G.P. Giani (*), O. Del Greco (*),
B.Astori (**), P.P.Rossi (***)

Analysis and control of the static behaviour of the Consolazione Church in Todi, Italy

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(*) Department of "Georisorse e Territorio", Technical University of Turin, Italy
(**) Institute of Topography, Technical University of Milan, Italy
(***) Ismes, Bergamo, Italy

ismes s.p.a.
viale Giulio Cesare, 29 - 24100 BERGAMO - tel. 035/358111 - telex 301249 - BG

ABSTRACT: The stability of Santa Maria della Consolazione Renaissance church in Todi has suffered from cracks and damages which have continously recurred in the apse and structure foundation since the end of construction. The church rises from a square on the southern slope of the Todi hill.
A supported study on the static behaviour of the church is now in progress.
Because the damage does not only depend on the yielding of structural elements, but also on differential settlements and slope movements, a geothecnical study is now in progress. In situ measurements, laboratory tests and a 3-D finite element method (FEM) stress analysis of the masonry foundations and subsoil of the church are the main preliminary results reported in this paper.

RESUME: Dans l'église de la Renaissance de Santa Maria della Consolazione à Todi des fentes et des dommages se sont manifestés dès la fin des travaux de bâtiment dans l'abside et dans les fondations. L'église s'élève dans une petit place sur la pente méridionale de la colline de Todi.
Un'etude sur le comportement statique de l'église est actuellement en cours. Puisque les dommages ne dépendent seulement du rupture des éléments structuraux, mais aussi de mouvement différentiel des terrains des fondations, un'étude géotechnique est aussi en cours. La mémoire illustre les mesures "in situ", les essais au laboratoire et l'analyse des contraintes dans les fondations en maçonnerie et dans le sous-sol de l'église avec la méthode des élèments finis.

1. HISTORICAL INTRODUCTION AND STUDY PURPOSES

The construction of the Santa Maria della Consolazione (figure 1) church in Todi is due to the faith of the people in an image of the Virgin Mary. This image was painted on a road wall which skirted the Todi hill, where the church was erected.

The church construction began in the year 1508 and was finished in 1606.

The church is attributed to the architect Bramante and is in the purest Renaissance style in the form of the Greek cross, the aisles simplified in apses leaning out of the side of the square and the dome resting on the square (Bruschi A., Nofrini U., 1970). From 1600 up to the present day, the structure stability has continuously suffered from failures due to differential settlements of the soil foundation and from yielding of structural elements such as truss rods or tie irons.

In 1638 the tie iron, which contained the arch of the eastern aisle was broken; this tie iron was again broken twice during the next two centuries.

Some reference points were installed in 1670 in order to measure sliding movements between the embankment and the natural slope; a part of the church, in fact, is placed on an embankment constructed with the purpose of enlarging the surrounding

Fig. 1 - View of the church.

square. A gravity semicircular wall was constructed between 1836 and 1860 in order to substain this embankment; previous instability and washing away phenomena resulted in the undermining of the soutern apse and the discovery of foundations and sub foundations which had been constructed before 1792.

In 1926 consolidation works were carried out on the eastern apse and a sub foundation was constructed below the southern pillar, as a consequence of which a large crack appeared in the eastern tribune.

A technical committee was convened in 1953 with the purpose of examining and suggesting possible remedies of the instability phenomena which had caused large cracks along the half dome median of the eastern apse and damages to the perimetral walls.

Engineer Mastrodicasa revealed and documented these damages and cracks (Mastrodicasa S., 1943).

Consolidation works carried out in 1953 were mainly concerned with the injection of large quantities of mortar into the soil foundations.

This paper reports some preliminary results of a study supported by the "Sovrintendenza ai beni culturali, ambientali, architettonici, artistici e storici" of Perugia and the Technical University of Turin, which operates in cooperation with ISMES of Bergamo.

The main purposes of this study are:

1) a characterization of the existing damages and the damages which were previously documented and revealed ,in order to define damages which depend on the yielding of structural elements and damages which depend on soil differential settlements or slope movements;

2) the control of the structural elements and existing cracks by means of experimental measurements;

3) a loading analysis of the structure in agreement with experimental measurement and analytical computations;

4) a geotechnical characterization of the subsoil;

5) a stress-strain FEM analysis which, on the basis of the results obtained from points 3-4, is able to describe the stress history and to contribute a starting point for future examinations of a possible consolidation project.

One of the most important arguments for the stability control of the church which has not yet been examined and which will be carefully studied in this work is the slope stability on which the church is found.

This southern slope, the least steep of the Todi hill, was mainly involved in superficial instability phenomena. A deeper landslide, which originated at the slope toe from the erosion of the Naia flood, has already been documented, even though the sliding movements took place a certain distance from the church.

The northern slope, on the contrary, has been involved with different types and sizes of movements. Soil formations have been characterized and stability analysis have been carried out for these problems (Calabresi et al., 1980).

2. GEOLOGICAL SITE DESCRIPTION

The town of Todi is situated on a hill at the height of about 350 m a.s.l. The left bank of the river Tevere skirts the hill. The hill is made up of sedimentary formation, with a fluvial-lacustrine origin and plio-pleistocenic age, lying on a compact rock basement formed by alternating sandstone and "Villafranchiane" marl layers (figure 2) (Pialli G., Sabatini E., 1969).

The entire hill mass is a complex formation, with a large heterogeneity along the vertical direction. The upper part of the hill, which includes the church, is of a large hetereogeneity in the horizontal direction.

The structure can be idealized by subdividing it into three structural complexes:

- the lower complex is mainly formed by overconsolidated silty-sandy-clay, which has a grayish-bluish colour and thickness of about 150 m, few degree plunging to the NE direction;
- the intermediate complex is formed by sandy and clay oversoncolidated silts, which have a variable colour from gray to yellow-; a large number of gravel intercalations are here present. The layer is about 60 m thick with a sligtly dip in the eastern direction;
- the upper complex is the most heterogeneous, being mainly characterized from dense sands and gravel conglomerates which are alternated with typical intermediate complex layers.

The lower complex always shows a good consistence and regular stratification, while the upper and intermediate complexes often show irregular stratifications and recurrent discontinuities. These large morfological modifications are due to superficial actions, alterations and landslides on the upper part of the slopes.

A well defined interpretation of the complex formations below the "Santa Maria della Consolazione" church has been possible thanks to the stratigraphics data of 8 boreholes carried out in 1953 on points equal in distance to the church perimeter and of 3 new boreholes.

New boreholes have also been carried out for this study with the purpose of gathering undisturbed specimens for laboratory tests and geothecnical characterizations of the different soil formations.

The material stratigraphy, directly underlying the church can be defined as follows, even though, as reported above, some irreguliarlity can be found:

- filling layer and sandstone (2-6 m);
- yellow-ochrous clay-sand and yellow compact clay (4-9 m);

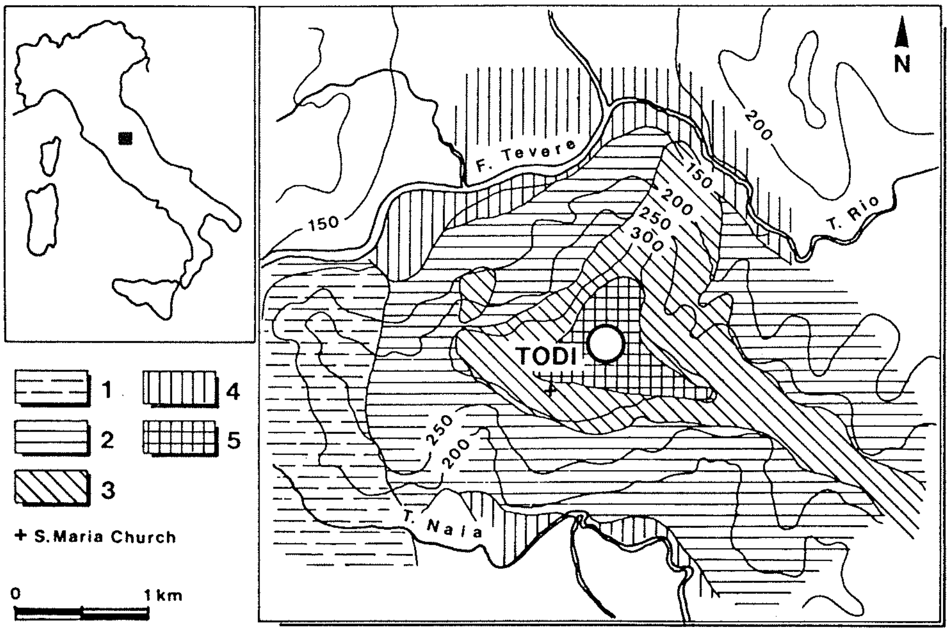

Fig.2 - Geological scheme of the Todi hill: 1) compact basal complex; 2) lower complex; 3) intermediate complex; 4) recent alluvials; 5) upper complex.

- plastic sandy-clay (1-5 m);
- coarse alluvial material with sandy-clay alteration (5-10 m);
- medium compact bluish clay with fine grain sand (at a depth of between 15 and 23 m from the ground level).

The hydrogeological situation of the hill reflects the complexity of the geological structure. Generally the upper complex is more permeable; the underground water contribution depends on the hill morfology and arrives exclusively from the meteoric infiltration on a rather small area.

Some seasonal springs confirm the irregular feature of the underground water circulation which preferentially follows the more permeable intercalations found in the clay-silts. The infiltration paths are rather short.

3. STRUCTURE DESCRIPTION AND LOADING ANALYSIS

The church rises on a large square which is constructed partly by excavation, partly by filling.

A gravity wall supports the filling materials of the southern part of the square.

The church, 50 m high, has 3 poligonal apses and 1 circular apse, the cross side is 43 m long.

The wall apses are 2 m thick, while the pillars rest on an area of about 25 m^2.

The wall foundations are only enlarged to about 20 centimeters from the wall apses and are 2 m deep.

The pillar foundations are 4 m deep.

Consolidation works, carried out subsequently to the construction, depened, as reported in the paragraph 1, the pillar foundation and a part of the northern and eastern apses with a subfoundation to a depth of 16.5 m.

The loading analysis was carried out by computing the weights lying on the pillars and on the wall apses.

The whole weight computed for the church is 17000 tons.

The average vertical stresses obtained at the base of the pillars and of the wall apses are 1 MPa and 0.46 MPa respectively.

The pillar and wall structures are formed externally in limestone block masonry and internally in an irregular small stones cemented with mortar. The external area of the masonry has been evaluated as 40 percent of the total area.

In the hypotesis in which only the external masonry is able to support the acting loads, the vertical stresses at the base of the pillars and wall apses become 2.5 MPa and 1.15 MPa respectivey.

4. TOPOGRAPHIC AND CONTROL MEASUREMENT PLAN AND RESULTS

A topographic grid has been planned, in order to control possible church movements and strains, by checking the only altimetric displacement of 14 points. These points have been located on the monument or in the surrounding area, on the southern slope at a maximum distance of 300 m from the church.

In the study continuation, a global topografic plan, which allows one to control absolute plane-altimetric displacements, including reference points located in zones considered as stable, by means of triangulations and levellings, will be set up.

The 14 examined points (figure 3) are:
- 4 reference points located inside the church, on the pillar sides;
- 4 reference points located outside the church, on the wall apses;
- 3 reference points located on the small wall delimiting the church square on the northern side;
- 2 reference points located on the ground level, the former in front to the eastern portal of the church, the latter in the area surrounding the semicircular southern gravity wall;
- 1 reference point located on the ground level at a distance of about 300 m from the road which joins the historical center to the church (point 1).

All the reference points, appropriately protected, are joined with a height precision geometric levelling ring.

External reference point 1 has been considered as conventional zero.

The measurement of the 4 pillar perpendicularity variations has been carried

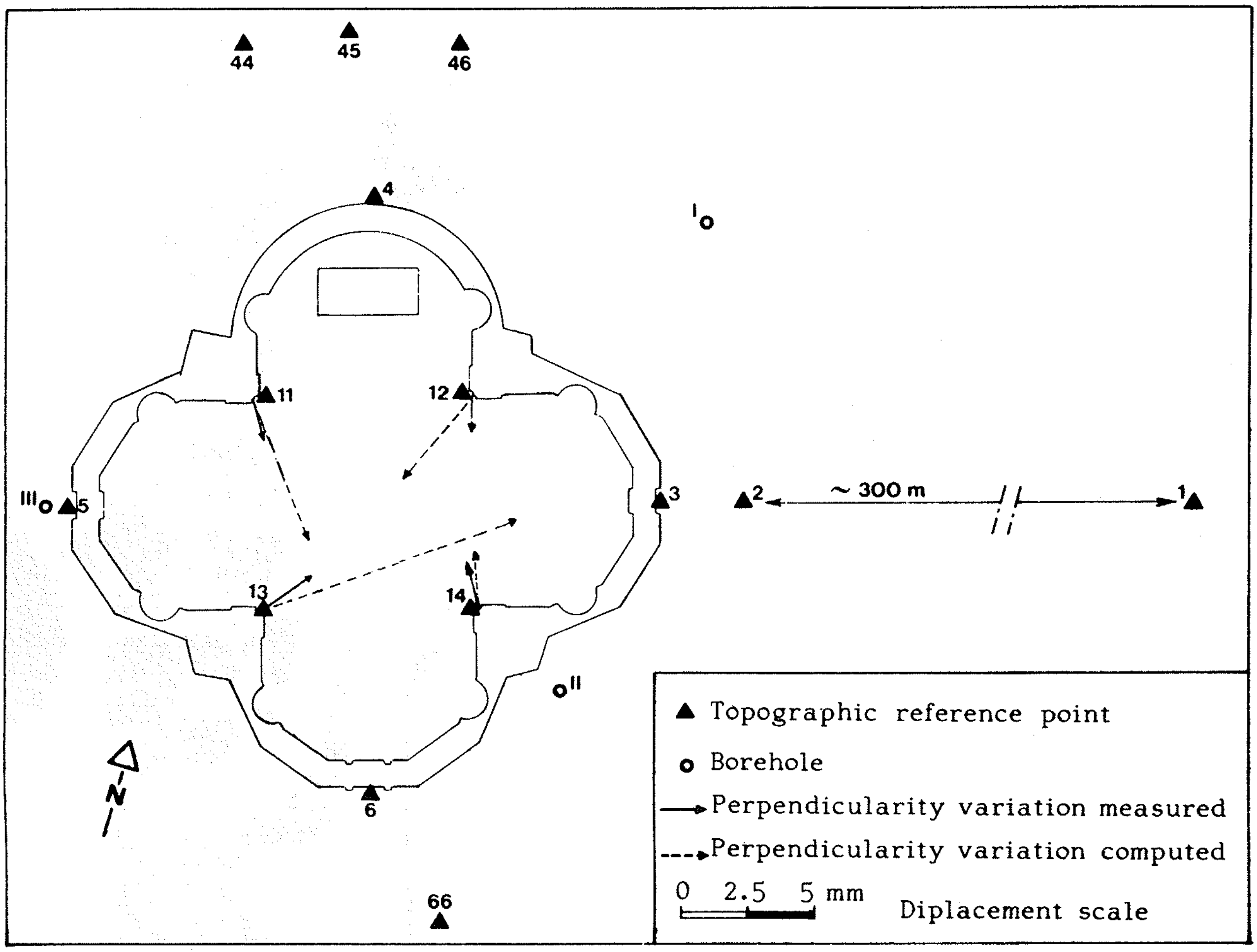

Fig. 3 - Measurements points on the church map.

out by utilizing already existing brackets which were installed by a control committee in 1953, at a height of 12.7 m, for plumb line measurements (with damped swingings). The measurement sensitivity has been determined as +- 0.2 mm.

Finally a millesimal extensimetric gauge has been used in order to check the size variations of crack in the Bramante dome. The measurement consists in determination of the length variation determination between two reference points and of the corresponding temperature values.

The topographic measurements have been carried out 8 times over a period of more than two years, by evidentiating the following phenomena:

a) All the examined reference points, except the three located on the northern wall of the square (n. 44, 45, 46) have been subjected to quota variations varying with in a range of +- 2 mm.

The 8 reference points placed on the masonry structure (n. 3, 4, 5, 6 and 11, 12, 13, 14) have particularly shown a uniform behaviour reaching, with progressive quota variations with in the range of +- 0.4 mm, complessive values of about 0.2 mm confirming a slow relative settlement of the zone (including reference point 1). The lack of a reference point located in a reliable stable zone does not allow one to determine the absolute displacements.

The two reference points located on the ground level in the southern and northern part of the square (n.2 and 66) have been subjected to the same progressive movement, even though the quota variation of the 8 measurements were of about +- 1 mm.

The three reference points 44, 45 and 46 have instead shown an anomalous behaviour. The quota variations have been subjected to homogeneous variations which derived from seasonal changes and in particular from dry to wet seasons. The quota variations varied in range between -9 mm and +13 mm showing seasonal excursions larger than 2 cm.

The study prosecution foresees new piezometric and inclinometric measurements in order to control and update the available measurements carried out, in previous studies in the last years. In particular, the new forecast plano-alimetric triangulation will be connected to a new inclinometer cup and to a reliable fixed point in a compact formation outcrop located to a 500 m distance from the examined area.

The perpendicularity pillar variation measurement has allowed one to determine the 4 pillar variations in the period between 1953 and 1985 and in the period between 1985 to the present day.

The perpendicularity pillar variation measurement carried out by the 1953 control commitee has been utilized as zero reference values. A further eight measurements have been carried out from the beginning of this study in 1985 up to the present day.

All the measurements of the perpendicularity variation increment are systematically concordant and convergent to the church square center. Figure 3 shows (in the same scale and in vectorial form) the thirty year perpendicularity variation and the complessive following two years perpendicularity variations for the 4 pillars.

b) The crack size variation measurements carried out with millesimal comparator in the dome have shown excursion variations lower than 0.1 mm, up to the present time.

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

The flat jack technique has been employed in order to determine the state of stress in the pillars and in the wall apses of the church (Rossi P.P., 1982, 1985 - Binda et al., 1983). The deformability characteristics of the examined structural elements are also determined by interpreting the test for the state of stress. The testing technique has also been reported

Fig.4 - Flat-jack test carried out on a pillar.

in a paper presented at this Conference (Astori et al., 1988).

18 flat jack tests were carried out for the stress determination on the pillar and wall masonry at a height of about 1.5 m inside the church and at an elevation of about 2.0 m on the outside of the church (figure 4).

Figure 5 reports the location of the testing points and the corresponding determined vertical stress values on the church map. The range obtained for the most frequent values of the deformability moduli of the masonry varies between 22000 MPa and 28000 MPa.

The stresses obtained from the flat jack technique tests are averagely in good agreement with the results of the loading analysis. The average value obtained for the vertical stress of the pillars is in fact 2.39 MPa. The average value for the

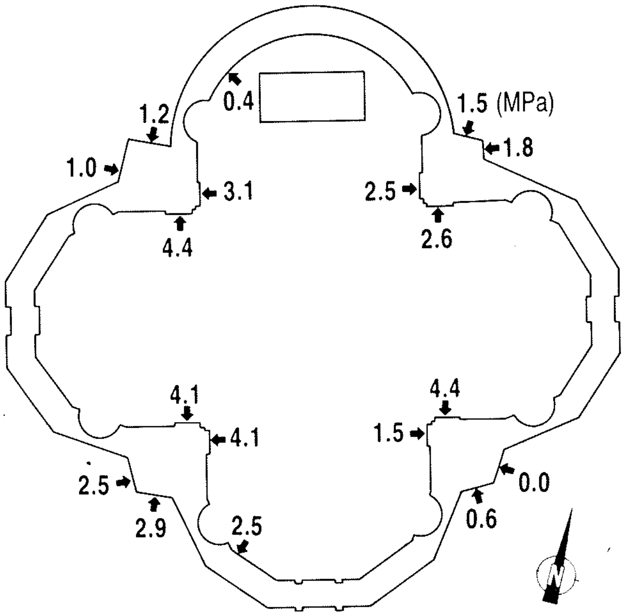

Fig. 5 - Stress values measured by flat-jack on wall apses and pillars.

vertical stress of the wall apses (only the results of two tests are available) is 1.43 MPa. These values are similar to the values obtained from the loading analysis (2.5 MPa in the pillar and 1.15 in the wall apses).

6. LABORATORY TESTS AND GEOTECHNICAL CHARACTERIZATION

Three boreholes have been carried out, as reported in paragraph 2, in order to complete the geological data obtained from the boreholes previously executed. Laboratory tests have been carried out, by utilizing these borehole cores, in order to improve the geotechnical characterization of the materials belonging to the soil formation on which the "Santa Maria della Consolazione" church is found.

The three boreholes have been located in the area immediatly surrounding the church (figure 3). Moreover the average bulk density values determined are 2.035, 1.998 and 2.070 for the materials a, b and c indicated in table 1.

The analyzed specimens have been extracted at different depths, in correspondence to the more significant layers.

The results of the granulometric analysis and of the Atterberg limits, carried out on the materials,are given in table 1.

Oedometric tests-have been carried out for the over consolidated clay (the yellowish clay and the grayish-bluish clay).

The deformation moduli of the clay materials are defined in terms of effective stress on the basis of the obtained oedometric moduli by taking into account the evaluated values (Lancellotta, 1987; Battaglio, Jamiolkowski 1987) for the vertical effective stress and for the over consolidation pressure.

In the stress-strain analysis carried out in terms of effective pressure the materials are considered as isotropic.

The constitutive law adopted for the sandstone formation, the alluvial materials and the sandy-clay is the elastic-perfectly plastic law, while is the elastic plastic with hardening cap law for the yellowish clay and grayish-bluish clay. No tensile strength has been assumed for all the materials.

The Drucker Prager yield function has been assumed for the materials 1, 2 and 4:

$$f_1 = \sqrt{J_{2D}} - \alpha J_1 - K$$

where J_I and J_2 are the first invariant of the stress tensor and the second invariant of the deviatoric stress tensor respectively.

The α and K parameters can be expressed in the case of a 3-D problem as:

$$\alpha = \frac{2 \sin \phi'}{\sqrt{3}(3-\sin \phi')} \qquad K = \frac{6 c' \cos \phi'}{\sqrt{3}(3-\sin \phi')}$$

where c' and ϕ' are the coesion and the internal friction angle.

The hardening behaviour of the materials has been defined by coupling a second function called yield cap to the above reported yield function f_1 (fig;6) which is expressed as:

$$f_2 = R^2 J_{2D} + (J_1 - C)^2 = R^2 b^2$$

where Rb = (X-C), and R is the ratio of the major to minor axis of the ellipse, X is the value of J_I at the intersection of the cap with the J_1 axis, C is the va-

TAB. 1

borehole n. depth (m)	Layer	Granulometry (%)							W_L (%)	W_P (%)	W_N (%)
		>0,25	0,25 – 0,125	0,125– 0,075	0,075– 0,04	0,04 – 0,02	0,02 – 0,002	<0,002			
I 7,5–8	a	0,86	5,32	8,64	17,68	14,26	27,25	25,99	37,18	27,78	
I 15,5–16	b	1,81	3,71	2,98	5,30	10,17	34,65	41,38	51,15	24,15	28,69
III 7,5–8	a	3,80	2,06	3,26	7,36	10,44	35,91	37,17			15,65
III 10,5–11	a	0,80	2,88	3,97	8,40	15,53	24,77	43,65	51,50	25,75	25,64
III 13,5–14	a	5,74	2,38	6,23	17,28	18,97	29,57	19,82			
III 16,5–17	c	3,30	16,30	10,14	12,50	11,67	23,68	22,41	34,20	21,42	9,41

a) yellow-ochrous clay-sand and yellow compact clay
b) bluish clay with five grain sand
c) sand clay alterations in coarse alluvial material.

TAB. 2

Materials	Ed (MPa)	ν (–)	c' (MPa)	Ø' (°)	D (Mpa^{-1})	W (–)	X (MPa)
Sandstone formation	200	0.3	0.05	35°	–	–	–
Alluvial materials	50	0.3	0.001	38°	–	–	–
Yellowish clay	5	0.12	0.03	26.5	0.2	–0.094	0.2
Sandy clay	2.5	0.12	0.001	34	–	–	–
Gray-blue clay	13	0.12	0.1	30	0.2	–0.094	0.2

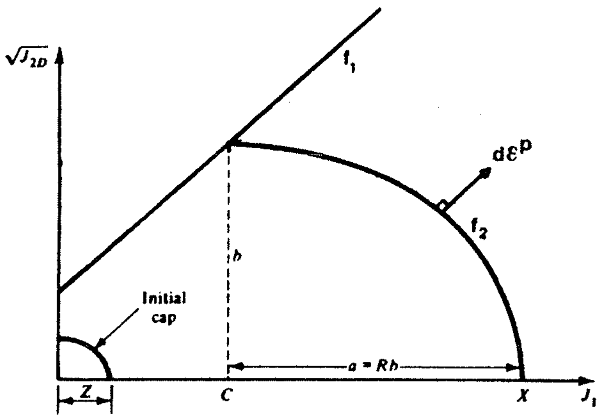

Fig. 6 - Yield surface for cap model.

lue of J_1 of the ellipse centre and b is the value of when $J_1 = C$.
The value of the hardening parameter X depends on the plastic volumetric strain and is expressed as:

$$X = -\frac{1}{D}\ln\left(1-\frac{\varepsilon_v^p}{W}\right) + Z$$

where D, Z and W have been determinated (Desal C.S., Siriwardane H.J., 1984) from hydrostatic test data.

The strength and deformability parameters adopted for the 5 materials are reported in table 2 (Calabresi G., 1987).

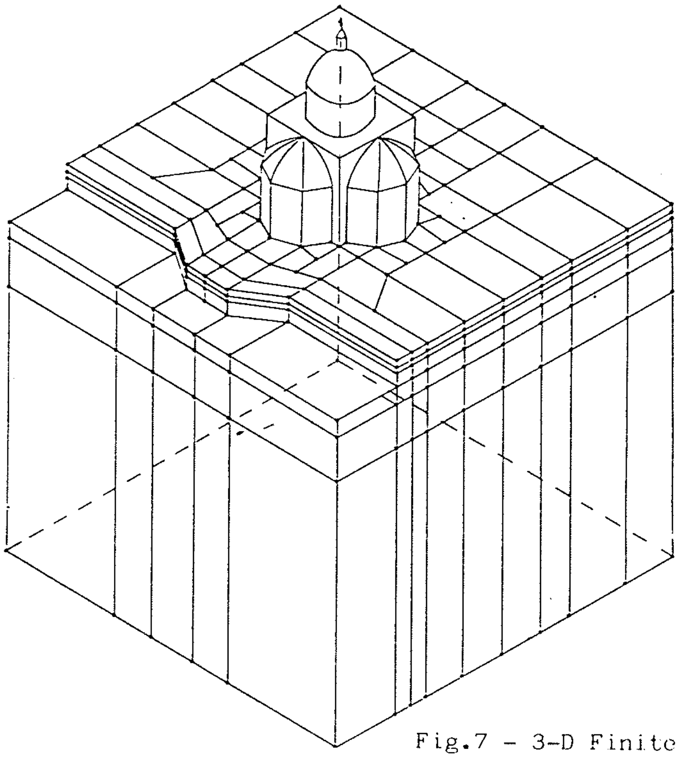

Fig.7 - 3-D Finite element mesh.

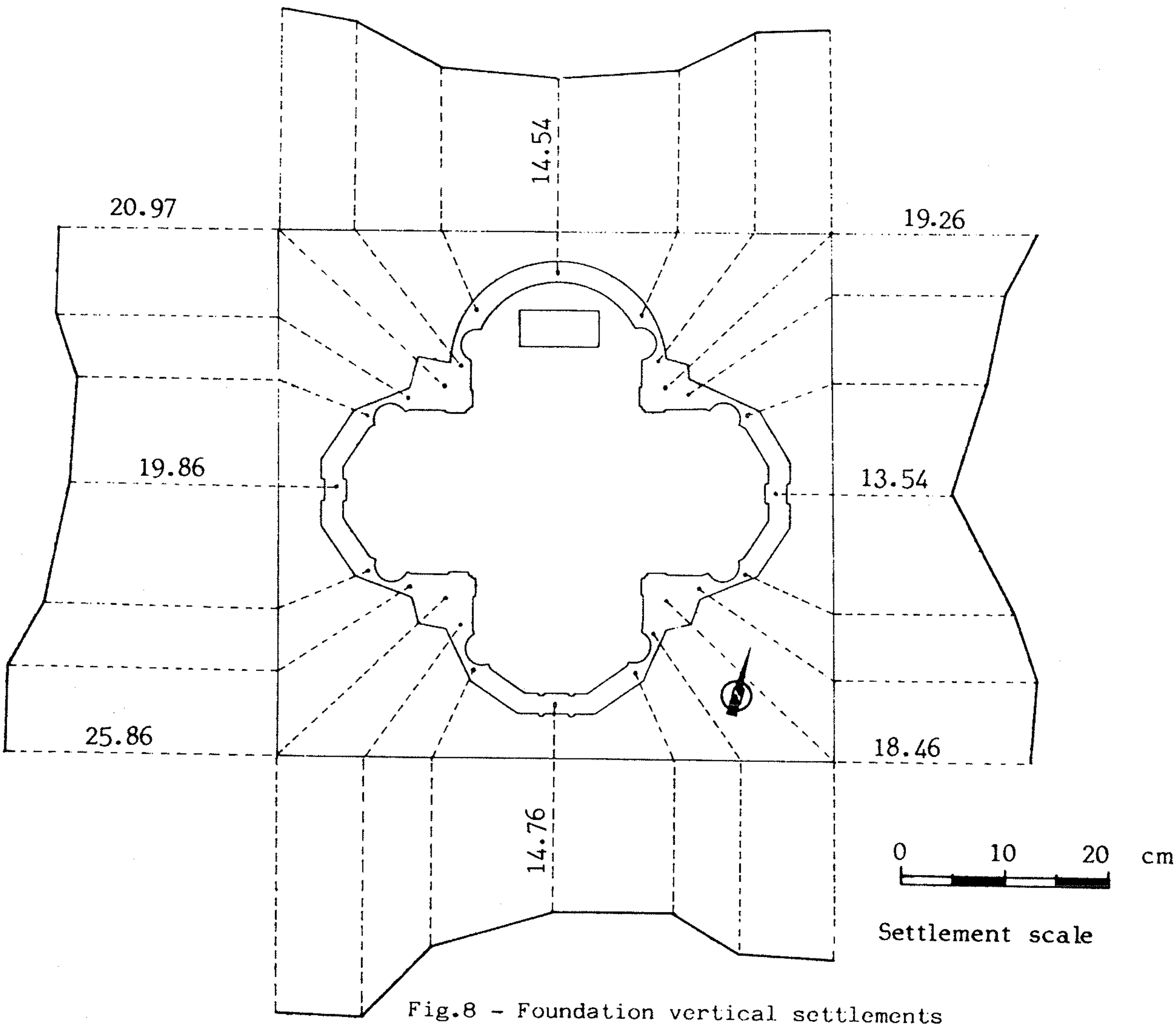

Fig.8 - Foundation vertical settlements

7. FEM stress -strain analysis

A FEM model (Bathe K.J., 1982) has been set up in order to reproduce the Church foundation soil stress history and to determine wall apses and pillar foundation settlements and base pillar foundation rotations.

The structure loading acting on the foundations are loadings obtained from loading analysis and non-destructive tests.

The FEM model (figure 7) is three-dimensional and discretizes a square side parallelepiped structure and soil volume. The Model, 100 m high and 105 m long, is formed by 969 nodal points and 579 sub or isoparametric elements. 579 elements discretize the subsoil, while 50 elements discretize the semicircular gravity wall and the masonry foundations.

The total number of freedom degrees in the 3-D FEM analysis is 2400, taking into account the boundary condition assigned to the model sides.

A FEM simulation has been carried out by executing the following phases in different computation stens;

1) loading and unloading computation steps state in agreement with the results of the oedometric tests, in the soil formation;

2) embankment and gravity wall construction, soil foundation excavation and masonry foundation construction;

3) correspondent church weight application on the pillar and apse foundations.

The 3 phase simulation has been carried out in 12 elastic-plastic computation steps; the FEM stiffness matrix bas been updated and 40 stress-transfer iterations have been carried out for each step.

The complete FEM simulation took 11000 sec of a 550 Prime computer CPU time.

8. STRESS-STRAIN ANALYSIS RESULTS AND CONCLUSIONS

Figure 3 reports the perpendicularity variation in the pillars obtained from FEM analysis. These values have been homogeneized to the topographic measurement values by referring the pillar base rotations to the pillar height.

The vectors which represent the perpendicularity variation for the FEM analysis have concordant direction with the vectors obtained from the topographic measurements.

The topographic measurements have provided the determination of the perpendicularity variation induced by the application of the large eight, for consolidation purpose, is 1953 by means of large mortar quantity injection.

These loading application effects on the subsoil are similar, even though of a lower intensity, to those which were induced by the church load application.

The consequent pillar leg divarication in then the cause of the yielding of the truss rods which contain the pillar at the top.

Figure 8 shows the pillar and wall apse foundation settlement diagram. This diagram shows the wide opening of the whole structure from the church center to the outside, above all in the N-S direction consequently largest cracks are on the dome and wall apses documented from an eastern western direction as has already been documented.

In conclusion, the numerical model set up seems to be a usefull means of studing the studying reliability of a future church consolidation project. An improvement of the geotechnical material characterization and of the control measurements is one of the mains aspects which will be examined in the future.

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