infolge Zwängungen b) verringern oder verhindern. Wirkungsvoll sind für diesen Zweck Stahlglieder. Sollen diese wirkungsvoll sein, dann kommt nur gezielte Vorspannung in Frage, denn dadurch kann eine sofortige Übernahme von Kraftwirkungen, wie etwa die Aufnahme von Horizontalschüben, erfolgen. Ein weiterer Vorteil der Vorspannung besteht darin, daß in Querschnitten zusätzliche Druckspannungen gezielt und meßbar eingebracht werden können, so daß die Aufnahmefähigkeit von Zugspannungen ohne Rißbildung möglich oder zumindest vergrößert wird.

6. BEISPIELE FÜR AUSGEFÜHRTE SANIERUNGEN

Aus dem Rißbild der Konstruktion gem. Fig. 3 ist zu ersehen, daß zwei Vorspannsysteme erforderlich sind: eines in der Längsrichtung und einige in der Querrichtung zur Kirchenachse. Zur Stabilisierung der Seitenwände des Kirchenschiffes ist es im allgemeinen immer möglich, einen Weg zu finden, das gesamte Gebäude durch horizontale Vorspannkabel in einem oberen Niveau der Außenwände zu umfassen. In Fig. 3 liegen die Spannglieder LC in Schlitzen, die in dem Außenmauerwerk des Kirchenschiffes verlegt werden und abschnittsweise, wie die Abbildung zeigt, durch Bohrlöcher zu führen sind. Diese Umfassung der Apsis bietet den zusätzlichen Vorteil, den Umfangszugspannungen des Gewölbes der Apsis entgegen zu wirken. In Fig. 3 verlaufen die Längszugglieder L im Bereich des Kirchenschiffes nahe an der inneren Seite der Mauern im Dachraum. In diesem Fall muß etwas gegen die Wirkung der horizontalen Ausmittigkeit zwischen der Achse des Spanngliedes und der Achse der Wand getan werden. Fig. 3b und Fig. 6 zeigen eine mögliche Lösung durch die Ausbildung eines Stahlbetonbalkens HS in dem Zwickel zwischen der Mauerkrone und dem Gewölbe im Dachraum des Kirchenschiffbereiches. Damit werden zwei Wirkungen erzielt: Zentrierung der Vorspannkraft und eine Verstärkung der horizontalen Biegesteifigkeit der oberen Bereiche der Wand des Kirchenschiffes. Letzteres ist günstig für die horizontale Abstützung der Gewölbe zwischen den Gurtbogen. Die Stabilisierung der Risse C1, C2, C4 und C5 der Fig. 1 und Fig. 3 in den Gewölben erfordert es, primär horizontale, nach außen gerichtete Bewegungen der Kämpfer zu verhindern. Die einfachste Lösung, nämlich horizontale, im Kirchenraum aber sichtbare Zugstangen, wird z. B. in Österreich oft als optisch störend abgelehnt.

Die Fig. 6 zeigt drei Beispiele, wie man die gleiche Wirkung durch Konstruktionen erreichen kann, die im Kirchenschiff vom Benutzer der Kirche nicht gesehen werden können. Der Druck gegen den Horizontalschub des Bogens wird durch die Reaktionskräfte R des gekrümmten Abschnittes von Spannkabeln S erreicht.

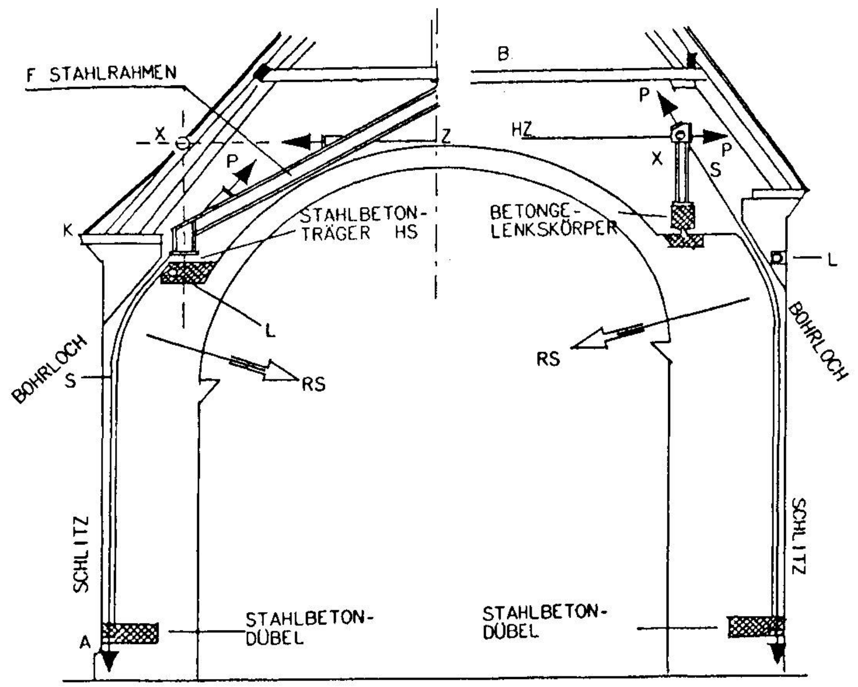


Fig. 7: Konstruktion für "unsichtbare Zugbänder" HZ

Die Endverankerung dieser Spannglieder befindet sich möglichst tief unten am Punkt A. Von dort aus läuft das Spannkabel in einem lotrechten Schlitz in der Wand und in einem geneigten Abschnitt in einem Bohrloch zur Stützenkonstruktion. Punkte P in Fig. 6 bezeichnen Stellen, wo die Spannglieder S vorgespannt werden. Wenn es die räumlichen Verhältnisse erlaubten, wurde die in Fig. 6, rechte Hälfte, dargestellte als einfachste Lösung ausgeführt. Das Vorspannglied HZ stellt ein "hochliegendes Zugband" dar. Um jede horizontale Reaktionskraft auf die Lager der Stützen für P - dargestellt auf der rechten

Seite der Fig. 6 - auf die Widerlager, besonders in der Richtung nach außen, zu vermeiden, sind die Stützen als vertikale Pendelglieder ausgeführt. Durch entsprechende Vorspannung der Zugstange HZ ist es möglich, deren Verlängerung derart zu regeln, daß die Stützen für Punkt X nach den Vorspannarbeiten stets lotrecht stehen.

Wenn der Lichtraum innerhalb des Dachgeschoßes die Ausführung entsprechend der rechten Seite der Fig. 6 - maßgebend ist der Punkt X - nicht erlaubt, so wurde das System entsprechend der linken Seite der Fig. 6 ausgeführt. Es handelt es sich um eine biegesteife Stahlkonstruktion F, die auf Rollen oder Gleitlagern ruht, und das Lager P für die Vorspannpressen bildet. Zur Erhöhung der Biegesteifigkeit der Stahlkonstruktion ist noch ein horizontales Zugglied Z angeordnet, das vorgespannt wird, und der Verringerung der horizontalen Verschiebung der Lager der Stahlkonstruktion während des Vorspannens der Spannglieder S dient. Andere Beispiele und Methoden sind in [3] und [5] enthalten.

7. WAHL DER KRÄFTE FÜR QUERVORSPANNUNG

Als Grundwert für die Ablenkkraft RS in Fig. 6 wurde angenommen, daß sie mit den Horizontalschüben HG und HD möglichst gleichförmige Bodenpressung unter den Längswänden ergeben, wobei die Höhenlage von RS ein festzulegender Parameter ist. Spannkraftverluste durch Kriechen und Wärmedehnungsunterschiede zwischen Spannglied und Mauerwerk sollen zugeschlagen werden, wobei über eine obere Grenze an Vorspannkraft, die wieder Risse oder zu große Druckspannungen erzeugt, nicht hinausgegangen werden darf.

In einem Forschungsvorhaben [5], [6] wurde unter anderem der Zusammenhang von Höhenlage und Größe der Vorspannkraft für das Kreistonnengewölbe ähnlich Fig. 2 für den Lastfall Eigengewicht und für den Lastfall Eigengewicht plus Vorspannung hinsichtlich der Spannungen im Gewölbe unter folgenden Optimierungsbedingungen untersucht:

- Keine Zugspannungen an der unteren Leibung im Scheitel
- Möglichst geringe Zugspannungen im Verlauf der Randspannungen über den Bogen

Es treten dann nur unwesentliche Zugspannungen an anderen Stellen auf, die für eine weitere Optimierung den Aufwand nicht lohnen. Die FEM-Untersuchungen wurden sowohl als ebenes Problem ohne Stichkappen und als 3D-Problem mit Stichkappen und Fensteröffnungen, Fig. 7, am Institut für Festigkeitslehre und Flächentragwerke der Universität Innsbruck im Kapitel, das von Dr. Stark behandelt wurde, durchgeführt [6].

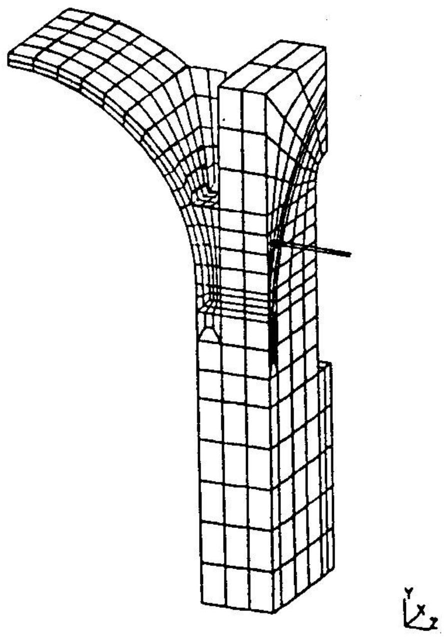


Fig. 7: FEM-Netz zur 3D-Berechnung

Höhenlage der Horizontalkomp. v. RS	8,95	8,49	8,05	7,59
Vorspannkraft 2D-Modell [kN/m]	18,5	20,1	21,8	23,9
Vorspannkraft 3D-Modell [kN/m]	17,8	19,2	20,9	22,9

Nebenstehende Tabelle zeigt die Ergebnisse für einige Vorspann-Höhenlagen. Die Wand wurde dabei auf Niveau ± 0,0 als eingespannt angenommen.

Dazu wäre zu bemerken:
- Die Tendenz, daß die Tiefersetzung der horizontalen Spannkraft, deren Vergrößerung erfordert, entspricht der erstgenannten Forderung, daß die Sohlpressungen möglichst gleichförmig gemacht werden sollen.
- Die Vorspannkraft von z.B. 18,5 kN/m entspricht dem eingangs genannten Wert 20 kN/m für starre Einspannung des Gewölbebogens gut. Der Wert in der Tabelle ist kleiner, weil die Gewölbeeinspannung dort elastisch ist.
- Die Höhenlage von R in Fig. 7 ist daher den räumlichen Gegebenheiten entsprechend in brauchbaren Grenzen wählbar.

8. ABSCHLIESSENDE HINWEISE

Zu der Frage, wie weit man über die den Horizontalschüben entsprechende Größe mit der Vorspannung zur Deckung der genannten zeitabhängigen Verluste hinausgehen darf, gibt beim Spannen auch das Bauwerk. Etwa 10 bis 20% über den Grundwert hinausgehende Spannkräfte können zu leichten kleinen Abblätterungen von Farbschichten führen. Bei mehr als 30% wurden in der Nähe von Rissen in Gurtbögen Loslösungen von Putzstücken beobachtet.

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Early Modern Architecture: How to Prolong a Lifespan

L'architecture moderne précoce: Comment prolonger une durée de vie

Frühmoderne Architektur: Verlängern der Lebensdauer

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SUMMARY

In Modern Movement architecture, the structural elements of a building - mostly concrete or steel frames - form an indissoluble part of the original design approach. Therefore, such elements are part of the historic value of these buildings. At the same time, modern architects strove after minimal constructions. Our problem today is, how to preserve these slender constructions in harmony with the original architectural concepts, so that the ideas of the original designers remain perceptible.

RÉSUMÉ

Dans l'architecture moderne, les éléments structuraux d'un bâtiment - essentiellement des cadres en béton ou en acier - représentent un élément fondamental du projet architectural original. Ainsi, ces éléments font partie de la valeur historique de ces bâtiments. En même temps, les architectes modernes visent à une construction minimale. Le problème actuel est de préserver ces structures minces en harmonie avec la conception architecturale originale, afin de respecter la conception primaire de l'architecte.

ZUSAMMENFASSUNG

In der modernen Architektur bilden die Tragwerkselemente eines Gebäudes, mehrheitlich Beton- oder Stahlrahmen, einen unverwechselbaren Bestandteil des ursprünglichen Projektes. Sie sind denn auch wichtig für den geschichtlichen Wert des Gebäudes. Gleichzeitig strebt die moderne Architektur nach minimalen Konstruktionen. Unser heutiges Problem besteht in der Erhaltung dieser schlanken Konstruktionen, denn die Harmonie mit der ursprünglichen architektonischen Konzeption soll gewahrt werden.

1. THE MODERN MOVEMENT

1.1. Previous history

The building tradition underwent great changes in the 19th century with the onset of industrialisation. In the old days, a few buildingtypes were fit for a variety of uses due to their neutral lay out. Therefore, buildings could easily be adapted to new functions, which resulted in a relatively long functional lifespan. Generally, the technical lifespan was in harmony with this principle, by producing firm and flexible constructions.
During the Industrial Revolution the briefs became more diverse and specific, and so did the buildings. Steammills, sanatoriums, locks and officeblocks, each required its own characteristic features. But steam was being ousted by other mediums, sanatoria became hospitals, wordprocessors are replacing the typewriter. Therefore it was not only the nature of the brief that changed, but also the period of such a use. Another important aspect was of course the application of new materials and new construction types that made use of the specific properties of these. This development had a major impact on the building practice and a tremendous effect on architecture.The techniques required for preserving buildings from the Industrial Age are therefore different from those appropriate to older ones.

1.2. Requirements and performance: a limited lifespan

In the period between the World Wars, these developments ultimately led to the pioneering work and revolutionary ideas of the designers of the Modern Movement. Around 1920 architects started to establish a direct link between the design, the technical lifespan of a building and user requirements. The consequent translation of these ideas into practice produced a specific movement in architecture, that came to be known in the Netherlands as 'het Nieuwe Bouwen'. Jan Duiker was among its protagonists. 'Het Nieuwe Bouwen' was not referred to as a style, as an aesthetic principle, but rather as a workingmethod, a way of thinking about building. Duiker set great value on the connection between form, function, applied materials, economy and time. User requirements and economy are the causes, form the result. If the function changes, the form looses its right of being and the building, as was frequently explained by themselves, should be either adapted or demolished. They regarded buildings as utilities with a limited lifespan by definition, sometime even as 'throw away' articles. This way of thinking formed the basis of what we consider normal practice today, but what represented a totally new and revolutionary point of view in those days. Therefore many of their buildings are highpoints in 20th Century cultural history .
Some famous examples in Holland are the Van Nelle factories in Rotterdam, by Brinkman and Van der Vlugt from 1925-29, the Gooiland Hotel, designed by Duiker for Hilversum in 1934, and his Zonnestraal Sanatorium in Hilversum, erected by the Diamondworkers Union in 1926-28 and now also a main monument of social history in our country.

Fig. 1. Sanatorium Zonnestraal (1926-28).

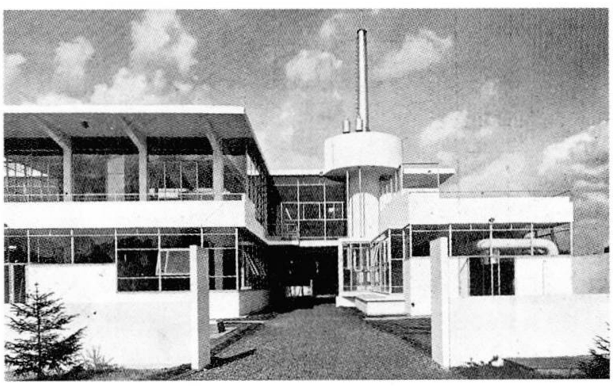

Fig.2. Gooiland Hotel (1934) in 1986.

1.3. Transitoriness of Modern Movement architecture

Duiker's work does not excel in properly detailed construction. Many of his buildings are now either dilapidated or even already demolished. This is largely because of his choice of materials and in the way these were used. Yet, there has too often been assumed that his constructions, that do not satisfy nowadays standards, arose from professional ignorance of the designers. For me that's too easy, since from documents as well as the constructions themselves can be learned that these designers must have been quite well aware of what they were doing. Apparently, other motives were in it as well, such as the acceptance of a limited technical lifespan as an answer to limited financial means, especially when also the functional life expectancy was limited. The issue of transitoriness of Modern Movement architecture should therefore be understood as being largely defined by the designer's approach. Obviously, this should have a great impact on how to restore these buildings.

1.4. Optimalisation

A rigorous distinction between loadbearing parts and infills was followed out, in order to attune each part to its proper function, making everything as 'monofunctional' as possible. For instance facades traditionally combined a loadbearing function with shutting out weather conditions. Obviously, this resulted in limitations as regards how and where the inside could be supplied with fresh air and daylight. By introducing a frame, the principles of the Modern Movement literally disclosed the buildings.
Another basic element was the use of prefabricated parts. This was closely related to the idea of varied lifespans, since it allowed the replacement of deteriorated parts without damaging others. Although common practice today, the prefabricated concrete parapet panels of the sanatorium are likely the first ones ever to be applied in the Netherlands.
By taking advantage of the specific qualities of materials, it was sought after to construct with a minimum of material used. The dimensions of the concrete beams for the sanatorium follow the moment diagram. Obviously, a lot of carpentry was needed to make the complicated shuttering, but in a period when labour was cheaper than materials, that was not uneconomic.
This strive after optimal construction is referred to by Duiker as 'spiritual economy', that 'leads to the ultimate construction (...) and develops towards the immaterial, the spiritual.' The 'art' of architecture is not in ornamentation, but in technology itself. The search for the optimum in materials and dimensions was considered a process combining the artists' inspiration and the engineers knowledge, an 'engineers-art'. He compared this with lucid Medieval cathedrals, Bach's fugues and the 'horrifying magnitude' of Einstein's theories.
The constructions he used can sometimes only be regarded as optimal, if a short technical life expectancy was being accepted as a starting point. In the case of the sanatorium this could be accepted, since tuberculosis was expected to be exterminated on short term.
Through this optimalisation, buildings were designed with an extreme sensitiveness concerning building physics. On the other hand - or therefore? - building physics were seriously studied by Duiker, Wiebenga, Van Loghem and some other designers of the Modern Movement in the Netherlands. Although identified as 'a great triumph in building construction', Van Loghem warned his collegues in 1936 'that the elimination of the loadbearing function true eliminated one problem, but that the requirements of 'Het Nieuwe Bouwen' on the other hand created at least ten new problems'.
In Holland there was a great interest in constructions as used in the USA, where the frame went through its adolescence. Wiebenga, structural engineer for a number of pinnacles in modern architecture in Holland, published a number of articles on buildings physics in frame-constructed buildings after his return from the USA. Van Loghem, as a convinced

socialist, meanwhile worked in Siberia and followed out series of experiments in building physics. Afterwards, he published a first reference work on building physics in Holland, since, according to him, many modern designers were not able to cope with the problems in building physics that had been introduced with 'het Nieuwe Bouwen' in the first place.

1.5. Duiker dilemma

It is easily understood that the idea of conservation is totally contrary to the concepts of the Modern Movement. According to these, a building's right to exist is not determined by its history, but by its usefulness. By deciding in favour of conservation of Duiker's buildings, we act against his principles at the same time: the 'Duiker dilemma'.
Still, I think there's a number of reasons to preserve this patrimony. First, the Modern Movement has been a fundamental turning point in building history. The quest for fundamental change in architecture in the late 19th century, for architecture that reflected the achievements of the industrial society, was principally answered for the first time.
The second point is, that contemporary architecture will loose an essential part of its history if we do not safeguard some of the early products of this unique approach. Even today, architecture is still deeply rooted in these principles.
Now how about Duiker's own remarks to do away with architecture after it performed its duties? Wasn't it Kafka who layed down in his will to have all his manuscripts destroyed after his death ...... and aren't we glad they didn't? Of course it makes no sense to use the provocative points of view of an emerging group of dissidents as a starting point for our attitude with respect to preservation of their built pamphlets in our time. What counts now, is the cultural impact of their works for today and for the future.

2. PRESERVATION

2.1. New approach required.

In view of these specific architectural principles, preservation of these buildings requires not only other techniques but just as well a totally new approach as compared to traditional conservation. First, a serious problem arises from the fact that many of these designers experimented with materials, constructions and details. Not only did they obviously lacked certain knowledge as compared to what we know today, but a problem just as well is that we do not yet know what exactly was their knowledge and what was beyond it.
I believe that the possibility to detect this original design approach in a restored building should be a determining aspect. Often, the value of a building is taken to mean nothing more than its perceptible impact, or even simply its appearance. If restoration does justice only to the appearance of a building, historical continuity will not adequately be guaranteed. So, just as well for the structural engineer dealing with the preservation of such constructions, quite a few questions emerge. How to deal with buildings where the bare constructions themselves are main part of the original design approach? How to restore them without the aspect of transitoriness being covered by advanced restoration technology for eternity, leaving an artificial memento behind? What can be learned by future students and architects about the concepts of functionality - according to me the key 'message' of the Modern Movement - when going through a restored Zonnestraal ?

2.2. Research

Buildings are sometimes altered so drastically that they do no longer bear any resemblance to their original state, let alone the conceptions they emerged from. Still, some of these buildings should be preserved, one way or another, in order to ensure historical continuity.

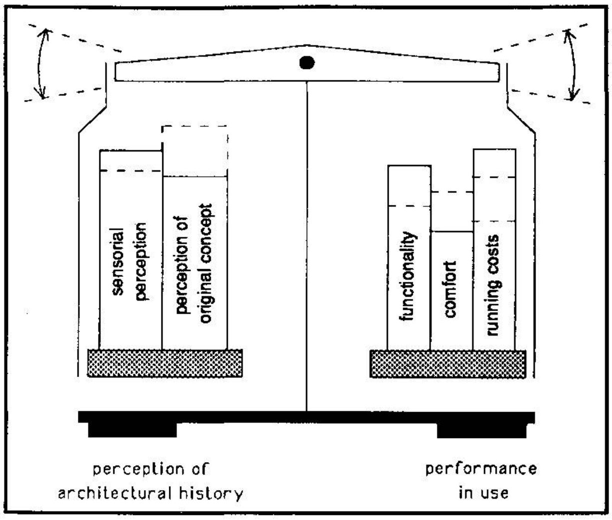

Fig. 3. The key problem is to maintain the correct balance between performance in use and the perception of architectural history. The original design approach is an indispensable element.

But which buildings should be protected and how can this best be achieved?
Numerous parties are involved in the decision-making process concerning conservation: owners and users, authorities and interested outsiders, each with their own legitimate requirements. Today, discussion between these parties can only take place if the objective aspects are being quantified and the subjective aspects are being identified and verified.
Professor Henket and myself, researchers at the TU Eindhoven, developed a method for preparing technical intervention models for restoration and for assessing and weighing up the consequences for a building, in the event of its being restored according to such a model. Numerous aspects are thereby being identified and surveyed, varying from architectural history to building physics, building technology and the annual running costs.
Each intervention has an impact on the balance between perception and use. The key problem is to maintain the correct balance between the intangible factors and the practical use. In order to determine this balance the numerous conceivable forms of intervention are classified and reduced to a number of intervention models, each with another philosophy:
- Model I: Restoring it uncompromisingly to its original state. This might be impossible technically. Still, it is an interesting option for comparison with other models.
- Model II: A variant of model I, but with small, imperceptible technical improvements.
- Model III: Pragmatic restoration, where changes in the nature of the building are introduced using contemporary methods and conceptions of architecture.
- Model IV: 'Ordinary' reuse for economic reasons; there is no question of restoration. This is interesting for comparing the costs of restoration with an everyday case of reuse.

Within these models, forms of intervention are outlined for facades, roofs and each of the different parts of the building. A selection of a particular model is based on the assessment of each models' properties as mentioned before. In 1990, the results of our research have been published, including an extended English summary (see litt. 1). The method has been tested in theory on the Zonnestraal Sanatorium and in practice on the Gooiland Hotel.

3. THEORY AND PRACTICE

3.1. Zonnestraal: a concrete frame

The sanatorium was founded by the Union of Diamond Workers. Money was extremely short, necessitating a cheap building method. Though the reason for not adopting the timber frame, eminently suitable for sanatoria, is still unknown, it is obvious that the building's function benefits from the slender, open construction. The pavilions illustrate how Duiker and Wiebenga incorporated the frame into their architecture. Each pavilion consists of two wings linked by a lounge, set at an angle of 45 ° to eachother to provide plenty of sun and an unhampered view. On the sunward side the rooms have balconies. A corridor on the north side connects the rooms. Across the corridor, a section is devoted to services. From layout to detail, the structure of the sanatorium is based on a 1m50 module. Even the floor-height comes up to the module, creating a three dimensional grid. Until recently, the origin of this module was unknown but we have found a clue in the Dutch requirements for concrete of 1918. If slabs span 3m or less, the formwork might be taken away already after one week, instead of four. In view of the strict 6 months construction schedule, it is likely that the module emerged from this condition.

Generally, the frame consists in long girders, supported every 9m, that support the floors. Beams are haunched at the bearings to follow the moments and to take up shear forces. Consistent with the general principle, the girders cantilever 3m but here without a taper, to match the form of the steel window frames. Floorslabs span 3m, with a 1m50 cantilever at both sides. Between the girders are the patient rooms, the northern cantilever is the corridor, the southern one the balconies.

Being in such harmony with the function, the superstructure is a clear illustration of the design approach. The frame has been designed very light. If we consider the moment diagram of the girders, the combination of 9m spans with 3m cantilevers seems rather optimal and allowed the designers to reduce the dimensions of structural elements, thereby saving precious concrete. The combination of 3m spans with 1m50 cantilevers for floorslabs seems less economic where momentreductions are concerned. Yet it allowed slabs being 12cm thick at their supports and a mere 8cm in the middle of spans and at the cantilever's edges.

Even these thin floors are made with two layers of rebar, each with its own layer of light orthagonal rebar to spread tensions. One can easily imagine that there can hardly be any covering on the reinforcement in such cases.

3.2. Concrete technology

To fill the narrow and complicated shuttering, the concrete has been diluted considerably to make it more fluid. The undesirable watercement ratio as well as the inhomogeneous composition of the concrete resulted locally in an extreme low compression strength, in some columns not even more than that of wet sand. Also, alarming concentrations of gravel have been found. In the 1918 concrete regulations there were no restrictions as to the amount of water. Very fluid concrete was even favoured for narrow shuttering, to guarantee for the rebar to be fully covered. According to the existing literature at the time, very fluid concrete had a lower resistance to compression at first, but in the long run there was hardly any difference to be expected. In contrast, Wiebenga wrote in 1925 that the strength was reduced by adding water, so at the time of construction this was a known fact.

Due to the same reason, also the porosity is a serious problem. The carbonization of the concrete reaches beyond the rebar in most cases. In the upper floorslab also chlorides have been added, presumably to advance curing in winter situations.

After being abandoned, the windows of the pavilion have been broken and the frame is fully being exposed to the climate. The damage caused by corroding rebar is enormous and parts of the sanatorium are now certainly unsafe for an excursion of IABSE's members. The safety coefficient is negative for numerous elements of the frame and the pavilion has collapsed in theory. It is being supported by the light separation walls that, of course, were never meant to do so.

3.3. Restoration options for Zonnestraal

The pavilion could be demolished from the ground floor up and rebuilt with advanced contemporary techniques, using the original dimensions. The appearance of that replica could match the original to the full extend. This would come up to the the demand that one should be able to detect the original design approach. But are we talking about restoration here? In the internationally accepted Charter of Venice the authenticity of material is a main

item, but if we select this option, almost all materials will be renewed.
Another option is to repair and reinforce the frame with contemporary techniques. First of all, that will be far more expensive, but the authenticity of materials will be better guaranteed. Yet, then we have to accept principal and visible changes of the building, such as an increase of dimensions of the frame. So where's Duiker's concept of optimalisation? Also, we have to accept the unintended loadbearing function of the partition walls, which brings about a conflict with Duiker's ideas about frame and infill.
The conflict between the concepts of the Modern Movement and the restoring tradition are clearly illustrated by this example. Today, the future of Duiker's masterpiece Zonnestraal still is uncertain and the buildings stand unoccupied on the moors of Hilversum.

3.4. Gooiland: a corroded image

Duiker got the commission for the luxureous Gooiland Hotel and Theater in 1934. In the late 1980's, a serious threat was the technical condition of the facades: steelframed windows and cork-isolated steelpanels were corroded, the tiles were damaged by frost and the superstructure incidentally heavily corroded as well.
The steelframe is partly placed in a brickwork cavity wall: a peculiar mixture of a 'modern' frame with a traditional construction, that again looks like it was given a functionalist' image by the tile covering. The cavity was meant only to provide thermal isolation and was therefore not ventilated. The tiles were necessary to keep the water out, since by lack of ventilation, water could not vaporize and leave. But when the facades collect heat in the summer, the steel shows a relatively greater extension as compared to the surroundings and cracked the tiled surface. Through these cracks, water entered after all. Since the lead strips on the beams, meant to lead water outside, were cut shorter than originally intended, water could reach the steel. The beams corroded, increasing their volume and pushing the outer wall away.

3.5. The risk of thermal isolation

When testing our research method on the Zonnestraal Sanatorium and later in practice on the Gooiland Hotel, some remarkable conclusions could be drawn for both.
With buildings like this, of which the outer skin in fact is one enormous thermal leak, one should be very careful with increasing thermal comfort and to improving thermal behaviour of the constructions. Intervention in one part of the building, for instance the addition of double glazing, often makes other measures necessary elsewhere, such as the isolation of columns and beams in the facade by adding layers of plastered foam to avoid condensation. Quite easily, the lucid character of such a building will be completely lost this way.
Also, such interventions could have a negative effect on the running costs, as was the case for both buildings. In the first place, double glazing is expensive and lasts only approx. 19 years. This is a heavy burden on the annual budget, that is often not compensated with the savings in energy. Then, if we want this addition to comply with the building's historic character, so many individual technical solutions for windowframes etc. are necessary, that the costs of intervention easily become disproportioned. Finally, the sealing of contemporary windows is so effective, that mechanical ventilation could be required to avoid a condensation surplus, resulting in another increase of investments and running costs.
Obviously, with respect to both history and the running costs, it proves to be most advantageous to retain the original design virtually intact, in accordance with model II. Then, annual running costs over a fifty-year period are lowest of all real restoration options (models I, II, III). The level of comfort is not very high under model II, so a use must be found which is appropriate to these characteristics.
The abovementioned factors led to the decision to restore Gooiland for its re-use as a

cultural centre, according to model II, applying single glazing in repaired steelframed windows. When the windows were being re-mounted, some 30 mm extra space was provided to apply incidental isolation to avoid direct thermal leaks to the superstructure. Also in the tiled parts some thermal isolation was provided where beams and columns are located, to prevent the steel from extreme extension. The foam is covered by a layer of cement on stainless steel mesh. To provide room for this incidental isolation and also to allow the tiles being atached in a slightly thicker layer of mortar than originally, the outer surface has again been moved outward approx. 30 mm. Also some ventilation was provided. In harmony with the choice for single glazing also the closed facades remained unisolated. One third of the steel in the facades had to be replaced.

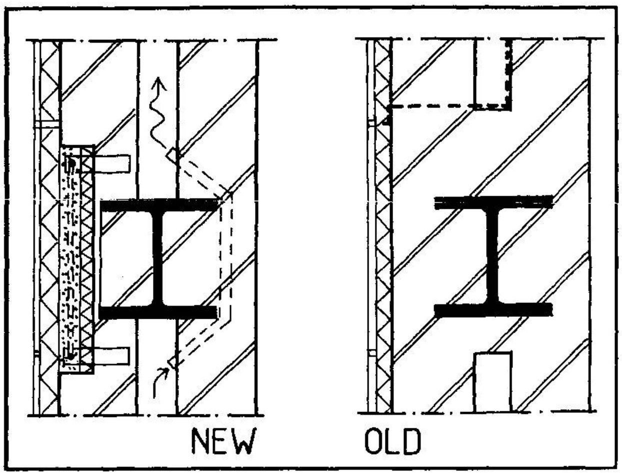

Fig. 4. The steel frame of the Gooiland hotel is placed in a brick cavity wall, a solution that caused severe damage. Today the steel is protected against extreme temperature changes

The main advantage of using our survey of models proved to be, that conclusions on the appropriate type of restoration can be drawn before restoration plans are actually developed. Also, the results of restoration with respect to maintenance and running costs will be known in advance. This way, a plan for restoration of Gooiland could be executed, that is in harmony with both its historic character and the contemporary requirements of the owner, as well as entailing the lowest possible annual costs. We hope to achieve such positive results for Zonnestraal one day as well.

5. DOCOMOMO

To conclude I would like to mention that an interdisciplinary and international network of experts exists that specifically studies the problems related to the restoration of Modern Movement buildings. DOCOMOMO is the International Working party for DOcumentation and COnservation of buildings, sites and neighbourhoods of the MOdern MOvement. The very diversity of participating countries and regions is indicative of the fact that no single universally applicable solution for the conservation of this architecture can be assumed. I have tried to illustrate that with the Zonnestraal case. On the contrary, it is the intention of DOCOMOMO to start and continue the necessary debate over the years to come, in order to arrive at a better understanding of this issue.
An international secretariat is hosted by our university, that publishes a newsletter twice a year. Our latest international conference was last year at the Bauhaus Dessau, Germany, and we hope to meet again in Barcelona in 1994. As secretary general of DOCOMOMO, I am happy to say that at the moment over 30 countries participate in DOCOMOMO, including a total of over 300 members.

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Posters - Session 4

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Protection Buildings for Ruins and Monuments

Bâtiments de protection pour des ruines et monuments

Schutzbauten für Ruinen und Monumente

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INTRODUCTION

The degradation of our cultural heritage has been increasing rapidly during the last decades. Examples from Italy clearly demonstrate the increased rate of degradation.

There are a number of cases that warrant the concern shown in recent years, and if urgent measures are not taken, historical buildings of great significance will inevitably be lost.

The obvious conclusion is that the increasing air pollution must be the cause of this disturbing development. Therefore, the problem of preservation has met with new challenges.

During the last decade a few proposals for protection buildings for cultural objects have been presented, e.g. a shed roof over Parthenon on Acropolis, Athens, and a protective shell over the Column of Marcus Aurelius in Rome, (Museum, Quart. rev., Unesco, 153, 1987).

In Norway, the ruins of the ancient cathedral at Hamar have been degrading since 1567, when the church roof burned down. During the last decade the Norwegian Central Office of Historical Monuments and Sites decided to build a protection building over the ruin, and an architectural competition was held in 1987.

In 1990 a research project, Eurocare Carebuild, was started, having the objective of developing a technology package which may serve custodians having objects that need protection.

THE PROTECTION BUILDING AT HAMAR

The protection building at Hamar, designed by the architects Lund & Slaatto is an aluminium/glass building having warped, skew glass walls, see Fig. 1.
For the project a new aluminium space deck system for triangular glass panels has been developed. The system can adjust itself to form a warped surface, see Fig. 2.

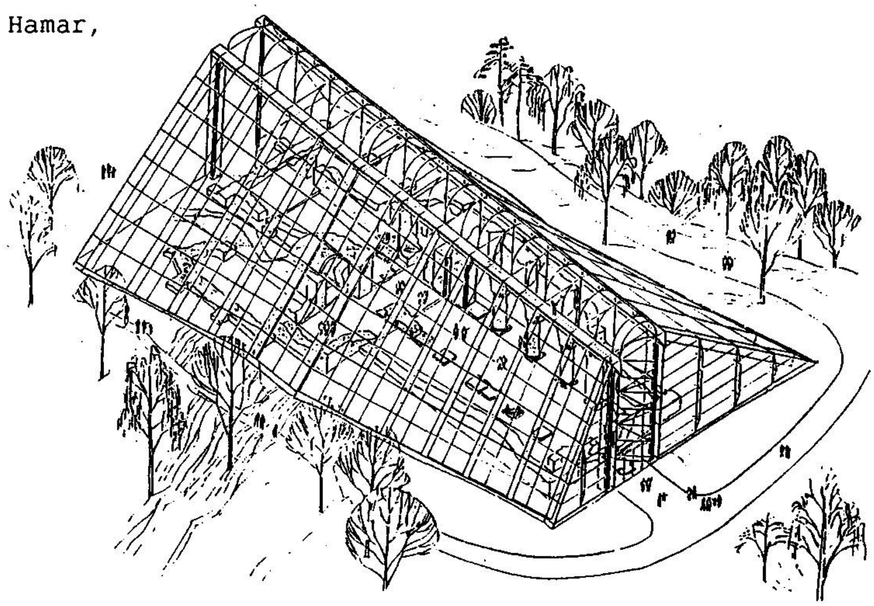

Fig. 1 Drawing of the protection building

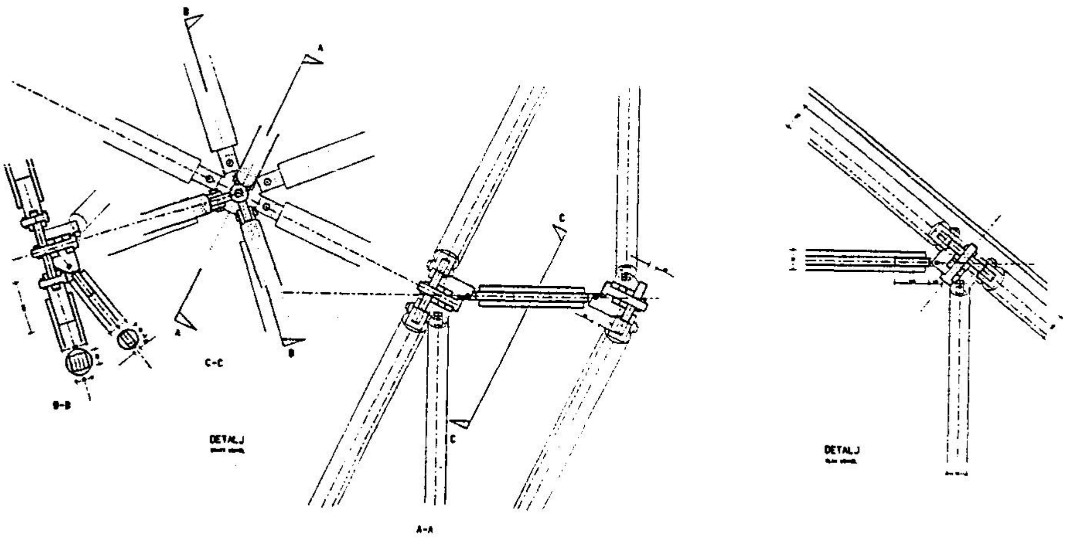

Fig. 2 Joints of the designed protection building at Hamar

EU 446 EUROCARE CAREBUILD

The research project has the following project profile, see Fig. 3, and is planned to run till 1995.

Title	: EUROCARE CAREBUILD Envelope buildings for historic buildings, monuments, stone ruins, etc.
Supported by	: Norwegian Council for Scientific and Industrial Research
Participants	: Norwegian Central Office of Historical Monuments and Sites NILU (Norwegian Institute of Air Research) Lund & Slaatto Arkitekter A/S Erichsen & Horgen A/S, HVAC-consultants Dr.techn. Kristoffer Apeland A/S, structural consultants University of Lund, Sweden (Professor Bo Adamson)
Project leader	: Professor Kristoffer Apeland, Oslo School of Architecture

Fig. 3 EU 446 Eurocare Carebuild Project Profile

So far, interesting results have come up in connection with:

- Consequences for archeological layers when a protective shell is built over the layers (published in Norwegian).
- Special energy use and conservation aspects for protective shells over stone ruins and buildings (to be published).
- Application of RILEM/CIB method for prediction of service life.
- New design of space decks in aluminium for warped surfaces.

The technology package will be further developed during the project.

Two Aspects of Structural Reinforcement of Historic Monuments

Deux aspects du renforcement structural des monuments historiques

Zwei Aspekte struktureller Verstärkung historischer Bauwerke

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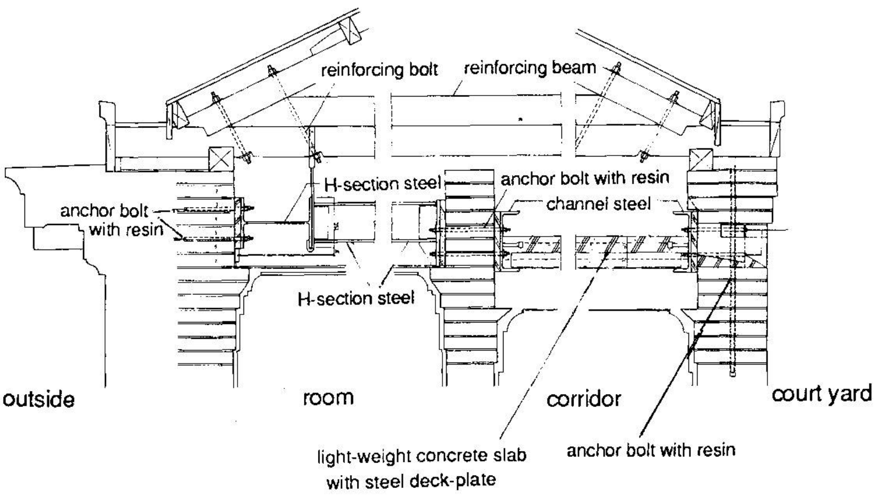
Structural reinforcement of historic monuments can be inducted to preserve buildings either in their authentic condition or only in their outer appearance. Among recent restoration-conservation in Japan, remarkable examples are introduced in the following:

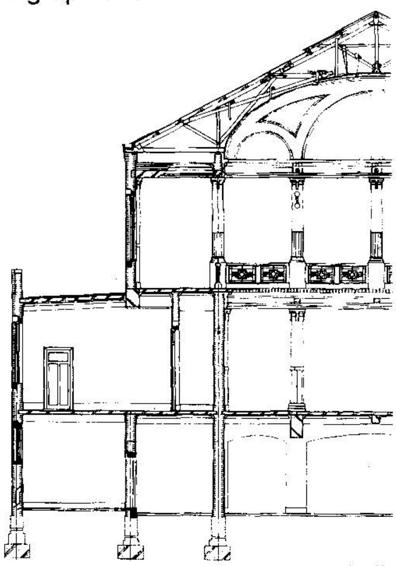
Example 1. Nagoya High Court and District Court Building (Important Cultural Property)
Completed in 1922, Baroque Revival Style, building area: 2,241.8m^2, 3 stories, base: brick & concrete, wall: brick, slab: reinforced concrete, beam: reinforced concrete, arch: brick, roof truss: timber, roof: copper, restoration: commenced in 1984 and completed in 1989,

Reference: Japanese Association for Conservation of Architectural Monuments, "Restoration of Nagoya High Court and District Court Building 1984-1989", Nov. 1989, Nagoya City.

On the poster, the whole structural reinforcement will be shown in detailed graphic form.



reinforcement on the top of longitudinal walls



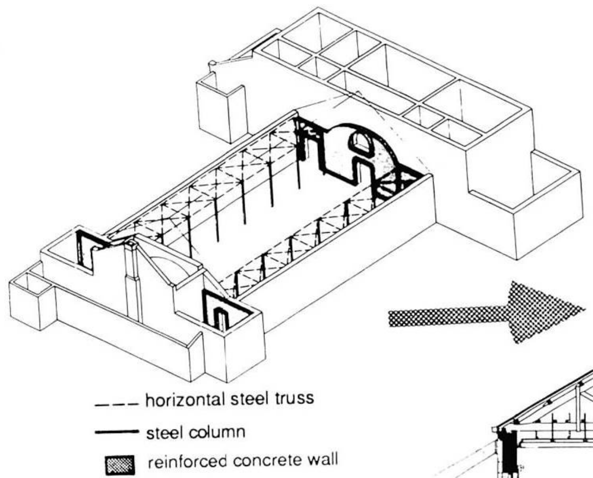
section of hall

The highlight of this building is the central hall, a square of approximately 17x11 meters, which has an open space, 10.9x8.2 meters, from the ground floor to the ceiling of the 2nd. floor covered with a big top light. This space supported by brick columns and walls bound together with reinforced concrete beams and slabs was insufficient concerning resistance against earthquake. In order to prevent at the top of brick wall from bending, it was necessary to achieve a monolithic horizontal reinforcing structure. As important as the hall's spatial qualities as a cultural property are, as weak they are seen from the structural point. After thoroughly examination and discussion, they innovated careful and proper ways that the original appearance of hall was maintained without reminding any trace of reinforcement. To achieve a monolithic structure against the earthquake, the top of hall and roof trusses were reinforced by using buttresses, reinforcing bar, anchor bolt with resin and steel. The buttresses along the 2nd floor's wall face to the court yard can be seen only from the 2nd floor's veranda behid the hall. To connect walls/columns to beams/slabs, reinforcing bars and anchor bolts with resin were inserted in the brick columns, walls and beams, channel steel beams were used along the top of the brick walls and H-section or channel steel beams were used along the top of the façade and back walls. These reinforcements were put inside the sturctural frame and roof trusses.

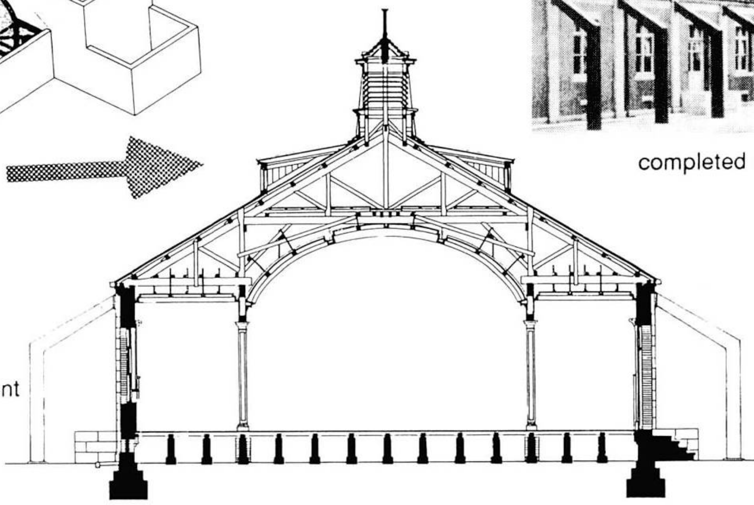
Example 2. Yamagata Prefectural Government Assembly Hall (Important Cultural Property)
Completed in 1916, Gothic Revival Style, building area: 866.4m², 1 story/partially 2 stories,
base: brick, wall: brick, floor framing: timber, roof truss: timber, roof: slate,
restoration: commenced in 1986 and completed in 1990,

Reference: Japanese Association for Conservation of Architectural Monuments, "Restoration of Yamagata Prefectural Government Building and Assembly Hall Vol.1 / The Assembly Hall Report on the work 1986-1990", Mar. 1991, Yamagata Pref.

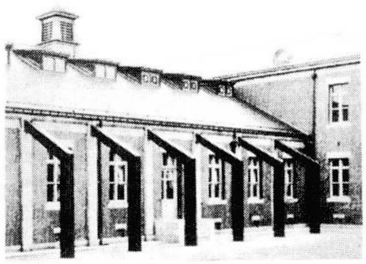
On the poster, the whole structural reinforcement will be shown in detailed graphic form.



horizontal steel trusses reinforcement
not realized



buttresses reinforcemment realized



completed exterior

The design of this building, which has a Basilica style vaulted ceiling with top lights, is unique for governmental assembly halls in Japan. Although the interior had been changed, the close investigation during dismantling made it possible to reconstruct the original condition with original materials. Reinforcing bars and structural steel were needed to strengthen the structure against earthquakes. For there was not adequate space to put horizontal reinforcing trusses along the top of the brick walls, the common reinforcing way like Ex.1. could not be applied. It would have been inevitable to change the original interior condition with additional new walls to support the trusses and they would have overlaid important vestiges on the original wall. After many examinations and discussions, it was decided that the walls of the hall were to be reinforced by using exposed structural steel such as flying buttresses on the exterior. Although buttresses are very striking and disturbing on the exterior, it was judged that the authenticity of the interior with its historical materials is more valuable and has to be given preference against an outer appearance with no reinforcing additions. This was the first time of this kind of restoration of a designated cultural property. In almost all previous restoration cases, preserving the historical outer appearance as a landmark in the townscape and preserving the historical materials was considered as equally important. The way of reinforcement in this case has given the chance to reconsider when restoring and preserving historical architecture.

Recently, conservation of Western style architecture built of brick during the Meiji and Taisho periods (1868 ~1912/1912~ 1926) is increasing. Brick structures which are relatively new to Japanese architectural tradition were not as highly developed as wooden structures. As was the case with wooden buildings in Japan, brick structures have also been refined in a principle that conceals the reinforcement in the interior and façade appearance, using reinforcing bars, braces and trusses in the roof or floor framing. But reinforcement sometimes deprives the architecture of some authenticity, in which the historical meaning which shows the particularity of that period is inherent. Reinforcement and authenticity are often contradictionary.

When we decide what and how we must conserve, it is inevitable to define in detail what is authentic in the architectural construction, i.e., authentic style and authentic design. Yamagata's example, where the reinforcing structural steel is exposed, is in this sense remarkable. Planning the conservation and active reuse of historic monuments, we must primarily consider their "authenticity". It is very important to identify the historical value and meaning from various points of view. Suitable reinforcement should be decided according to these considerations.

Seismic Evaluation of a Brick Masonry Building of 1895

Comportement aux séismes d'un bâtiment en maçonnerie de 1895

Erbebenverhalten eines Backsteingebäudes von 1895

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1. Introduction

This papar describes the seismic appraisal of exisiting masonry building and the measures needed to ensure the structural meets modern Tokyo seismic requirements. Fig. 1 shows the first plan.

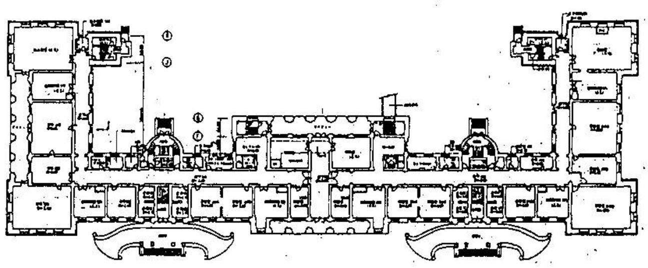

Fig 1 Building Plan (1st Floor)

2. Response Analysis of the building

As the structural characteristic in the plan, X and Y directions are different, separate models were created for each direction (see Fig. 2). Each floor was assumed to consists of 5 lumped masses, connected by assumed stiffness for floor slab derived from the test-recorded stiffness value for the wall (see Part 1) . Thus vertically, the masses are connected by the brick wall stiffness value based on the shear modulus, and horizontally the masses connected by the floor slab stiffness having both shear and axial components.
The calculation models are shown Fig. 3.

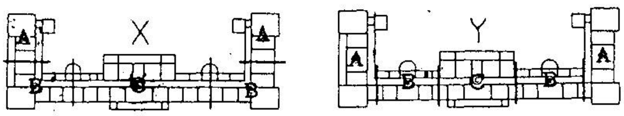

Fig 2 Building Sub-division for Modeling

By Comparing the buildings dynamic characteristics, the input seismic waves adopted for analysis were EL CENTRO (1940 NS), HACHINOHE (1968 NS), TAFT (1952 EW) and TOKYO (1956 NS).

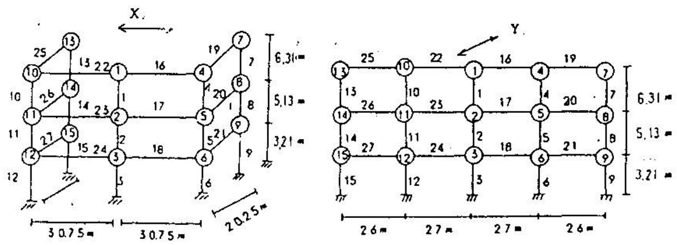

Fig 3 Building Model for Calculation

The fundamental natural frequency of the structure was calculated as 5 Hz (approx.) and the peak value of input acceleration normalized to 200 cm/s^2. The base of the structure's foundation was assumed as fixed against rotation in consideration of the restraint provided by the soil and the soil's damping factor ratio assumed as 7%.

From the analysis, the maximum response anaylsis in the X direction was 561 cm/s^2 (TAFT), representing an amplification factor of 2.81, and in the Y direction was 610 cm/s^2 (HACHINOHE), an amplification of 3.05. (Table 1)

Table 1 Maximum Response of Mass Points

Max. Response		X- direction TAFT 1952 EW 200cm/s²						Y- direction HACHINOHE 1968 NS 200cm/s²					
Block	No.	DISP.	(TIME)	VEL.	(TIME)	ACC.	(TIME)	DISP.	(TIME)	VEL.	(TIME)	ACC.	(TIME)
C	1	0.33	(4.69)	11.1	(4.64)	406.	(4.60)	0.64	(4.16)	14.2	(4.10)	580.	(4.15)
	2	0.20	(4.69)	6.6	(4.64)	272.	(4.69)	0.46	(4.16)	10.3	(4.10)	462.	(4.16)
	3	0.08	(4.69)	2.5	(4.64)	202.	(3.71)	0.15	(4.16)	3.1	(4.10)	278.	(4.16)
B	4	0.37	(4.69)	12.4	(4.65)	451.	(4.69)	0.67	(4.16)	14.9	(4.10)	610.	(4.15)
	5	0.25	(4.69)	8.0	(4.65)	319.	(4.69)	0.43	(4.16)	9.6	(4.10)	438.	(4.15)
	6	0.09	(4.69)	2.9	(4.65)	205.	(6.55)	0.18	(4.16)	4.0	(4.10)	295.	(4.16)
A	7	0.47	(4.70)	15.7	(4.65)	561.	(4.69)	0.50	(4.16)	11.1	(4.10)	490.	(4.15)
	8	0.31	(4.70)	10.1	(4.65)	386.	(4.69)	0.34	(4.16)	7.5	(4.10)	389.	(4.15)
	9	0.10	(4.70)	3.3	(4.65)	214.	(6.56)	0.13	(4.16)	2.7	(4.10)	265.	(4.15)
B'	10	0.40	(4.69)	13.4	(4.65)	484.	(4.69)	0.63	(4.16)	14.1	(4.10)	587.	(4.15)
	11	0.26	(4.69)	8.6	(4.65)	339.	(4.69)	0.40	(4.16)	8.9	(4.10)	417.	(4.15)
	12	0.10	(4.69)	3.1	(4.65)	210.	(6.55)	0.17	(4.16)	3.7	(4.10)	286.	(4.16)
A'	13	0.47	(4.70)	15.7	(4.65)	558.	(4.69)	0.47	(4.16)	10.4	(4.10)	472.	(4.15)
	14	0.31	(4.70)	10.1	(4.65)	387.	(4.69)	0.29	(4.16)	6.4	(4.10)	355.	(4.15)
	15	0.11	(4.69)	3.3	(4.65)	216.	(6.55)	0.11	(4.16)	2.3	(4.09)	253.	(4.15)

3. Structual Assessmemt from Results of Response Analysis

Masonry allowable stresses are obtained directly from testing and divided by a safety factor of 1.5 for short term (seismic) conditions. (Table 2)

Maximum responses shear forces and average shear stresses, based on the 200cm/s² input acceleration, are shown in Table 3.

Areas exceeding the allowable stress are also indicated (mark *).

The stresses from the maximum response forces in the slab are in all cases less than allowable stresses.

From the results discussed, it was desided to reinforce those walls which were shown to be over stressed, by constructing reinforced concrete strengthning walls · connected by shear stud bolts to the existing walls. The maximum shear stress in the upgraded wall,which in all cases are less than the allowable stresses.

Regarding out-of-plane direction (perpendicular to masonry walls),shear forces based on the maximum response acceleration of inplane direction are adopted as the external forces to check the wall bending bearing capacity (Fig.4). By means of this calculation, at thin walls such as 380mm THK.,510mm THK. steel plates (3.2mm THK.) are installed at both sides of the wall surface to strenghthen flexural capacity.

Table 2 Allowable Stress (MPa)

		Testing Value	Short Term
Compression		6.0	4.0
Bending		0.15	0.10
Tension		0.15	0.10
Shear	3rd fl.	0.30	0.20
	2nd fl.	0.35	0.23
	1st fl.	0.40	0.27

Table 3 Maximum Shear Stresses in Wall

X-Dir	FL	Mem.No.	Weight (KN)	Shear Area (m²)	Sh. Force (KN)	Shear Stress (MPa)	
A	3	7	8480	20.7	4150	0.20	
	2	8	9650	24.1	7520	0.31	*
	1	9	8750	35.4	9500	0.27	
A'	3	13	8780	22.5	4390	0.20	
	2	14	10570	26.0	8040	0.31	*
	1	15	9720	37.6	10300	0.27	*
B	3	4	11360	34.6	5650	0.16	
	2	5	13820	41.3	10010	0.24	*
	1	6	11830	51.9	12990	0.25	
B'	3	10	11430	31.9	5610	0.18	
	2	11	14010	39.7	10040	0.25	*
	1	12	13060	49.7	13290	0.27	
C	3	1	26410	75.8	13240	0.17	
	2	2	33150	115.9	23730	0.20	
	1	3	29230	140.1	31330	0.22	

Y-Dir	FL	Mem.No.	Weight (KN)	Shear Area (m²)	Sh. Force (KN)	Shear Stress (MPa)	
A	3	7	14660	49.6	8700	0.18	
	2	8	16750	54.1	16070	0.30	*
	1	9	14990	73.3	20630	0.28	*
A'	3	13	15020	43.2	8510	0.20	
	2	14	18490	64.5	16180	0.25	*
	1	15	17200	87.9	21180	0.24	
B	3	4	10350	17.6	4740	0.27	*
	2	5	13440	29.7	10190	0.34	*
	1	6	11180	31.6	12790	0.40	*
B'	3	10	10370	18.2	4790	0.26	*
	2	11	12180	29.7	9370	0.32	*
	1	12	11160	31.6	11890	0.38	*
C	3	1	16050	49.3	9700	0.20	
	2	2	20340	43.0	18680	0.43	*
	1	3	18060	74.6	24220	0.32	*

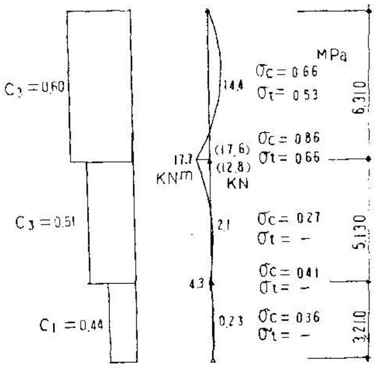

Fig 4 Bending Diagram Perpendicular to Wall

4. Conclusion

From the response analysis, it was shown the the natural period of the structure is 0.2 seconds as compared to 0.33 seconds for the surrounding soil. This large difference would appear to partly explain why the building didn't suffer any severe damage when struck by the Kanto earthquake.

Thus, structural stability is maintained for an input level up to 200 cm/s² at the ground surface. Further, if the ultimate strength is assumed to be equivalent to the material strength obtained from testing and some of the walls are upgraded as described above, the structure should withstand ground surface accelerations up to 300-400cm/s².

Despite the building's 100 years of age,it can be seen that this famous old building can remain in their masonry building for many years to come. This study also illustrates how masonry (or indeed other materials) can be engineered to create seicmic resistant structures.

Restoration of an Ancient Masonry Building after the 1990 Earthquake in Macedonia

Restauration d'un bâtiment ancien en maçonnerie après le séisme de 1990 en Macédonie

Instandstellung eines antiken Mauerwerksgebäudes nach dem Erdbeben von 1990 in Mazedonien

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1. STRUCTURAL SYSTEM AND STATE OF DAMAGE OF THE BUILDING

The analysed two storey residential building, dating from the beginning of the 20th century, was one of the buildings to suffer most from the earthquake in Gevgelija in December, 1990, that struck the frontier regions between Macedonia and Greece (intensity VIII - IX).
The building is quite impressive itself, measuring 100 meters in length and 11.8 meters in width, with a decorated entry on the south side and a well preserved elegant roof wooden structure.
Its structural system is quite complex: at the ground level a cast in situ concrete slab connects the massive brick facade walls with a row of steel profiled columns at midspan; at the first level a wooden floor strucure is supported by the walls and the columns.
The partition walls are of different dating and origin: the recently introduced ones are made of concrete, while the original ones consist of wooden frames filled with compacted earth.
The structure survived the earthquake but was considerably damaged. The damage was concentrated at the first level and at the roof, the concrete slab at the ground level preserving the lower part of the structure from any considerable damage. In the upper part the partition walls have been seriously damaged, some have collapsed. The facade brick walls, although quite massive, have cracked, especially at the roof level. The wooden floor structure has been seriously damaged, and in places has collapsed. The gable walls have been dislocated at the corners and have cracked, especially at the roof level. This changes were detrimental to the integrity of the structure as a whole.

2. PROPOSALS FOR REPAIR AND STRENGTHENING OF THE STRUCTURE

The project for repair consists of partial grouting at the places of damage on the facade walls, respecting at the same time the original facade and wall finishes. At the corners where important cracking and dislocation have taken place, vertical cast in situ elements have been embedded. All the original partition walls have to be replaced by new modern walls, made of brick. The old wooden floor structure has to be entirely replaced by a new one.

As for the strentghening of the structure genuiting of the key perpendicular walls with a 5 cm layer of concrete has been proposed. Thus, a sufficient capacity of stength and deformability in the perpendicular direction of the structure can be achieved, the capacity of the structure in its original state due to the limited length of the walls (11.8 m.) being very low.The stiffening of the structure at the level of the first floor is to be provided with genuiting of all the facade walls, from the inner side between the celling and the lentels, a vertical distance of 70 cm. In such a way the wooden floor structure is going to be braced by a concrete belt all around the perimeter of the building. Futhermore, a thin concrete slab is to be embedded at the top of the structure, connecting the individual wall elements and thus contributing to the overall stiffness of the structure. With these measures a synchronized behaviour of the individual wall elements will be obtained, thus enlarging the posteleastic capacity of the structure and preventing its collapse.

3. STABILITY VERIFICATION OF THE RETROFITTED STRUCTURE TO A MAXIMUM EXPECTED SEISMICITY LEVEL

The structure in its original and repaired state has been modeled using a simple cantilever system fixed at the base, with masses concentrated at the two floor levels. With this mathematical model a dynamic time history analysis with different earthquake records has been performed . The level of the maximum expected earthquake action, as well as the different types of seismic records to be applied, have been determined with a special seismological study. In the analysis, certain ductility capacity has been allowed to the masonry walls (1.8 for the genuited perpendicular walls and 1.5 for the longitudinal facade walls).
Even though the maximum expected earthquake level is very high (42 % of "g"), a satisfactory response for the retrofitted structure has been obtained. The required ductility by the earthquake does not exceed the ductility capacity of the walls, in longitudinal and perpendicular directions.
The analysis shows that the retrofitted structure posseses sufficient capacity of strength and deformability - ductility to survive strong ground motions without considerable damage.

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Repair and Strengthening of a Medieval Brickwork Bell Tower

Réparation et renforcement d'un clocher médiéval en maçonnerie

Reparatur und Bewehrung eines mittelalterlichen Glockenturmes aus Mauerwerk

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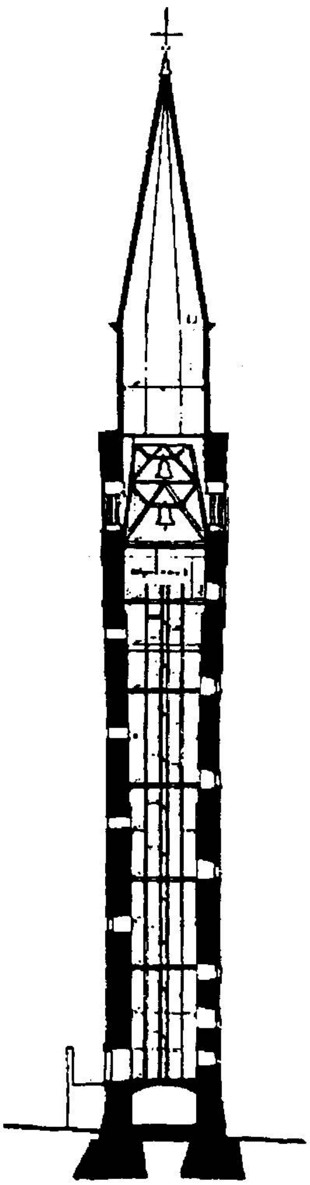


Figure 1

The described case study is the evaluation, repair and strengthening of the bell tower of the Cathedral of Pordenone, in Friuli, the Italy's region stricken by a strong earthquake in 1976. The structure, built during the last years of the 13th century, is 72 m high (Figure 1) and dominates with its imposing mass the historical center of the town.

The masonry walls are sufficiently strong and generally well constructed and preserved, and demonstrated to be tough enough to survive the earthquake.

Some major concerns about its actual safety level were however justified by the not negligible inclination (about 1%) and by evident traces of past heavy damages, often caused by lightnings, which were repaired between the end of the last century and the beginning of the present one.

The tower history and the repairs performed during the centuries have been found to be well documented.

In particular, pictures exist of the damages at the northern corner of the tower, which was near to collapse at the beginning of this century, and the inner part of such corner still presented damages and cracks in 1990.

The previous interventions appeared to be in general very efficient. Such are in particular the strong iron ties applied at five different levels and the reconstruction of the masonry in the outer part of the northern corner.

The preliminary investigations made in order to decide if new strengthening interventions were necessary were first of all based on the study of the existing documentation and on the accurate survey for detecting all the damages and cracks of the masonry walls.

Soil characteristics were investigated by spt tests and water table was monitored with piezometers.

The mechanical characteristics of the existing materials, masonry and iron ties, were then assessed by means of adequate in situ tests (flat-jacks for masonry).

Measures were also made of the dynamic response properties of the overall structure. Based on the obtained data FEM numerical models were constructed and used for analysing the structural behaviour of the whole structure and the relevant local states of stress in the masonry walls: in static conditions, during the bells motions and during the earthquake. The results showed the need of a strength improvement. Interventions were finally decided and executed aimed to improve the actual conditions of the structure without however substantial changes.

This was obtained first by reconstructing damaged masonry portions using traditional materials and techniques. In particular hand made solid clay units and mortars and injections made with hydrated lime mixed with "cocciopesto", which gives hydraulic properties to the admixtures, have been used.

Then, the inactive ties have been substituted and some new ties have been added. Finally, the confinement of the inner parts of the masonry walls have been substantially improved where an external confinement is already provided by the existing iron ties by means of five steel diaphragms in the positions indicated in Figure1. They are included in the system of the internal stairs as shown in Figure 2 a. Their principal characteristics are shown in Figure 2 b.

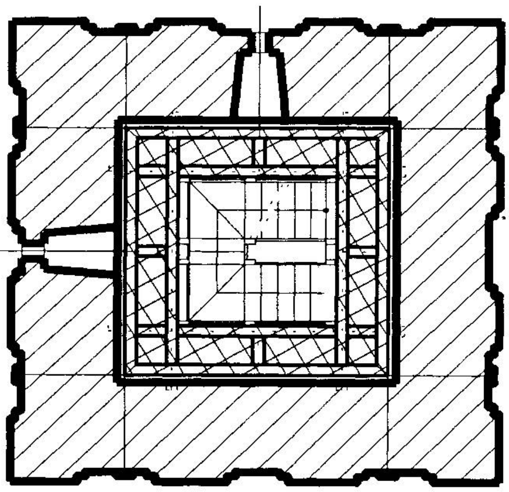


Figure 2 a: section of the tower at the level of one of the diaphragms.

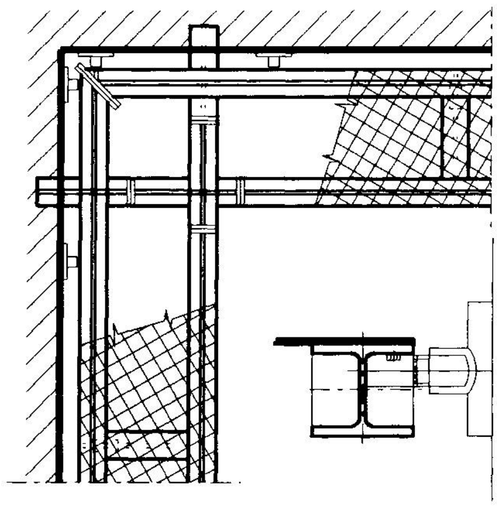
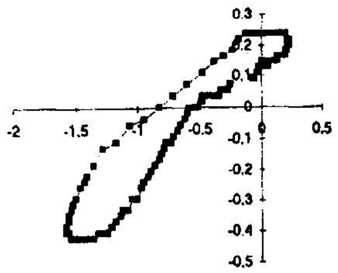


Figure 2 b: details of the steel diaphragms and of the devices used to put them in contact with the masonry walls.

The installation of the diaphragms inside the tower has been made possible by the use of several bolted joints. This construction technique allows the maintenance and, in case, the removal or substitution to be easily executed.



A monitoring system was finally installed to realise an automatic control of: tower inclination, cracks movements, temperature in the walls thickness, water table level and dynamic excitations.

In Figure 3 the daily variation of the tower inclination is shown, which is typically measured through the displacements of the pendulum, in sunny days.

Strengthening of Pisa Tower by External Post-Tensioning

Renforcement de la Tour de Pise par précontrainte extérieure

Verstärkung des Turmes zu Pisa mittels externer Vorspannung

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Mech. Eng.
VSL International AG
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In 1991, VSL International Ltd. was entrusted with a study of the temporary circumferential prestressing of the Pisa Tower.

The job was to present a solution with temporary hoop prestressing tendons at the first level "Loggia" of the Tower. Those tendons were intended to prevent buckling of the masonry on the South Side of the Tower.

The idea was to wrap a number of prestressing strands around the circumference of the tower in the area of the first loggia as shown in Fig. 1. It was decided to place monostrands distributed over the height of the loggia wall. In addition, 8 monostrands were placed above the arches of the lowest level columns. The tendons were initially stressed to 50 % of their guaranteed ultimate strength. Very strict requirements with regard to the visual and functional effect on the tower were set:

- temporary tendons
- small visual impact on the tower
- no detrimental effects on the marble surface
- long term corrosion protection
- no grease in the monostrand
- strand overlength nicely hidden
- resistance against microorganisms and similar
- resistance against UV-radiation
- proven technology must be used
- stressing system insensitive to human error

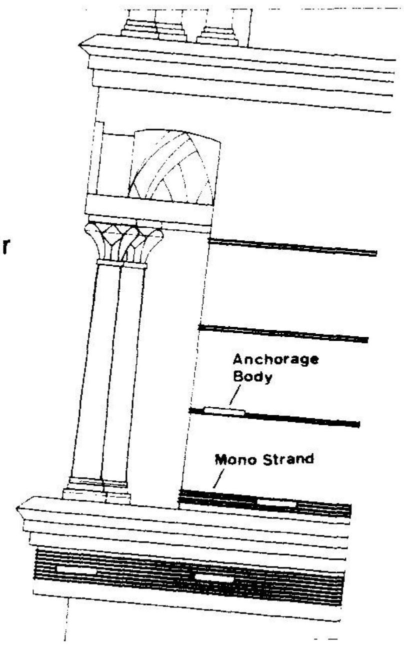

Fig. 1: Tendon Lay-Out at First Loggia

These requirements called for a design which allows the tendons to be force-monitored, retensioned, detensioned and removed. The chosen prestressing system does not interfere with existing materials. The tendons are finally held in place by their prestressing force.

The anchorage bodies were machined out of full material St 52-3 and then hot dip galvanized with a zinc thickness of min 80 μm.

The 7-wire prestressing strand according to Euronorm 138-79 is hot dip galvanized after drawing of wire with a zinc coating of min. 50 μm.
The strand is sheathed by a sleeve made of PVDF (Polivinylidenflouride), diameter 20/16.2 mm. This material is highly resistant against chemicals and weathering, and allows low-friction sliding of the strand in the sleeve inspite of the absence of any grease. The wedges were treated with a new corrosion protection method called "Dacromet 320", similar to hot-dip galvanizing but 2.5 times more effective for the same coating thickness. Because of the requirement that there must not be any interference with existing materials the anchorage had to be a "flying" anchorage, i.e. it slides with respect to the marble surface while stressing. A special PVDF pad was therefore provided between the anchorage body and the structure. This pad allowed low friction sliding, at the same time protecting the marble.

The anchorage system has minimized outside dimensions, thus causing hardly any visual impact on the structure. After stressing, the strand overlength is hidden in the anchorage body and covered by closing the lid as shown in Fig. 2.

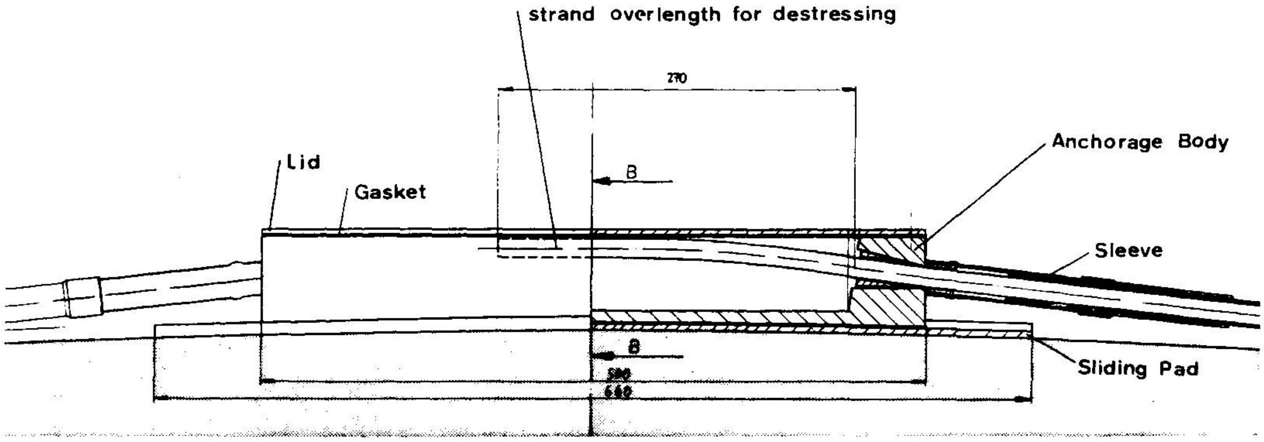

Fig. 2: Specially Designed Monostrand Anchorage Body on Sliding Pad

Although "proven technology" was used, the importance of the structure justified the execution of extensive testing. A full-scale stressing test was carried out on a circular silo structure with a radius similar to that of the Pisa Tower. The aim of the test was to demonstrate the adequacy of the entire system, in particular the anchorage body and the PVDF sliding pad. The experience gained during the installation and stressing confirmed that the special anchorage body, and the stressing procedures work as intended. The friction coefficient of the assembly of non greased strands was approximately $\mu = 0.11$.

The installation of the hoop tendons was executed in June 1992 by the VSL licensee PRECO.

Structural Reinforcement of the Cathedral in Cava dei Tirreni

Renforcement structural de la cathédrale de Cava dei Tirreni

Statische Festigung der Kathedrale von Cava dei Tirreni

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1. SUBJECT OF THE INTERVENTION.

The main building of St. Adiutore's Cathedral in Cava dei Tirreni is composed of a large broad nave, two side aisles, a transept and an apseidal area. The construction of the Church, lasted about 55 years, begun in the second decade of the XVI century. However, many subsequent interventions caused during the time a remarkable "static confusion" of the resisting elements which, added to the original high seismic vulnerability of the structure, led to the complete closing of the monument after the Irpinia earthquake of November 23, 1980.

2. DIAGNOSIS OF THE DAMAGES.

The main diseases were: deep fractures in the arches and in the vaults of both the side aisles, compression failure of some masonry pillars and a substantial foundation settlements in a wide zone between the principal facade and the right side aisle.

The masonry buttresses, built to contrast the seismic lateral loads, fulfilled their function during last earthquake event, but disjoined from the bearing walls and plasticized in the cross sections of lowest strength. The principal facade presented dangerous slipping surfaces and disjunction from the aisles walls, and the outbuildings were diffusely damaged. The wooden coverings were in a severely degraded state.

3. SOLUTION ADOPTED.

The proposed intervention therapy, aimed at the reduction of the building seismic vulnerability through retrofitting and strengthening the existing structures.

First of all, with reference to the central nave and the lateral aisles, it was decided to prop the fractured vaults and arches, to demolish the heavy masonry buttresses, and to disassemble the wooden covering.

Retrofitting of the existing masonry structures has been performed by:

- confinement of the damaged masonry pillars, using closed steel plates as stirrups and an external cover of spritz-beton;
- sewing of the arches fractures through insertion of radial steel bars and injection of grouting mortar;
- strengthening of the vaults through superposition of a reinforced concrete slab, connected to the vaults by means of glued steel nails.

The task of contrasting the seismic lateral loads is now performed by reinforced concrete frames effectively shaped, which are much lighter than the demolished tuff buttresses and have a much higher strenght per unit volume. These elements bear the new r.c. pitch roof of the lateral aisle, and allow a complete connection between the pillars-arches complex and the masonry walls of the central nave: r.c. and masonry are linked together by means of steel nails.

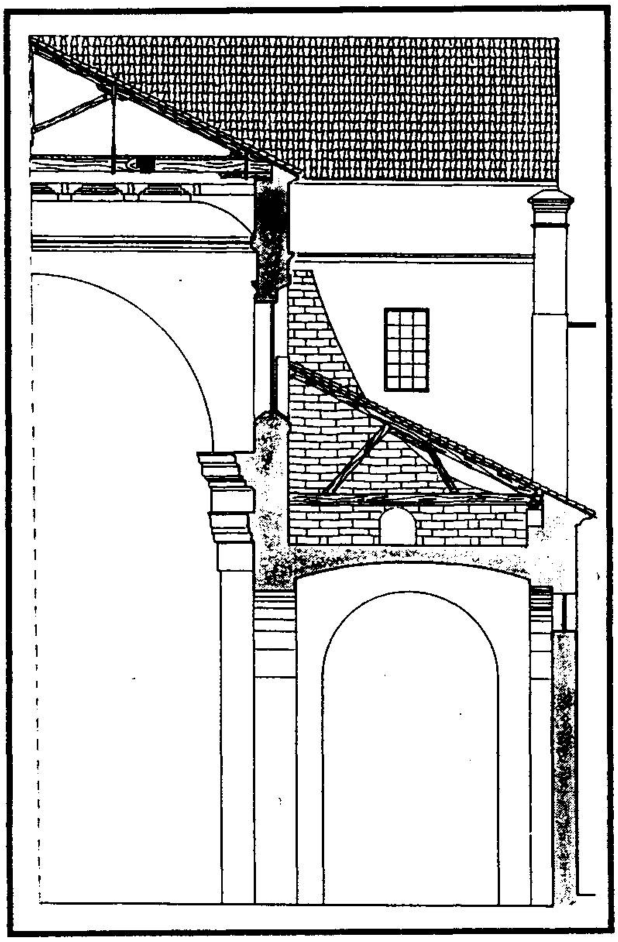

Fig. 1 Cross-section of the right side aisle before structural rehabilitation

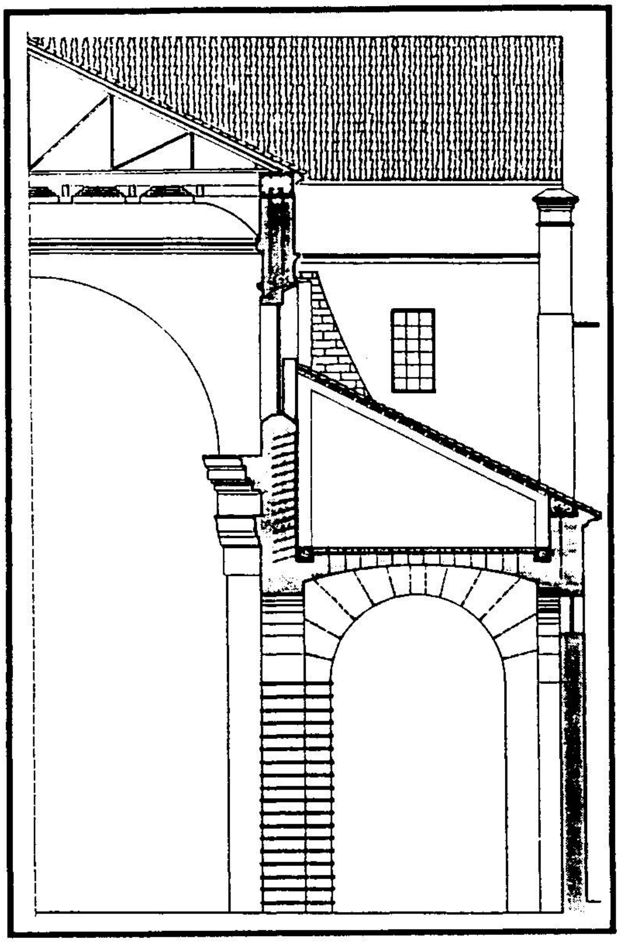

Fig. 2 Cross-section of the right side aisle after structural rehabilitation

A finite element model of the entire system has been developed, assuming that masonry elements are unable to resist tractions. An equivalent linear dynamic analysis has been performed, selecting the design earthquake time histories among the accelerograms recorded in the nearby stations during the Nov. 23, 1980 event. Results show a satisfactory reduction of the seismic vulnerability.

The new roof structures of the central nave consist of steel trusses, which were built assembling with bolts at the Cathedral site shop-welded elements, and then hoisting them onto the r.c. curb built on top of the existing walls.

The static recovery of the monument ended with the strengthening of the principal facade, performed through insertion of vertical and horizontal steel bars.

Strengthening of the Minutolo Chapel in Naples Cathedral

Renforcement de la chapelle Minutolo de la cathédrale de Naples

Verstärkung der Minutolo-Kappelle in der Kathedrale Neapels

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Historical notes

The Minutolo Chapel was probably built during the same period as the opposite Illustrissimi Chapel. Before the construction of the Cathedral was finished, it was decided to increase its size and importance by widening the transept and adding the two chapels to either side of the apsis.
The date of the frescos in the Minutolo Chapel, almost wedged in between the already existing structure of the right apsis, leads us to believe that it was therefore built after the Cathedral rostrum, though not after the last 10 years in the 13th century. Inside the chapel the underlying hypogeum is even clearer, the external buttresses and the lower base inclined towards the upper cornice of the siding and the square tower of the transept are evident.These walls, in square tufa blocks,show signs of a lenghty period exposed to atmospheric conditions. This leads us to believe that the Minutolo Chapel, already in use as originally designed on the aisle, was brought about after the construction of the apsis.
The inside of the chapel, built with a rectangular design with an ogivally crossed vault, ends with a polygon apsis added in the second phase of construction, which took place in the beginning of the 15th century. Even the small vestry is an addition, exploiting an area between the Cathedral apsis and that of the chapel. Nearly all of the pavement is decorated with precious marquetry with policrome marble inlays of the Cosmos. Throughout the various centuries the internal walls have been completely covered with frescos.

State of conservation and erosion.

As can be gathered from the history of the chapels' construction, the walls in the supporting structures were built in various periods and with various characteristics. The wall on the left and that of the entrance, is the transept pillar in the shape of a polygon, also in tufa blocks. The nave is roofed by two bay-vaults with crossed ribbing, while the apsis is formed by a large gothic style arch and a semivault with ogival ribbing.
On the left wall, the irregular external apsis buttress is hidden by part of a wall situated between the two pillars and the cornice of a depressed arch. A small chapel has been built under this arch.
The above floor is vaulted in tufa stones with a strongly depressed curve that is thin and flat at the extrados, almost like a "plate". Considering the use of

small stones of an excellent quality, there are signs of considerable erosion. The technical solution used in the past was that of a supporting element in relation to the crown by a large pillar in stone masonry.
Two large brick arches were then added, converging towards the centre of the pillar, creating a larger support area in the vault. About 30 years ago, in the hope to block the increasing fissures, it was considered necessary to increase the width of the pillar (by at least 4 square meters), without considering the need for a suitable foundation for this heavy structure.
In this period, structural work was also carried out on the previous pillar that had obviously pulled away from the vault intrados trhough sinking. The foundations for the converging walls in the south corner were also realized.

Intervention for consolidation.

The problem concerning the static restoration of the main structure of the Calpestian vault in the chapel, was formed with the birth of the actual vault due to its dimensions and form. The bulky supporting walls, built in the past to the hy pogeum quota, have continually proved to be not only insufficient, but also damaging by creating ad excessive weight to the supporting ground, thus causing sinking.
The object of this restoration, carried out by the Superintendant for the Architectural and Environmental Treasures of Naples, and Suburban Areas, is to eliminate the enormous pillars and to use a crypt method that will allow the chapel to be opened to the public and, with time to ensure the definitive strenghtening of the vault. Apart from being a piece of particular architectural work with precious mosaics of the Cosmos, it is without a doubt a significant structural element in the original building.
The technical solution used was that planned with the advise of Eng. Bruno Pandolfi and Paolo Falasca. This method created the supporting structure (for both above and below ground) by a frame of metal beams, positioned on the shortest side of the chapel, and interconnected at the right-angles by another series of steel sections placed according to where they are required. These steel sections were welded to the inferior sides of the beams and supported the vault by stays bolted above and a distribution plate at the intrados. The use of this metallic frame, limited in width and disposition, has given both support to the fissured vault and to an overload equivalent to that determined by overcrowding, (a rare occurance but not to be excluded). This solution has been positively valued, because it does not detract from the architectural image and has provided a quick and easy execution. The last phase was to progressively demolish the preceding reinforcement structures in the crypt. This operation was carried out by evaluating and controlling, with the use of a computerized monitoring system, the eventual static alterations in the vault structure.

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Restoration of the Altemps Palace in Rome

Restauration du Palais Altemps à Rome

Restaurierung des Altemps-Palastes in Rom

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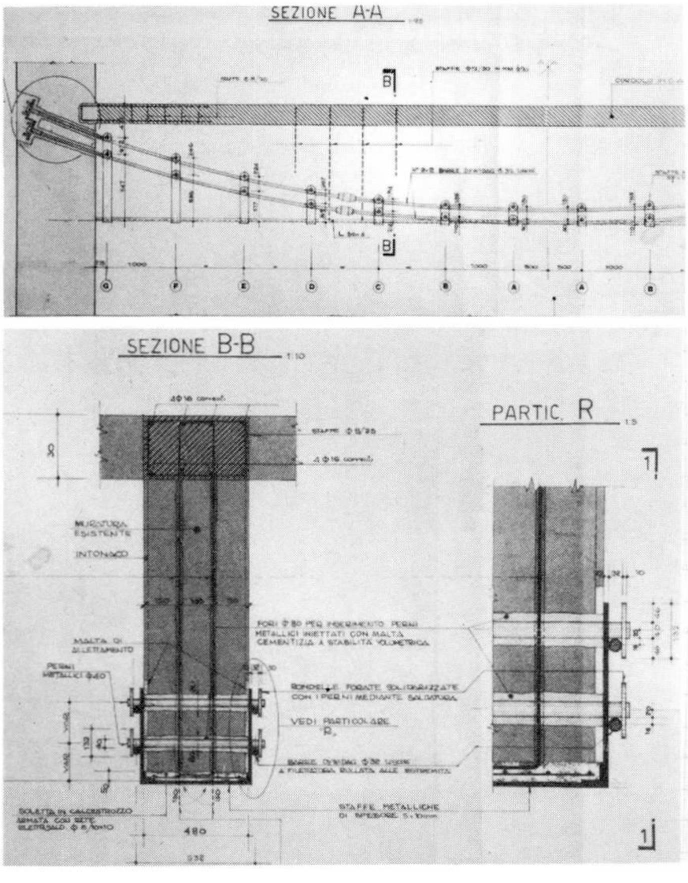
1. INTRODUCTION

The restoration of this historic, medieval-renaissance, block in centre of Rome involved the work of different disciplines: from Engineering to Architecture, from the History of Art to Archaeology. The project of structural consolidation was carried out by Prof. G. Croci, of the University of Rome with the collaboration of Engineer M. Biritognolo.
The problems of restoration and consolidation were tackled with an inter-disciplinary approach.
In particular, the structural intervention concerning the general restoration of the building, was preceded by an analysis using a mathematical model. Besides this there were a series of local analyses and interventions carried out on single structural elements.
It is important to specify the artisan nature of the restoration works, which is present in the individual details as well as in the global approach.
Some local structural interventions rise to a particular importance from a structural point of view. These interventions are related with the strengthening of structural elements (to assure the bearing capacity requested for the future function of the Palace) whilst preserving of the original structural and architectural typologies.
A monitoring system to watch the behaviour of a strengthened masonry element was also installed.

2. LOCAL INTERVENTIONS

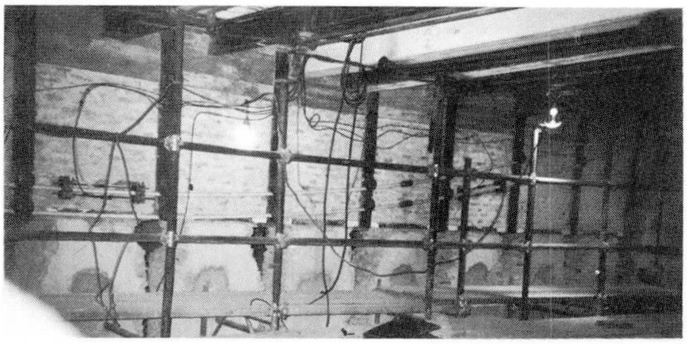
2.1 Strengthening of the masonry walls and of the floor structure in the hall

Three adjacent rooms separated by two internal walls, bearing the upper floor structures, had to be enclosed to obtain a hall of great dimension, originally named "Sala delle feste".
The intervention carried out in each one of the two walls was the demolition of the lower portion and the creation of a masonry beam supporting the upper floor.
This beam is composed by a wall panel having height of about 2,00 meters and span length of about 13.50 meters; over this beam, at the level of the upper floor, a concrete beam was cast (pic. 1, 2 and 3).
Four parabola shaped diwidag bars, (two at each surface of the wall), were placed as reinforcement to ensure the required bearing capacity.
A similar intervention was carried out on the principal structure of the upper floor of the hall. This structure is composed of wooden or steel beams. (see pic. 7)

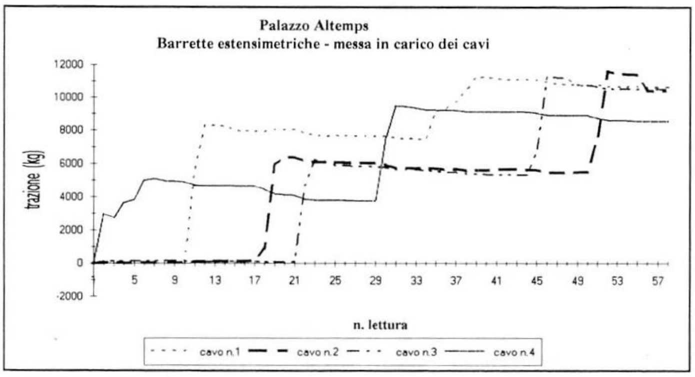


Pic. 1 and 2

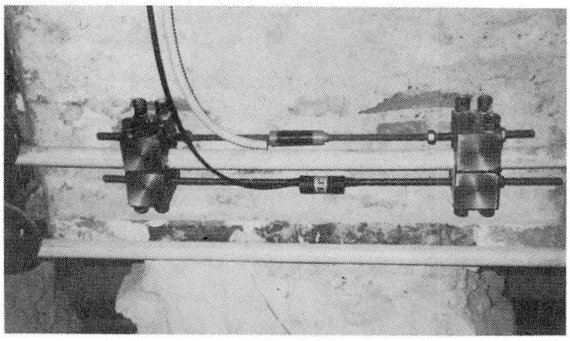
A monitoring system was installed to observe the values of the tensile forces in the dywidag bars in the principal masonry beams during and after the works.
Four pairs of strain guages (pic. 4) were attached to the bars, as well as a thermal sensor to measure the temperature variations, which permitted the control of the structural behaviour, comparing it with the theorical analysis (pic. 5).



Pic. 3



Pic. 5

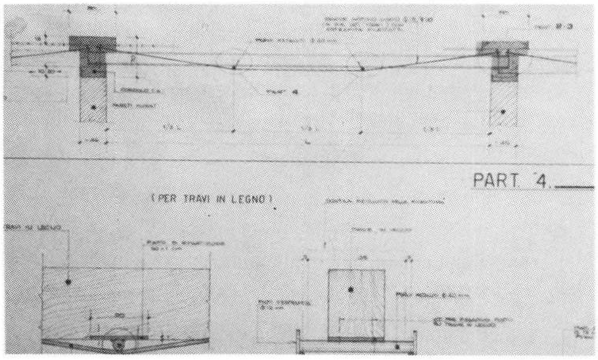


Pic. 4

2.2 Strengthening of the timber beams supporting the floor of the "Perspective Hall"

The floor of the "perspective hall" is supported by two timber beams, over these there is a secondary wooden structure bearing floorboards and an upper paving.
The bearing capacity of timber beams had to increased according to the foreseen utilization of the hall.
In order to maintain the original timber elements, a load text and an extensive examination of their state of conservation were performed.
The increase in bearing capacity was obtained by a steel section placed over the principal beam within the height of, and connected to, the secondary structure.

Pic. 6



Pic. 7

2.3 Reinforcement of some masonry panels with external steel bars

These interventions shown in the photo 6, allow the avoidance of any interference and damage the reinforced structure.

Strengthening and Control of the Dome of Vicoforte Sanctuary

Renforcement et contrôle de la coupole du sanctuaire Vicoforte

Verstärkung und Kontrolle der Kuppel des Vicoforte-Heiligtums

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The elliptical masonry dome of the Sanctuary of Vicoforte near Mondovì, Italy, built in 1731 is the largest of its kind (major axis "37,15 m, minor axis 24,80 m, maximum height of the monument 84 m) and is in absolute the fith largest dome in the world. The original project of the monument is due to Ascanio Vitozzi (1539-1615) who was responsible of the early part of the construction.
The unfortunate selection of the site from a geothechnical view point is responsible of the structural damages that the monument suffered throughout his life. Only the north-east section is founded in fact on sufficiently consistent marls, whereas the remaining parts of the monument, and in particular the south-western sections, rest on compressible clay-silt layers of variable thickness (up to 3÷3,5 m). The monument was therefore exposed to the effects of large differential setlements during the various phases of its construction and life. Construction itself - started by Vitozzi in 1596 - after continuous compensation of initial settlements during the construction process and the establishment of a drainage system of clay layers, was practically abandoned at elevation 11,10 m in 1600, with a slow prosecution untill el. 19 m during the whole XVIIth century.
Architect Francesco Gallo (1672-1750), after new levelling of structures to compensate further settlements due to progressive consolidation of clay layers, as well as to unsufficient maintenance of the drainage system, started again construction works in 1701 and completed in 1731 the daring large elliptical dome, inspite of a negative opinion expressed by Filippo Juvarra asked for consultancy. New settlements due to immediate and delayed effects of large added loads, magnified in time by recurrent lack of maintenance of the drainage system, were responsible in the following centuries of statical disorders with the appearence of large cracks in the dome and in the lower parts of the monument. Maximum differential settlements of the west-side foundations with respect to the north-east side, developed during the whole history of the monument, were estimated in 1962 to be the order of 55 cm. Maximum amplitude of cracks measured at the base of the dome was 82 mm with a total amplitude of 413 mm on the dome perimeter. Total increment of major west side cracks (extending from el. 14 m to the top of the dome) was of 14 mm in the period 1935-60.
A monitoring, rehabilitation and structural strengthening program was started in 1976 with the following objects:
- consolidation and stabilization of foundations,
- structural strengthening of the dome through the formation of a post-tensio-

ning ring at the base of the drum,
- monitoring of the principal parameters characterizing the structural disorder and of the response of the monument after strengthening (with particular regard to the time-dependent stress response of the post-tensioning ring).

The post-tensioning ring is formed by 14 interconnected tangential tie-rods, each one consisting of 4 prestressing bars hidden in the drum masonry tensioned to a convenient limited fraction of the calculated circumferential stresses in the dome.

Diagrams of the variation in time of the stresses in the bars over 5 years (as obtained by the monitoring system) have been analysed and a separation between the relaxation of the stresses in the bars (due to plastic flow of the masonry) and parasite stress effects due to the differential thermal dilatation of masonry and steel bars has been tried. Maximum value of mean relaxation of stresses in the 56 tensioning bars has reached the order of 20% of initial values (stress decrease from 50 to 40 KN per bar) in the four years following initial tensioning. (fig.1)

Correlation of these data with monitored movements of the main cracks of the dome leads to the conclusion of the need for a recalibration of the stress levels in the post-tensioning system to be performed in 1993. The need of more reliable data on the internal temperatures in the dome masonry has led to a program for measuring and monitoring these parameters to be operative also in 1993.

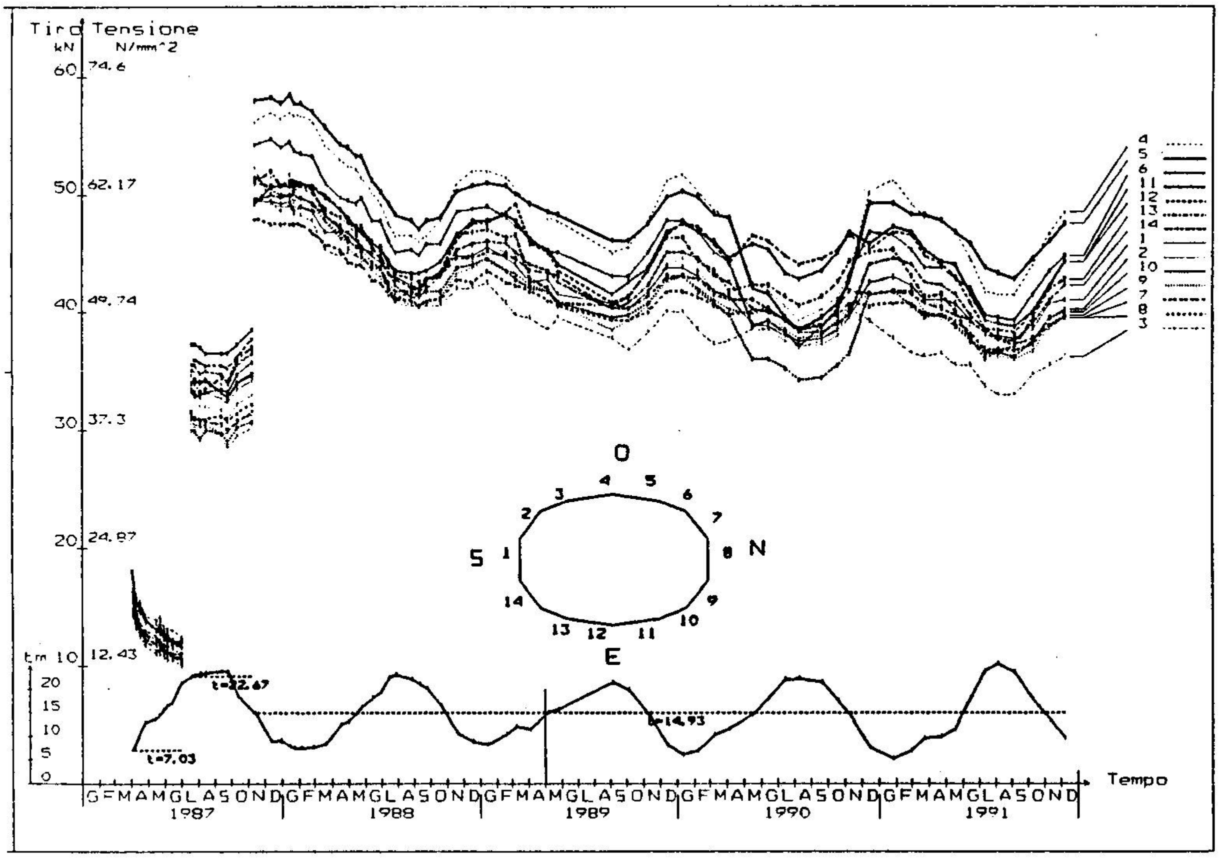

Fig.1 Variation in time of forces and stresses in the post-tensioning ring (mean values of 4 bars for each of the 14 sections of the ring) and variation of inside temperatures.

Conservation of the Lions Court at the Alhambra of Granada

Conservation de la Cour des Lions à l'Alhambra de Grenade

Erhaltung des Löwenhofs in der Alhambra von Granada

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1. INTRODUCTION

Decay studies on marble artifacts, relative to different architectonic structures of the Arab Palaces in Alhambra, show that the severe deterioration processes have been primed by static displacements. It is evident in the Court of the Lions [1], in which the environmental thermohygrometric influences have an important, [2] and [3], but secondary part on the decay evolution. Weathering processes weaken, above all, the marble artifacts already affected by micro-cracking conditions [1].

2. MARBLE STRUCTURES

2.1 Material

The Macael marble, widely used in the monumental complex of the Alhambra, is an anisotropic material both in texture and physical-mechanical properties, as the relative petrofabric analyses and investigations of the directionality of certain physical and mechanical parameters (Vp, E, σc and σt) have verified.
This anisotropic behaviour and its relative magnitude have been preliminarly determined in laboratory on correlated quarry and monument marble specimens (from working scraps of the "fuente de los Leones" unexposed copy, and from damaged original artifacts, removed during ancient restorations).

2.2 Colonnades in "Patio de los Leones"

On-site valuations of the physical-mechanical behaviour and decay evolution on marble artifacts (under external influences and loading stresses) have been based on the following non-destructive investigations: i) Textural orientations of the single structural elements, related with their specific geometrical conformation, working and laying. Generally the foliation plane, mechanical weakness plane, lies horizontally for base, capital and architrave elements, while it is in vertical placement for the shafts. ii) Ultrasonic pulse velocity measurements, to value the relative decrements due to incipient or moderate and severe decay.
The whole of the Macael marble artifacts, in different structures of the Arab Palaces, show a satisfactory conservation stage, with some exceptions: e.g., colonnades in "Patio de los Leones"(figure 1).
Selective decay evolution, relatively to columnar structures in the Court of the Lions has been verified

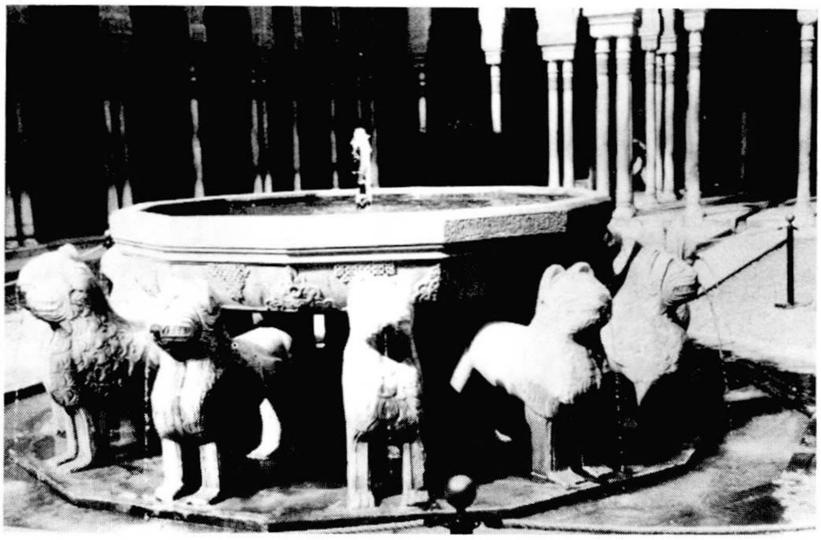

Fig. 1 "Patio de los Leones", Alhambra.

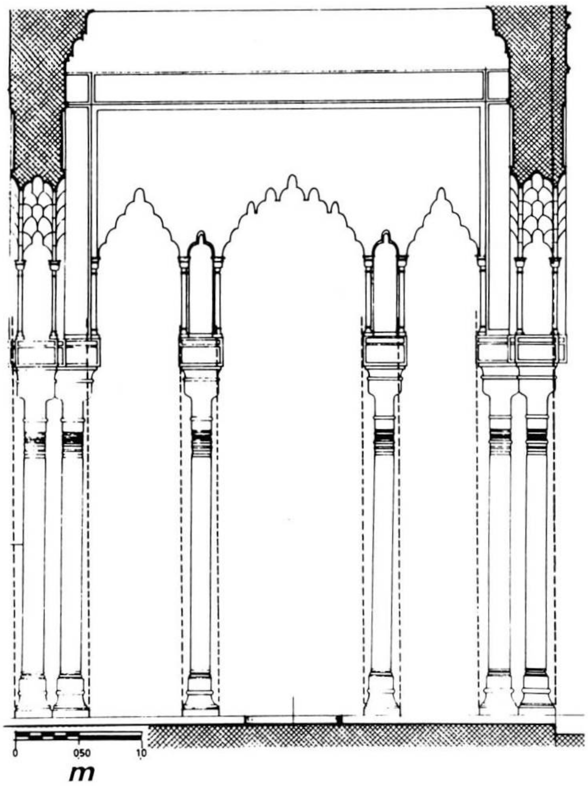

Fig. 2 Static situation, drawn in 1976, for "Templete de los Mocarabes", Court of the Lions.

Fig. 3 Progressive reduction in height of bases, and damaged shafts according to mechanical weakness plane.

by direct correlations between the increases of the apparent decay degree and total anisotropy index, measured in marble artifacts. Moreover, this evolution has been related to: a) rotation of the foliation plane, consistent with the shaft placement, influencing the durability of the artifacts under the same exposure conditions; and b) evident displacements of the original static conditions (figure 2), and new signs of differential settlement and plumb-line diversifications in severe damaged shafts, recognized particularly in the "Galeria de Dos Hermanas" colonnade (figure 3).

3. CONCLUSIVE REMARKS

A convenient control on stability conditions of the colonnades appears useful to found the adequate maintenance interventions, before carrying out replacements of damaged marble artifacts and/or protective treatments.

(This research was supported by contract EV4V-0108-I from the Commission of European Communities, and Research Group N° 4065 of the "Junta de Andalucia" Government).

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Structural Preservation of St. Paul's Facade at Macau

Conservation structurale de la façade de St. Paul à Macao

Strukturerhaltung der St-Pauls-Fassade in Macao

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1 - INTRODUCTION

The St. Paul's Facade at Macau is part of a church whose construction was concluded in 1644. Its architect was the italian jesuit Carlo Spinola who leaded a team of christian japanese workers.

The facade built with pink granitic stones is very imposing due not only to its dimensions, but also to the fact of being marvellously sculptured with decorations both in western and eastern style.

The church, whose interior (columns and ceiling) was made of wood, was destroyed by a fire in 1835 and only the stone facade remained. The fire deteriorated some stones namely those placed around the window frames of the facade. So, afterwards, these frames were strengthened with brick arches placed in their interior.

In 1935 a general restoration campaign was carried out and most of the joints between stone blocks were refilled with a strong cement mortar that covered part of the block edges contiguous to the joints.

By 1990 the facade had the following pathology:
- displacement of some stones from their original positions
- infestation by plants rooted in the joints between stone blocks
- bad drainage conditions
- dirtiness due to traffic pollution, rain water and "graffitti".

Thus the Macau Municipality and the Macau Cultural Institute formed a joint-venture in order to promote the following actions for the facade rehabilitation:
- stability studies
- preservation works

In order to valorize the monument and to establish a future museum on site an archaeological search was carried out in order to find the remains of the interior foundations of the original church.

2 - STABILITY STUDIES

Studies were made in order to evaluate the facade stability conditions.

First a radar search was undertaken in order to evaluate the soundness of the facade stone blocks and to verify if there were voids between them.

Then a survey to the foundations bottom was made by excavating inspection pits.

Finally, penetrometer tests were executed in order to quantify the soil capacity bellow the seating level of the foundations.

With all the collected data a stability analysis was made mainly for the severe local wind conditions (typhoons). The strongest wind ever recorded in Macau was in September 1964 (typhoon Ruby) with peak velocities of 211 km/h. The "Code of Practice on Wind Effects" of Hong Kong, edited in 1983, prescribes wind preassures that correspond to peak velocities of 214 km/h.

The conclusions obtained from the referred studies were:

- a strengthening of the foundations should be executed since only one half of its width was in contact with the granitic bed-rock. A solution with micropiles was proposed
- the resistance of the superstructure is sufficient provided that actions to correct the detected pathology are undertaken.

3 - PRESERVATION WORKS

The preservation works carried out were mainly:

- replacement of displaced stones using hydraulic jacks. After reaching the convenient position stones were kept by using stone wedges thus allowing the removal of the jacks before the refilling of joints with mortar
- removing of the existing vegetation by using adequated chemical produtcts (herbicides)
- repair of damaged joints between stone blocks using lime mortar. Particular care was taken in order to place the mortar withdrawn from the vertical plan of the facade
- improvement of the drainage conditions by correction of existing mortar slopes and execution of new ones
- hand cleaning of the facade mainly using low pressure water and plastic or nylon brushes. In the case of greasy stains detergents were also employed. For the removal of the "graffitti" some organic solvents (chloride of methylene, for instance) were required
- removal of the exceeding part of the cement mortar placed in the joints during the 1935 restoration campaign. This was made by cleaning the joints to a depth larger than their width with a minimum of 25 mm. Metallic hand tools, as scratchers, hammers and chisels or wire brushes, were employed carefully in order to avoid damages on the granitic blocks. After the excessive cement mortar was removed the joints were just cleaned with compressed air.

Preserving Frescos while Substituting the Wooden-Beam Floor

Conservation des fresques lors du remplacement de planchers en bois

Erhaltung der Fresken während dem Ersetzen des Holzbodens

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In many old buildings of the "Ancient Centre" in Naples, the false ceilings carried out to mask the intrados of the wooden floors, were usually vault shaped and painted in fresco, in order to give greater solemnity to entertainment rooms and greater prestige to the owuer. The usual shape was a "gaveta" vault that gives larger plane surface. The false ceiling, originally, was made of reeds ("canne") with a wooden trellis, from which derive the Italian term "incannucciata", then, in the course of time, the reeds were replaced with canvas. The wooden trellis was fixed to the wooden beams of the floor through little wooden bars, which was a remaining of the wooden trellis working. The trellis was usually made of chestnut.the section of the wooden elements, which constitued the trellis, was from about cm 3 x 3 to about cm 7 x 7, even if the section usually employed was about cm 3 x 5. The reeds or the canvas were coated with gesso and painted in fresco. The interstice resulted from wooden floor and the "incannucciata", to present that the latter went rotten, was ventilated through air intakes, called "ventarole", that shut by wire gauzes, connected the several rooms among them and outside.

Nowadays the engineers often are asked for giving their opinion on the stability of these old wooden floors. The problems of diagnosis of impairment due to the impossibility of access to the floor soffit because of the "incannucciata" are omitted in this paper in order to speak about the possible intervention to make on the floors once they are considered unsafe. To make this procedure clear, the traditional thecnics of construction of the wooden floors in Naples is described briefly. The floors were made of raw beams, usually of chestnut, leaned on longitudinal walls for a lenght not less than 1/20 of the span. The distance of the floors from one axis to another was 80 - 90 cm. On the beams, in the zone included between two adjacent beams, called "valera", some halfround pieces, usually made of chestnut, called "panconcelli" or most generally "chiancarelle" were put orthogonally and in contact. On these "chiancarelle", a layer of scraps or masonry debris ("sfabbricina") mixed with scanty mortar was laid. This layer is called "riccio". A layer of good mortar or a light mix of lapillus, which formed a sort of cover, called "masso" was spread on the "riccio". On this "masso" the proper flooring is placed. Once the floor must be replaced, the choice inevitably fall on the steel beam floor. It is made of steel beams, hollow flat tiles and a light concrete filling casting. The beams are connected together with a net, o 8 with a mesh of cm 20 x 20, soldered to the beams.Everething will be completed by a cm 4 concrete slab casting. The choise is inevitable for a different reasons. First of all, the lightness of this kind of floor, next its pecularity of having, like the wooden floor, points of suppot, and finally, as we can see latery, the possibility of assembling it into parts connecting it to the "incannucciata".

After making the dimensional choices and reducing the members with the well known methods of the Science and Technics of costruction, we examine the operative phases of sustitution. The first phase consists in removing the flooring, next the "masso" and finally the "ricco" in order to lay the "chiancarelle" bare. This is the most dilcate phase, because, generally the "chiancarelle" are very damaged, so any sharp movement or the use of wrong tools, can give rise to the fall of "sfabbricina" on the "incannucciata" with the its resulting damage. Thus the operation must be performed with great care using only bush-hammer and trowel and not other tools, such as shovels or picks. To preserve the "incannucciata" the floor should not be uncovered as a whole, but only a "valera" at a time, while at the beginning, two at a time in order to have two spans opened. After this, the "chiancarelle" should be removed. The "incannucciata" could be very damaged, in this case a skilled restorer should be called. Nevertheless if the trellis and the canvas, or reeds, are in good conditions, the only advisable operation is to stiffen the connections between the reeds or canvas and the wood. At the vault extrados, the connection of the reeds to the trellis takes place sticking strips of glass fibre cloth, firstly impregnated with bicomponent exposy resin, or more economically, using strips of cloth sticked with no watery glue. Before removing the beams it is necessary to create temporany supports for the beam and to protect the canvas from possible small masonry debris and above all from the water casting. The temporany supports for the canvas could be made with wires fixed on bars orthogonally placed to the floor frame. The simpler and cheaper protection of the canvas is to put first a plywood sheet on the trells, second cardboards and papers and finally a sawdust layer. To remove the beams it is necessary to widen the hole of housing to sling the beam and to saw in two parts. Now the wooden beams is replaced with the steel beam. The profile used is generally a hot rolled steel Fe 360 section IPE or NP. The profile must not be leaned directly against the tuff masonry, because, obviously, the pressure concentration could break the support stone. That's why a concrete or solid bricks bearing is created to support the beams. The support of the beams must be about 1/20 of the span and never less than cm 15. After placing the beams, the trellis, which support the "incannucciata", is connected to them. The connection, in the case of particulary deformed "incannucciata", is made with steel wires and thread-tensioner which will be used to settle the vault again. But if the vault is in good condition it is sufficient to bring back the connection into use through zinc plated steel wires tied to the trellis and the beam. Then the hollow flat tiles are placed and the holes of housing are sealed with a sand and cement mixture. When the concrete is set, the completing concrete is cast. After the setting of the last one, the plywood steels are removed, the cardboards and the sawdnust are sustituted and the operation for the next "valera" is repeted. At the last bay the cardboards and the sawdnust are not employed, except the playwood sheets which will not be got back after casting. After completing the last span, the electro-saldered net is placed and the slab of cm 4 is cast, taking care of not mixing the concrete with the surplus water compared to the stoicheiometrical necessary water in order to avoid damp stains on the "incannucciata".

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Controlling Methods Applied to a Roman Dome to be Restored

Méthodes de contrôle pour une coupole romaine en cours de réparation

Kontrollmethoden für eine römische Domkuppel

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1. BACKGROUND

The Temple of Romulus, named in honor of Romulus, the son of the Emperor Maxentius, was built in 311 AD on the site of the Temple of the Penates, which had been torn down to make way for the emperor's great basilica on the Via Sacra in the Roman Forum [1]. Much of the original building is extant, including a cylindrical entrance hall, approximately 15 meters in diameter, topped by a semicircular dome, the main hall, and two smaller halls, which, however, are in ruins. In the 6th century, the temple was transformed into the atrium of the church of Sts. Cosmas and Damian. In 1631, a Baroque vaulted ceiling was added, which is sustained by the outer drum walls and inner pillars at the church floor level. During this campaign, the temple was seriously damaged when a communicating door was opened to connect the rear of the church to the temple dome area.

2. PRESENT STATE OF THE DOME

The static condition of the dome is poor. This is the result of several factors: the collapse of the smaller halls, which laterally sustain the pressure of the dome; deterioration wrought by time; the 19th century excavations in the Forum; and the various modifications made to the building.

An examination of the cracking patterns has revealed the static and mechanical changes undergone by the dome over the centuries. The building contains serious lesions, which start at the top of the dome and continue downwards, almost vertically, to the ground. Moreover, the drum walls are rotated outward, visibly diverging from vertical. A major lesion, which from the arch keystone propagates throughout the dome up to the lantern, is the result of the construction of the communicating door. Also, the vaulted ceiling may also have contributed to the poor condition of the whole.

3. RESTORATION AND STRUCTURAL REPAIR OF THE BUILDING

Restoration of the Temple of Romulus is being undertaken by the Soprintendenza Archeologica in Rome, in conjunction with the Soprintendenza ai Beni Ambientali e

Architettonici. The project will entail restoration of the domed hall and demolition of the Baroque vaulting. The church communicating door will be left. An automatic monitoring system will be installed. In view of the precarious state of the whole, special care will be taken in the removal of the vaulting. To limit the effects of the modification, a temporary external encircling of the dome will be added at drum level. The encircling could be made permanent, should the results of the automatic monitoring warrant.

4. NUMERICAL FINITE ELEMENT METHOD USED IN RESTORATION

The mechanical history of the building will be determined as part of the forthcoming restoration project. A finite element model has been developed to accurately duplicate the geometry of the structure. Using a numerical analysis carried out on the virtual model of the integral structure, the static state of the building in its original geometry has been investigated. This has made it possible to determine the mechanical consequences of the evolutions from modifications (the church communicating door, the vaulted ceiling), structural cracking, and the collapse of various elements. Hence, using structural identification methods, we have been able to arrive at an exhaustive representation of the monument's present cracking state. This has entailed:

1. Carrying out tests for determining the stress state using flat jacks and assessing the deformability and resistance of the walls

2. Developing a permanent monitoring system to check the progress of crack propagation and structural movements.

The finite element numerical model together with mechanical measurements has enabled diagnosing the building's physical condition. The numerical model has given indications regarding the effects of removing the vaulted ceiling and the encircling system on the damaged structure. In simulating the some encircling system, beneficial effects have also emerged with regard to the existing state of cracking.

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Reconstruction of Bridges in the Historical Centre of St. Petersburg

Reconstruction de ponts dans le centre historique de Saint Petersbourg

Wiederaufbau von Brücken im historischen Zentrum von Sankt Petersburg

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1. INTRODUCTION

St-Petersburgh, founded in 1703, occupied the territory in the mouth of the Neva river, which included about 40 islands, separated by rivers, channels and canals with their total number now reaching 86. Due to such situation, the builders had to erect bridges for connection of different parts of the new city from the very beginning. First of them were wooden-made and don't exist. First stone bridges appeared in 1760ies and some of them are still working. Later on first russian cast-iron, steel and concrete bridges were built in St-Petersburgh, forming distinctive outlook of the city's centre, contributing much to it's graceful silhouette.
For the last period of time city authorities have faced the necessity of reconstruction of the bridges, built between 1760 and 1916, because they were no longer capable to withstand the needs of transport from the points of their width, reliability and shipping conditions underneath. Serious deformations and damages made further service of some bridges dangerous. During the restoration work there appeared several methods of approach, depending upon architectural and transport requirements and conditions of existing structures.

2. PRESERVATION OF EXISTING STRUCTURES

This method was applied, when it was possible to use old structures after their strengthening. Examples:
- the Laundry bridge across the Fontanka river, built in 1769 as 3-span stone arch, faced with granite. It's piers' foundations were strengthened after the appearance of serious deformations and cracks of stone superstructure;
- the Upper Swan bridge across the Swan canal, built in 1768 as one-span stone arch. All it's constructions were remained and only new water-protection and asphalt were laid;
- the Kazan bridge across the Catherine canal, built in 1805 as one-span brick-work arch. After the guniting of the arch it's still working at the main city's thoroughfare-the Nevsky avenue.

3. RESTORATION OF OLD CONSTRUCTION IN NEW MATERIAL

This method was applied in the cases, when serious damages made further usage of old material impossible, but the old bridge had architectural value. Examples:
- the Hermitage bridge across the Winter canal, built in 1766 as one-span stone arch. It's stone arch was changed for concrete one, faced with granite, of the precisely same dimensions, after the appearance of serious destructions of old superstructure and piers' foundations;

- the General Post Office pedestrian bridge across the Moika river, built in 1824 with one-span suspension superstructure. After serious damages of the chains and pylons two extra piers were built, completely spoiling it's appearance. Recently it received original outlook after capital reconstruction.

4. ERECTION OF A COMPLETELY NEW BRIDGE

This method was used, when existing bridge hadn't got any architectural value, but a new bridge had to provide harmonious unity with surrounding ensemble. Examples:

- the Italian pedestrian bridge across the Catherine canal. It's new one-span steel girder, decorated in the classical traditions of 19 century, changed wooden construction and became integral part of the heart of St-Petersburgh;
- the Second Winter bridge across the Winter canal. It's concrete arch, faced with granite, replaced old wooden superstructure, repeating the outline of the First Winter and the Hermitage bridges and giving an excellent finishing touch to the ensemble of the Winter canal.

5. CHANGING OF ELEMENTS OF BRIDGE'S CONSTRUCTION

This method was used, when a part of a bridge had to be replaced due to certain circumstances, with a new contruction becoming an integral part of the whole bridge. Examples:

- the Trinity bridge across the Neva, built in 1903. It's swing span failed to withstand the requirements of shipping, being only 22,8m wide. It was replaced by the bascule span, providing 43m clearance for ships, going by the Volga-Baltic water way. The outline of the new span produced the impression of continuation of old constant superstructure. Newly-built concrete arch, connecting the pier of the bascule span with the left bank of the Neva, made the whole construction completed, coinciding with the outline of the existing arches near the right bank;
- the Old Kalinkin bridge across the Fontanka river, built in 1780ies. It's removable central span, once used for shipping of sailing vessels, was changed by the stone arch, similar to the adjacent;
- the Big Okhta bridge across the Neva, built in 1911. During forthcoming reconstruction only the bascule span and the deck will be changed and it's 136m arch trusses will be preserved.

6. CHANGING OF THE WHOLE OLD SUPERSTRUCTURE

This method was applied during the reconstruction of the Liteiny bridge across the Neva, built in 1879. It's old piers were widened, using the starlings, which made possible to install new continuous steel girder instead of former iron arches, having the carriageway's width increased from 18 to 28m. The swing span with the width 19,8m was replaced by the bascule one, providing 50m clearance. The new superstructure had got the original length of the spans and curvilinear outline, resembling old arches. The new bascule span with the outline, similar to the constant one, provided the completed and continuous silhouette to the whole construction.

7. CONCLUSION

The choice of the method of reconstruction was made in each case after thorough inspection of the bridge's condition, archive studies, consultations with architects and art critics. This provided the opportunity to preserve unique architectural ensembles, strict and graceful view of St-Petersburgh.