Diagnostic Analysis of Masonry Buildings

Analyse et diagnostic d'un bâtiment en maçonnerie

Diagnostische Analyse von Backsteinbauten

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SUMMARY

This paper describes operative criteria for the stress analysis of masonry buildings whose data on local geometrical configuration, physical properties of materials and loading history are incomplete or only partly available. The use of experimental data obtained from non-destructive in-situ tests concerning material mechanical characteristics and local stress and strain states (stress relaxation methods) are discussed.

RESUME

Le rapport présente des critères d'analyse des contraintes d'ouvrages existants, dont la connaissance de la configuration géométrique, des caractéristiques physiques des matériaux et de l'histoire de chargement n'est pas complète. Il est fait mention de l'utilisation de données expérimentales obtenues à l'aide d'essais non destructifs «in situ» (méthode de libération des contraintes).

ZUSAMMENFASSUNG

Dieser Artikel beschreibt operative Grundsätze und Methoden für die statische Berechnung von bestehenden Backsteinkonstruktionen, von denen die physikalischen Eigenschaften der Materialien, die geometrische Beschaffenheit des Aufbaues und die Geschichte der Belastung nur teilweise bekannt sind. Wir diskutieren die Verwendung der experimentellen Daten die aus zerstörungsfreien «in-situ»-Versuchen über die Materialeigenschaften und die lokalen Beanspruchungszustände entnommen werden können.

1. DESCRIPTION OF ACTUAL SITUATION

This research, as in previous paper [1], aims at a systematic use of in situ investigation procedures capable of collecting helpful data for structural analysis.

As is known, the determination of geometrical features and their graphic display (Figs. 1, 2) together with an appropriate photographic campaign (Fig.3) are needed to supply a knowledge of the statical behaviour of the construction.

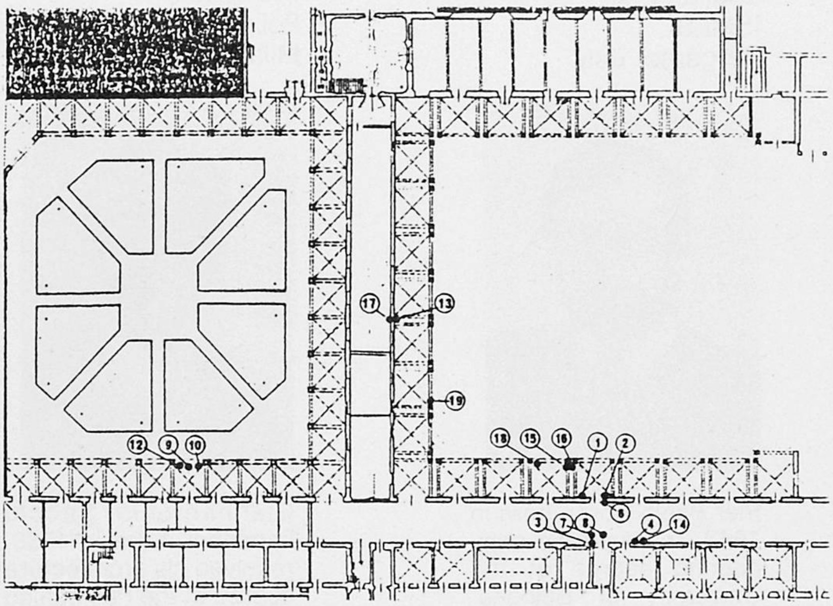

Fig. 1 Plan of the building.

Changes undergone by structure as a result of soil or structural element settling or else of rehabilitation works, cause modification in the stress-strain state of the structure. Relevant information may be given by historical research.

The photogrammetric survey and the crack pattern representation (Figs. 4, 5) may be quite useful: they aid in defining the actual static behaviour.

A direct measurement of stabilized settlements undergone by the structure in the past is often practically impossible. The setting-up of monitoring systems allows to control displacements

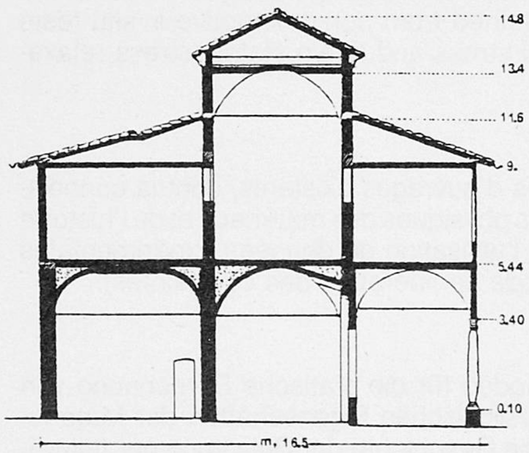

Fig. 2 Vertical section.

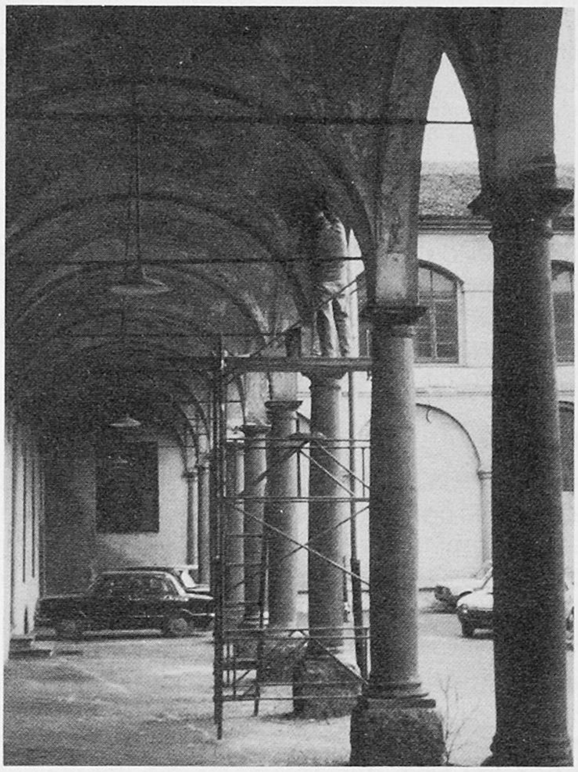

Fig. 3 Photographic survey.

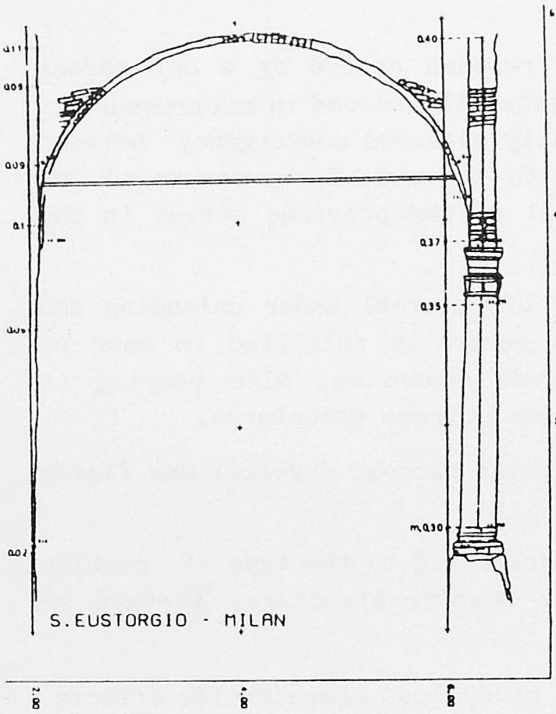

Fig. 4 Photogrammetric survey.
(Inst.of Topogr.& Photogr., Politecnico of Milan)

in progress. Of course, control should include soil behaviour.

Alteration of materials, especially of mortar joints, decreases the load carrying capacity of the structure. Knowledge of actual deformability charac teristics and strength limit of materials enables the designer to appraise both the structure response to loads and its safety margin. Laboratory tests call for large-size in situ sampling, which might be of no use if the sample is disturbed.

Non destructive in situ mechanical tests based on the insertion of special flat jacks in the masonry represent a useful tool to determine the mechanical behaviour of the material without extracting samples.

An experimental investigation on the masonry static condition of S. Eustorgio Cloister in Milan, was conducted through non-destructive testing technique.

Two are the S. Eustorgio cloisters, attached to the homonimous church. The construction of the first cloister dates back to the first half of 13th century; the construction of the second one started in 1380. Both cloisters were repeatedly damaged and restored in the following centuries during Spanish and French domination and rebuilt in 1600. After that time they underwent further damages and rehabilitations, the latest of which around 1950, when wooden floors were replaced with tile lintol floors and r.c. beams were placed. The superposition of the various ages may be seen in the surveys (Fig. 5).

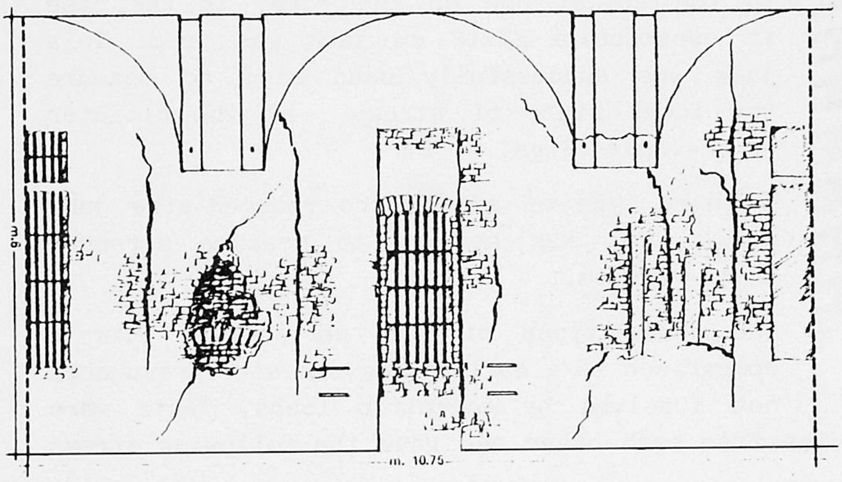

Fig. 5 Crack pattern representation.

2. MECHANICAL NON-DESTRUCTIVE TESTS

Mechanical in situ testing makes it possible to determine both the local stress state in the masonry and deformability properties and may also provide an appraisal of failure resistance of material.

2.1 Stress state measurement

Stress state measurement is based on stress release caused by a cut normal to the masonry surface. A flat jack is subsequently placed in and pressure increased gradually up to compensate the previously measured convergency between two points which are symmetrical in relation to the cut. The pressure of the properly calibrated jack provides an appraisal of the previous stress in the masonry in normal direction to cut plane.

The assumption that the mechanical behaviour of material under unloading and reloading conditions is of reversible type, is generally fulfilled in case of slightly fractured rock masses and may include masonries, also bearing in mind the low stress levels which generally exists in these structures.

Test reliability was checked at ISMES laboratories through physical and finite element tridimensional models [2].

Flat jack dimensions may easily be changed according to the type of problem to be tackled. In the case of high-thickness bearing structure, a 40x20 cm jack is generally used (Fig. 6).

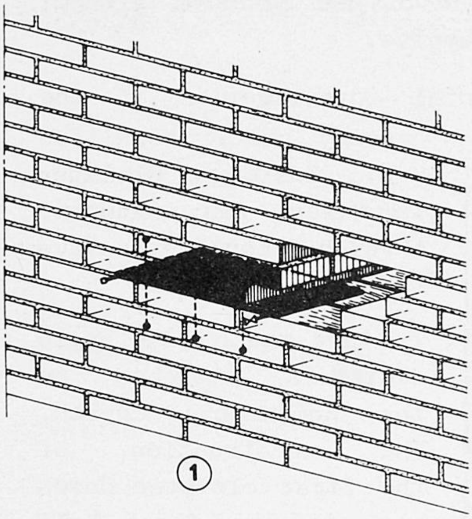

Fig. 6 Flat jack test.

If we wish to study load eccentricity effects through tests carried out on the two opposite faces of the masonry, the jack depth should be reduced to measure a stress value nearest to the actual one on the edge. In the case in question, a 24x12 cm jack was used. This jack was successfully used also to measure the local state of stress at the cloister cross-vault (Figs. 7, 8).

A third type of still more reduced-size jack (12x12 cm) was set up to measure stresses in the arches.

The three types of jack were set for a comparison in a wall of the cloister presumably not involved by eccentric loads. Tests were carried out at short distancies from each other and gave the following stress values:

JACK 1	(40 x 20 cm)	σ = 1.04 MPa
JACK 2	(24 x 12 cm)	σ = 1.12 MPa
JACK 3	(12 x 12 cm)	σ = 1.17 MPa

The good agreement between measurements made it possible to ascertain the reliability of the test carried out by using small-size jacks. The correct working of the 40x20 cm jack was indeed fully checked during calibration tests and several investigation campaigns conducted on monumental buildings (Palazzo della Ragione - Milan; "Classense" Library - Ravenna; building at "Piazzale Dateo" - Milan).

2.2 Comparison between measured and calculated stress values

In situ tests were carried out at the points shown in Fig. 1. Stresses

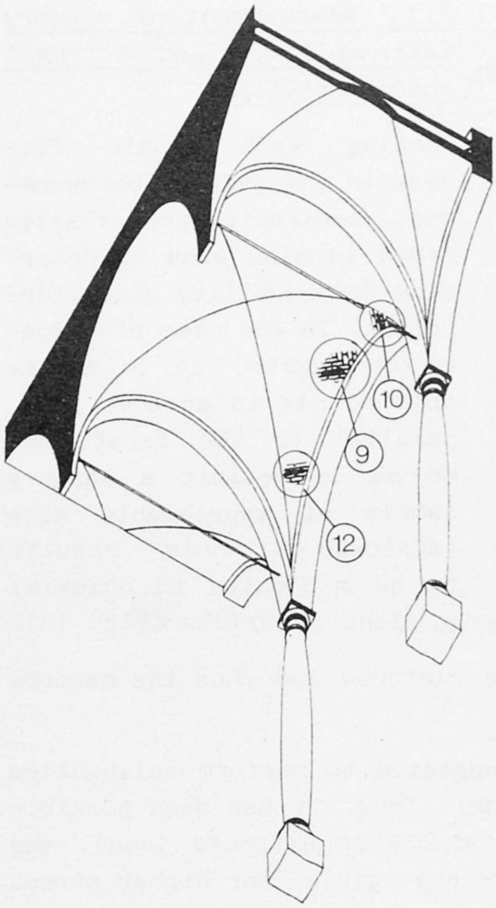

Fig. 7 Tests points on the cross vault.

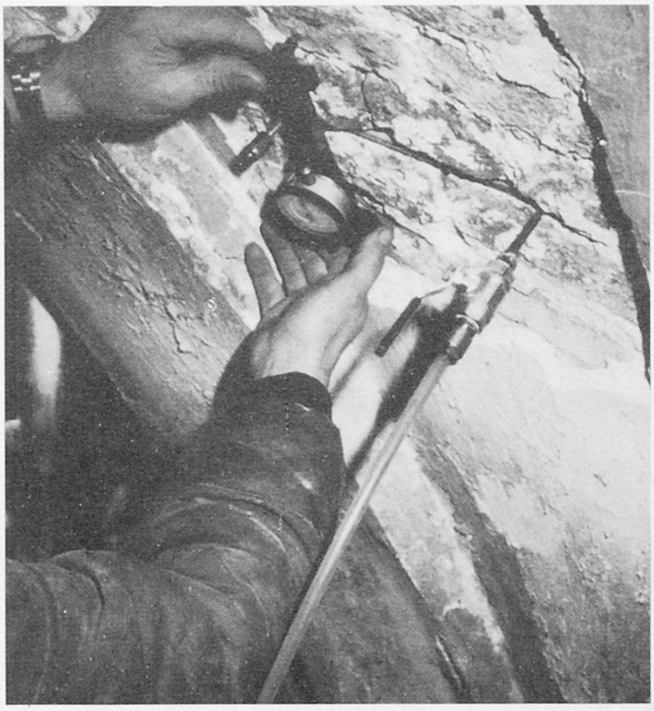

Fig. 8 Tests on the cross vault with a 24x12 cm flat jack.

corresponding to a statically determined solution, suggested by the crack pattern, were computed. In particular, Fig. 9 shows the solution adopted for the cross vaults. The results are reported in the Table I.

TABLE I: Comparison between in situ measured stresses and calculated stresses

Test no.		Calculated stress (MPa)	In-situ measured stress (MPa)
1	wall	0.51	0.4
2	wall		0.56
6	wall		0.4
3	wall	1.16	1.04
4	wall		1.12
14	wall		1.05
13	wall	0.78	1.6
17	wall		0.48
15	pulvino	1.73	2.0
16	pulvino		1.46
7	barrel vault		0.8
8	barrel vault		0.72
9	cross vault	0.06	0.16
10	cross vault	0.37	0.32
12	cross vault		0.24
18	cross vault		1.3

The evident correspondance between experimental and calculated values confirms the good choice of the statical solution.

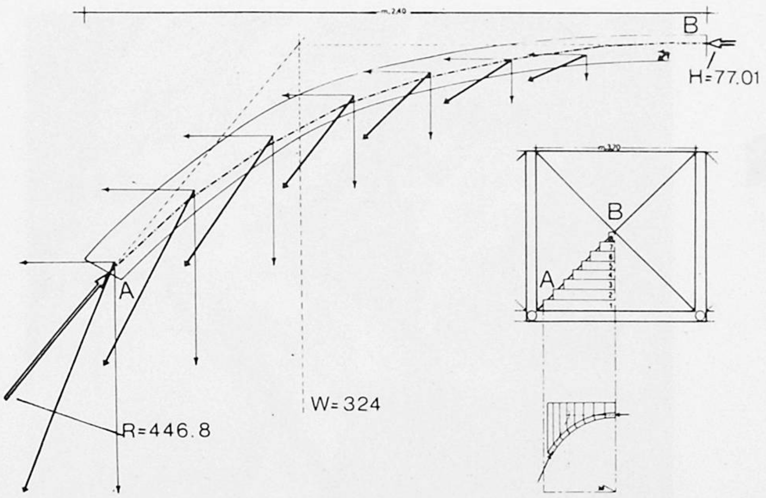

Fig. 9 Static solution adopted for cross vault.

2.3 Measurement of masonry deformability and strength characteristics

Testing with simple flat jack in the case of homogeneous, isotropic and elastic media is also used to determine deformability characteristics. In the case of masonry structures, it is advisable to set in another jack, parallel to the first one, so as to delimit a masonry sample of appreciable size (40x50x20 cm); this results to be subjected to uniaxial compressive test in direction normal to the laying plane of bricks (Fig. 10).

After the test was performed, mortar layers are restored and thus the masonry is returned to its original conditions.

Lateral confining conditions of sample have suggested to perform calibration tests by means of physical and mathematical model. Thus, it has been possible to ascertain that , for slight stress values (< 20% of ultimate load), the lateral confining effect may be considered quite negligible. For higher stress levels (< 50% of ultimate load), an increase in deformability modulus equal to about 10% of the value determined by unconfined compression tests was noticed.

Fig. 11 shows stress-strain diagrams of the sample delimited by the two jacks.

To estimate the strength limit, a test was carried out up to the appearance of the first cracks in the bricks (Fig. 12).

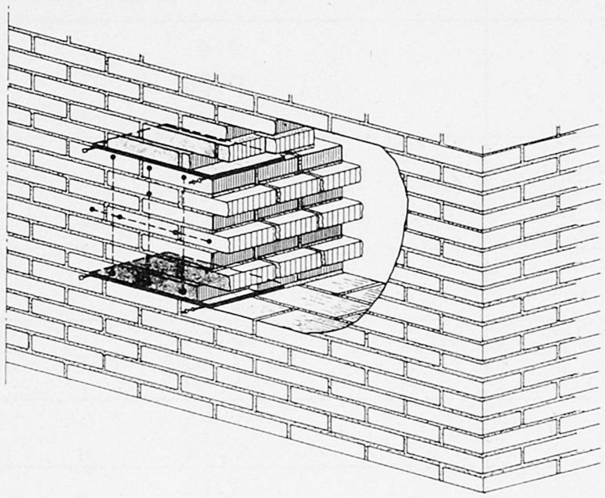

Fig. 10 Deformability test.

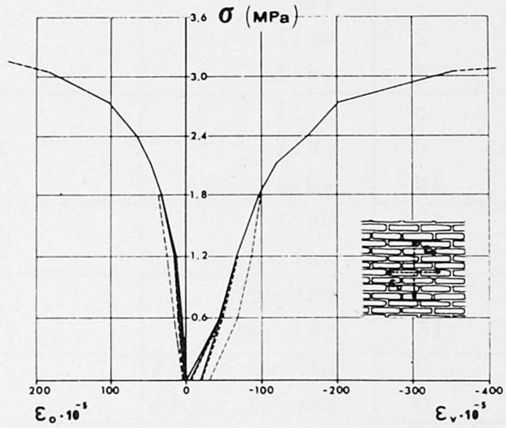

Fig. 11 σ - ε diagram.

The ultimate stress value measured was equal to 3.1 MPa.

Fig. 12 Photo showing the appearance of first cracks.

Ratio of deformability modulus E_d, computed in the load interval 0 - 0.5 σ_R to ultimate stress σ_R is (Fig. 10):

$$\frac{E_d}{\sigma_R} = \frac{1,990}{3,1} = 609$$

It may be pointed out that the evaluated ratio is included in the usual range for this kind of materials.

3. ANALYSIS OF A REAL STRUCTURE

Experimental in-situ investigation, which deals especially with geometric and mechanical aspects of the structure, is of particular importance because it makes it possible: to follow the soil settlements (monitoring); to notice whether geometric alterations exist, such as to change the original load condition; to appraise the strength of the material and the local stress state. These latter two conditions are particularly useful, as they provide an important hint for the formulation of equilibrated solutions.

The following remarks mainly emphasize the know-how degree of the structure stress state and therefore of its safety, versus available information on the behaviour of the material, on constraint conditions and on soil behaviour, and on the stationary condition or progress of alteration phenomena.

a) In case all this information is clear - which is an exceptional case - a step-by-step non-linear analysis may be applied. This, in spite of the numerical difficulty, will provide exhaustive solutions.

b) Constraint settlements are unknown, constraint conditions are uncertain; however, the material shows a behaviour, up to failure, characterized by ductility and associated flow rule. In this case, the classical limit analysis may be applied. The structural safety domain may be defined in the load space. For every load condition, the correspondent kinematic load P_K will be an upperbound on the limit load P_L; in turn, the latter is an upperbound on any statically admissible load P_S [3] [4]. A solution $P_S = P_L = P_K$ is proved to exist.

c) Preceeding conditions occur, but the material has no-tensile strength. Limit analysis may still be used, assuming the cracks as fictitious ductile strains [5], the normality rule being still considered existent.

d) A non-associated flow rule is known. Two fictitious standard domains f_G, f_F [6] [7] can be defined; f_F contains f_G and, using limit analysis techniques, inequalities $P_{LG} \leqslant P_L \leqslant P_{LF}$ can be proved. In short, an upper and a lower bound of P_L can be calculated; however, it will be impossible to know the exact value of P_L.

e) The material has no-tensile but very high compressive strength; that is the particular case of stone-masonry structures. Limit analysis may still be applied and is essentially reduced to a geometric problem [8] [9] .

f) The failure criterium of material is known as a function of stresses f_F; however, the flow rule is unknown. Referring to item d), we may say that the domain f_G is no longer defined and we only state that $P_L \leqslant P_{LF}$. Load domain built up with f_G is then to be considered only as "potentially safe". Points outside the domain f_F certainly represent collapse states. Therefore P_L may be accessed to only for kinematic approach - which involves a good experience on failure mechanisms of the various structures [10]. The capability of the static approach decreases, although it remains still useful. A coefficient of "true failure" can be defined - which is important in case a modification of the load conditions of the structure is required [11].

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