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Control measurements and stability analysis of the Cathedral of Orvieto, Italy

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ABSTRACT: The stability of the Cathedral of Orvieto suffers from the existing cracks in the columns and pillars. A wide research on the statical behaviour of the structure is in progress. This study involves experimental measurements and numerical analysis aiming at investigate the static conditions of the Cathedral and define the consolidation structures required. The experimental measurements mainly refer to topographic survey, in situ and laboratory tests, stress and deformability measurements in column, masonry and subsoil. The analytical and numerical analysis concerns the loading analysis and 3-D modelling of a part of the structure, its foundation and subsoil. The diagnostic analysis of the safety conditions of the structural elements is underway and only the preliminar results and future developments are given here.

RESUME: Actuellement d'importants études sur le comportement statique de la Cathédrale d'Orvieto sont en cours. Les études envisagent des mesures expérimentales et des analyses numériques ayant pour but la recherche des conditions statiques de la Cathédrale et le calcul des structures de consolidation nécessaires. Les mesures expérimentales sont réalisées principalement au moyen de la surveillance topographique, d'essais in situ et au laboratoire, de mesures directes de l'état de contrainte et des caractéristiques de déformabilité des colonnes, de la maçonnerie et du sous-sol. Les analyses analitiques et numériques concernent l'analyse des charges et la réalisation de modèles 3-D d'une partie de la structure, de la fondation et du sous-sol. L'analyse diagnostique des conditions de la sécurité des élements structurels étant actuellement en cours d'exécution, seuls les résultats préliminaires et les développements futurs sont ici présentés.

1 INTRODUCTION

The construction of the Cathedral of Orvieto took place over three centuries. It was begun in Romanic style in 1290 to commemorate the Eucaristic miracle of Bolsena of 1263, and was then continuated in Gothic style. Subsequently and up to the present day the Cathedral (Fig 1) has been subjected to continuous modifications. The first problems of stability were recorded in 1308, when, after the construction of the three aisles and the transept, Architect Maitani was commissioned for the consolidation works with rampant arches and buttresses. In 1620 some columns were bound in order to prevent the increasing of the opening of vertical cracks which had appeared earlier. During the past 30 years, clear signs of instability in the Cathedral have been observed and have worried the responsible authorities.

This paper records some results of the studies, supported by the "Soprintendenza" of Perugia, the Technical University of Turin and ISMES of Bergamo. The main purposes of this study are the following:

1) Statical analysis of the structures and of its state of conservation
2) Crack survey and monitoring;
3) Investigation of a possible interaction between the instability of the tufa stab on which the town of Orvieto is resting and the stability conditions of the Cathedral.
4) Localization of structural problems which can be solved or reduced by consolidation works.

In situ and laboratory investigations, numerical analysis and control measurements, were carried out in order to deeply investigate the static conditions of the structures.

2 GEOLOGICAL AND STRUCTURAL FEATURES OF THE ORVIETO SLAB

The town of Orvieto is resting on a tufa slab, having an average height of 50 m, which lies above a hill composed of "pliocenic clay". A mixed sequence, volcanic and sedimentary, 10 m thick, called "Albornoz formation", is located between the clay and tufa formation. The upper part of the tufa slab, which is strongly weathered, is composed by pozzolanic sand. Subvertical joint systems divide the rock walls of the cliff into columns.

The instability phenomena in the cliff and in clay slope have been known and recorded since the Middle Age. The most important instability phenomena in the cliff have been connected to rock falls, foliations and local subsidence due to excavation inside the cliff itself. The origin of the failures depends on the tension stress in the tufa cliff due to the greater deformability of the clay formation in respect to the rocky slab and to the instability of the slope below. The instability of the rock walls is increased by the static fatigue and the weathering due to climatic action. Since 1979, consolidation works of the cliff are in progress as well as the regulation of the underground water.

3 SUBSOIL AND FOUNDATION SURVEY

The Cathedral of Orvieto was founded on the upper pozzolanic-sand layer. The survey of the subsoil of the Cathedral included a set of drillings in order to define the stratigraphy of the sequence below the foundation geometry and the depth of the separation surface between pozzolanic sand and tufa formation. The subsoil survey was extended by means of a detailed exploration of the caves excavated in the area surrounding the square. The position of the boreholes and of the caves is reported in the map of Fig. 2.

Fig. 3 shows a vertical section of the Cathedral foundation, perpendicular to the direction of the aisle, with the indication of the stratigraphy determined by the boreholes. The tufa formation was found at a depth of 23 m in reference to the Cathedral floor elevation (borehole 3). The caves were excavation only in the

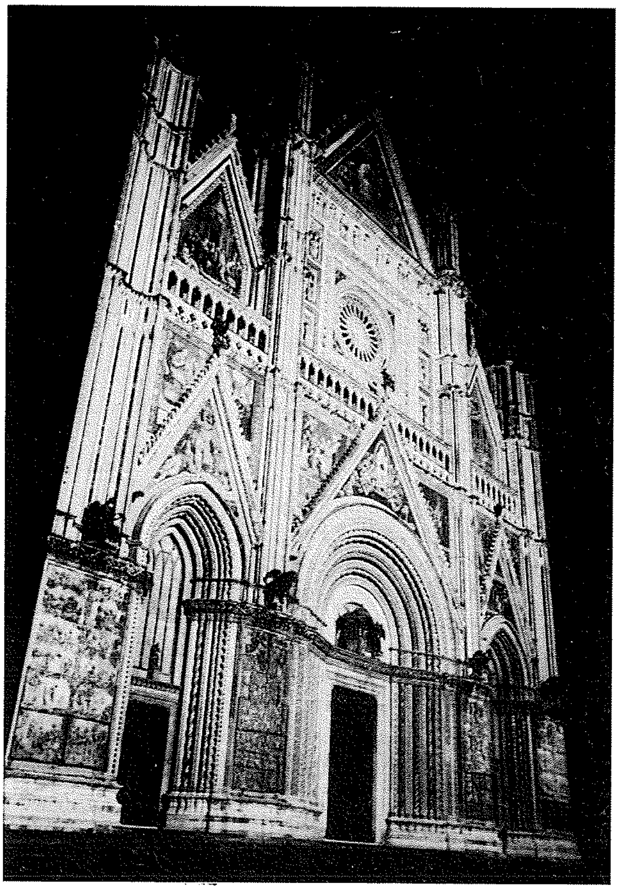

Figure 1. Frontal view of the Cathedral of Orvieto.

pozzolanic formation at a depth of about 12-15 m. It must be observed that the thickness of the pozzolanic sand layer is decreasing going to the edge of the tufa slab.

4 MASONRY STRUCTURES DESCRIPTION

4.1 Foundation structures

The Cathedral foundation is built with continuous masonry, composed by irregular tufa blocks with an average size of 10x20x30 cmc, cemented with mortar, lime and pozzolanic sand. The size of the foundation walls has been determined analysing boreholes 2 and 3. They are about 2.2 m thick, 8.3 m high and rest entirely on the pozzolana. Drilling 3, having a 30 degrees slope, penetrated the perimetral wall foundation of the left aisle for a length of 6.5 m in the zone of the 4th Chapel. This wall has an external lining, formed by alternating layers of travertine and "leucite tefritica" commonly called "basaltina".

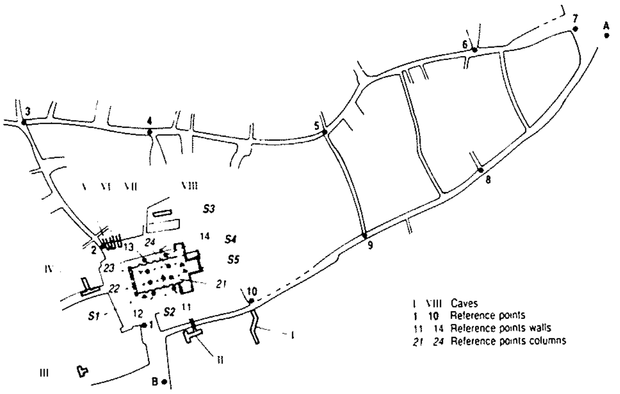

Figure 2. General layout with the indication of the drillings, the reference points for the topographic survey and the caves used for soil investigation.

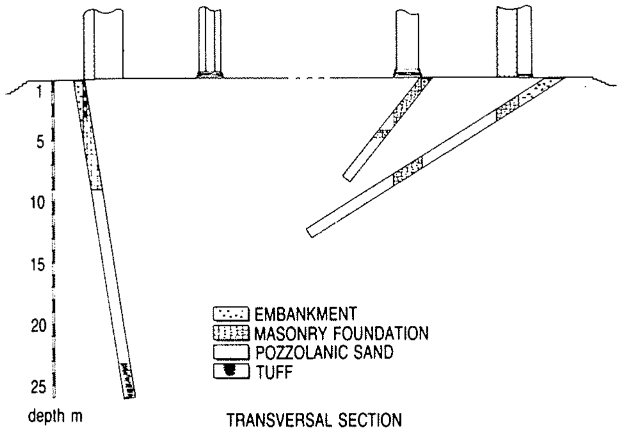

Figure 3. Vertical section with the indication of the stratigraphy determined by the drillings.

4.2 Masonry and arches

Alternating layers of basaltina (black) and travertine (white) were used for the external side of the perimetral walls but only partially for the internal one. Only columns, pillars and the walls below the arches are completely stone covered. The internal wall of the facade is covered with regularly squared stone from the floor to the open gallery; the side walls are covered with stone, but only to a height of 1.6 m. The other surfaces are made with tufa blocks plastered and painted in stripes simulating curtains of stones.

4.3 Columns and pillars

The columns, 13.5 m high with diameter 1.6 m, are formed externally by alternating layers of basaltina and travertine and

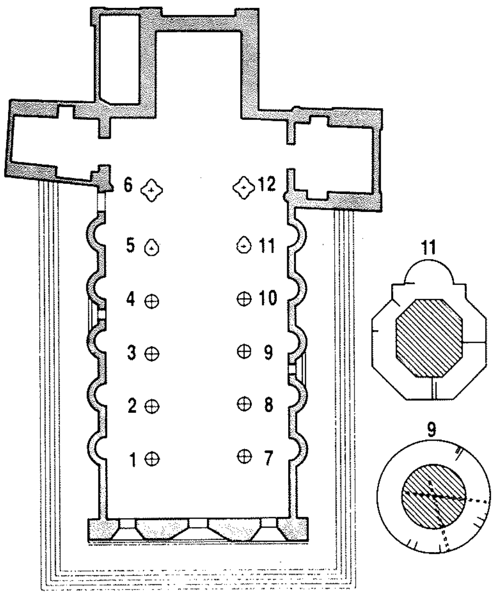

Figure 4. Plan of the Cathedral. Typical sections of the pillars and columns are shown with the indication of the observed cracks.

internally by a masonry composed of small stones. The stones are 26.5 cm high and about 36 cm thick. The masonry of the inner part is formed by irregular stones of tuff, basaltina and travertine of decimetric sizes layed with a great quantity of mortar. This structural typology was investigated by means of two boreholes drilled from the surface of the column 9 (Fig. 4). Some voids were found in the inner part of the column. An overall view of the Cathedral is shown in Fig. 5.

5 FAILURES DESCRIPTION

A series of cracks in the columns unexpectedly appeared in 1619; in the following year the most damaged columns were hooped. The cracks are subvertical and are concentrated in the upper and lower part of the columns. A systematic survey of the cracks was carried out at all the columns and the pillars starting from the floor up to the height of 2 m and position, opening and length of each crack were recorded.

The failures cross one or more levels of stones generally passing through the vertical joints between two adiacent stones. The frequency of cracks is greater in the basaltic stones in comparison to that in the travertine ones. The only undamaged elements are the optagonal pillar and the polystyle pillar of the left aisle. Fig. 4 illustrates the planimetric map of the Cathedral and a graphic presentation of the cracks in two columns.

The survey carried out on the balcony above the arches and surrounding the perimeter of the central aisle evidenced two failures at the contact between the facade and the side walls overhanging the colonnade. Only small cracks are observed on the sidewalls at the contact between the stone blocks and tufa masonry at the contour of the windows.

6 LABORATORY TESTS

The mechanical and physical characterization of the different types of masonry structures and of the subsoil was carried out by means of laboratory tests. The laboratory tests were carried out on specimens of pozzolana, tuff, masonry foundation, travertine, basaltina and mixed material travertine-basaltina. The mixed specimens were composed of two alternating cylinders of basaltina and travertine. The following determinations were carried out:

- bulk density
- sonic velocity
- uniaxial compressive strength and deformability
- brasilian test.

The results of laboratory tests are reported in Table 1.

Table 1. Results of laboratory tests.

	pozzolanic sand	tuff	masonry foundation	travertine	basaltina	mixed traver. + basal.	masonry inside the column
Bulk density [t/m^3]	1.35	1.15	1.20	2.34	2.17	2.22	1.20
Sonic velocity [m/s]	1495	1735	1710	4750	4350	2290	–
Young Modulus [MPa]	500	2000	1450	47500	37500	14000	800
Poisson secant ratio	0.33	0.27	0.30	0.27	0.23	0.25	0.25
Uniaxial compression [MPa]	–	–	36	68	80	85	
Brasilian tensile strength [MPa]				9.7	8.4		

7 NON DESTRUCTIVE IN SITU TESTS FOR THE MECHANICAL CHARACTERIZATION OF MASONRY STRUCTURE

The determination of the parameters necessary to evaluate the static

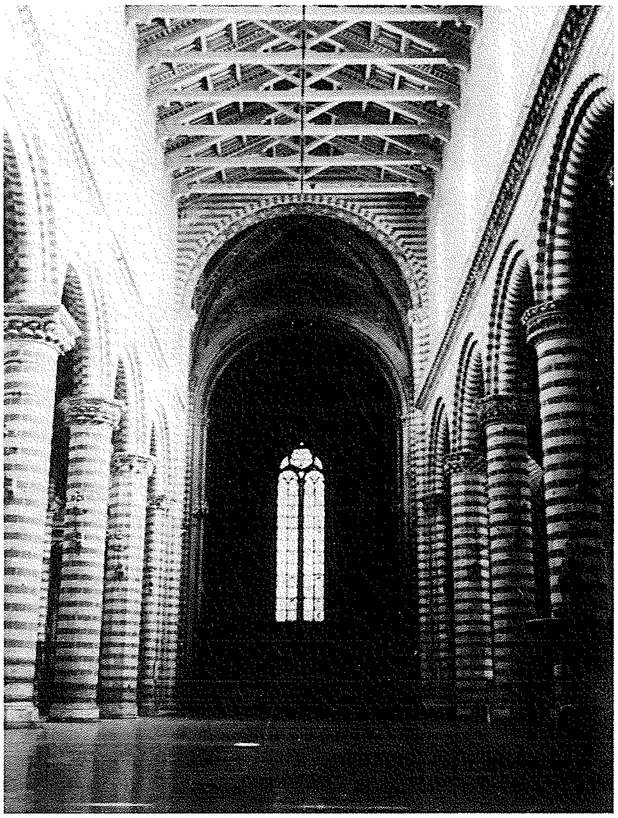

Figure 5 Overall view inside the Cathedral.

conditions of the Cathedral was carried out by means of a special "non destructive" test based on the use of thin flat-jacks inserted into the mortar layers.

This testing technique, which was developed some years ago by ISMES, has been applied in recent years for the study of more than 30 monuments with brick and stone masonries. This technique has been set up in order to give reliable information concerning the following parameters:

- measure of the state of stress
- determination of deformability and strength characteristics

The test is devided into two separate phases. At first, one flat-jack is used to measure the state of stress and, in the second phase, a second jack is inserted in order to determine the deformability modulus and the compressive strength of the masonry.

7.1 Description of the flat-jack testing technique

a) Measurement of the state of stress

The determination of the state of stress

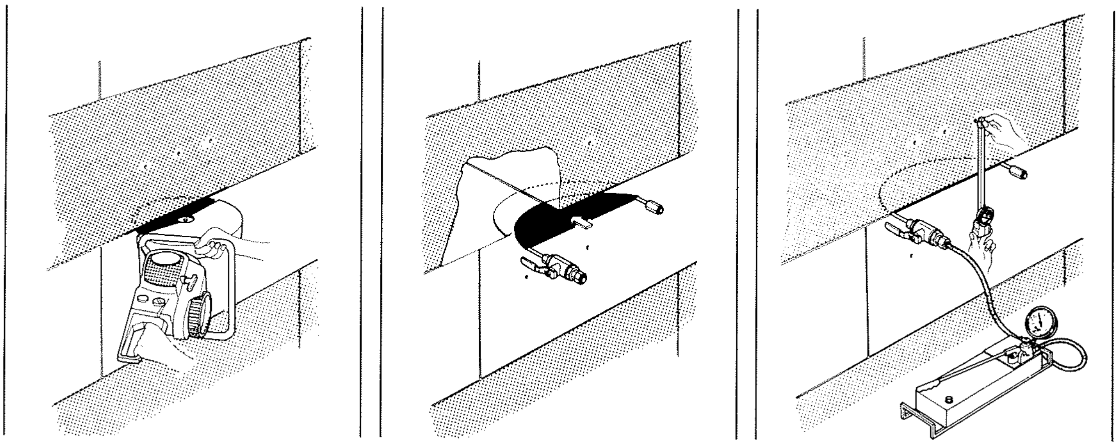

Figure 6. Flat-jack testing phases.

is based on the stress release caused by a plane cutting, perpendicular to the surface of the wall. Two reference points are installed on the wall surface and the initial distance (di) between the two points is measured. A cut perpendicular to the wall surface is then made and the stress release determines a partial closing of the cutting, the distance (d) after the cutting being d<di. A thin flat-jack is set inside the cutting, and the pressure is gradually increased up to cancel the previously measured convergency. In this condition, the pressure p inside the jack is equal to the pre-existing state of stress in a direction perpendicular to the plane of the cutting. The values obtained must by corrected by a coefficient which depends on the ratio between the flat-jack surface and the cutting surface and on the rigidity of the welded boundary. The value of the state of stress (σ)in the testing point is given by: $\sigma = p.Km.Ka$
were: p is the oil pressure, Km is the jack constant which must be determined by means of laboratory calibration and Ka=Aj/Ac is the ratio between the surface of the jack and the surface of the cutting.

In brick or stone masonries with mortar layers thickness higher than 10 mm, the plane cutting can easily be made by means of overlapping holes made by simple hand tools. In this case rectangular flat-jacks are used. In a stone masonry with very thin mortar layers (less than 10 mm) the cut is made by means of a steel plate with diamond tools and the jack has the same shape of the cutting (circular segment with length 32 cm, height 12 cm and thickness 4 mm). Fig. 6 shows the different testing phases on a stone masonry.

b) Determination of deformability and strength characteristics

In homogeneous isotropic material, the test above described can also be used to determine the deformability characteristics. In the case of a masonry, which is a highly anisotropic material, it is advisable to introduce some changes in the testing technique. For this purpose, a second cutting is made, parallel to the first one, and a second jack is set in it at a distance of about 50 cm from the other jack. The two jacks delimit, therefore, a masonry sample of appreciable size to which they apply a uniaxial compression stress (Fig. 7).

Several measurement bases for removable mechanical strain-gauge, installed on the sample free face, make it possible to obtain a complete picture of axial and transversal deformation of the sample. In this way a uniaxial compression tests is carried out on an undisturbed sample of large size which is certainly representative of the behaviour of the structure as a whole. Several loading cycles are carried out at gradually increasing stress levels in order to determine the deformability modulus of the masonry in loading and unloading phases.

The loading test with two flat-jacks can also be used to evaluate the compressive strength of the masonry. For this purpose the load is increased until the first cracks in the brick appear, and the strength limit of the masonry can be estimated to a very good approximation by extrapolating the stress-strain curve. The effect of lateral confinement of the specimen may by taken into account by means of calibration tests carried out in the laboratory. It must be noted that, although nearing failure conditions some cracks appear in the bricks, the damage suffered by the masonry is quite negligible and the restoration is easy.

The reliability of the flat-jack test was evaluated by means of a wide range of calibration tests carried out at ISMES laboratory in collaboration with ENEL's Research Dpt.

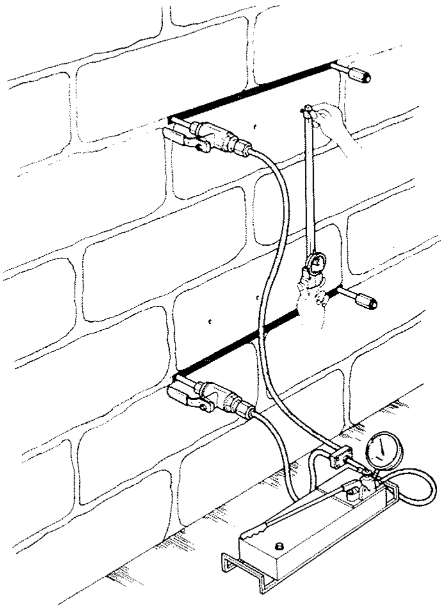

Figure 7. Deformability test with two flat-jacks.

7.2 Results of the tests carried out on the masonry structures of the Cathedral of Orvieto

a) Stress analysis

Flat-jack tests were carried out in 17 points of the perimetral walls using the circular shaped jacks shown in Fig 6 and in 17 points of the pillars or columns using smaller jacks (12x12 cm and 24x12 cm). The testing points on walls and pillars were chosen in order to check eventual load eccentricity. Fig 8 shows the plan of the Cathedral with the indication of the testing points and the stress values measured by the flat-jack tests.

As regard the two lateral walls, a quite sÿmmetrical load eccentricity, with an average stress value of 0.3 MPa on the internal surface and 1.1 MPa on the external one, was observed. This load eccentricity seems to be ascribed to the structural characteristics of the two faces of the lateral walls. In fact the external face is made by alternating layers of basaltina and travertine while the internal one is composed of tufa stones (see paragraph 4.2) therefore its deformability is much higher than the former one.

At the base of the columns 1, 3, 7, 9 a constant load eccentricity was measured with an average value of 2.7 MPa on the side towards the main aisle amd 5.5 MPa on the opposite side. Particular attention must be devoted to the pillar 11 which is involved by some deep cracks shown in the section of Fig 4. The flat-jack test carried out on the portion between the two cracks (see Fig 9) seems to indicate that this zone is completely destressed. This reduction of the section of the pillar induces a considerable increase of the value of the maximum compressive stress (6.9 MPa) on the apposite side. Uniform stress distribution has been observed on the two pillars (6, 12) near the transepth with an average value of 4.2 MPa.

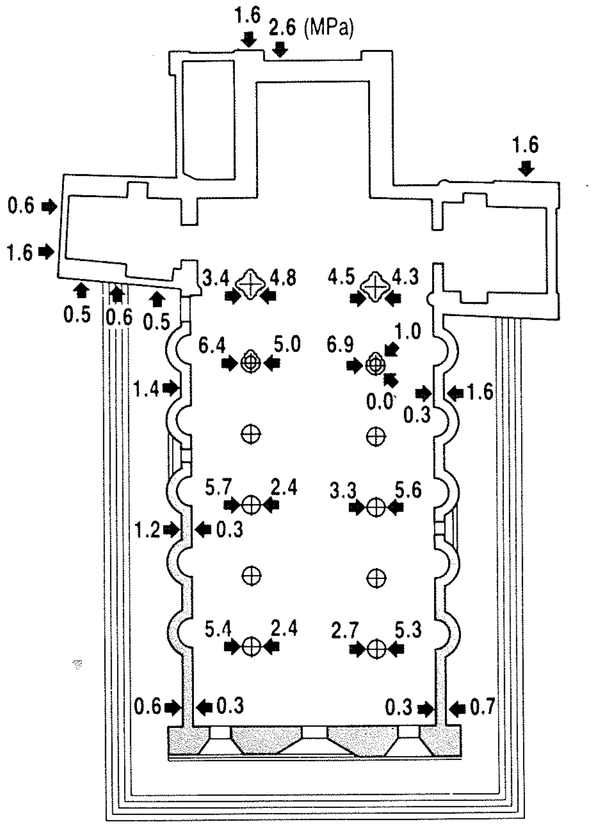

Figure 8. Stress values measured by flat-jack test on perimetral walls and columns.

b) Determination of deformability characteristics

The tests with one flat-jack on the perimetral wall were also used to evaluate the deformability modulus of the masonry composed by alternating layers of basaltina and travertine. The very thin mortar layers of this kind of masonry justify the assumption of homogeneity and isotrophy which are required for this calculation. The average value of deformability modulus evaluated by flat-jack test on this type of masonry was:

$E_1 = 13000$ MPa.

The deformability of the foundation masonry (made by tufa stone) under perimetral walls and pillars, was on the contrary determined by means of two parallel flat-jacks. This test was carried out on the foundation masonry of the pillar 11. The deformability modulus, determined for a stress range 0-12 MPa, was: $E_2 = 1800$ MPa. The stress-strain diagram of this test shows a linear elastic behaviour of the masonry up to a stress level of 1.2 MPa. For higher stress levels a plastic behaviour is observed.

The same technique was used to determine the deformability of the pozzolanic material which represents the foundation of the masonry structures. Two tests were carried out using small caves excavated near the Cathedral (Fig10). The following value of deformability modulus was obtained:

$E_3 = 500$ MPa

The mechanical parameters determined by flat-jack tests were used as input data in the FEM model described at paragraph 10.

8. TOPOGRAPHY SURVEY AND CONTROL MEASUREMENTS

8.1 Plan of measurements

The measurements carried out for checking movements of the Cathedral are as follows:

- high precision levelling grid;
- reciprocal trigonometric levelling;
- measurements of verticality of the columns;
- control measurements of the main cracks of the columns.

The main geometrical levelling line is about 2400 m and is formed by 12 reference bases (1-10, A and B of Fig 2). A direct connection line between the reference points 5 and 9 represents a transversal stiffening of the main levelling line. The considerable difference of level (about 39 m) between the Cathedral square and the reference point (A) placed in the proximity of the famous Saint Patrick well requires 70 levelling measurements to complete the levelling line.

A second high precision geometrical levelling line was planned in order to check some structural elements of the Cathedral (4 points on the columns amd 4 points on the external walls).

The reciprocal trigonometric levelling was effected in order to check the cliff movements relating to the surrounding plane. The trigonometric levelling

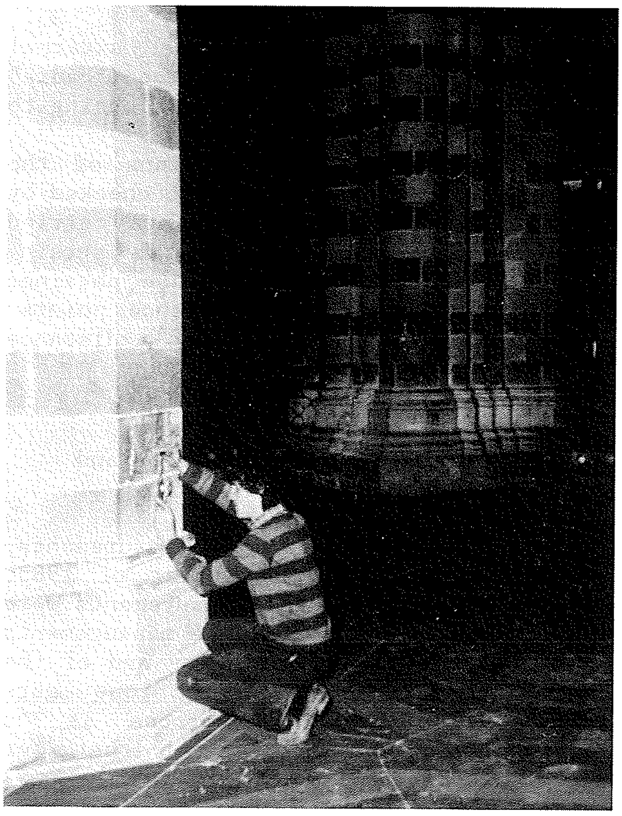

Figure 9. View of a flat-jack test carried out on pillar 11.

Figure 10. Determination of deformability characteristics of pozzolanic layer by means of flat-jack testing technique.

connections are between the external reference point placed on the "Badia" tower and two points A and B, belonging to the main levelling.

Finally the opening variations of the main cracks on the columns were checked by means of periodical measurements carried out by a removable mechanical strain-gauge. The measuring bases were obtained by cementing two steel reference points, at both sides of the crack, at a distance of 20 cm.

8.2 Results of topographic survey and control measurements

The first set of reference measurements, was carried out in the spring of 1985. Subsequently, periodical measurements were executed every six month. The measurements of the column perpendicularity and of the crack opening were carried out more frequently. The final purpose is to continue these measurements, at least, for a ten year period.

The geometrical levelling has not evidenced, at the moment, progressive deformation processes. Closure errors have been limited in each set of measurements. The measurement error has always been lower than 0.7 mm for the historical center line, and lower than 0.3 mm for the ring surrounding the Cathedral. The grids are rigorously compensated. Every reference point located on the Cathedral structure denotes stability conditions with quota variations less than ± 0.1 mm. All the reference points belonging to the main levelling indicate quota variations lower than the average kilometric standard deviation (± 1.4 mm). This shows that relative movements in the area surrounding the Cathedral are negligible.

The quota variations between the reference points A,B and C are lower than the precision limit of this type of measurement which is about ± 10 mm depending on the grid measurement adopted and the distance (1000 m) between the reference points.

The perpendicularity control measurement carried out on 4 columns, showed horizontal displacements lower than 1 mm and consequently of the same magnitude as the instrument precision.

The main cracks, belonging to columns 1, 2, 3, 8, 9, 11 and 12 have been checked by means of a set of measurements from June '86 up to now. The diagram of Fig 11 shows, for each set of measurements, the length variation of measuring bases versus time. The maximum variation of the opening

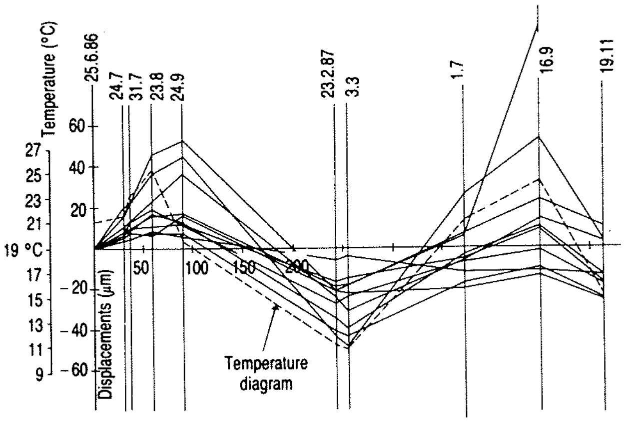

Figure 11. Control measurements of the main cracks. Diagrams of deformations versus time.

of the cracks was less than 0.1 mm mainly depending on the thermal variations. At the end of the first year of measurement, in fact, the displacement variations return to zero values with a difference of about 0.02 mm. The period of measurements taken up to now is about two years. This period is too short to formulate any reliable hypothesis on the evolution of deformation processes.

9 NUMERICAL MODELLING

A Finite Element model has been set up with the purpose of analyzing the statical behaviour of the structural part formed by column, masonry foundation, pozzolanic sand and tuff. The input data for the FEM model are the geometrical and mechanical features of the structural elements and of the geological formations. The results of the experimental measurements utilized in the procedure of calibration and control of the model by means of a statical back analysis simulation are: the measurements of the state of stress in the columns, and the load eccentricities. The geometrical features of the structural elements and of the geological sequences have been obtained from the drillings in the columns and in the subsoil.

Deformability and strength parameters of the finite elements forming the columns, the masonry foundation and the subsoil were determined by means of laboratory and in situ tests. The constitutive law adopted to describe the stress-strain relation for all the materials is elastic perfectly plastic. The yielding criterion which defines the limit between elastic

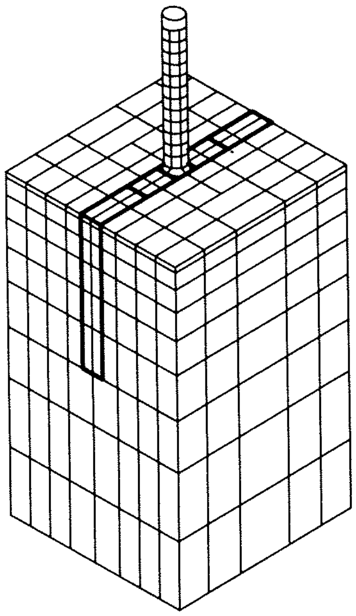

Figure 12. Three-dimensional Finite Element mesh.

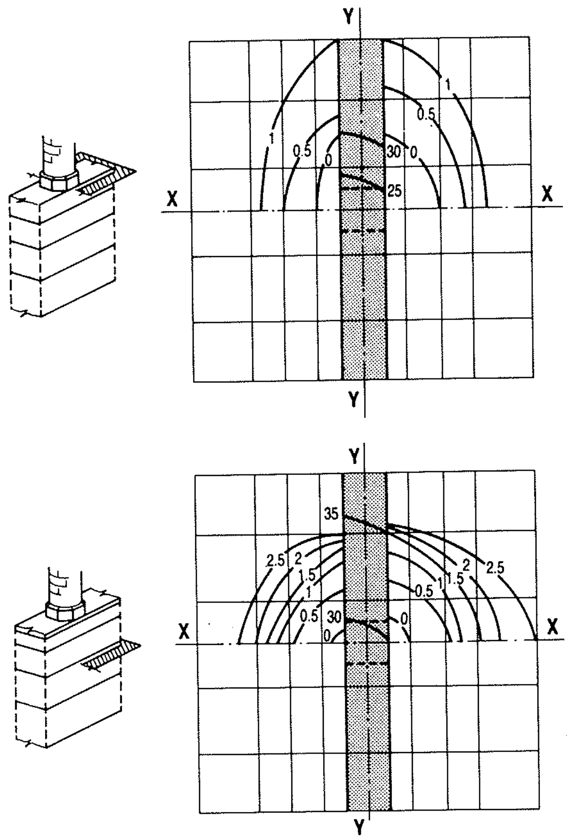

Figure 13. Stress isocurves in the subsoil determined by FEM model (10^{-1} MPa).

and plastic phase is taken from the Druker-Prager condition.

All the FEM analysis were carried out in the conservative hypothesis of no-tension resistence of all materials.

In Fig 12 the FEM model mesh reproducing a standard column, the masonry foundation, the pozzolanic subsoil and the tufa formation (down to depth of 28 m) is shown. The number of nodal points of the model is 1017, the number of elements, hexahedral subparametric, is 614.

The FEM analysis was carried out assigning to the columns, masonry and pozzolanic soil surrounding the wall foundation, their own weight and subsequently applying to the top of the capital four different loading configurations. The FEM analysis which best fits analytical and experimental data is that which corresponds to a vertical trapezoidal loading configuration acting on the capital with extreme values of the pressure equal to 1.1 MPa and 6.0 MPa.

The FEM results are nodal displacements, stress and yield function values in the integration points of the elements. The yield function can be assimilated to the difference between deviatoric stress mobilized and deviatoric stress available.

Fig. 13 represents for the foundation masonry and pozzolanic sand the yield isocurves (MPa).

Fig. 14 reports the plasticized areas and the vertical stress values (MPa) in different sections of the column.

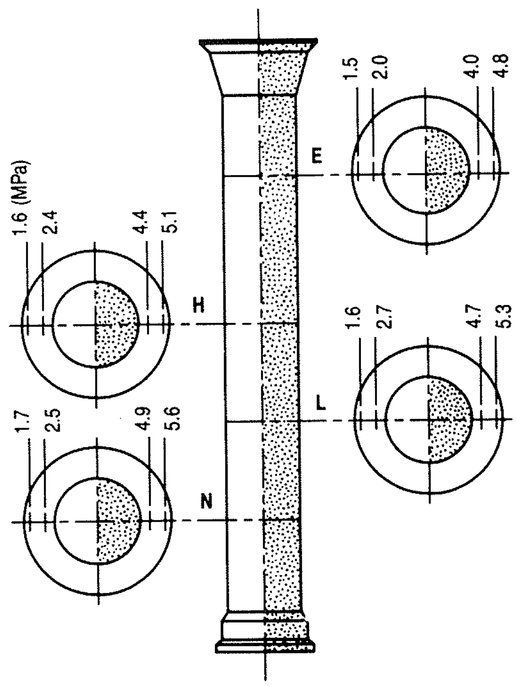

Figure 14. Plasticized zones in the column with the indication of the vertical stress values determined by FEM model.

10 RESULTS AND CONCLUDING REMARKS

The results of the FEM analysis put into evidence the completely satisfactory static conditions of the masonry foundation which seems to be very well designed. The resulting subsoil plasticity zones (Fig 12) are limited, extending down to less than half of the masonry depth while masonry and other parts of the subsoil are remaining in the elastic field.

The inner part of the columns, formed by small stones, resulted partially plasticized. These results are in agreement with the hypothesis assumed in analytical loading analysis were only the external ring of stones of the column is actively resistant.

Fig. 15 report a standard vertical section of the Cathedral at the right aisle. This figure also reports stress values computed and measured at el. 1.6 m from the floor. The measured stress values reported in Fig 15 are the averages values of the stresses measured by flat-jack test on the columns n. 1, 3, 7, 9; the computed ones refer to the loading analysis and FEM computations. It can be observed that the experimental and computed stresses are in a good agreement. The stress values are conclusively lower than the strength characteristics of the stones and the loading eccentricities are in the range foreseen by the constructive typology.

Pillar 11 on the right aisle is almost unloaded on the right part of its section; two subvertical relevant failures delimite an unloaded part as shown by experimental measurements. This results confirm the need of urgent consolidation works on this pillars.

The development of future studies foresees the repetition of topographic and control measurements, as well as a set of geophysical measurements. These measurements will be mainly devoted to columns and pillars in order to determine the real section of resistant elements.

Experimental investigations and numerical models are in progress with the purpose of determining the specific origin of cracks by means of analysis devoted to the determination of tensile zones induced by different deformability ratios between basaltina, travertine and mortar and by means of the determination of the stress concentration effect produced by non-homogeneous contact between stones. Finally, physical and numerical models of damaged columns will be prepared in order to check the effectiveness of the consolidation works.

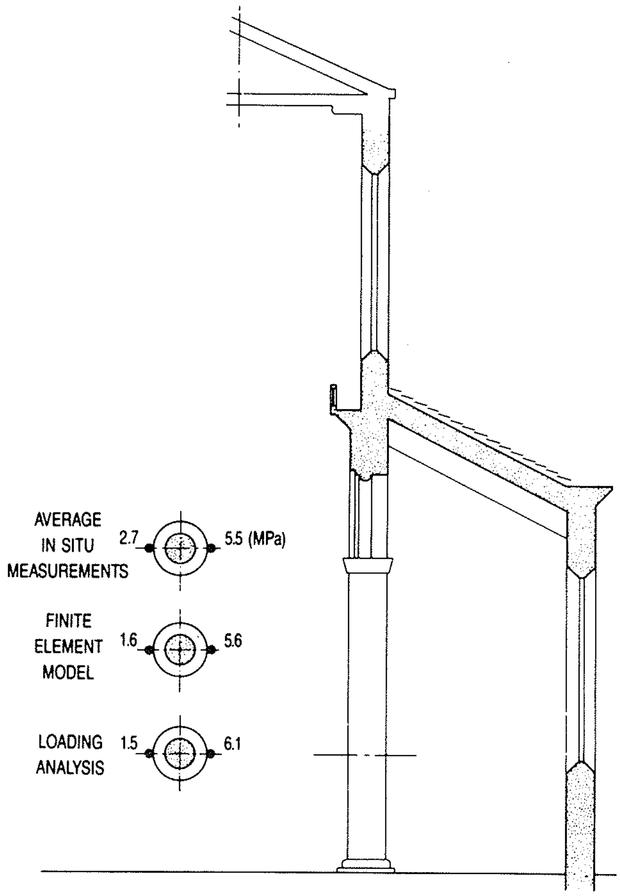

Figure 15. Standard vertical section with the comparison between measured and computed stress values on a column.

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