Session 9

Saving Cities and Old Buildings

Sauvetage des Cités et des Bâtiments Anciens

Chairman	J. Kerisel (France)
Co-Chairman	A. Croce (Italy)
General Reporter	U. Smoltczyk (FRG)
Co-Reporter	R. Butterfield (UK)
Technical Secretaries	S-E. Rehnman and H. Bohm (Sweden)
Panelists	M. Gonzáles Flores (Mexico), J. Huder (Switzerland), A. Loizos (Greece)

J. Kerisel, Chairman

DISCOURS

Paul VALERY a dit : " Le difficile est de durer . Seule la chaîne des humains peut nous sauver du temps, semeur d'oubli " .

Je voulais citer cette pensée de Paul VALERY en ouvrant cette séance consacrée au sauvetage du patrimoine artistique mondial menacé dans ses fondations, et marquer ainsi toute la portée humaniste du travail qu'ont apporté et que vont apporter tous ceux qui collaborent au succès de cette Session 9 : ils vont faire partie de cette chaîne des hommes qui aident à transmettre aux générations futures le message spirituel des monuments du passé .

Une deuxième remarque que je voudrais faire est que dans notre travail , il y a beaucoup plus qu'ailleurs une prise de conscience de la durée . Bien des édifices religieux qui nous sont présentés sont fondés sur les restes de 2, 3 ou 4 églises , ou sur d'anciens cimetières , et dans cette superposition de générations nous prenons conscience de la tenue d'un édifice à travers des siècles . Pour ne prendre qu'un seul exemple à travers les communications qui nous sont présentées , LORD nous propose dans ce graphique Fig. 1 une évolution pendant 9 siècles des tassements de la Tour Centrale de York Minster .

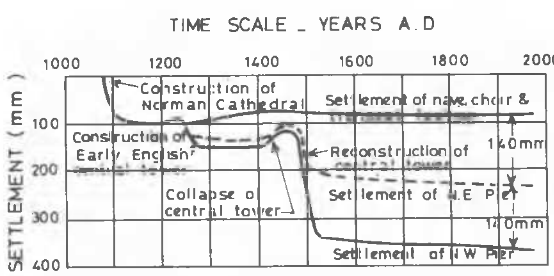

Fig. 1. Hypothetical reconstruction of settlements since 1080 A.D. York Minster (Lord)

Tous ces édifices du passé constituent un merveilleux champ d'observation des déformations de structures pesantes , dont certaines imposaient hardiment à des sols, cependant argileux , des contraintes élevées de 500 KPa (Tour de Pise) , 750 KPa (Tour Centrale de York Minster, piliers du Dôme de Milan , etc ...) et ceci sur des surfaces qui étaient assez étendues . La lecture des fissures , l'analyse des tassements , des rotations, distorsions et cambrures des maçonneries sont d'une richesse incomparable .

Tout aussi fascinante est la variété des causes des incidents qu'ont connus ces édifices : fissures des piliers du Dôme de Milan par suite d'une trop longue durée de la construction et de taux de travail élevés et très différents du début à la fin de la construction ; points durs dans les restes d'une ancienne église sur lesquels repose une nouvelle église (York Minster) , pompages sauvages comme à Venise , Milan , Mexico , Helsinski, et bien d'autres lieux hélas ! (ce qui est la rançon d'une civilisation effervescente et inconsciente) , fluage vers les berges de canaux (Vieux Stockholm, Venise, Tour de Nevyansk en U.R.S.S.) , transgressions marines (Stockholm) mouvements alternés provoqués par les variations de la nappe (Palais de Justice à Rome) , gonflement des schistes (tombeaux de la Vallée des Rois en Egypte) , prédominance de vents (Campanile de Burano), mutations lentes dûes aux intempéries (St-Léo , Orvieto en Italie, l'Acropole en Grèce), enfin effondrements de cavités (à Cracovie et sous les vieilles églises de Hongrie) .

Mais aussi , énigmes au premier rang desquelles se situe la Tour de Pise . Que cette tour continue à pivoter 7 sièces après sa construction , à vitesse presque constante (10 à 12" par an depuis 80 ans) et à se visser dans le sol en s'y enfonçant sans se fissurer et sans tomber, est vraiment extraordinaire , et il y a là un problème auquel , à mon avis, aucune explication parfaitement satisfaisante n'a été apportée . Et puisque je parle d'un monument italien , je voudrais dire que je suis encore marqué par la richesse des discussions au dernier Congrès National Italien de Géotechnique qui s'est tenu à Florence fin Octobre 1980 consacré aux mêmes sujets que notre session 9 .

Non moins varié est l'éventail des remèdes pour le sauvetage de ces vieux édifices : vous le verrez tout au long de cette session . Mais cette variété n'en laisse pas moins un choix difficile pour leur traitement : les vieux édifices étaient en effet, souples à l'origine car les mortiers étaient à base de chaux grasse à prise très lente ; aujourd'hui , ces chaux ayant fait pr les édifices sont devenus fragiles et la marge de tance à des tassements différentiels dans les rep en sous-oeuvre est devenue très réduite .

Ch. Comte (Invited discussion)

La présentation des travaux de confortation de la cathédrale St Pierre à Genève, dans le cadre nécessairement limité réservé à chaque communication au congrès ne pouvait que chercher à signaler les traits essentiels de ce travail.

Chaque entreprise de ce genre soulève cependant un grand nombre de questions fondamentales que le Président de la session et le Rapporteur général ont rappelées, en ouvrant la discussion sur certaines d'entr'elles.

Comme il a été mentionné, le choix de pieux éxécutés par le procédé ROPRESS résulte d'une comparaison avec six autres méthodes de reprise en sous-oeuvre.

L'étude de détail de chacune, sur la base d'applications antérieures réellement exécutées, a permis d'arriver à la conclusion que cette méthode nouvelle comportait de véritables avantages sur des procédés plus connus. Ceci paraît en contradiction avec les commentaires de Lord concernant les travaux pour York Minster qu'il a présentés. Il faut cependant tenir compte de conditions de travail locales différentes et aussi d'une évolution indiscutable de la technique entre ces deux réalisations, la plus ancienne, celle de York, étant connue dans ses grandes lignes des auteurs de la seconde.

Quant à la question de principe traitée par le Président Kérisel sur la valeur des tassements que peuvent supporter sans danger des monuments historiques, on peut affirmer après l'étude de l'état de la structure de St Pierre, que celle-ci souffrait de tassements relativement faibles, absolument sans importance pour des immeubles ordinaires dans les environs.

Ceci s'explique par la rigidité des vieilles maçonneries soulignée par le Président Kérisel et par la nécessité d'envisager la stabilité de la cathédrale au cours des siècles, pendant lesquels les immeubles utilitaires sont plusieurs fois rénovés ou même reconstruits.

Le fait de procéder à des travaux d'excavation et à des fouilles archéologiques n'a été que le facteur déterminant le début des études de stabilité et la découverte des causes réelles de défaillances observées précédemment et auxquelles on avait cherché à remédier avec des moyens inadéquats.

Ainsi que le remarque le Prof Huder il faut reconnaître les vrais origines des désordres apparus dans un monument et posséder une connaissance exacte de la géologie du sous-sol pour proposer une confortation durable et efficace.

Les constructeurs du XIIe S, les rénovateurs des XVIIIe et XIXe S n'avaient pas les moyens matériels nécessaires pour apprécier les conditions de fondation. A St Pierre, ce sont elles qui les ont trahi.

Les voûtes peu épaisses en tuf, très légères, ne peuvent en aucun cas exercer une poussée suffisante pour expliquer les déversements de lourdes maçonneries romanes de la nef. Les vieilles chroniques sont explicites, "la voûte abandonnait la muraille".

Comparaison des Procédés de Confortation

	1 Chaînage	2 Encastrement des porteurs	3 Centrage des charges	4 Sécurité à la rupture	5 Nuisances	6 Protection site	7 Rapidité
1. Pieux ROPRESS et dalle B.A.	+	+	+	+	+	+	+
1bis Pieux aiguilles et dalle B.A.	+	+	0	+	+	+	+
2. Injection et dalle B.A.	+	0	0	+	+	0	+
3. Sous-oeuvres traditionnelles et dalle B.A.	+	+	0	0	+	0	0
4. Terre armée et dalle B.A.	+	0	0	+	0	0	+
5. Pieux vérinés et dalle B.A.	+	+	+	+	0	0	0
6. Pieux forés et dalle B.A.	+	+	0	+	0	0	+

+ = respect des conditions impératives
0 = non respect des conditions impératives

St Pierre de Genève, avec ses croisées d'ogives porteuses, qui sont des arcs en plein cintre, comme dans le grand nombre des églises gothiques de France, témoigne du sens aigu des bâtisseurs du XIIIe S pour alléger les constructions et assurer la descente des charges des structures conçues pour travailler uniquement en compression.

C'est lorsque une défaillance des matériaux ou des fondations entraine des tassements ou des déformations que la stabilité est compromise.

En donnant le moyen aux fondations de St Pierre de suivre d'éventuels tassements profonds, sans se déformer et en réduisant les tassements différentiels, on a cherché à améliorer la stabilité générale de la structure sans avoir à intervenir dans celle-ci et sans altérer la pureté de sa conception comme ouvrage en pierres de taille.

On a cherché non seulement à conserver l'aspect du monument mais aussi son caractère.

Les archéologues ont beaucoup apprécié un système de confortation qui leur assurait toute sécurité dans leurs travaux, sans interférer dans ceux-ci.

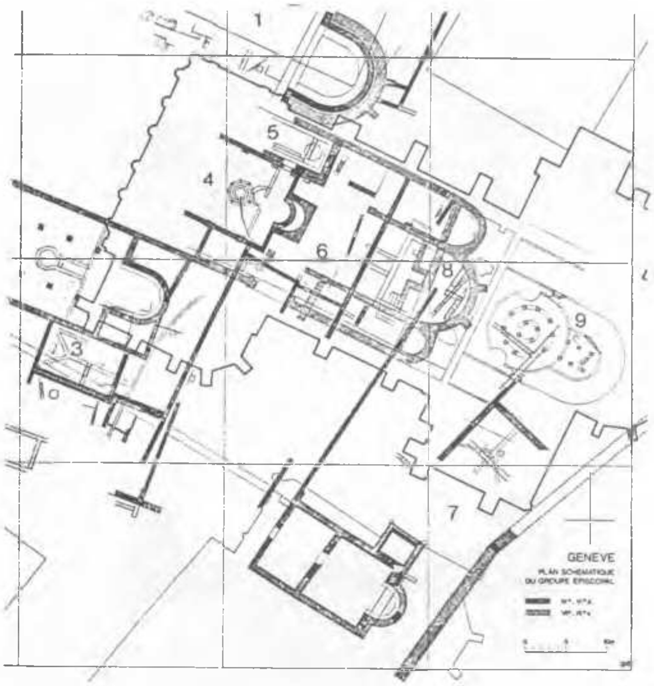

Fig. 1. Restitution des premiers résultats des fouilles archéologiques sous la cathédrale St Pierre à Genève.

A. Croce, Co-Chairman

INTRODUCTION TO THE TOPIC MONUMENTS AND OLD CITIES

1.- The present Conference is considering geotechnical engineering from two different points of view. The first one is the traditional, let me say *classical geotechnics*, a powerful tool to understand and control the mechanical behaviour of soils and rocks.
Recently, however, geotechnical engineering has taken a new aspect; it is called to use its scientific and technological knowledge in a wider horizon.
In fact the present Session is concerned about the heritage of the past; another Session n. 6, was concerned about the present environment: both Sessions, however, have to look ahead in the distant future.
The specific aspects of our problems are the *pre-existence* of *structures* and the large *time-scale*.
When the construction already exists, the engineer needs, first of all, to look to the old construction with a large-minded approach.
Apart from unforeseen difficulties which his studies bring to light, he sets himself an ambitious goal: the reconstruction of the history of the mechanical behaviour of both the subsoil and the overlying structures.
The engineer has firstly to go back in time with the help of every source of information, secondly to consider architectural criteria and construction methods wich differ more from present ones, the further back in time he goes.
He also has to analyse the loads, deformations and displacements. In addition, he finds the signs left by various agents which have operated slowly and slowly throughout the centuries.
At the end, when he succeedes, he makes his judgment on the actual state of the construction.
The next step to be done is to design and carry out the plan; it is the most difficult and dangerous step, like therapy following diagnosis.
Two fundamental requirements are to be met.
The first is a cultural requirement, that is, to hand on the future generations the possibility of *reading* the quintessential message of the old constructions.
The second requirement is a technical one, that is, an acceptable level of static safety.
These two basic requirements are in contrast.
This is the main reason why very often therapy to save old buildings and monuments is more difficult than diagnosis.

2.- The discussion, which is now to begin, will be developed following the course set by the Chairman.
The range of problems is quite wide. Is it possible to discover some methodological aspects of

the theme of this Session ?
Prof. Kerisel, Prof. Smoltzick and Dr. Butterfield have shown that the answer should be affirmative.
I hope that the discussion will contribute to this aim. I will try to do the same introducing the topic of Saving Monumental Buildings.

I'd like to take a few examples from my own country.
We Italians have to deal not only with individual monuments, but also with groups of monuments as well as entire villages, towns and cities.
In fact the historical and artistic significance of many cities lies not only in their single monuments, but also in the harmony of buildings, squares, roads, canals as a whole.
I will speak of cities. Each city is a separate case. However, both history and geomorphology show the possibility to confront separately cities on the plains and cities on hills and mountains.
I will speak of cities on the plains.
However the plains in Southern Italy are very small and lie along the sea costs; the influence of the old Greece is the most impressive.
In Northern Italy there is only one very large plain: this is the Po Valley; here the historical relations with Central Europe are predominant.
I will speak of three cities in Northern Italy: Venice, Como and Milan. They have completely different problems, but at least one point in common, that is, the subsidence of the ground.

Venice

The city of Venice developed between the XI and the XVI centuries. It extends over islands and isles within a large lagoon which is separated from the sea by a narrow line of dunes.
The urban layout is characterised by a dense network of big, small and very small canals which separate buildings or groups of buildings. Therefore, from the earliest times, the main concern of Venetians was about the superficial waters.
Water in the canals moves up and down following the daily sea movements; moreover it is continually agitated by boats of every tipe. And,thirdly, buildings and squares are more and more frequently flooded by the *high waters* .
A general, imperceptible but continuous deterioration spread over the city, in spite of all the efforts of the authorities and citizens.
Today, Venice's problems remain the same, but more serious than in the past, since the factors of deterioration are more intense.
Apart from the problem of the water flowing from the sea into the lagoon and viceversa, the main geotechnical problems are of two kinds.

The first one concerns the foundations. In general they are shallow foundations. Soils directly below the structure exibit poor mechanical pro-

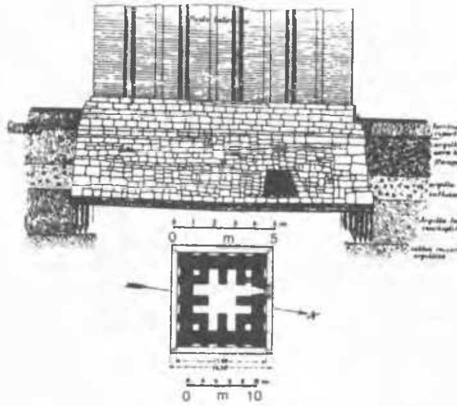

Fig. 1

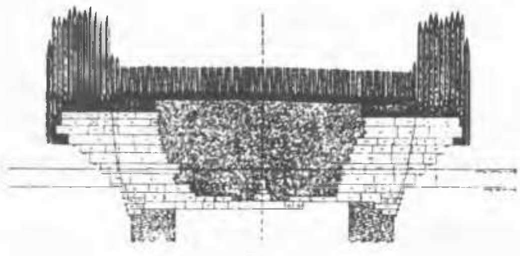
Fig. 2

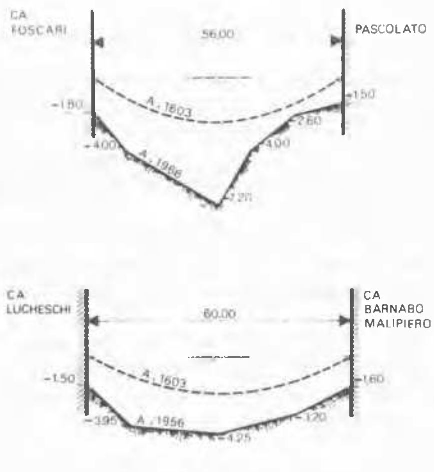

Fig. 3

perties; better soils, however,are present a little lower down.
The loads are relatively high, since they are of the order of 1,5-3,5 kg/cm^2.
On the other side, it has to be noted that the construction of the buildings proceeded very slowly so that it was possible to adjust the structures to the large differential settlements which occurred in the meantime.
The foundation of the Bell Tower of Saint Mark (Fig.1) can be considered as a typical example

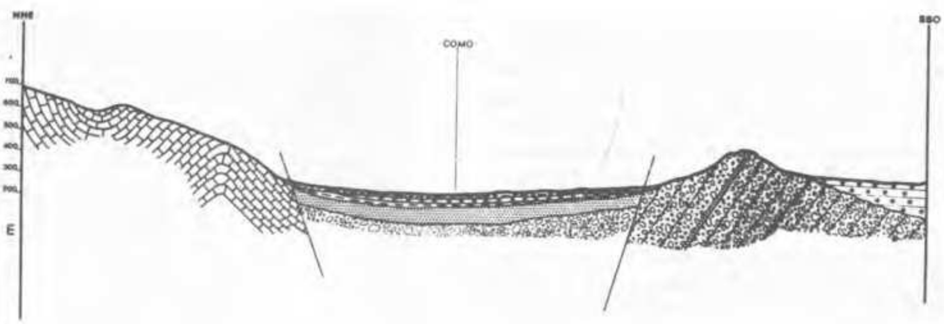

Fig. 4

of Venetian foundations, with its short wooden piles, the wooden raft and the masonry socle
The load applied to the soil was 6,5 kg/cm^2.
This Bell Tower, as indeed others in the past, collapsed. This happened in 1902. The Bell Tower was immediately reconstructed on the previous foundation which was reinforced with a little longer piles around the perimeter (Fig.2). In addition to the original deficiencies in the foundations, there has been, in the last decades, the destructive action of the canal waters. The fig.3 shows the erosion in the Canal Grande from 1603 to 1966. This is another cause of deterioration, which requires constant maintenance work.
The effects of all these negative factors on the behaviour of the entire building are evident.
Improving the conditions of both the foundations and the structures is a difficult job. But the real difficulties are of a quite different character. It would be useless, indeed, to carry out work on the buildings without doing simultaneous work on the defense structures along the canals, the canal banks themselves, the acqueducts and the sewerage system.

The second type of geotechnical problems are connected with subsidence.
We know with reasonable accuracy that the Venetian ground level has sunk at an average rate of 1 mm per year in the last millennium.
In this century the rate of subsidence increased, especially in the period from 1950 to 1970. One can estimate that the subsidence in this period was around 3 to 4 mm per year.
In the last decade (1970-1980) many wells in Venice and in the surrounding area have been closed and the subsidence rate is slowing down.
Therefore, although the subsidence in Venice is very slow, we have to look to Venice in the future. The predictions as to how much the ground will settle in the next century are extremely uncertain for the time being.
As a consequence, it is necessary to carry on more and more surveys and studies on Venetian subsidence.
Moreover, it is to wonder about the possibility of an *active protection* of the city. Research and studies should be developed for this purpose. In any case, it is imperative that methods and techniques be congenial to the whole of the physical ambient and the city itself, which both constitute Venice.

Como

It is a delightful busy center in the most beautiful countryside, on the shores of Lake Como which straddles the Italo-Swiss border.
The geomorphological aspects of the area are typical of mountain country: a relatively narrow valley hemmed in by steep, high mountains (Fig.4).
The Romans built a fortress in the valley which became the nucleus of later development over the centuries.
Recently the relationship between the town and the Lake is not as happy as it has been in the past.
In fact the town has been flooded more and more frequently.
Geodetic measurements have shown that an intense subsidence took place in Como from 1950 to 1975: the largest settlements, measured near the shores, were around 70 cm. Such values are roughly ten times the values of Venice and very close to the high values of Ravenna, another italian town famous for its past history as well as for its present subsidence.
But, on the other side, the same measurements showed that the subsidence was slowing down in the last years. Therefore new measurements were performed in 1979; the slowing down of the settlements was clearly confirmed.
In the meantime geotechnical studies were carried out into the possible causes. The usual antropic causes were considered, but the answers were negative.
Moreover buildings, squares, streets don't show any sign of distress.
The town has to be and will be protected against flood, but this is not a difficult task. What is really needed is to know the cause which gave rise to the subsidence in order to look to the future without uncertainties.
This is the true, unusual protection which is required to save Como.

Milan

Milan is a quite modern city, yet it includes monuments of great artistic and historical value. In the period between 1950 and 1970 the city has been subject to a light subsidence, which didn't cause any inconvenient to recent constructions. On the contrary the Cathedral, the most famous monument in Milan, underwent serious consequences.
In fact, the differential settlements caused by the subsidence, although small, appeared unacceptable by the bold architectural design of the building.
The diagnosis was fairly simple and the cause of the subsidence has been easily removed by gradually closing the pumping wells in a wide area.
I will then omit the specific aspects of geotechnical engineering and I'd rather mention some aspects of the work in progress to reinforce the structures.

Fig. 5

During the six centuries past from the foundation of the Cathedral, small superficial cracks occasionally occurred in the columns and vaults. Owing to the subsidence, this deterioration of the monument remarkably increased, so that the risk of a sudden collapse was feared.
Therefore it was necessary to transfer the loads transmitted by the domes and the vaults directly to the foundations. In this way it was possible tc substantially reduce the stress-field in the columns, whose material had seriously weakened (Fig. 5).
Instead of using traditional shoring methods, the Architect of the Cathedral followed a new course. He designed, tested on models and then carried out different types of reinforcements to be superimposed on the original compromised structures (Fig.6).

With the protection provided by these support sy

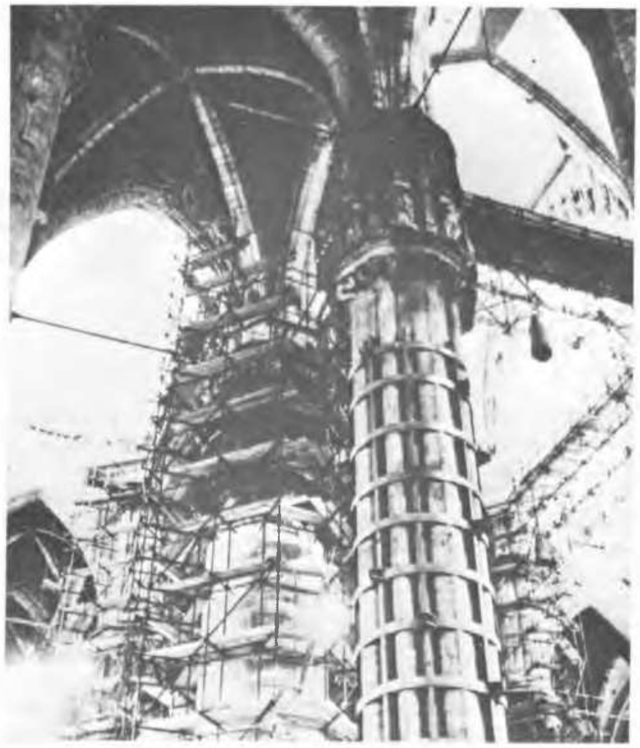

Fig. 6

stems, the Architect will now turn towards the execution of an inverse operation.
He will gradually eliminate the superimposed reinforcements, while proceeding to improve and reconstruct the material of the original structures.

I cited the problem of the temporary support of the monuments, since it happenes sometimes that the overlying structures are in good static conditions and the work has to be carried out on the foundations or on the subsoil. However the monument requires temporary support.
This could be the case of the Leaning Tower of Pisa.

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S. Martinetti (Oral discussion)

SAVING OLD TOWNS RISING ON HILL TOPS
Petites Villes Bâties sur les Sommets des Collines

SYNOPSIS Scattered throughout the Centre and South of Italy are a large number of beautiful ancient small towns of great environmental, cultural and historic value; the peculiarity of such towns is that they rise on hill tops that due to progressive erosion and degradation are affected by instability phenomena. This paper gives an overview of the problems related to the saving of such towns. The fundamental aspects of their stability have been identified within a general framework that can be applied to individual geologic and morphologic situations with only slight changes. The beautiful town of Orvieto has been chosen as an example and its situation is the subject dealt with by this paper.

Almost 80% of the Italian territory is hilly or mountaineous; two-thirds of the remaining flat land is situated in the Po Valley (Northern Italy). In the centre-south of Italy many towns are located in areas where a relatively rapid geomorphologic evolution is underway with a varyingly intense impact on the stability conditions of the towns themselves.

Figure 1 (D'Elia, 1981) synthetically shows the results of a survey carried out by the Ministry of Public Works; the figures relating to each Region stand for the ratio expressed in percentage between the towns where "landslides" are underway versus the total number of towns in that Region. This figure by itself is sufficient to understand how highly recurrent this situation is in Central and Southern Italy.

Many of these towns whose origins date back centuries and centuries rise on hill tops. The ancient inhabitants would choose such highlands to build their towns in order to defend themselves from invaders and also because the valleys were swampy and unhealthy.

A rather widespread situation is that of towns built on slabs of competent rock bound by nearly vertical escarpments which overlie clayey formations with milder escarpments. The competent rocks that form the slab are of various kinds: tuffs, lavas, conglomerates, sandstones, limestones and so on.

A regional example of this type of situation is illustrated in Fig. 2 (Manfredini et al, 1981); the schematic geologic map outlines an area lying immediately to the North of Rome, called Southern Tuscia, which corresponds to a part of ancient Etruria. The map also shows the stratigraphic successions that are typical of the towns of that Region, most of which were founded before or during the Etruscan period.

Among the towns scattered throughout this region, Orvieto is quite probably the most famous one in the world. It is lavishly endowed with invaluable monuments, and as a whole it is a town of great artistic value.

Figure 3 shows a global view of the town where the gentle slopes of the basal clay formation and the almost vertical cliffs in the overlying tuffaceous slab can clearly be seen. The top of the hill is entirely covered with buildings. The hill on which the town rises is completely isolated from the surrounding reliefs. The top is elliptical in shape, its longer axis lying in the EEN-WWS direction (1600 m) and its shorter axis being about 800 m long.

Figure 4 provides a schematic explanation of the geologic situation. Essentially the upper part of the hill is form-

Fig. 1 - Percentages of towns affected by major instability phenomena

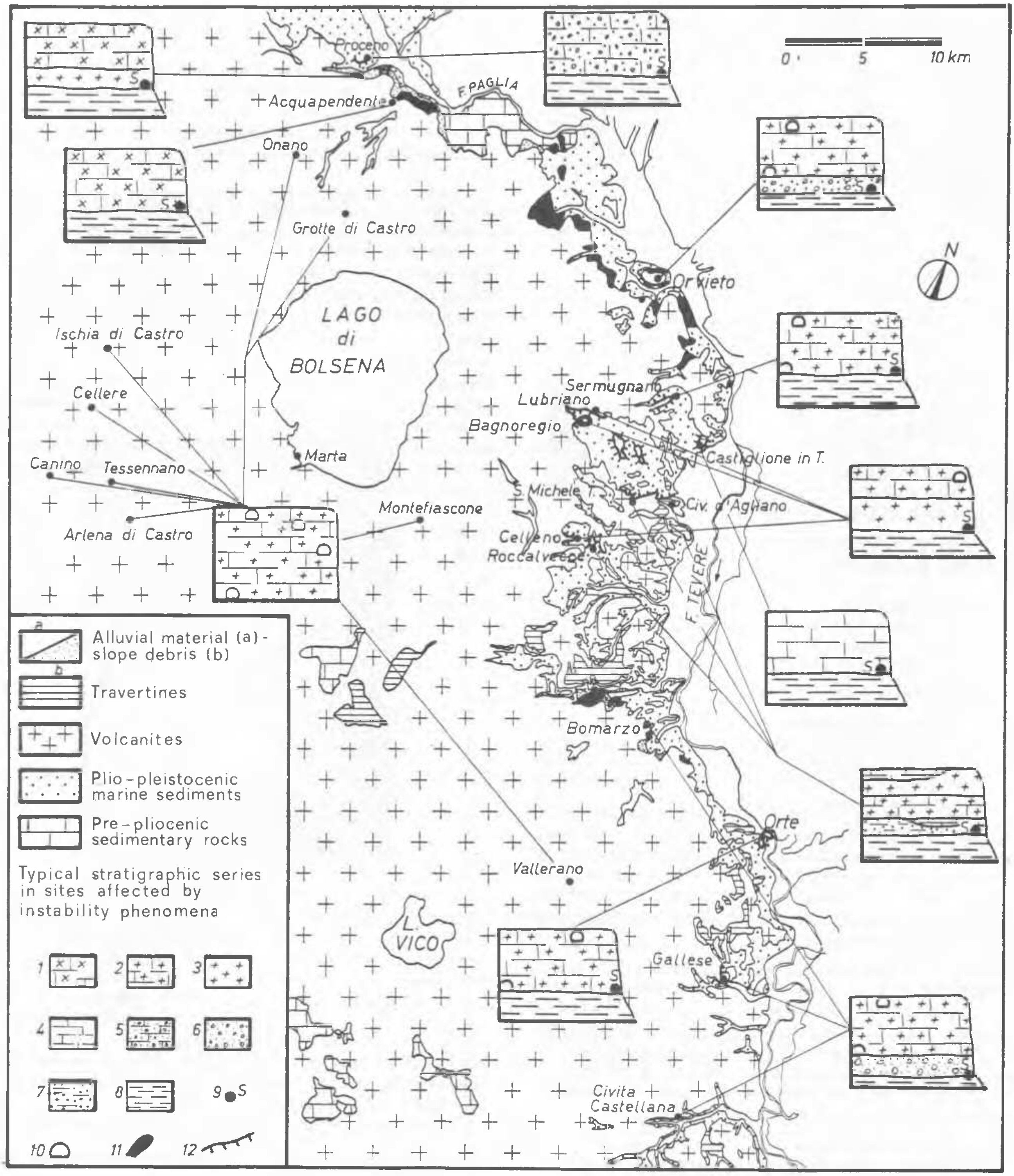

Fig. 2 - Schematic geologic map of Southern Tuscia and typical stratigraphic successions at the sites affected by instability phenomena. (1) Lava; (2) Competent Tuff; (3) Loose Tuff; (4) Travertine; (5) Conglomerates; (6) Sands and Gravels; (7) Sands and Silts; (8) Plio-Pleistocene Clays; (9) Springs; (10) Tunnels and Caves; (11) Landsliding Areas; (12) Edge of the slab affected by major instability phenomena

Fig. 3 - Panoramic view of the southern side of Orvieto Hill. The following can be noted in the picture: jointed lithoid tuffs passing to pozzolana in the cliff that shows its typical bee-hive structure; a portion of the cliff affected by a landslide in the more pozzolanic material; the "step-like" configuration at the edge of the tuffaceous slab

ed by a "lithoid tuff with black scoriae" (mean thickness being 50 m), which consists of a fine-grained matrix with inclusions of pumices of varying sizes; it is found both in lithoid facies with a reddish matrix (predominating) and in weakly coherent facies (pozzolana) with a grey matrix. The uniaxial compressive strength (Manfredini et al, 1981) of the lithoid tuff is about 3 MPa, whereas for the pozzolana it is about 0.9 MPa.

The underlying clays are stiff overconsolidated clays which in depth are not highly fissured. Their undrained cohesion is about 1.2 MPa. Nearer the surface the clay is softened and has poorer mechanical properties.

The presence of a thin layer (5-6 m thick on average) of sedimentary and remoulded volcanic materials (Albornoz) can be noticed between the tuffs and the clays; the presence of this layer proved to be very useful for stratigraphic reconstructions.

The stability problems that the towns of Tuscia and Orvieto are confronted with are quite similar to those facing other Italian towns in the centre and south of the Country, even if the formations involved are different. The remarks made in the following therefore are of a general nature even though the exemplification is based on the observations regarding Orvieto.

The very peculiar situations that favoured the development of those towns and of their flourishing civilizations, are the outcome of very rapid geomorphological evolutions. In fact, the inclination of the clay slopes exceeds the value that would ensure long-term stability and consequently, the slopes evolve through a succession of instability phenomena occurring in different zones and at different times, which consist mainly of landslides and creep deformations within the superficial softened part of the clay formation.

As regards Orvieto, (Sciotti, 1981) aereal photography has shown that the whole of the slope underneath the rock has the typical features of land subject to landslides; the older ones can clearly be identified by the presence of slide terraces; whereas, for the more recent ones, the area involved is cut up into slump blocks and at times the head is tilted into the hill.

An ongoing study is looking into the details of one such slide("Cannicella") that occurred very recently (Manfredini et al., 1981) and the results obtained so far are rather meaningful. Fig. 5 shows the scarp of such a landslide; in

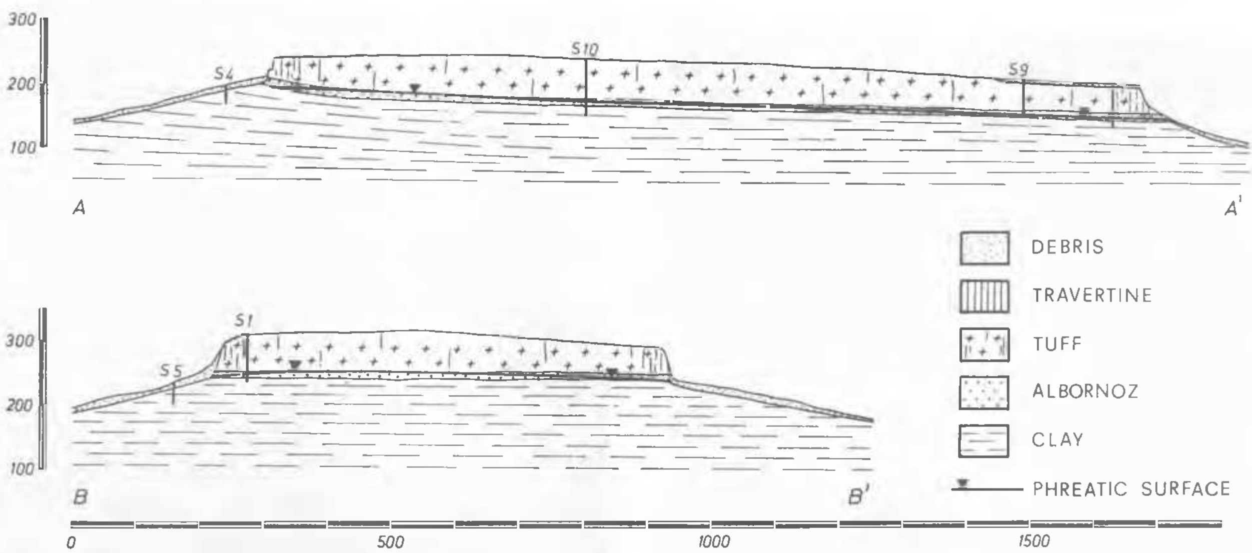

Fig. 4 - Geologic Cross-section of Orvieto Hill

Fig. 5 - Scarp of the "Cannicella" landslide

the back of the figure the base of the cliff is shown to be very close to the scarp. In Fig. 6 the results of the back analysis of the landslide are compared with the strength parameters determined by laboratory tests; the mobilized strength is very close to that of the softened materials.

All the observations made at Orvieto agree on acknowledging that the instability phenomena in the tuffaceous cliff are strictly related to the movements in the clay formation through a complex mechanism of interaction between the stiff rock slab and the underlying clay.

At Orvieto for example, there is a close relationship between the orientation of the joints in the lapideous tuff and that of the cliff faces; the main sets of fractures are all subvertical; one of them is parallel to the faces, one is perpendicular and the other two form angles of about 45° (conjugated) with the faces. The latter two sets of joints are found mainly where the border of the cliff forms projecting edges or spurs; where the latter are frequent the rock appears to have a column-like structure (Fig. 7).

The close relationship between the position of the joints and the orientation of the face points out that the sets of joints were originated mainly by the stress conditions induced by the morphologic situation; in particular the deformability of the clay, which increases towards the border of the cliff, is very likely to enhance fracturing.

Another instability phenomenon that appears to be induced by similar mechanisms (viscoplastic shear deformations in the clay formation), the one that produces the "step-

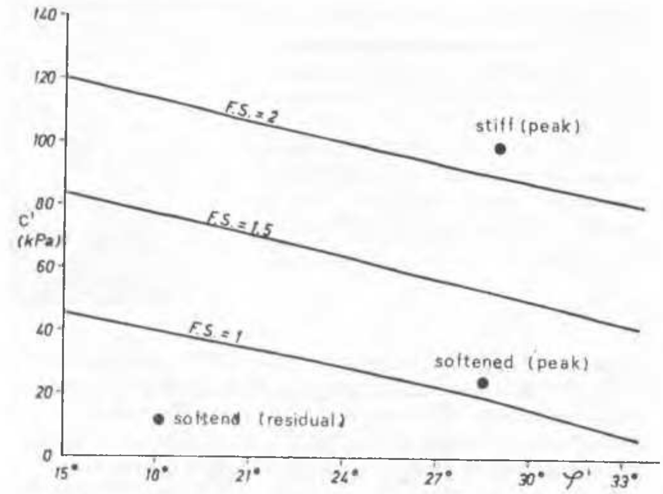

Fig. 6 - Back analysis of the "Cannicella" landslide

like" configuration of the border zones, consists in the lowering of the marginal clods of the tuffaceous cliff with respect to the back rock mass. This phenomenon is visible not only directly (Figs. 3, 7) but also through the stratigraphic reconstruction (Fig. 8). Another type of instability occurring in the tuffaceous cliff which is obviously related to the movements and deformations in the clay formation is the "disarrangement" of the tuff mass (Fig. 9), caused by a number of irregular fractures which tend to propagate from bottom to top. With time, slices of tuff reaching right up to the top of the cliff or involving only part of it, detach and then fall. In the latter case overhanging portions of tuff are left (Fig. 10).

Fig. 7 - "Column" fracturing of a spur and "step-like" configuration of border areas

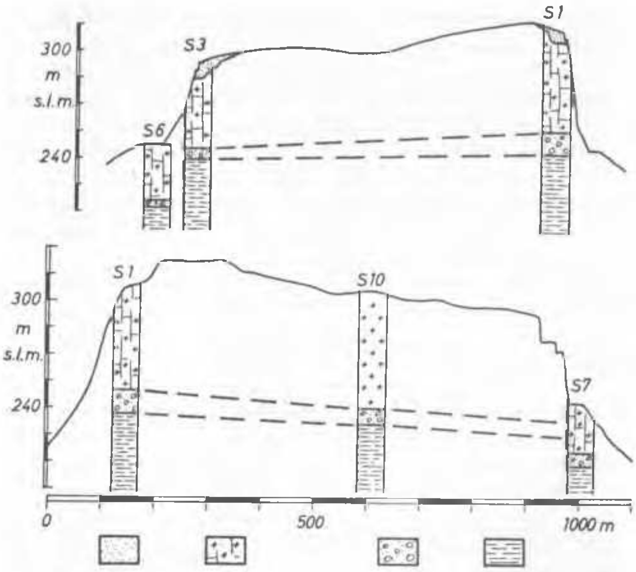

Fig. 8 - Cross-sections of Orvieto Hill (For legenda see Fig. 3)

To conclude, the most important stability problems of the Orvieto cliff, and, in general of all rigid rock slabs on clay slopes, are closely related to the deformation and failure phenomena that occur in the underlying clays. With reference to the human time scale, the rate of the natural evolution of hillside configuration can be considered to be a slow one; however, it must be borne in mind that man's activity may significantly increase the rate of

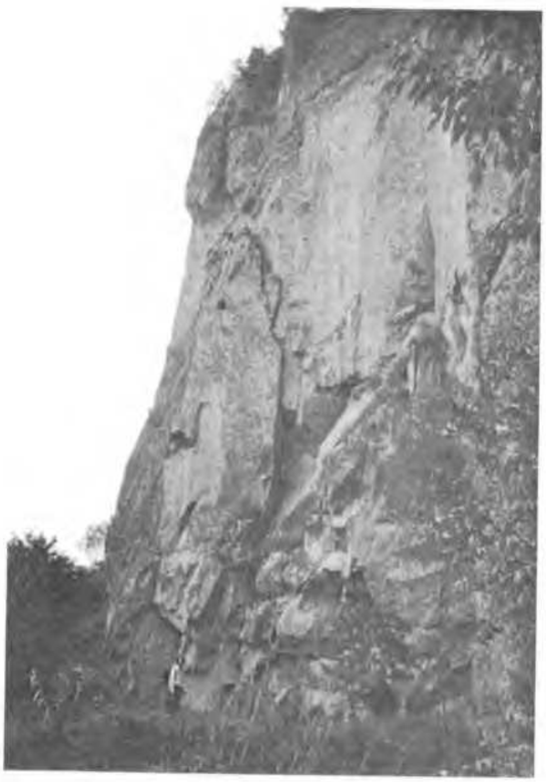

Fig. 9 - "Disarrangement" of the tuffaceous mass

Fig. 10 - Portions of overhanging rock

these changes.
Regarding Orvieto there is a general feeling that the rate of the changes in slope configuration has hastened over the last decades, but unfortunately well-founded data are totally lacking, and thus it is impossible for the time being to relate such changes directly to anthropic factors.

However, there are no doubts whatever that one of the main factors influencing the rate of slope evolution is the changes in the hydrogeological situation brought about by greater availability of water for the towns that have expanded considerably as well as by the growth in needs of the citizens; indeed, there ensues an increase in the quan-

Fig. 11 - Broken and displaced check-dam; water infiltrating into the clay formation

tity of infiltration water both from leaks in the aqueducts and sewers and from the discharge of waste waters along hill slopes.

The remarks made in the foregoing give an idea as to the severity of the problems brought about in Italy by the need to make sure that the countless number of towns on hill-tops can go on living in acceptable conditions of safety, in particular those that by centuries of history have been provided with an artistic and monumental patrimony of great value.

The economic implications of these issues are self-evident, and they demand that the measures to be taken be planned very carefully keeping account of all the relevant factors, assessing the objectives that the measures must achieve and choosing the best solutions for each practical situation.

The following provides a guideline for dealing with this problem:

- first of all an assessment should be made of the evolution rate of the slopes and of its correlation to anthropic factors;
- secondly the remedial actions should mainly aim at controlling the primary causes for instability in the clay formation rather than at reinforcing the zones of the rock cliff where its effects are more conspicuous; indeed, measures of the latter type can be urgently taken to help save crucial local situations, but they never represent solutions capable of changing the general trend of the phenomenon;
- thirdly, the remedial measures should be conceived of "dynamically", bearing in mind that continuous interventions will be required to maintain the functionality of the works, and also to avoid that self-enhancing deterioration processes be triggered. It must be pointed out that as regards Orvieto, Sciotti (1981) provided clear evidence that many of the more recent slides in the clay slopes initiated from gullies as a consequence of the increase in the uncontrolled circulation of water; Fig. 11 shows that a check-dam built only a few years ago along one of these gullies has already lost its essential functions; this could have been avoided without incurring large expenses by merely providing an adequate inspection service;
- finally, the effectiveness of each measure should always be controlled by appropriately monitoring the behaviour of the clay slopes and of the rock slab.

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J. Huder, Panelist

CONFORTEMENT DU CLOCHER DE ST MORITZ

L'édification de nouvelles constructions est en général la tâche de l'ingénieur, car il a été formé pour cela.

Il connaît:

- les propriétés des matériaux de construction,
- la statique resp. les conditions d'équilibre des forces qui doivent être remplies
- et les conditions pour le sous-sol, afin qu'une telle construction soit résistante.

Comment doit procéder l'ingénieur lors de la conservation de monuments architecturaux? Souvent dans les vieilles constructions l'observation de ces trois exigences fait défaut, mais il y avait autrefois des professionnels qui pratiquaient l'art de la construction avec leur seule intuition.

Il s'agit certainement d'un travail difficile qui demande aussi de l'intuition de la part de l'ingénieur, lorsque celui est prié de restaurer de vieilles constructions. L'état des constructions peut se présenter ou par des fissures ou des inclinaisons, cependant les tensions internes sont difficilement à reconstituer.

Les problèmes qui ont été soulevés dans le cadre de notre session seront illustrés par l'exemple du clocher penché de St. Moritz.

Si l'on considère la figure 1, on constate très facilement que tout ne va pas pour le mieux pour le clocher, mais quoi? L'inclinaison atteint 6° et la déviation de la force résultante environ 1,50 m c.à.d. environ le quart de la largeur de la fondation. L'inclinaison est aussi importante que celle de la tour de Pise. Bien que les deux tours aient plus ou moins le même âge, c.à.d. construites durant le 12e siècle, l'histoire des déformations est très dissemblable. Les premières notes sur l'inclinaison du clocher de St. Moritz remontent seulement à 1797. A la suite d'un tremblement de terre, la tour se déplaça et menaça de s'écrouler. En 1890, l'église à cause de ses fissures fut démolie et simultanément on ôta les cloches du clocher. Il est intéressant de noter que l'église en amont du clocher penchait vers l'amont au contraire du clocher qui penche vers l'aval.

Vers les années 1928 on discuta de la démolition ou de la consolidation du clocher, après que les

Fig. 1 Le clocher penché de St. Moritz

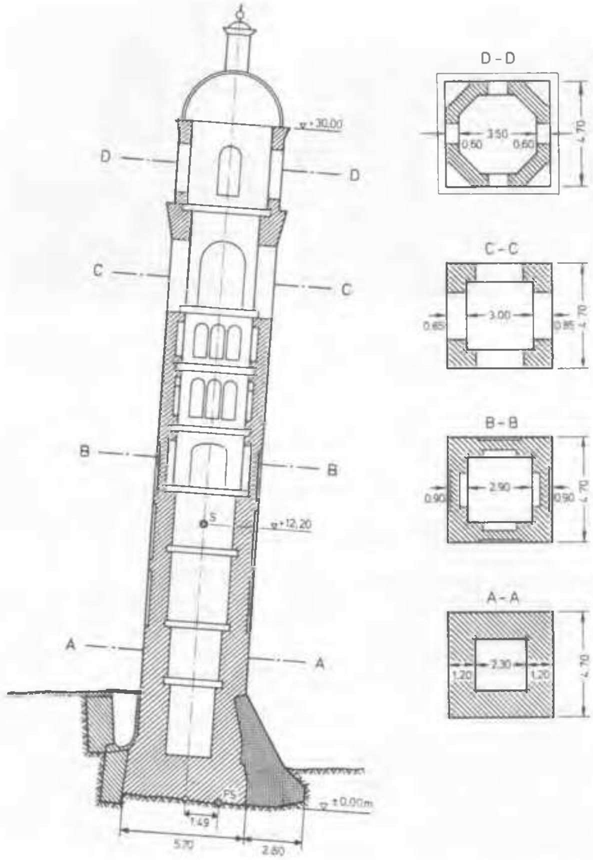


Fig. 2 Différentes coupes à travers le clocher

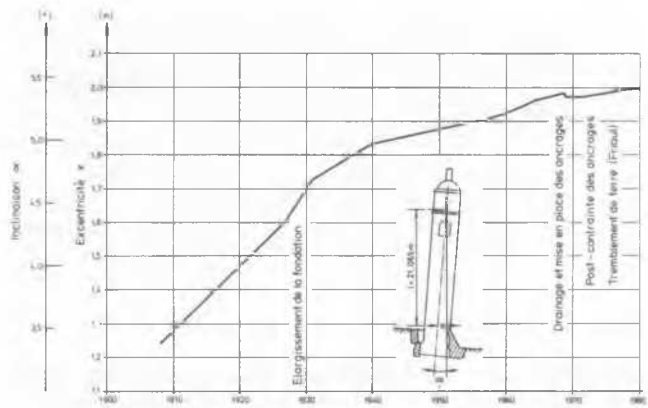


Fig. 3 Déformations entre 1907 et 1980

déformations qui furent enregistrées pour la première fois en 1908, aient pris des proportions menaçantes. Voir figure 3. Le célèbre ingénieur suisse R. Maillart et l'entrepreneur F.L. Prader furent chargés de la sauvegarde de l'édifice. Après le contrôle de la maçonnerie du clocher (fig.2) il apparut indiqué de renforcer la fondation. Cette mesure était aussi relativement bon marché et l'exécution en fut décidée et menée à bien. L'influence de cette reprise en sous-oeuvre est aussi nettement visible sur les déformations (fig.2), cependant une stabilisation complète ne fut pas atteinte.

Dans les années suivantes et jusqu'à 1967 le clocher se déplaça de 1,50 cm par année vers l'aval et l'augmentation de l'inclination fut, dans cet intervalle, de 0,24% par an.

Sur la proposition de R. Haefeli on a alors mis en place 4 tirants à 300 kN de 20 mètres de long qu'on a mis sous tension. En plus on a encore aménagé 3 sondages horizontaux en drains. Le renforcement de Maillart permet l'exécution des tirants.

Durant cette période l'état d'esprit par rapport aux vieux édifices changea de même. La démolition du clocher n'est pratiquement plus mentionnée cependant les déformations, en constante progression, incitent à une nouvelle redéfinition des mesures de sauvetage.

Les installations de mesure furent affinées et les mesures intensifiées. La cause de ces déformations dut être à nouveau étudiée. Elle réside dans le matériau d'éboulis qui remplit un profond sillon d'érosion. La situation topographique et une nappe phréatique haute, comme on le remarque à la figure 5, sont responsables pour les déplacements. Les vecteurs des déplacements sont aussi donnés à la figure 5. Les

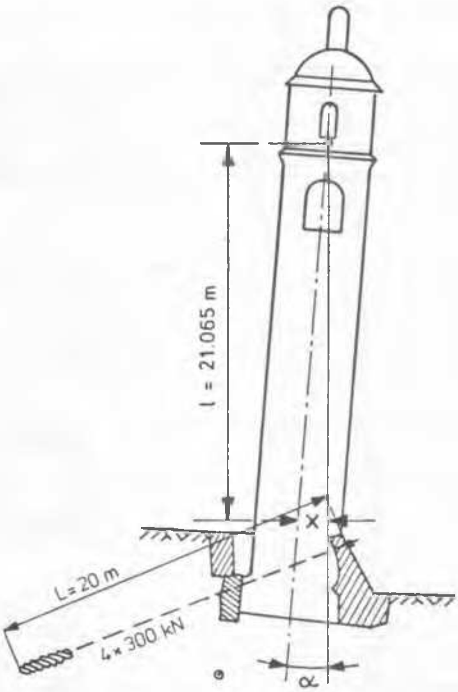


Fig. 4 Confortement du clocher et situation des tirants

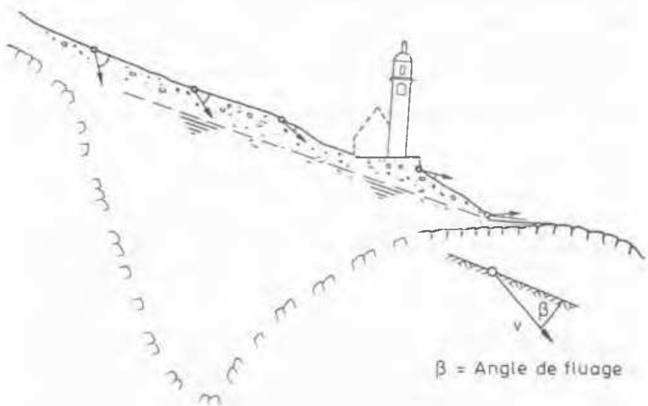


Fig. 5 Coupe à travers la pente

grandeurs des déplacements mesurés varient très fortement d'un point à un autre et atteignent jusqu'à 20 cm.

Pour mieux évaluer les différentes influences sur l'ensemble des déformations, on a étudié plusieurs modèles de calcul différents. A la figure 6 on a représenté le mouvement que le terrain autour du clocher doit effectuer, si l'on considère uniquement le pivotement de la tour. In reconnaît nettement le centre de rotation de ce mouvement. La figure 6 montre aussi distinctement pourquoi les ancrages n'ont pas eu l'effet désiré. A cela s'ajoute que les zones d'ancrages des tirants sont fixées dans la masse qui est elle-même en mouvement.

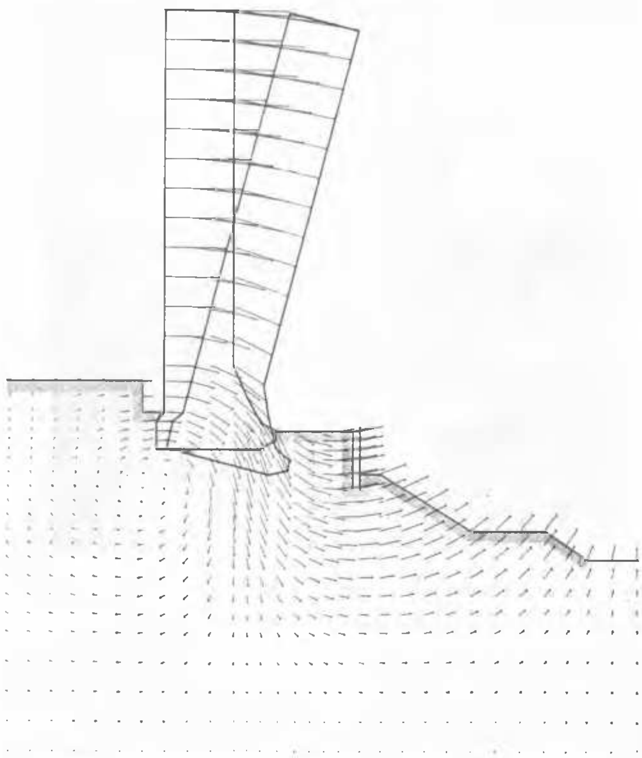


Fig. 6 Résultat des déformations subies par le poids propre du clocher

Il est certain que la reprise en sous-oeuvre et partiellement aussi les ancrages ont aidé à la conservation du clocher, cependant un assainissement total n'est possible qu'en considérant l'ensemble de la pente. Ceci signifierait un important drainage de la pente, une entreprise qui seulement pour le sauvetage du clocher entre à peine en considération. Au contraire on prévoit à nouveau une consolidation locale seulement. On a prévu des voiles en parois moulée avec appui réglable du clocher.

W.X. Huang and X.Z. Chen (Oral discussion)

TIGER HILL PAGODA OF SUZHUO, THE PISA OF CHINA

Tiger Hill Pagoda of Suzhou, China was built around 959-961 AD.. It has 7 stories and its total height is about 47.1m. The plane of the pagoda (Fig.1) consists of an hollow octagon with 8 exterior or wall columns and 4 interior columns. The thickness of the wall is about 1.8m, and the base width of the pagoda is about 14m.

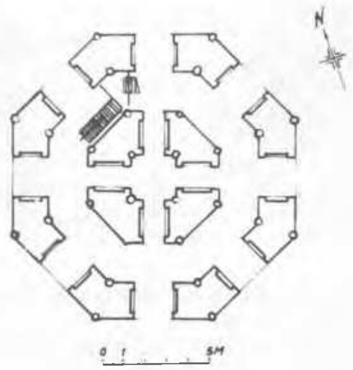


Fig.1 Cross Section through the Base Story, as well as the Foundation Plane of the Pagoda.

Brick was the only material used for construction. The total weight of the pagoda is estimated about 5000 tons. Both the wall and interior columns were directly built on a layer of broken brick and clay mixture of about 1.0m thickness with no extended footings. Below this supposed foundation base is a layer of broken brick and stone of about 1.0m thickness, and further underneath is a layer of clay and broken stone mixture with a thickness of 1.57m. Between this layer and the sloped base rock, which is inclined with a slope of about 1:5 from south towards north, is a layer of light clay mixed with some rocks and boulders. The maximun thickness of this layer which occurs on the northern side of the pagoda reaches 2.27m. Thus the total thickness of the compressible stratum underneath the pagoda varies from about an

Fig.2 Tiger Hill Pagoda of Suzhou

Fig.3 Plumb Line showing inclination of 7th Story.

estimated depth of 2.4-2.8m on the southern side to 5.8m on the northern side of the pagoda. This difference in the thickness of the foundation soil stratum may be the principal cause of the tipping of the pagoda.

In the memory of the people this tower is always inclined. The displacement of the top from the plumb line passing through the center of the foundation base now reaches 2.32m (see Fig.2). The inclination is towards NNE. 2 interior columns and many door openings are badly cracked, thus restoration appears to be imminent to most engineers.

It was recorded in history that the 7th story has been reconstructed in 1638 AD, but this story has now been found also inclined towards NNE. (see Fig.3). This indicates that during the past 340 years the tipping of the pagoda is still continuing. This pagoda has been restored many times since its construction, but no detailed record has been found. In 1957 this pagoda has been strengthened with reinforced concrete loops, thus the total weight of the pagoda has now been increased 520 tons as compared with the original. Restoration by underpinning and other foundation treatment measures are now under active study.

Acknowledgement is due to Prof. T.M. Yu, Mr. Y. Z. Tao, Mr. J. Zhao, Mr. Q.L. Pan and the Commission of Historic Relics of Suzhou for their cordial assistance in the preparation of this manuscript.

F. Lizzi (Oral discussion)

THE LEANING MOSUL MINARET RESTORATION WORKS
La Consolidation Statique du Minaret Penchant de Mosul (Iraq)

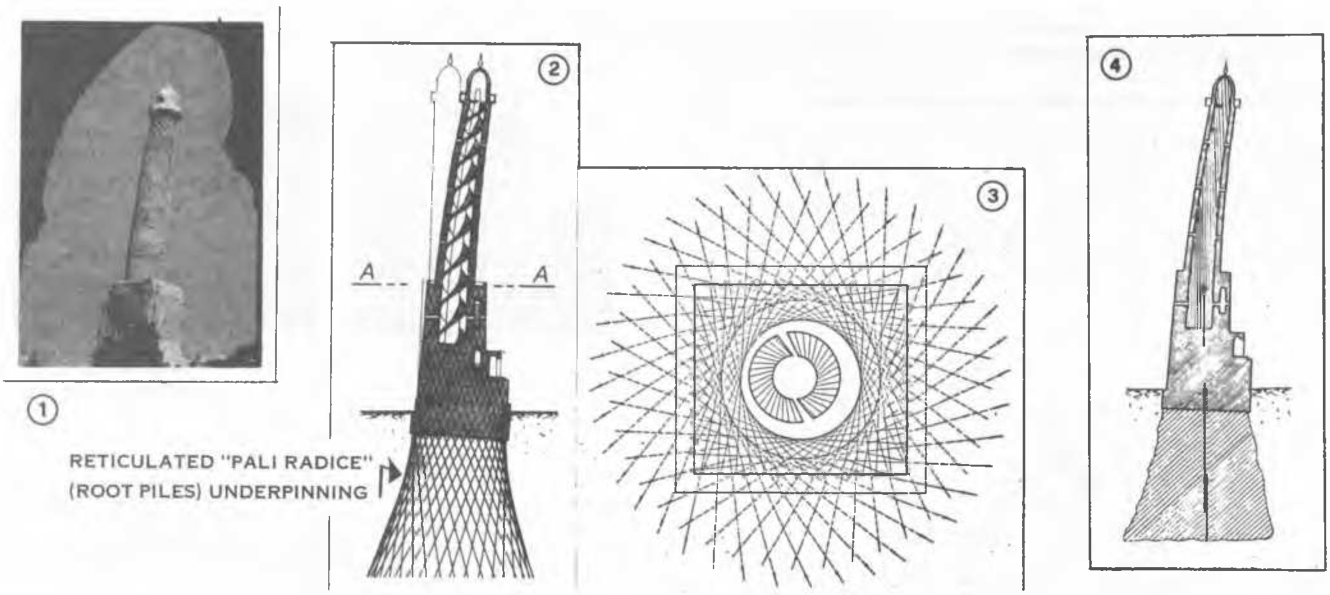
The AL-HADBA Minaret in Mosul (XI century) is well known all over the world as the "Pisa Tower" of the Middle East (fig. 1).

The progressive inclination of the structure threatened, lately, very seriously its stability and asked for immediate intervention.

The restoration works, designed by the Italian Company FONDEDILE S.p.A. and carried out in approx. 10 months (1980-81) by the same Firm, consisted in (figg. 2 and 3):

- the masonry reinforcement with a network of steel bars grouted in the walls ("reticolo cementato"), in order to obtain some sort of "reinforced masonry";
- Underpinning, carried out with "pali radice" (also known as "root piles", "micropiles", "pieux racine", "Wurzelpfähle", "estacas raiz" ecc.) according to the original patents obtained by the Author in 1952.

  The number of piles was proportioned to the



full vertical loads, like in normal foundations or underpinning. On the other hand, it is to be noted that the piles were distributed, with opposed raking, according to two revolution surfaces in order to obtain a "reticulated pali radice structure", that is a mixed structure in which the soil is encompassed by a network of piles. According to this scheme, the plurality of piles forming the underpinning can be considered also as an "in-situ reinforced soil" associated with the upper structure.

The soil reinforcement is, therefore, not separated from the structure, but it becomes part of the same, according to a unique gravimetric scheme, whose gravity center is very close to the soil surface. The advantage is evident (fig. 4).

No stresses were introduced, neither in the masonry nor in the soil.

No propping structures were used, except for normal scaffolding.

The outer parts of masonry did not suffer any aesthetical alteration, except for some already crushed decorations which had to be restored.

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Loizos, A.A. (Oral discussion)

STABILITY OF THE SPILLIANI MONASTERY, NISSIROS, GREECE

1. GENERAL

The Spiliani Monastery is situated on a ridge, located on the island of Nissiros, which is surrounded partly by the sea and partly (on the other side) by the Mandraki district.

The slopes of the ridge are very steep, almost vertical and they have a height of approximately 25 - 30 m.

It has already been ovserved that slides of rocky material occur from the slopes towards the sea. Due to this there is a possibility that large scale slides could occur, a fact which would be dangerous for the stability of the foundation of the monastery perimeter wall facing the sea.

Slumping has already been observed from the slopes towards the sea and the risk of large scale failures has often been pointed out in reference to the foundation of the sea-side part of the monastery.

2. GEOLOGICAL CONDITIONS

From the site reconaissance, it could be concluded, that the ridge has been formed by submarine volcanic outflows of andesite nature (pillow-lava). This material contains iron oxides and sulphate compounds. The andesitic cobbles are rounded and have dimensions which range from a few centimeters up to about one meter in diameter and are cemented by volcanic material consisting mainly of weathered material of volcanic origin derived by postvolcanic activity of warm fluids.

This process is responsible for the "watertight" of the volcanic rocks which the monastery has been built on. This "watertight" is easily observed from the artificial concave (cave) which lies within the church of the monastery having a ceiling depth of approximately 1.5m. under the surrounding soil surface.

So far, no rain water has penetrated into the concave and no humidity has been observed, a fact which is indicative of the "watertight" of the whole rock mass,which constitutes a "compact mass" together with the andesitic cemented cobbles.

On the contrary the monastery yard, which is situated at the top of the ridge, is composed mainly of losse fine-grained material of volcanic origin, the looseness of the grains being due to external factors, as agents of erosion to which the material is exposed. Similar erosion processes have been observed in several locations at the foot of the slope as well as over its surface.

3. NOTE OF LOCAL DANGEROUS DISCONTINUITIES AND CRACKS

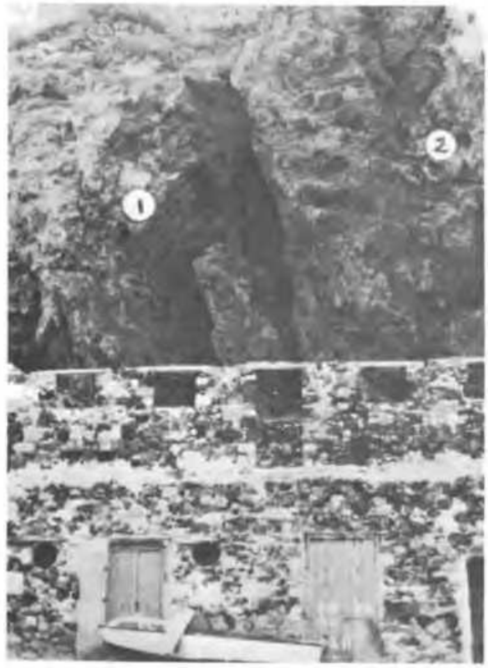
Within the concave, the rock is intact, as it is not subjected to atmospheric processes. On the other hand wherever it is exposed, as on the surface, the medium loses its cohesion. This weathering process is most probably a phenomenon at the surface.

Several faults or cracks have been noted along the side of the slope towards the inhabited area of the sea side, the most important being as follows:

a. There is a natural triangular concave under the wall of the stairs of the monastery. This extends over a large part of the total height of the slope, having a base of approx. 10m. and a depth of 4 to 5m (see Photos 1,2 and 4 point (1)).

b. Under the steeple of the monastery, there is a surface system of small faults, surrounding a wedge-like zone. This zone extends almost over the whole height of the slope and has a width of 4-5m. under the monstery yard (see Photos 1,2 and 4 point (2)).

Phot. 1



Phot. 2.

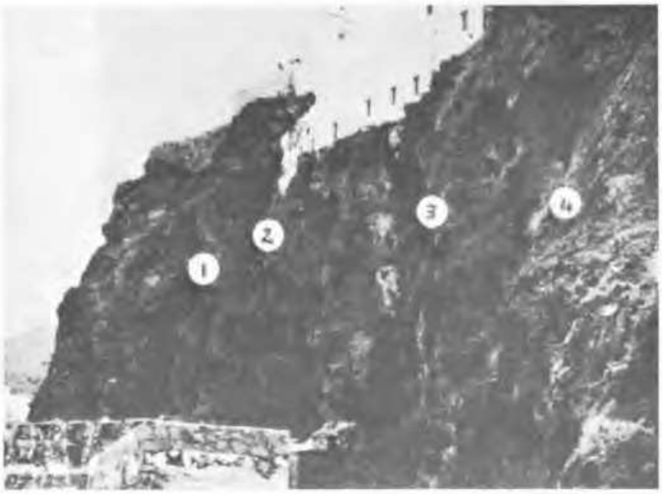
Phot. 3

c. At a distance of approximately 10m from the zone of point 2, there is another zone of small faults which extends from the basis of the wall of the monastery up to the ceiling of the artificial concave, the width of this zone being up to 5 to 6m. The erosion is intense at the surface of the zone and it could be attributed to a certain degree to the waterpipe of the monastery which discharges freely at the upper basis of the zone (see Photos 3 and 4 point (3)).

d. Far beyond the monastery yard (at the top of the ridge) towards the direction "Hohlaki" there have been observed indications of slumping of a large volume rocky block which is probably due to faulting.

In general it should be noted that no extension of the surface erosion phenomena should be expected along the basis of the monastery.

Since 1973 (when the corresponding investigation has been carried out) no serious slumping has been noted in the area under discussion.



Phot. 4

4. REMEDIAL MEASURES

From the site reconnaisance, it seems that the faults are mostly superficial and are due to erosional processes. We have been told that after rainfalls followed by periods of sunshine, slumping of small blocks of rock is observed.

It seems that no extensive discontinuities exist which could affect the stability of large blocks. However, remedial measures should be taken in order to protect the slope in the near future against erosional effects and to improve the conditions of the stability.

In particular, the following measures have been suggested:

a. Point (3). Loose material should be tentatively washed out from the outer zone

Phot.5

and after this the area should be protected from possible slumping by means of anchors. The wider part of this zone, should be protected against erossion by using gunite reinforced by iron mesh. Furthermore, the water pipe of the monastery should be withdrawn from this particular area.

b. Point (2). Wedge zone of faulting. Loose material should also be washed out from this zone, and special measures of improving the stability of the slope by means of anchors as it has been suggested previously. Also measures should be taken against errosion by using the Gunite method.

c. Point (1). Natural Concave Improvements of the stability of the slope by means of anchors and partial internal protection by using the gunite method.

d. Area between points(1) and (2). Installation of mesh reinforcement where necessary and protection by using gunite.

e. Point (4). Rocky block partly slumped. Anchors as in the case of point (3) and filling with cement grout the slumping gap (Photo (5)).

P. Bonaldi and L. Jurina (Oral discussion)

DETERMINATION OF FOUNDATION SETTLEMENTS THROUGH ANALYSIS OF CRACKS IN OLD MASONRY BUILDINGS
Détermination du Tassement à travers l'Analyse de Fissures dans un Vieux Bâtiment en Maçonnerie

In order to design structural reinforcements in old masonry buildings an appropriate knowledge of the actual state of stress, mainly due to dead weight and to foundation settlements, is required. As settlements are sometimes difficult to identify properly even by means of accurate in situ and laboratory tests, and complete historical reconstruction is seldom possible, an auxiliary procedure can be proposed. It is based on the simulation of the crack pattern present on the masonry walls, using a numerical model of the whole building.

This procedure was firstly proposed in Jurina et al. (1980) for the stability analysis of Palazzo della Ragione, a 13th Century building in Milan (Fig.1). The main steps are here described.

An accurate survey of the existing cracks and their openings was previously prepared while complete information about geometry, loads and constitutive laws of the materials was introduced in a finite element model of the structure (Fig.1).

As a first approximation, masonry was considered as orthotropic elastic and the main cracks were introduced simply disconnecting the nodal points.

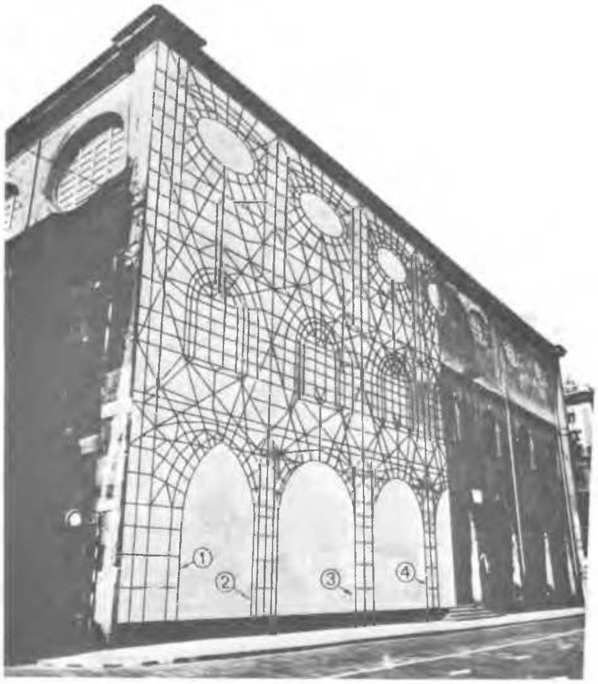


FIG.1 - Palazzo della Ragione in Milan (Italy) and adopted Finite Element model.

Unit distortions imposed to the basis of each column cause relative displacements of the opposite sides of all the cracks, which can be calculated and recorded. This was done for every column and for three different types of distortions, i.e. vertical settlements, horizontal settlements and rotations. If we amplify properly the effects of each unit settlement, it is possible to obtain, by addition, a distribution of openings in the cracks as similar as possible to the actual one.

COLUMNS	1	2	3	4
VERTICAL SETTLEMENTS [CM]	3.60	.14	0.	.38
HORIZONTAL SETTLEMENTS [CM]	.10	0.	0.	0.
ROTATIONS [DEGREES]	0.	.49	0.	0.

FIG.2 - Computed settlements in the identification process.

A simple procedure, based on the least square method, allows to find the amplification coefficients which minimize the discrepancy between calculated and observed response of the structure. In fig.2 the obtained settlements distribution is reported. The calculated crack pattern reproduces the actual one in quite an accurate way. It can be observed that the main damages are principally due to an anomalous settlement of column 1 and to a remarkable rotation of column 2 caused by the horizontal thrust of the no-constrained masonry arches. Repeating the same procedure for the adjacent masonry wall the identified settlement of the common corner column (column 1) is practically the same. The calculated out of perpendicularity displacements of the walls are in good agreement with the measured ones in both cases. Geotechnical in situ tests have also confirmed that the nature of the underlying soil is particularly poor expecially under the corner zone.

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E.W. Brand (Oral discussion)

INVESTIGATIONS FOR THE RESTORATION OF THE PHRA PATHOM CHEDI (PAGODA), BANGKOK; THAILAND

The Phra Pathom Chedi (pagoda) is an important Buddhist monument situated 60 km southwest of Bangkok, Thailand. It consists of a bell-shaped structure, 95 m in base diameter, built on a circular platform 158 m diameter and 9 m high. The top of the Chedi is 115.5 m above ground level. This huge structure, reputed to be the world's largest pagoda, is founded on the soft Bangkok clay which covers the extensive flat deltaic plain of the Chao Phraya River. Beneath the 15 m of soft clay are layers of medium clay, stiff clay, and sand and gravel to great depth, bedrock being at a depth in excess of 400 m.

The Chedi is of ancient origin and was built in three stages (Fig.1). The original structure on the site was erected about 300 B.C.; it was probably hemispherical in shape and took the form of an Indian 'stupa'. Some time in the twelth century, when the Khmers controlled Thailand, a 'prang' was erected on top of the stupa. The third stage, which is visible today, was built in the 1860s to cover the existing two structures.

In 1974, advanced deterioration of the Chedi fabric was evident, and it was decided that a full restoration of the structure should be undertaken. As part of this work, a geotechnical investigation was conducted, the main objective of which was to obtain geotechnical engineering information sufficient for an assessment of the past foundation performance of the Chedi and for its imminent restoration.

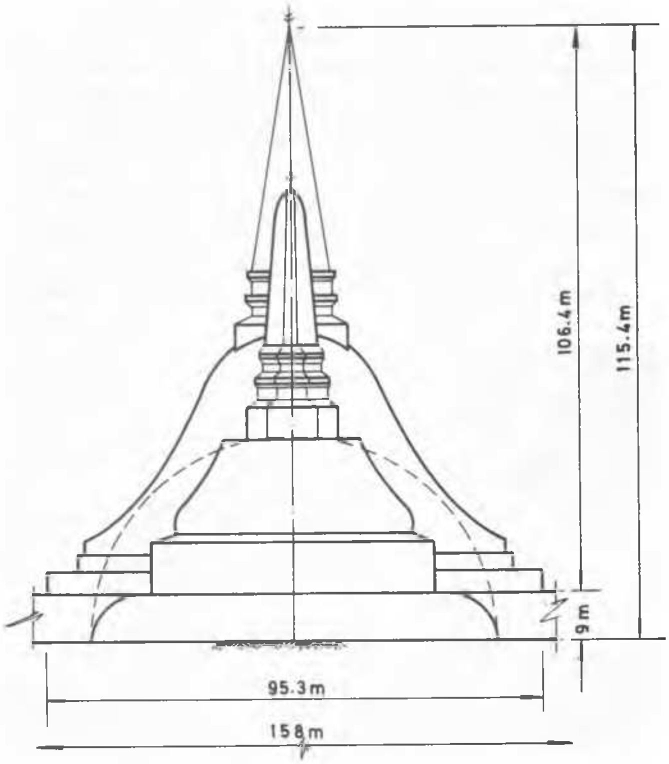


Fig.1 Cross-section of Chedi showing the three stages of construction

The scope of the investigation was :

(i) determination of the soil profile and its variations at the site of the Chedi,

(ii) determination of the engineering properties of the soils beneath the Chedi,

(iii) installation of some simple instrumentation to enable future monitoring of the Chedi settlements and in-situ pore pressures, and

(iv) evaluation of the settlement and stability history of the Chedi, and assessment of the effects of any restoration works on future settlements and stability.

The programme of investigation revolved largely around boreholes and laboratory testing. A great deal of data was obtained, and extensive analyses were carried out, details of which cannot be given here.

Vertical boreholes on the platform revealed that the foundation supporting the structure consisted only of a 'raft' of unmortared bricks several metres thick. Holes drilled horizontally through the Chedi proved that the structure was completely solid and was composed entirely of bricks. Samples of the bricks enabled the total weight of the Chedi and its platform to be estimated as in excess of 5 million kN!

Typical of some of the calculations carried out to trace the settlement history of the three stages of the Chedi are those shown in Fig.2. It can be seen that settlements were calculated for both rigid and flexible raft conditions.

Although a slight tilt of the Chedi was found to exist, settlement and deformation observations taken over a period of a year established that no movements were still taking place. Back analysis suggested that a total settlement perhaps as high as 2.5 m had occurred throughout the life of the Chedi. Because of its existing vast weight, it was decided that the addition of about 25,000 kN as part of the surface restoration would not cause appreciable movements: in the event, this proved to be correct.

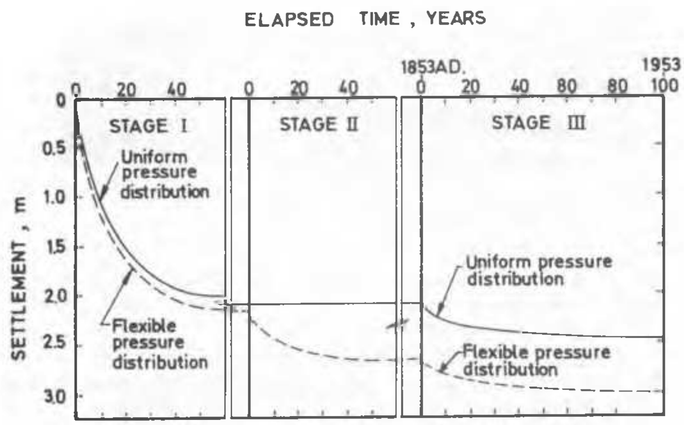


Fig.2 Deduced settlement history of Chedi for 'uniform' and 'flexible' assumptions

B.S. Browzin (Oral discussion)

CALCULS DYNAMIQUES DES STRUCTURES ANCIENNES EN REGION SISMIQUE

It appears that seismic load on old structures, such as cathedrals, monuments, monasteries, castles, and palaces should be evaluated, particularly in Italy, and dynamic analysis performed. This will provide information on the expected performance of ancient buildings during severe earthquakes that may occur in the future. It may also indicate that preventive structural measures are necessary at present in order to protect the ancient religious and architectural treasures for mankind for the future.

The discusser was happy to learn that the Italian experts promised to report on this subject in San Francisco at the XI ICSMFE four years from now.

C.A. Mascardi (Oral discussion)

Few comments can be added to the contributions by Martinetti and Diamanti on the geotechnical problems of the old towns on the cliffs, particularly Orvieto, and on their solutions.

THE MECHANISM OF FAILURES.

The cliff of Orvieto, as many others, is originated by the geological condition of a rigid "slab" (tuff) resting on a relatively deformable "bed" (o.c. clay) labelled resp. A and B in Fig. 1. At the toe of the vertical walls limiting the slab, the debris produced by local or general failures in the tuff create a steep slope of heterogeneous material (C) resting on the more gentle slope typical of the clay.
The stress relief due to progressive dismantling of the tuff and erosion of the clay tends to cause vertical (upwards) and horizontal (outwards) strains in the clay: hence bending and stretching actions arise in the tuff slab and, from both, tensile horizontal stresses accumulate at its bottom. This situation was put into evidence by a FEM analysis roughly simulating the natural process. As the tensile strength of the tuff is reached, a series of vertical fissures appear in the lower portion of the tuffaceous mass. At Orvieto, a number of horizontal boreholes 60 m long, carefully inspected by a t.v. camera, have shown that width and spacing of the fissures at the lower third of the slab height do not vary significantly along the inspected lenght. The inspection of the many tunnels and caverns dug since many centuries in the upper part of the slab has shown comparingly very few fissures; those in the lower part of the tuffaceous layer are generally open but empty, as they close gradually upwards and hence cannot receive secondary filling from above.
A weak point in the system is the debris slope at the toe of the cliff: water coming from above percolates vertically in the rather permeable tuff and accumulates at the bottom, seeping then horizontally through the debris mass and emerging at the contact with the much less permeable clays. Very high piezometric levels and gradients occurring in the rainy season in this zone, play a dominant role in developing landslides involving also the upper region of the clayey bed.
As the tuffaceous prism limited by the cliff and by the plane of the first fissure is undermined by a landslide under its toe, the shearing action

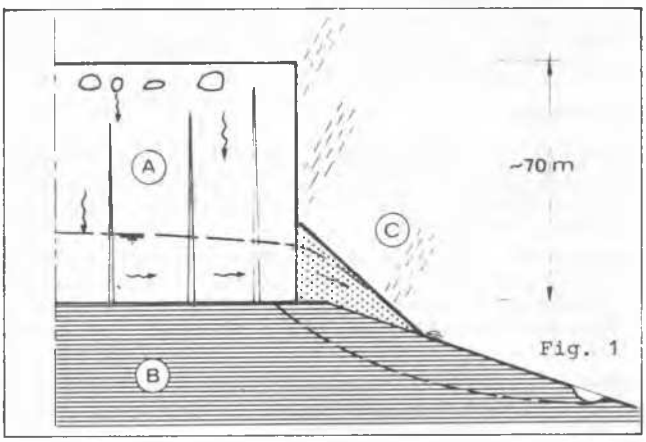


Fig. 1

extends the fissure upwards until the prism is completely isolated from the mass and it eventually falls by toppling. The fluctuation of water table level also causes a decrease in the strength of the tuff, as it was clearly shown by compression tests on specimens undergoing many wetting and drying cycles.

A SUGGESTED SOLUTION.

In spite of the general difficulty of opposing massive natural processes, a solution for the problem of Orvieto and similar cliffs can be effective, if all the causes of failure are opposed and not only their apparent effects. So it is believed necessary to modify the pattern of water flowing into the subsoil, not only by limiting the leakage from water and sewer ducts in the town, but also eliminating the outwards seepage pressure of water in the debris heap and erosion by surface water along the clayey slope. This can be achieved by a deep drainage system

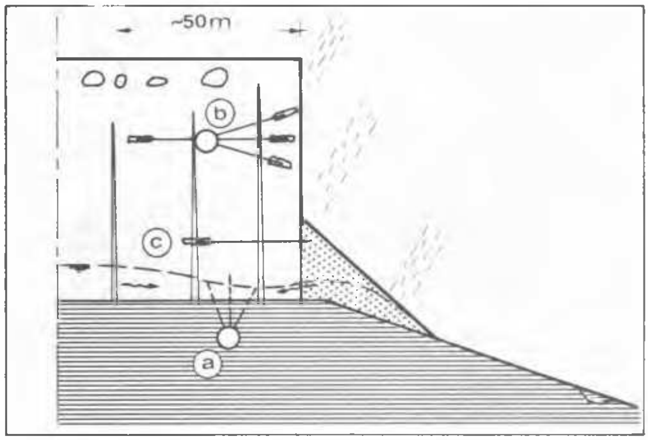


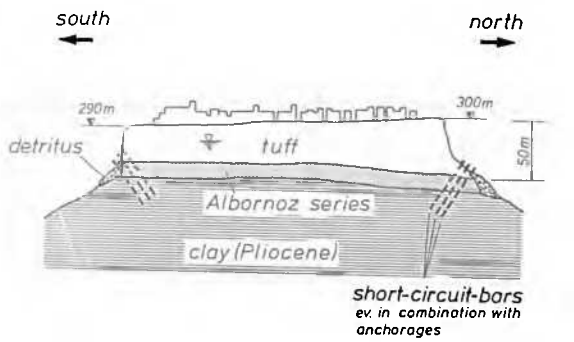
Fig. 2

comprising a tunnel and bored drains, as shown in Fig. 2(a). A secondary benefit would be an important reduction of the level fluctuation of the water table in the tuff, so eliminating its progressive weakening.
A second tunnel (b) at the upper third of the height would have permitted the execution of anchors both outwards and inwards, linked by the lining of the tunnel itself. The large total anchor lengths required could be achieved without difficulties and the aspect of the cliff could be preserved intact, without the marks of anchors heads, as it happens if they are bored from outside.
While the lower drainage should be built under almost the whole periphery of the cliff, the upper tunnel and the anchors could be provided only at the sites where the failure mechanism is more active.
A lower row of anchors (c) would eliminate, where necessary, any further horizontal strain at the tuff bottom.
These suggestions were not accepted by the "Technical Scientific Commission" which selected other solutions; curiously enough also the experimental data provided (t.v. inspection, effect of cyclic wetting) were completely disregarded.

Ch. Veder (Oral discussion)

PROPOSAL FOR AN INVESTIGATION WHETHER THE CLAY LAYER SUBJACENT TO THE CLIFF ON WHICH THE TOWN OF ORVIETO IS SITUATED CAN BE CONSOLIDATED BY ELIMINATING THE ELECTRIC POTENTIAL DIFFERENCES BETWEEN TUFF AND CLAY

As already known from numerous reports (P. Lunari, 1980; L. Diamanti, 1981), the more than 2500 years old town of Orvieto is endangered in its existence because the about 50 m high cliff of tuff on which it stands keeps breaking off in almost vertical slabs. This tuff, which turns in the lower layers to a detritus of different granulometry at different stages of cementation (Albornoz series), rests on a layer of clay. This detachment process is caused by a lateral movement of the clay that robs the tuff of its base.



The town of Orvieto, situated on a rock of tuff

The main cause for this fairly recent phenomenon, no doubt, is a general increase of water inflow to the clay layer, due to the increased water consumption of the population, as already pointed out by previous authors (P. Lunari, 1981; L. Diamanti, 1981). This leads to a swelling of the clay, making it deformable, and thus to the detachment of the tuff slabs.

Several alternatives for a stabilization have already been designed, such as the revision of the hydraulic and sewage plants, drainage in the bedding plane, horizontal and sub-vertical drains to reduce water pressure, reinforcement of the cliff with rock bolts and very long, prestressed anchors, and the construction of retaining structures that prevent to erosion of the clay layer.

In my opinion, an additional stabilization measure has to be carried out, based on the following theoretical considerations. The damaging action of the water in the clay layer is enhanced by two factors which are electro-chemical phenomena that have already been observed in other places. Electric potential differences will build up at the zone of contact between layers of different chemical nature, in this case tuff, Albornoz, and clay, which cause a flow of water from one layer to the other, and most probably this happens at Orvieto. Not only is there more water present in the tuff than in former times but the inflow of water to the clay is speeded up by potential differences.

One chemical factor are the feces contained in the sewage, which make the reducing layer (clay) even more reducing, thus increasing its negative charge (Veder, 1981). Another factor could be the recent use of detergents for laundry and dishwashing. The water has a high content of Na-ions and these settle between the layers of the layered clay-mineral packs, dilating them and causing the clay to swell, thus reducing its strength (Ch. Veder, 1981, Fig. 7-8).

The stabilization proposals aim at eliminating to some extent the damaging water inflow and at fixing the vertical tuff slabs with anchors, but there still remain the potential differences between the different geological strata. Even very extensive drainage lines will eliminate only the gross flow of water from the tuff to the clay but not the danger of the swelling phenomena at the zone of contact due to potential differences. If the clay continues to swell and deform, this may exert strong additional stresses upon the anchors holding the tuff slabs.

In my opinion, it is first of all - even before any other works - necessary to eliminate the potential differences with short-circuit conductors. Prerequisite for this is that the existing potential differences are measured with a special sound from boreholes (d = 8 cm). For investigation purposes, in this case, the special sound needs to be inserted to depths between 30 and 50 m near the foot of the tuff cliff; the depths, inclination, and distance between the borings for the measurements must be varied.

The results of theses measurements provide the basis for placing the short-circuit conductors (Ch. Veder, 1981). As a rule, the drying-up process takes effect very quickly, mostly within a month. Where this appears necessary, subhorizontal drainage holes should be bored. Then a permanent anchoring can be designed and carried out. If anchorings or other stabilization works have already been carried out, then any change in anchor stresses due to the swelling of the clay, caused by the permanent moisture in the tuff, can be kept much better under control than without short-circuit conductors.

I would suggest that by way of experiment first an about 100 m long row of short-circuit conductors is installed, following the toe of the tuff cliff and then, depending upon the success of this measure, the definite installation could be designed. Unfortunately, the anchors will not act as short-circuit conductors; the concrete surrounding the steel of the anchors acts like an insulation.

The proposed measures are relatively economical and for the sake of Orvieto it could be wished that the potential measurements with the sound are carried out soon.

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J. Kerisel, Chairman

CONCLUSION AU NOM DU PANEL

Certes , nous ne sommes ni les seuls , ni les premiers à nous occuper du sauvetage des vieux monuments et vieilles villes : le monde est déjà sensibilisé à ce problème et un grand nombre d'architectes, d'historiens, d'archéologues se sont déjà penchés sur le problème en maintes occasions .

Mais la question s'aggrave pour des raisons politiques , sociales et financières .

Politiques parce que les guerres amènent l'abandon : à ANGKOR , il n'y a pas seulement le fameux temple hindouiste d'ANGKOR Vat semblable à celui que nous a montré notre collègue BRAND , mais 287 monuments dont une quinzaine ont une taille qui les rend comparables à nos cathédrales . Or aujourd'hui , ils sont à l'abandon : les pluies abondantes et l'absence d'entretien des drains, conjuguée au poids considérable des monuments les destabilisent,ainsi que les remblais qui les entourent dont les murs ne peuvent plus contenir la poussée .

Raisons sociales : les vieilles villes ne veulent pas mourir . Mais pour que la vie s'y maintienne et les irrigue , il faut aménager sous celles-ci des circulations souterraines , créer des accès à celles-ci et des parcs souterrains, et tout ceci avec des dommages aux vieux édifices .

Raisons financières évidentes car les pays sous-développés par exemple , où se trouve un certain nombre de ces monuments , ont à faire face à d'autres nécessités .

Le sauvetage des vieux monuments exige donc un effort accru et efficace . Or, un des éléments essentiels de leur survie est la maîtrise de leurs fondations à la fois en termes techniques et financiers : maîtrise très spéciale qui appelle le concours de géotechniciens dont la compétence puisse s'affirmer en face des considérations suivantes :

1°/ L'éventail des solutions est très large et leur choix est délicat .

2°/ Le diagnostic dépend au premier chef d'une reconstitution de l'histoire de l'édifice (phases de construction, modifications à l'oeuvre , travaux au voisinage, etc...) et d'une observation des mouvements (tassements, translations , rotations , etc ...) pendant une longue période couvrant plusieurs saisons .

3°/ Le diagnostic dépend par ailleurs d'une étude complète des caractéristiques du sol (à long terme notamment) et des variations de la nappe , éventuellement des vitesses de dissolution des sols ou roches .

4°/ Pour les problèmes présentant une certaine ampleur , il convient de dresser des cartes rapprochant les nivellements du sol et du toit de la nappe d'une part, de la géotechnique d'autre part .

5°/ Avant tout choix de solution , il faut rechercher si les nuisances dûes à l'activité de l'homme (pompages sauvages, variations de la nappe, fontis , infiltrations d'eaux nocives , gonflements , vibrations, érosion dûe aux agents atmosphériques, etc ...) ne sont pas responsables des dommages.

6°/ Il y a toujours avantage à éviter des reprises en sous-oeuvre trop partielles afin d'éviter que les parties sauvées ne jouent le rôle de points durs .

7°/ Si les moyens financiers ne permettent pas une reprise en sous-oeuvre généralisée , il ne faut pas hésiter , pour limiter cet effet de point dur , à asseoir les parties restaurées sur des fondations qui puissent concéder quelques tassements (en adoptant par exemple des pieux à coefficient de sécurité limité par exemple) .

8°/ L'électronique permet aujourd'hui de régler automatiquement les pressions développées sous la fondation en fonction de déformations mesurées sur l'édifice dans les différentes phases de la reprise en sous-oeuvre.

9°/ Pour savoir quels tassements ou soulèvements différentiels sont admissibles , il convient d'étudier la raideur des constituants (mortiers , briques , pierres, etc ...) et le système d'éléments porteurs compte-tenu des fissures existantes .

10°/ On doit tenir compte de ce que toute reprise en sous-oeuvre produit des tassements avant sauvetage .

11°/ La valeur de l'équipe qui met en oeuvre un procédé classique est aussi importante à considérer que le caractère novateur d'un procédé .

12°/ Le géotechnicien doit connaître de mieux en mieux les conditions de pourrissement des bois , de corrosion d'acier , d'altération des bétons au-dessous et au-dessus d'une nappe .

En conclusion , s'il a fallu des artistes pour la construction des vieux édifices , il en faut aussi pour assurer leur sauvegarde . Et jamais le mot art de l'ingénieur n'a pris autant de sens . Nous autres, ingénieurs géotechniciens , devons avoir un rôle important dans cette restauration . Il serait donc intéressant que les Sociétés Nationales de Mécanique des Sols et Travaux de Fondations réunissent une documentation comportant un abrégé de cette session 9 et des cas typiques nationaux pour la présenter aux autorités administratives de leur pays chargées de cette restauration .

Il est recommandé également à la Société Internationale d'adresser un abrégé des comptes-rendus de cette session 9 et en particulier des présentes conclusions à l'UNESCO avec l'annuaire de nos sociétés, en lui proposant la collaboration de nos membres partout où s'impose la sauvegarde de vieux édifices ou de vieilles cités .

Toutes ces conclusions sont le fruit des réflexions de tous les membres de notre panel, et en particulier de notre vice-président le Prof. A. CROCE et de notre rapporteur général le Prof. U. SMOLTCZYK et du co-rapporteur le Prof.R.BUTTERFIELD . Je les présente en leur nom à tous .

.I.W. Ellis (Written discussion)

(Discussion on Paper 9/81 by J.A. Lord)

With reference to the paper submitted by J.A. Lord and referred to in the General Report of Prof. Butterfield it was stated that for the underpinning of York Minster a number of different methods were considered. These included Pali Radice piles, Pynford Stools, ground freezing and bored piles reinforced with steel H sections. All of these methods were rejected in favour of extended concrete pad foundations.

Further on in the paper we are informed the piers supporting the Central Tower settled between 20 and 30 mm during the three year underpinning period.

While this work was in progress, Bootham Bar, another Medieval building only 200 metres away, was also underpinned. (See Fig. 1) Bootham Bar is one of the four ancient gateways into York.

Bootham Bar also had a serious settlement problem and had been monitored over a number of years. The subsoil was grossly overloaded, the avgarge bearing pressures being in the order of 400 KN/m^2. In addition the foundations were only 600 mm deep founded on wet silty clay. One hundred 114 mm Ø Pali Radice piles, 12 metres long were installed through the structure in a period of two months. No appreciable settlement was recorded during, or since the underpinning.

The reason for the little, if any settlement compared with that of the piers under the Central Tower of York Minster can be appreciated if consideration is given to the differences in the method of underpinning.

At Bootham Bar as each pile was constructed, part of the underpinning was completed thus gradually improving the foundations efficiency and factors of safety from the very first day. Also because of the large number of small piles no 'hard spots' were created and additionally the structure itself was slowly strengthened by the grout and reinforcement extending from the piles.

At York Minster, the excavation, cutting of the Norman foundations, drilling horizontally below the piers, stressing and finally jacking were all operations which contribute to further settlement of an already weakened structure.

Even settlement of a structure is not normally a problem but differential settlement, as was the case at York Minster, should be avoided - particularly when these movements can be suddenly induced by the cutting of existing masonry - or the removal of shoring etc.

The major consideration when examining proposals for carrying out the strengthening of foundations of structures such as York Minster, is that the system chosen should cause the least disturbance to the existing soil/structure interaction.

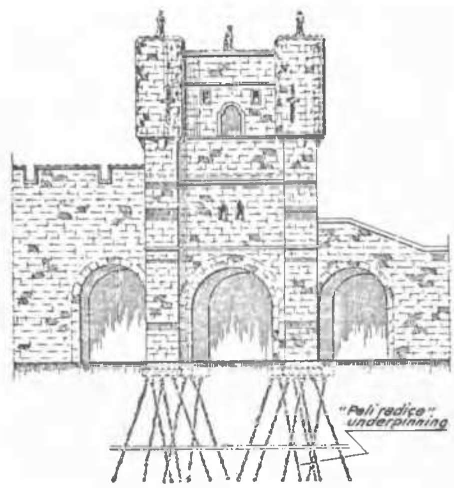

Fig. 1 Underpinning of Bootham Bar York

E. Togrol (Written discussion)

The safety of old structures sometimes had been secured by early but efficient repairs and reinforcements. During our research into the construction of an eighteenth century drydock (Fig.1) at Golden Horn Shipyard (Paper 9/26) Mr. I.H. Aksoy and I came across some valuable documents dealing with such a reinforcement.

From an archive document dated 1814 (i.e. after fifteen years of the completion of the construction) we learn displacements took place at the northern corner of the drydock. According to that document a number of stones of a sidewall about 0.75 m. to 1.50 m. below the sea level were moved inwards. Also water was observed percolating through the masonary wall.

The soil profile at this location consists of a shallow artificial fill which is underlain by a marine origined sandy silty clay which rests on hard graywacke. The type of deformation and its location suggest that some sort of softening or liquifaction had taken place below the artificial fill.

Various measures were discussed and recorded at that time. Finally, an interesting and rather sophisticated remedy was applied. An area about 15 m. long and 12 m. wide behind the damaged section of the wall was excavated down to about the deformed level. Then a number of wooden piles were driven to refusal. The lengths of these piles are now estimated to be about 8-9 meters.

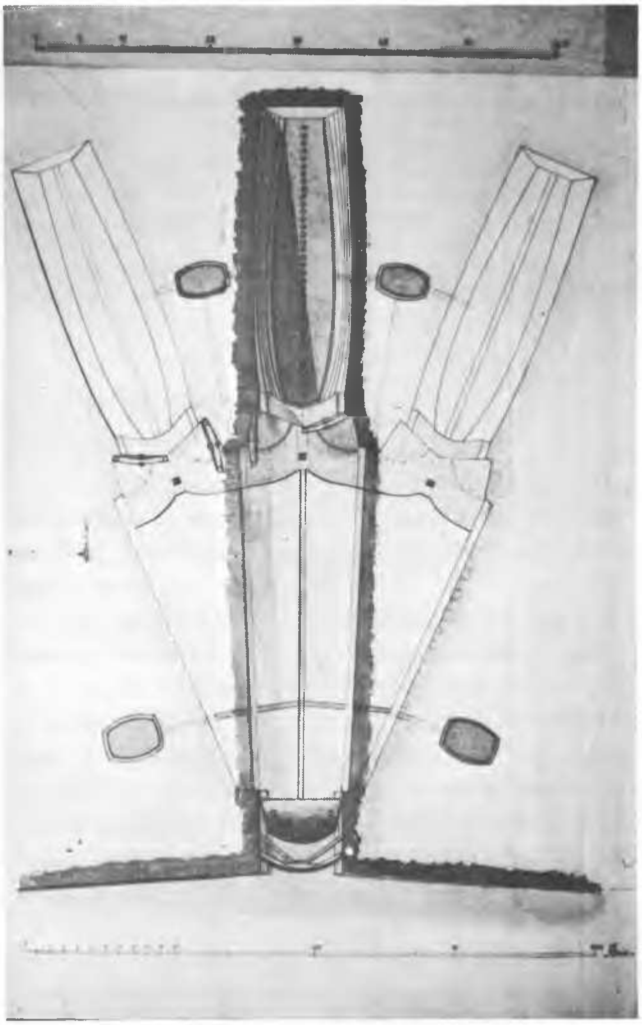

FIG. 1 - Original drawing of the 1796 drydock by Swedish engineer Rhodé showing the drydock, dock entrance and two possible extensions.

Then a large mass of mortar and rubble was placed on top of them. Large flat stones were placed on the mortar and rubble mass, in a grid formation. By doing so, builders presumably expected to achieve various things. They had had (a) a high strength backfill, (b) a fairly rigid pile cap against lateral movements, (c) an impermeable plug behind the deformed section. Of course, one of the merits of this design was that the vertical loads and possible moments were transferred to the rock foundation through the piles. This arrangement proved to be successful and no further repairs or reinforcements were required (Fig.2).

FIG. 2 - Present view of the once repaired corner.