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# Mechanical Characteristics of Ancient Roman Masonry

## Propriétés mécaniques de la maçonnerie antique romaine

## Mechanische Eigenschaften antiken römischen Mauerwerks

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## SUMMARY

The procedure for the analytical evaluation of the mechanical characteristics of Roman masonry and the strength of the bearing structure of the Rotunda of Thessaloniki is presented. The mechanical properties and failure envelope of ancient Roman masonry were predicted using a specific finite element program for the non-linear analysis of masonry micromodels. Using the predicted properties of masonry, the structural system of the monument was analysed and proved to be cracked meridionally under dead loads. Finally the ultimate seismic capacity of the supporting piers has been calculated.

## RÉSUMÉ

L'article présente une analyse des propriétés mécaniques de la maçonnerie romaine et de la résistance de la structure porteuse de la Rotonde à Thessalonique. Les propriétés mécaniques et la charge de rupture sont estimées à l'aide d'un programme d'éléments finis permettant l'analyse non-linéaire des micromodèles de la maçonnerie. Les propriétés estimées de la maçonnerie sont utilisées pour l'analyse du système de la structure du monument, lequel a montré des fissures méridiennes sous le poids propre. Finalement, la résistance ultime sismique des piliers qui portent le dôme est calculée.

## ZUSAMMENFASSUNG

Im Aufsatz wird eine Vorgehensweise zur analytischen Bestimmung der mechanischen Eigenschaften des römischen Mauerwerks und der Festigkeit des Tragsystems der Rotunda von Thessaloniki dargestellt. Die mechanischen Eigenschaften und die Bruchlast des römischen Mauerwerks werden mit Hilfe eines speziellen Finite-Elemente-Programms bestimmt. Unter Verwendung der berechneten Eigenschaften wird das für die nichtlineare Berechnung von Mikromauerwerksmodellen statische System des Denkmals analysiert und gezeigt, dass es unter Eigenlast in Meridialrichtung gerissen ist. Abschliessend wird die seismische Grenztragfähigkeit der Mauerwerkstützen berechnet.

## 1. INTRODUCTION - ROTUNDA OF THESSALONIKI

The Rotunda of Thessaloniki, built in about 300 A.D., is the greater surviving monument of the late Roman Empire in the Balkan region. It is an imposing circular building, resembling the Pantheon in Rome, covered by a huge brick masonry dome 24.50m in diameter. The dome is supported by a, 20.00m in height and 6.25m in thickness, cylindrical drum divided into eight strong piers,by barrel vaulted niches and openings at two levels (Fig.1,5). The structure, already extensively cracked due to earthquakes during its life, suffered serious damage due to the earthquake of June 20, 1978 (epicenter 30km from Thessaloniki, magnitude 6.2 grades in Richter scale). The dome was radially cracked in its lower zone. The two southern piers $P_2$, $P_3$, weakened due to internal helical staircases (Fig.1), had wide shear-compression inclined cracks which have been reactivated.

Extensive emergency works were undertaken after the earthquake because of the critical structural condition of the monument. During the 1980 a team of specialists, supervised by Pr. Penelis, carried out a wide research project on the repair and strengthening of the monument including extensive in-situ investigations and laboratory tests. The greater part of structural analyses were carried out considering the masonry as homogeneous, isotropic and linearly elastic material [1].

Recently at the Reinforced Concrete Lab. of Aristotle University of Thessaloniki extensive research has been carried out for a better approach to the problem of evaluation of nonlinear mechanical properties of reinforced concrete and masonry structures by means of F.E.M. The following specific F.E. computer programs have been the fruits of these efforts:
"MAFEA": For the in-plane nonlinear analysis of unreinforced masonry under monotonic loading until failure. The program is capable of predicting cracking, crushing, or transverse splitting of bricks and mortars, as well as sliding or unstucking at the joints and simulating propagation of damage. It must be pointed out that the program calculates and takes rationally into account the transverse third principal stresses that developed in bricks and mortar joints under the in-plane loading of masonry [2].
"AXICRACK": For the linear elastic analysis of isotropic or orthotropic axisymmetric structures. A nonlinear repeated process for the propagation or meridional cracking is included [3].
"RECOFIN": For the in-plane nonlinear analysis of reinforced or unreinforced concrete under monotonic loading until failure. The program is capable of predicting cracking or crushing of concrete as well as yielding or sliding of reinforcing bars and simulating propagation of damage [4].

In the following the three phases of limit analysis of the bearing structure of Rotunda in order to determine the ultimate strength of the monument under dead and seismic loading are presented. The mechanical properties and failure envelope of Roman masonry were predicted using the MAFEA program. The axisymmetric structural system of the monument was analysed using the AXICRACK program. Meridional cracks were predicted under dead loading at the upper ring which connects the piers. Finally the ultimate seismic capacity of the piers was calculated by the RECOFIN program using the previously predicted properties of masonry.

## 2. MECHANICAL CHARACTERISTICS OF MASONRY

The necessary input for the MAFEA program is the geometry of the masonry and the mechanical properties of bricks, mortar and joints. The most of them had been already determined by the in-situ and laboratory tests, as mentioned above, and are given in Table 1.

It is well known that in a structural member the dominant stress state under in-plane loading consists of a pair of compressive and tensile principal stresses ($\sigma_1<0$, $\sigma_2>0$) of various ratio. The stress state of masonry at the piers of

| Materials | Dimensions | $f_c$(MPa) | $f_t$(MPa) | $E_0$(MPa) | $v_0$ | $\varepsilon_u$(‰) |
|---|---|---|---|---|---|---|
| Bricks | 30x40x5cm | $f_{bc}$=9.00 | $f_{bt}$=1.20(2) | $E_{bo}$=10700 | 0.19(3) | 2.50(3) |
| Mortar | $t_{bed}$ ≈4cm | $f_{mc}$=1.20(1) | $f_{mt}$=0.40(2) | $E_{mo}$= 1350 | 0.19(3) | 2.50(3) |
| Joints | $t_{vert}$≈3cm | Shear strength:$f_{jso}$=0.17,Tensile strength:$f_{jt}$=0.075MPa(3) | | | | |

(1) Prismatic strength from the strength of a short specimen: $f^c_{mc}$=1.80MPa

(2) Direct tensile strength from the flexural strength: $f^{fl}_{bt}$=2.25, $f^{fl}_{mt}$=0.65MPa

(3) From the literature according to relative measurements.

Table 1 Mechanical properties of masonry materials,input for the MAFEA program

Rotunda, under dead and radial seismic loading, consists mainly of normal stresses $\sigma_n$ perpendicular to the bed joints and shear stresses $\tau$ in meridional level (Fig. 2a). The stress state ($\sigma_n$, $\tau$) is equivalent to the principal stress state ($\sigma_1$<0, $\sigma_2$>0) mentioned above.

In order to determine the mechanical characteristics of Roman masonry, the micromodel shown in Fig.2a was analysed under various normal stresses $\sigma_n$ increasing gradually to a defined value, followed by shear stresses $\tau$ increasing gradually until failure.

The model was analysed at first under uniaxial compression ($\sigma_n$, $\tau$=0) and pure shear ($\sigma_n$=0, $\tau$) in order to bound the ($\sigma_n$, $\tau$) failure envelope and on the other hand in order to determine the following basic mechanical properties of masonry:
-Compressive strength perpendicular to bed joints: $f^n_{wc}$=3.111MPa
-Modulus of elasticity perpendicular to bed joints (initial value): $E^n_{wo}$=2667MPa
-Normal stress-strain curve ($\sigma_n$-$\varepsilon_n$): See Fig.2b
-Shear modulus (initial value): $G_{wo}$=1090MPa
-Strength under pure shear loading: $f_{wso}$=0.133MPa
-Shear stress-strain curve ($\tau$-$\gamma$): Almost straight line.
The strong and early appeared nonlinear character of the stress-strain curve ($\sigma_n$-$\varepsilon_n$) must be attributed to the weakness of mortar which is almost the 50% of the masonry mass.

In order to define the failure envelope shown in Fig.2c, seven failure points ($\sigma_n$,$\tau_u$) were determined for various predefined values of $\sigma_n$ (points 2,3...8). It can be seen that the shear strength of masonry shows a serious reduction when the normal stress $\sigma_n$ exceeds the 50% of the compressive strength of masonry. The corresponding failure envelope in principal stresses at the (-,+) region ($\sigma_{1u}$<0, $\sigma_{2u}$>0) is shown in Fig.2d. Finally in order to bound this curve at the tensile axis, the masonry micromodel was analysed for tenth time under uniaxial tensile loading ($\sigma_1$=0, $\sigma_2$) parallel to the bed joints until failure ($f^p_{wt}$=0.135MPa, see Fig.2d, point 10). The initial value of elasticity modulus under this loading found to be $E^p_{wo}$=5555MPa which is more than double in comparison with $E^n_{wo}$. The anisotropic character of the masonty is obvious.

In Fig.2c and 2d, are also shown in dashed line the corresponding failure envelopes of a "concrete like" material having a compressive strength equal to the masonry. These curves had been used as failure criteria at the strengthening project of Rotunda in the 1980. The compressive strength of masonry had been calculated according to the following formula:

$$f^n_{wc} = \sqrt{f_{bc}}\ \sqrt[3]{f^c_{mc}} = \sqrt{90.0}\ \sqrt[3]{18.0} \approx 25.0\text{kg/cm}^2 \approx 2.50\text{MPa}$$

It must be pointed out the great overestimation of strength if masonry considered as a homogeneous material ignoring the shear weakness of joints.

In the Fig.3a,b the families of failure envelopes of masonry under principal stresses at the (-,+) region for various orientations of principal axes to the bed joints, experimentally determined by Samarasinghe-Hendry[5] and Page[6] respectively, are shown. On these figures the failure curves corresponding to the $(\sigma_n,\tau) \rightarrow (\sigma_{1u}<0,\ \sigma_{2u}>0)$ loading has been drawn with dashed line. On each curve the points ① till ⑨, corresponding to the analytically determined failure points of the Fig.2d have been marked. Although on the masonry of Rotunda the bricks are much more flattened and the mortar joints are much more thick than those of the experimental programs mentioned above, the qualitative similarity of the analytically determined failure envelope of ancient Roman masonry with the relative experimentally determined curves is remarkable.

The propagation of damage, until failure of the models, predicted by the MAFEA program is very interesting. The gradual dominance of joint damage and the brittle character of failure are evident as the precompression decreases. In Fig.4a the damage pattern at failure of the model No 4 is shown. The most of header joints were unstucked during the phase of step by step application of uniaxial compression till the final value of $\sigma_n$=1.555 MPa. Under the first steps of superadded shear loading, slidings and unstuckings of mortar elements at the bed-header joints crossings were occured. Finally failure occurs, under $\tau_u$= =0.153 MPa, due to successive inclined cracking of brick elements and sporadic cracking or crushing of mortar elements. In Fig.4b the damage pattern at failure of the model No 9, under pure shear loading, is shown. The failure has a strong brittle character. Ladder like slidings and unstuckings of mortar joints occured under $\tau_u$=0.133 MPa, without any warning damage at the previous loading steps.

## 3. STRENGTH EVALUATION OF THE MONUMENT

### 3.1 Axisymmetric Structural System under Dead Loading

In Fig.5 the axisymmetric F.E. model of the structural system of Rotunda, analysed using the AXICRACK program, is shown. The masonry at the dome and the connecting rings is considered as linearly elastic isotropic material ($E_{wo}$= =2667MPa), because the compressive stresses developed in monumental structures under service loading are almost one order of magnitude smaller than the material strength. At the three zones of openings of the cylindrical drum, masonry is considered to be orthotropic with Modulus of Elasticity equal to zero at the circumferential direction. The excess of tensile strength of masonry ($f^p_{wt}$= =0.135MPa) has been used as failure criterion for the prediction and propagation of meridional cracks.

Under the thrust of the dome the upper connecting ring has been progressively cut after three successive analyses of the model (see Fig.5b). The lower ring and the lower zone of the dome are also under circumferential tension and it is expected that they will be cracked as well under the superadded action of a medium sized earthquake. So the piers must be considered to act as independent free standing cantilevers. Consequently the ultimate strength of the monument depends on the strength of the piers.

### 3.2 Strength of the Piers under Dead and Seismic Loading

Two piers were modeled, one without ($P_1$) and another with internal staircase ($P_2$) and were analysed, under dead and seismic loading, until failure using the RECOFIN program. The radial distribution of normal force N and horizontal thrust V of the dome at the top of a pier (see Fig.6) were determined by circumferential integration of the normal and shear stresses respectively, calculated using the AXICRACK program. The 2D F.E. models refer to the middle meridional level of the piers. In order to simulate the variable cross section of the piers, due to the openings, the masonry rings at the top, medium and foundation level and the staircase hole, the thickness of each element has been estimated appropriatelly.

The mechanical properties and the failure envelope of masonry under biaxial compression-tension (-, +), determined previously using the MAFEA program, are introduced into the RECOFIN program. In order to form the complete biaxial failure envelope required by the program, the failure curves for the (+, +) and (-, -) regions of an "equivalent concrete" are used. The dead loading was applied step by step in two successive phases corresponding to erection phases. At first the dead weight of the piers ($W_{pier}$: see Fig.6) was applied in four steps, afterwards the weight and the horizontal thrust of the dome ($N_{dome}$, $V_{dome}$: Fig.6) were added at the top of the pier in two steps. Finally the seismic loading was applied step by step up to failure.

In Fig.65 the cracking propagation of the two models until failure is shown. The models sustained the dead loading and the seismic loading up to a value of 10% and 8% for the seismic coefficient (c), for the pier $P_1$ and the perforated pier $P_2$ respectively, without any damage. The first horizontal crack at the model $P_1$ appeared at the inner part of the base for $c_{cr}$=11% and propagated till the core of the pier (Fig.6a). At the next loading step (c=12%) no further damage occured. Finally for $c_u$=13% inclined flexural cracks appeared at higher levels of the inner face of the pier propagating to the core and failure occurs due to vertical splitting at the outer face of the base. The model $P_1$ shows a flexural, rather ductile type of failure. On the contrary the model $P_2$ shows a shear-compression brittle type of failure. At a seismic coefficient of $c_{dcr}$=9% inclined cracking occured firstly at the very thinner elements of the core of the pier (see Fig.6b) and rapidly propagated diagonally up and down cutting the thin elements of the staircase hole till the base. The thicker elements along the inner and outer faces of the model remained intact with the exception of the lowest inner element at the base which was horizontally cracked. At the next loading step ($c_u$=10%) the horizontal crack propagates to the core of the base, the diagonal cracking propagates upwards cutting the inner face and finally failure occured due to vertical splitting at the outer face of the base. The cracking pattern of the model $P_2$ simulates successfully the existing active inclined sliding surface at the piers $P_2$ and $P_3$ of the monument (see Fig.5b, 6b). Furthermore the base shear resistance of both types of piers was found to be close to the results found during the 1980 research project [1].

## 4. CONCLUSIONS

The successful analytical simulation of the structural behavior of a huge masonry monument, starting from the material properties and ending to the strength evaluation and the failure patterns, is very encouraging and the procedure followed seems to have good prospects for further development. Of course several points and results should be justified through extensive parametric analyses, before this procedure would be considered as a useful tool for practical purposes. The authors are convinced that the analysis by means of F.E.M. with microelements of bricks and mortar is the most promising one for the determination of the failure envelope and the constitutive law of old masonries as the only necessary inputs are the geometry of masonry and the mechanical properties of bricks, mortar and joints which can be rather easily experimentally determined.

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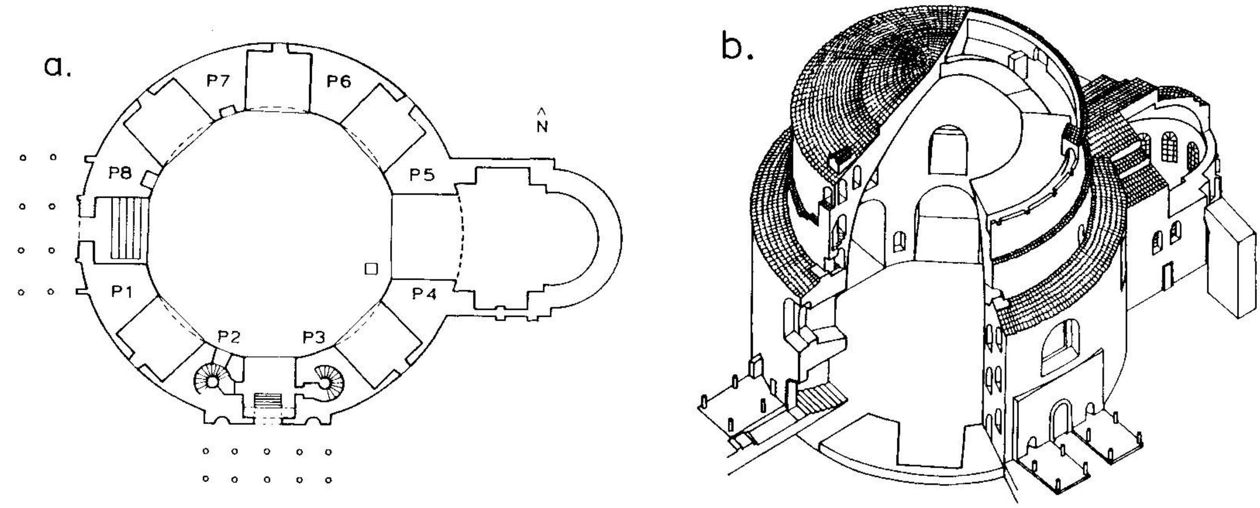


Fig.1 Rotunda of Thessaloniki: a.Plan at ground level, b.Isometric view

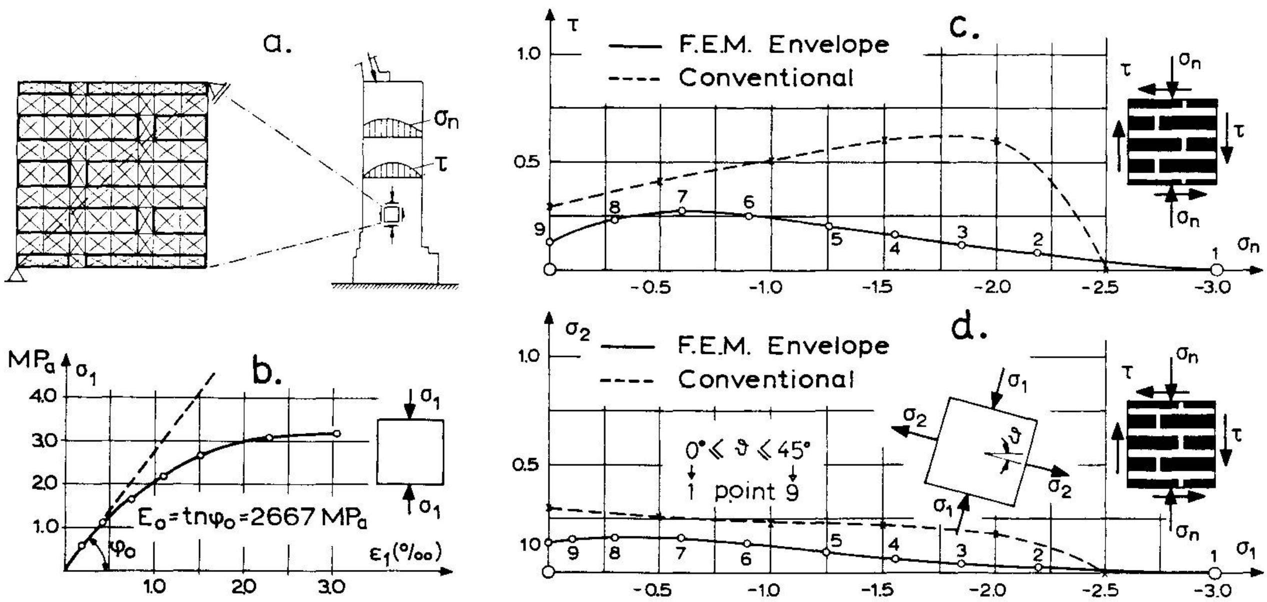


Fig.2 Analytical evaluation of masonry properties: a.Masonry micromodel, b.Stress-strain curve ($\sigma_n$-$\varepsilon_n$), c.Failure envelope ($\sigma_n$,$\tau$), d.Failure envelope ($\sigma_1$,$\sigma_2$)

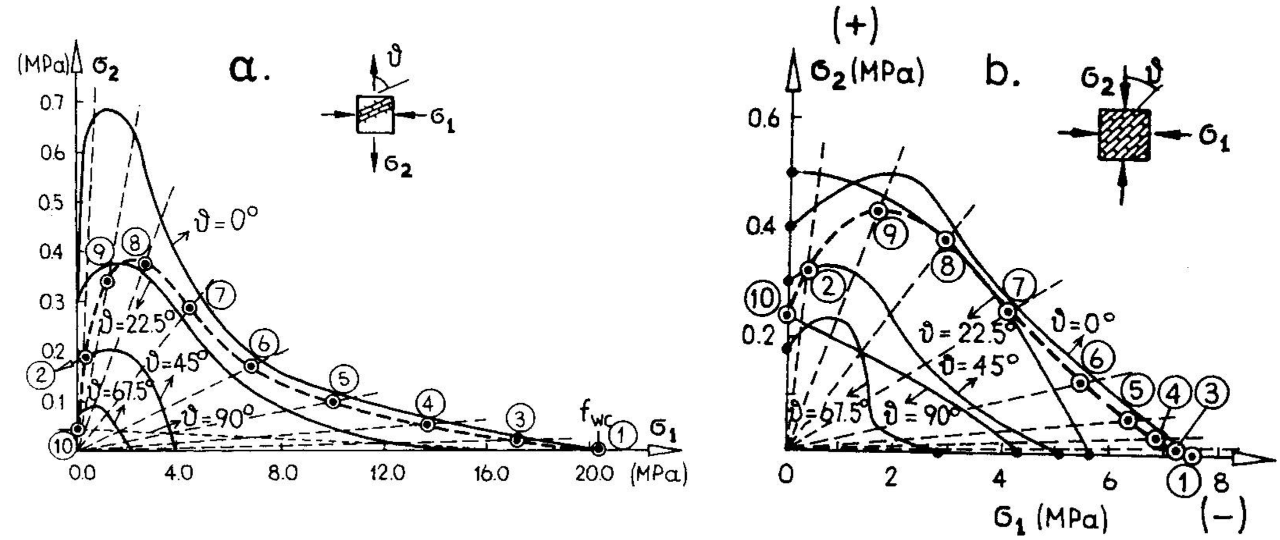


Fig.3 Experimentally determined failure envelopes of masonry: a.Samarasinghe and Hendry[5], b.Page[6]

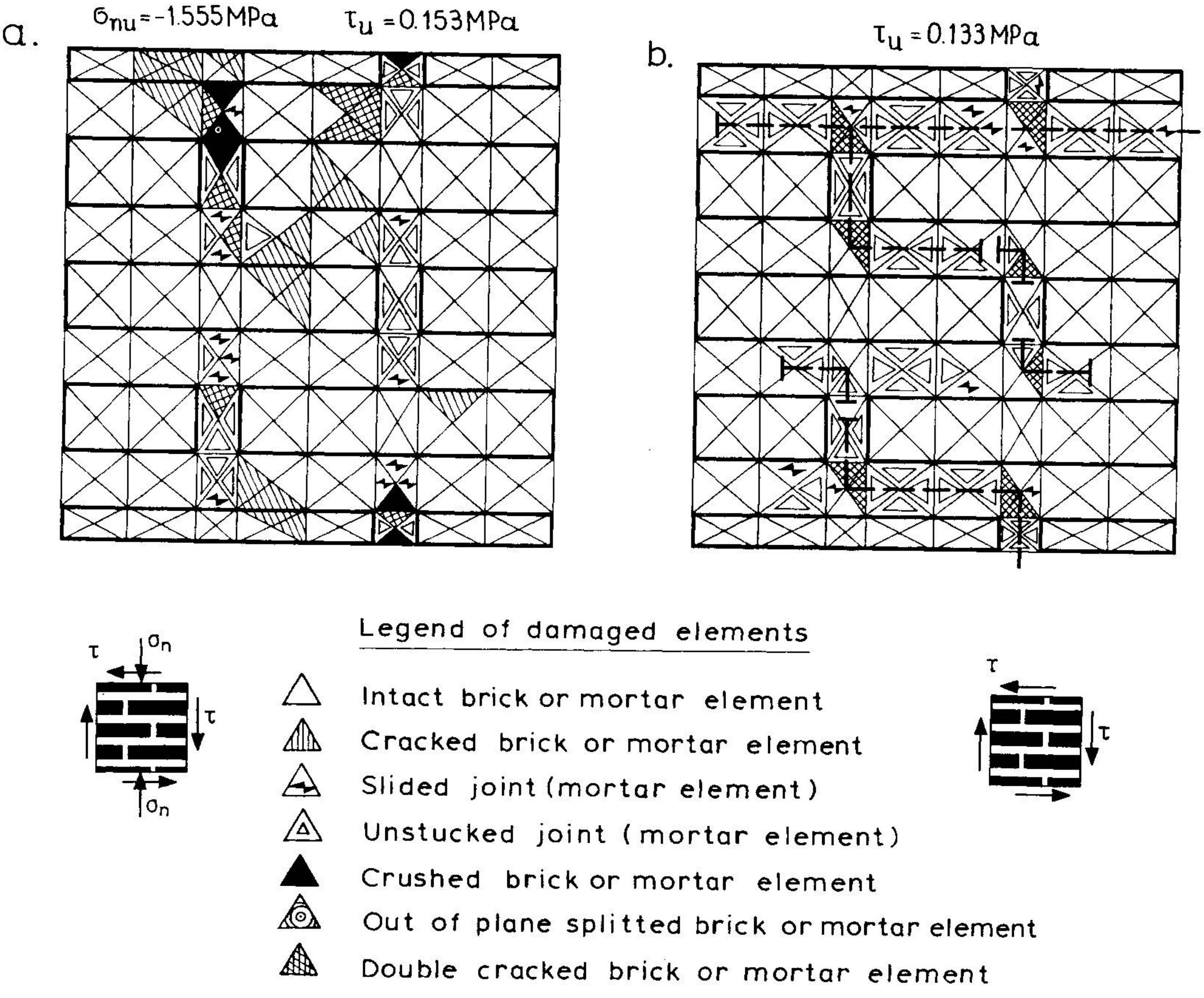


Fig.4 Failure patterns of masonry micromodels: a.Model No 4, b.Model No 9

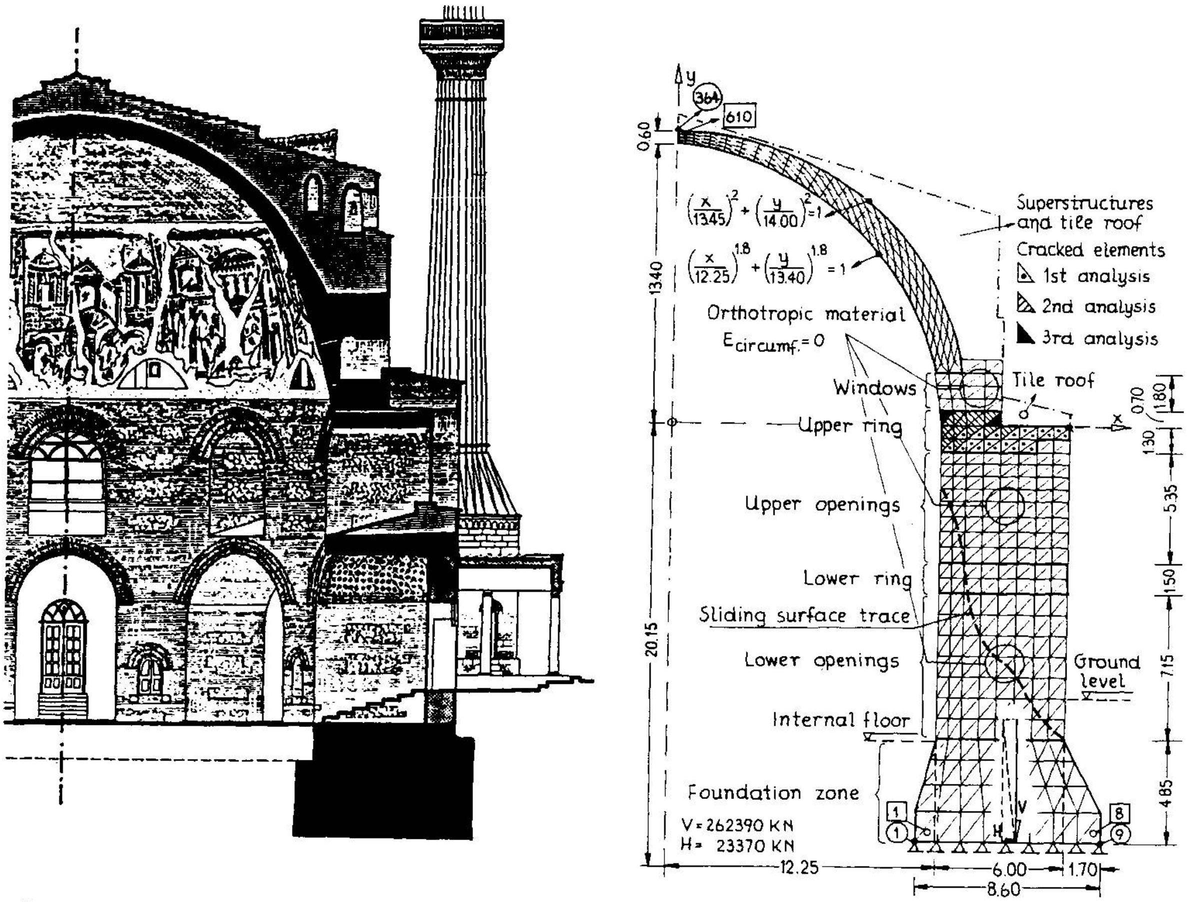


Fig.5 Rotunda: a.Meridional cross section, b.Axisymmetric F.E. model, predicted cracking

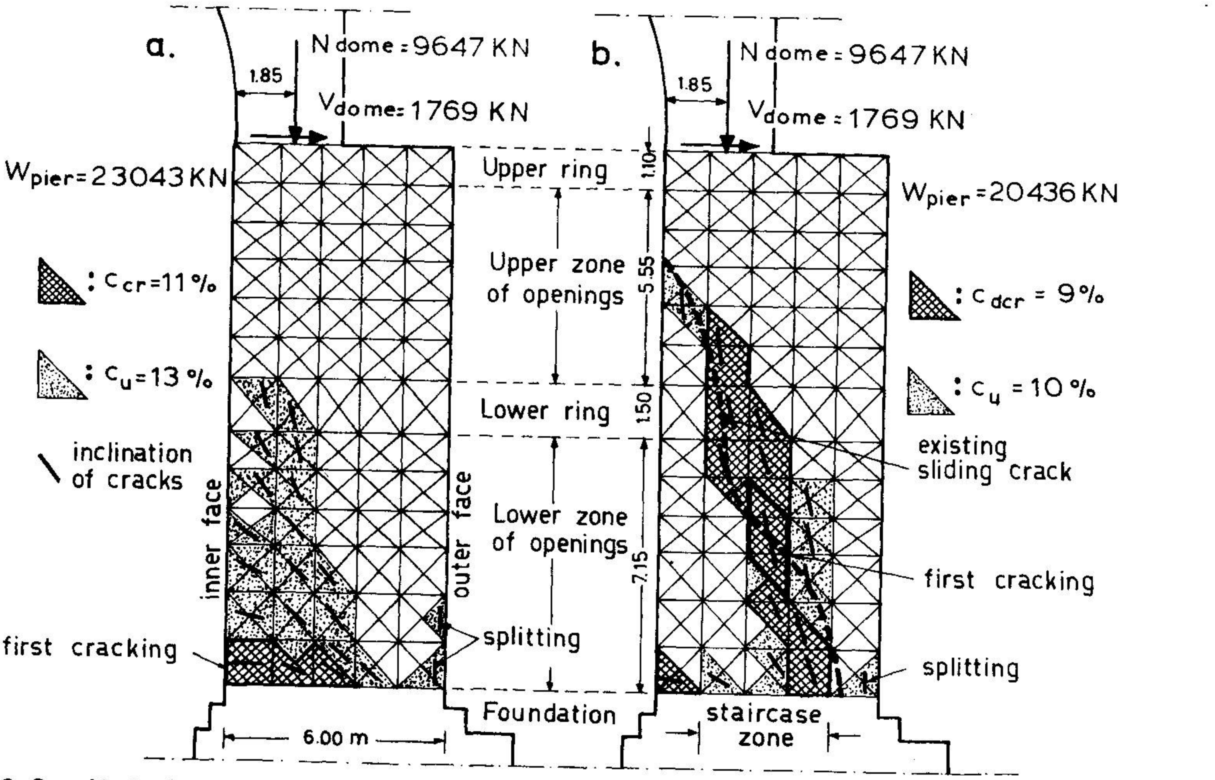


Fig.6 Predicted cracking patterns of the piers $P_1$ and $P_2$ till failure: a.Massive pier ($P_1$), b.Pier with staircase ($P_2$)