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Simulation of the Design Schemes for Architectural Monuments

Simulation de projets d'études pour des monuments architecturaux

Simulation von Entwurfsmodellen für architektonische Bauten

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The purpose of this paper is to propose of the general ideas on simulation of the historical architectural constructions (destroyed buildings as well) design schemes, also recommendations on their analysis and a more precise definition.

The simulation of the design schemes allows to obtain an obvious situation of the construction stress and deformation state both nowadays and originally. Besides it allows to reveal the reasons of the destruction and to forecast durability and reliability with a definite degree of approximation.

The simulation is implemented in such a sequence:
1. according to the remained part of the construction a space rod design scheme is created as being the most visual one;
2. the geometrical characteristics of the conventional rods are specified so as to bring the work of this design scheme to the work of the real construction as near as possible;
3. the massive parts of the construction are simulated by volume finite elements.

The finite element method and the superelement method are used for the analysis of constructions. It is recommended to use separate parts of the construction as superelements. These parts can be separated out of the construction according to the definite sign and calculated by any method indepedently.

It is proposed to consider the following kinds of superelements:
1. base superelements - the parts of the structure having sufficient strength and degree of stability;
2. unstable superelements - the parts of the structure working under unfavourable conditions, having heavy damages and increased value of the flexibility;
3. cantilivers - various parts of the structure, being in breakdown state, partly destroyed, having different cross sections, representing themselves as handing down parts of structure at different angles. They may be destroyed parts of archs, frames, plates and continuous beams, separately standing columns and other elements.

The cantilivers may be classified and reduced into several types for which research work can be done and recommendations can be made to determine their carrying capacity.

The analysis of constructions may be performed in two directions: by carring out strength verification of construction parts and by finding out possible reasons of its destruction. While realizing the first direction of analysis besides static calculations great attention is paid to dynamic and seismic calculations. It acquires especial significance if calculated constructions are located in seismic dangerous region such as Mediterranean, Transcaucasus, Middle Asia and other regions.

In the second case analysis is made by defining the possible variants of the foundation settling or by the strength factors application. According to these data theoretical situation of destruction is defined and these results are compared with real situation.

The paper presents analysis of continuous beams, frames and cantilivers with axis in the form of a broken space line, taking into account that geometrical characteristics are changed according to arbitrary law. Stiffness matrixis are determined for these cases.

Geometrical Shape of Stone Arches

Forme géometrique des arcs en maçonnerie

Geometrische Form von Mauerbögen

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Despite it could appear obvious, the effective structure of an arch made by stone voussoirs seldom fits its geometrical shape. In this paper it is shown that, in particular cases such as the round arches, structural stability almost always requires static collaboration of the backfill. As it is known, successful design of a masonry arch depends more on its geometrical shape than on the characteristics of the materials. Indeed, in order to state the stability of this kind of structures, a thrust line of acting loads lying wholly within the arch profile needs to be found (if it exists); in particular, the closer the thrust line fits the arch centreline, the better is the shape designed.
This kind of circumstance can never be verified dealing with round arches under gravitational loads, considering that the affinity between the thrust line of the loads and the respective bending moment diagram of a simply supported beam having the same span excludes vertical tangents at the ends. With reference to figure 1, let us consider the subsequent arches, featuring cross-section of uniform dimensions, obtained by increasing the angle α and leaving unchanged the bending radius R and the depth s. At the beginning, being α small, it is always possible to find a thrust line of the applied loads (self-weight, backfill and overload) lying wholly the arch outline; this is true, in general, up to a certain value α^*. The problem can be faced defining the minimum thickness needed to contain the thrust line, once fixed R and per each value of α.
The minimum depth can be defined as follows: let us suppose that depth is exactly equal to the minimum one, then just one line of thrust lying wholly within the masonry can be found. This line touches the arch profile in a finite number of cross-sections. Being α small, one can observe that the thrust line touching the arch profile intrados at the crown and at the springing is wholly above the intrados. This consideration suggests the way to find the minimum thickness: if the depth is equal to the minimum one, the thrust line touches the arch profile intrados at the crown and at the springing, as well as the extrados at the haunches. If the thrust line touches the arch outline in these points, the structure turns into a mechanism. Being α greater than a certain value α_1, the collapse mechanism concerns only a part of the structure, defined just by α_1, as, in general, the thrust line is included in the arch profile up to an angle value $\alpha_2 < \pi$; therefore the minimum thickness found for $\alpha=\alpha_1$ keeps unchanged up to this limit (figure 2). In order to determine the minimum thickness beyond this threshold (α_2), one should refer to the condition that the thrust line be within the arch outline near the springing, as there is no more question of mechanism type failure.
To the purposes of the present paper, a numerical code for the automatic research of the minimum thickness has been written, solving the problem both in the case of mechanism type criterion and in the case of thrust line exceeding the arch profile near the springing [3,4]. A comprehensive numerical investigation has been carried out to show the influence of each parameter concerning the problem, whose major results are shown in figures 3 and 4. In the first one (fig. 3) the lines corresponding to the values of s_{min}/R (minimum thickness over bending radius) obtained varying α are plotted for

different values of the ratio γ between the stone self-weight and the backfill weight; in this figure the overload height h is equal to 0. Figure 4 shows the same lines plotted for different values of h, representing an uniform load over the arch, being γ=0.5. As estimated, the curves show three ranges of α. In the first one s_{min}/R increases with α up to a certain value, say s_1, corresponding to α_1. Being α in the range defined by α_1 and α_2, s_{min}/R keeps unchanged to s_1. When α increases beyond α_2, and up to the maximum value π, s_{min}/R fast climbs to exaggerated values. As in general stone arches do not feature such large values of the thickness, one can state that, in these circumstances, just the static collaboration of the backfill assures the structural stability.

The threshold values α_1 and α_2 can be considered not very variable; they can be assumed to be equal respectively to $2\pi/3$ and to $\pi/1.1$.

The diagram showed can be used to assess the stability of an arch, once known its bending radius and the loads. The ratio between the actual depth and the minimum one provides a measure of the safety degree of the structure. Should the actual thickness be smaller than the minimum one, the structural stability can be thought entrusted to the static collaboration of the backfill near the springing. One has to stress the role played by the backfill in defining the acting loads as well as in participating to the effective structure of stone arches.

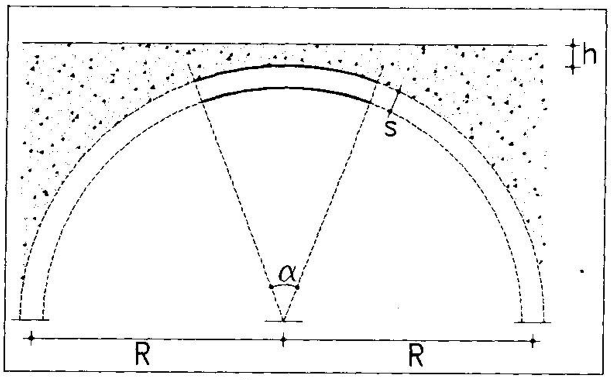

Fig. 1

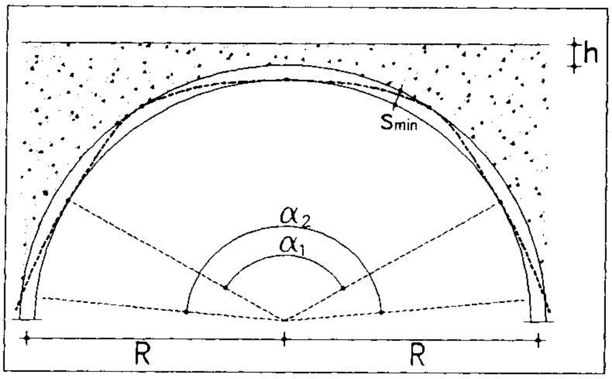

Fig. 2

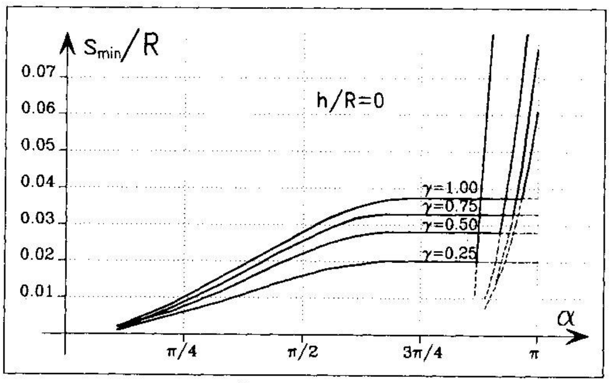

Fig. 3

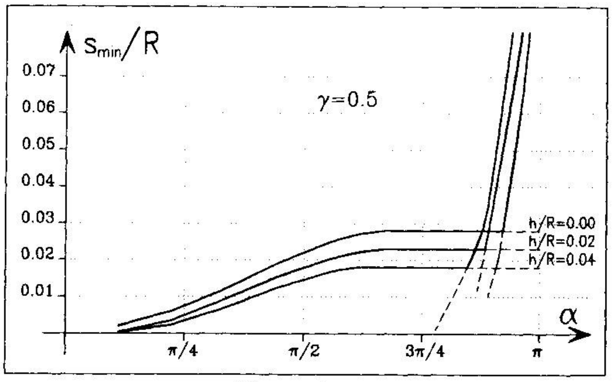

Fig. 4

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Dynamic Analysis of Church Masonry

Analyse dynamique d'une église en maçonnerie

Dynamische Berechnung von Mauerwerkskirchen

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1. INTRODUCTION

For the acient masonry wall, we neglect the tensile strength and fracture energy, due to the time damaging. Tensile failure occurs when one tensile principal stress tends to become positive. In this case a plane of failure develops at right angle to the previous principal direction and is conserved its orientation at onset of cracking for the whole loading process. Subsequent failure planes could be only orthogonal to the first and between them. After tensile failure, the normal coefficient in local stiffness matrix is abolished and shear coefficient is multiplied by a constant retention factor less than unity. The nonlinear uniaxial compressive behavior is defined by initial Young's modulus E_o, crushing point $C(\varepsilon_c, \sigma_c)$, ultimate point $U(\varepsilon_u, \sigma_u)$. In multiaxial compressive state compression, the crushing and the ultimate points of uniaxial test may be enhanced relating to the projection on the triaxial failure envelope. Unloading from a compressive state is parallel to the initial Young's modulus. Isoparametric plane elements with a maximum of eight nodes and four integration points in each direction are used. The tangent stiffness matrix is referred to:

- principal stress direction before tensile failure;
- failure coordinate system (axes parallel and transverse to the crack planes after tensile failure.

Various acient buildings are taken into account. Different accelerograms are used, a few real, other obtained by power spectra according to current rules. The Newmark's integration method is used obtaining: stress and strain fields, crack patterns, displacements velocities and accelerations in each node. In conclusion, a method apt to analyse dinamically any masonry building with great accurancy is proposed.

2. PROBLEM DESCRIPTION.

ADINA Code is used. A church masonry is taken into account, having span and height 10.20m, depth 3.90m. The vertical walls are stone made and in the upper arch and over there are bricks. The materials mean features are: E_o=1000. N/mmq, σ_c=-3. N/mmq, ε_c=-.01, at crushing and tension cut-off. The Model and F.E. mesh are represented in Fig. 1. Accelerations are applied as in Fig. 2, having maximum value 0.2g. The Newmark's integration method is used obtaining the nodal results as in Figs. 3-5.

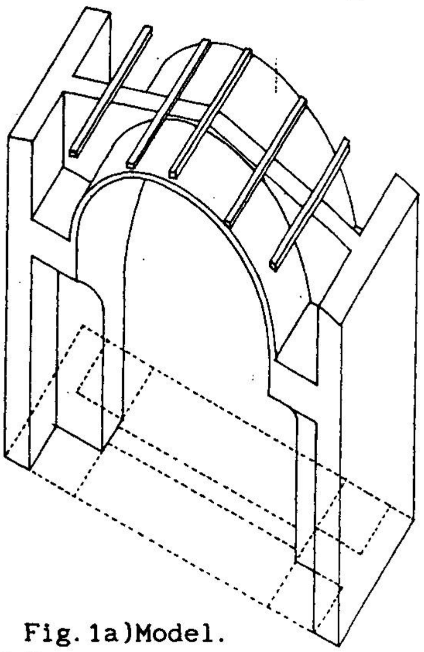

Fig. 1a)Model.

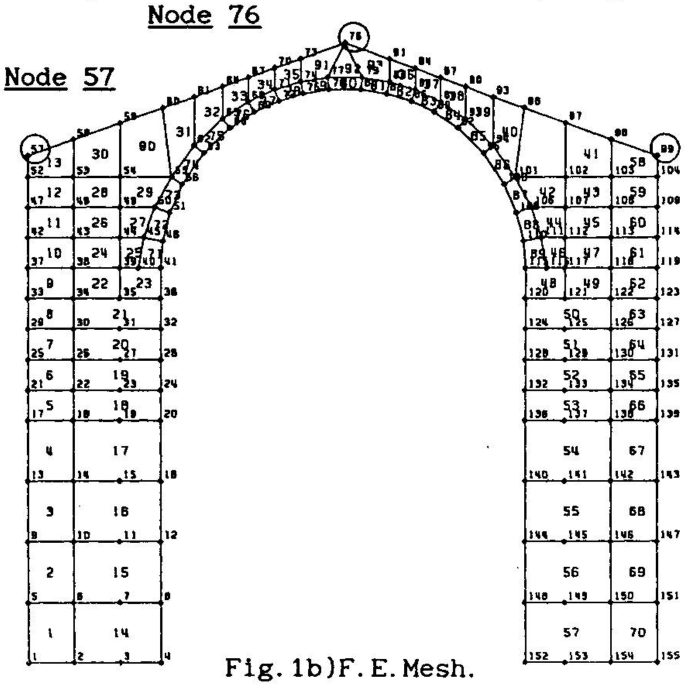

Fig. 1b)F.E. Mesh.

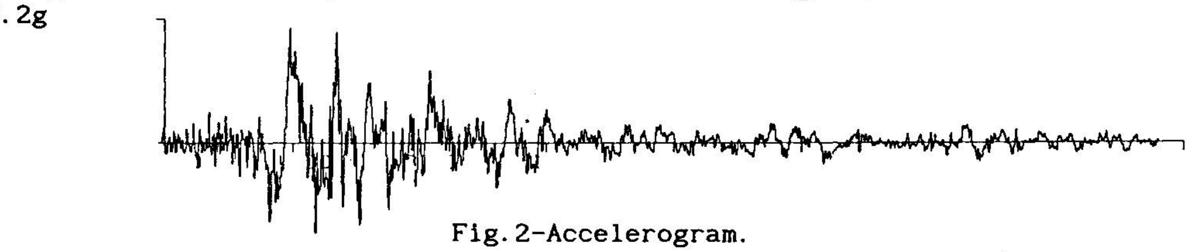

Fig. 2-Accelerogram.

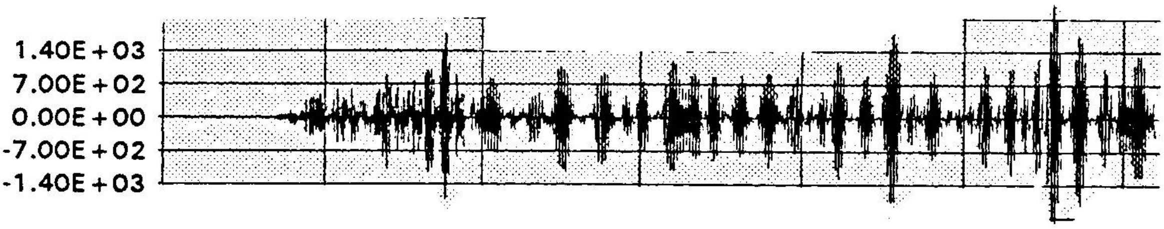

Fig. 3-Accelerations Node 76, [cm/sec^2].

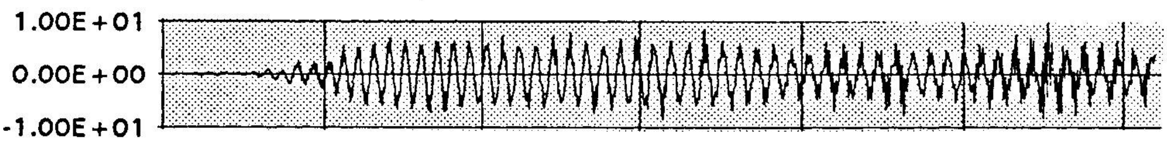

Fig. 4-Velocities Node 76, [cm/sec].

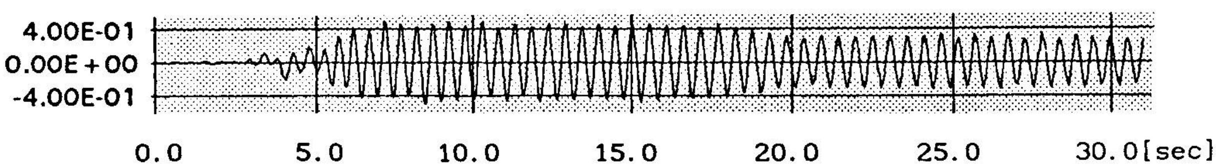

Fig. 5-Displacements Node 76; [cm].

Structural Analysis of the Damage at the Cathedral of Sibenik

Calcul statique des dégâts de la cathédrale de Sibenik

Statische Berechnung der Schäden an der Kathedrale von Sibenik

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Structural analysis of the octagonaly shaped ribbed dome of cathedral of Šibenik, a masterpiece of the Croatian Quattrocento architecture, damaged by a bomb-shell in 1991. contains a condensed review of its structure, a brief art-historical analysis and the analysis of the mechanical characteristics as well as behavior of its structural mechanism. Special emphasis is placed on the spatial, formal and structural significance of its vaults erected with the peculiar technique, unique in its kind. The barrel vaults with the thin one-layer plates, shells, which also create the roof of the church, are constructed by mounting the large thin stone slabs on the relatively slender semi-circular stone arches tightened with iron ties. It is clearly a prefabricated structure. This structure, which seems formally Renaissance, but is structurally Gothic, for its plain skeleton, its clear distinction and hierarchy of primary and secondary structural elements, is used even for the dome structure. The dome, generally a spatial structure par exellance, here is carried out as a system of planar arches, converging into one point, and the envelope, acting as a covering, consists of stone slabs, like that one of the barrel vaults.

The main objective of this research was the FE computer modeling and analysis of the dome, demaged by a bomb- projectile explosion. The modeling and the static FE analysis was carried out using the COSMOS/M FE program. As we have suspected, the demage, a circular hole in the upper part of the stone slab-shell dome system has not much consequences on the carrying capacity of the dome. Luckily that it is on the part of the dome where the stresses are low, and the deformations extremely small. Fortunately, it is not a structurally dangerous injury which could jeopardize the stability of the Chatedral because only a secondary element was damaged. This could be seen on the presented screens. Only by chance the projectile has missed the rib: there could be done a devastating effect to the dome stability, if the rib has been hit and demaged.

We have had another objective before the 1991. and the well known aggression on Croatia: to build up a comprehensive FE model of the whole Cathedral. But with the war going on, and the drastically reduced foundings, as well as the damage done to the dome, the attention was shifted to the macroelement modeling of the damaged dome only.

Because of the peculiarity of the problem (nonhomogeneous masonry structure, nonresistant to tensile stresses), different variations of modeling have been made and examined, to approximate as close as possible the real behavior of the stone structure. Some of the most comprehensive classical approach to the stability and force-stress distributions was done by M. Šimunić in her M.Sc. thesis (in 1990.), and was accompanied by some limited FE model simulations of the dome's stone slabs (shell) behavior (using IBM v. ICES STRUDL2 and COSMOS/M). The structural role of some elements of the fabric is examined in particular, creating alternative models of the structure by changing and eliminating those elements. The preliminary structural analysis of the whole structure have proved that the structure is very logical and purposeful and that every element, even the decorative ones, has a precisely determined structural function.

The further work on a more comprehensive analysis is still going on. Displayed are some of the obtained CAD/COSMOS/M screens of the work in progress.

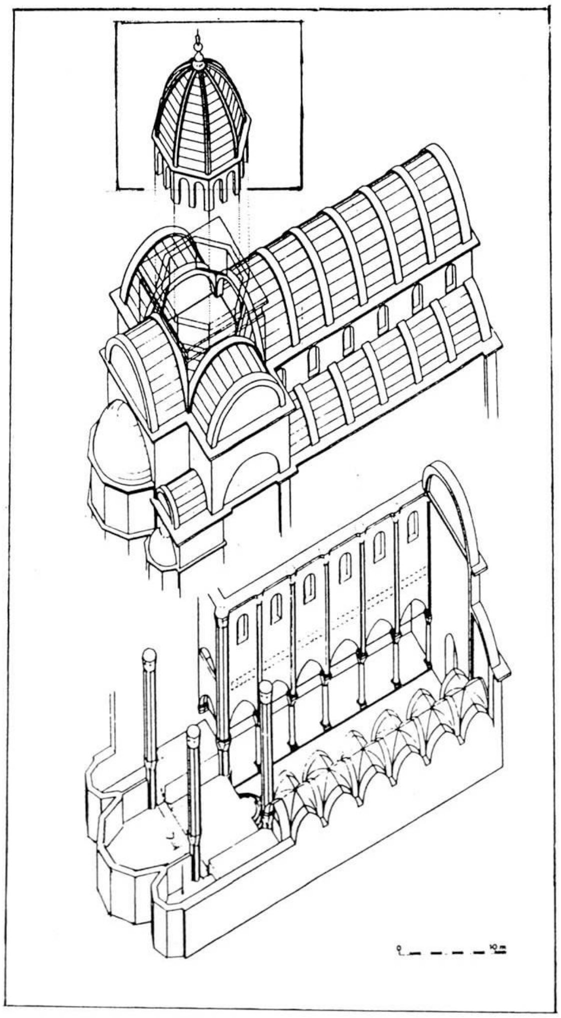

Fig. 1. Vaults of the Cathedral of Šibenik and the substructures.

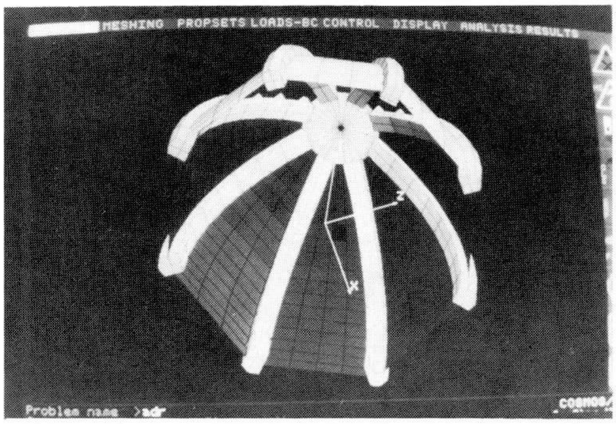

Fig. 2. The FE mesh of the dome-macroelement. The damaged part (a hole) is clearly visible.

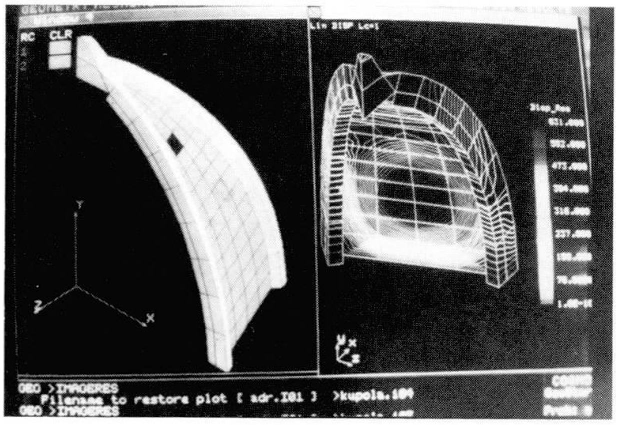

Fig. 3. The deflection lines of the substructure due to the selfeight.

Soil - Structure Interaction in the Leaning Tower of Pisa

Interaction du sol et de la structure de la Tour Penchée de Pise

Wechselwirkung Baugrund - Konstruktion beim schiefen Turm in Pisa

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The main aspects of the time-dependent behaviour of the leaning tower of Pisa are: creep and the non-linear response of the soil; second order effects due to the increase in leaning; the progressive reduction in the loaded area.

The model shown in Fig. 1 has been developed to take all these aspects adequately into account, by making the following assumptions: a) the structure of the tower is a rigid body; b) the foundation area is subdivided into 19 strips, arranged perpendicularly to the leaning plane; c) the soil under each strip is a Maxwell element, consisting of a non-linear spring and a non-linear dash-pot, applied to the strip's centre of gravity.

The response of the elements of the chain (for a unit of area) is illustrated in Figs. 2a and 2b (for non linear springs and non-linear dash-pots, respectively).

The displacement of a point of the foundation can then be expressed as:

$$y_i\,(t_K) = a\,(t_K) + b\,(t_K) \cdot x_i$$

where $a(t_K)$ and $b(t_K)$ denote the displacement of the foundation centre and the rotation at time t_K, respectively.

The equilibrium equations during the time step $\Delta t_K = t_K - t_{K-1}$ are:

$$\begin{aligned} \Delta a(t_K)\sum_{i=1}^{19} K_i A_i + \Delta b(t_K)\sum_{i=1}^{19} K_i A_i x_i - \sum_{i=1}^{19} \Delta y_i^c(t_K) K_i A_i &= \Delta N(t_K), \\ \Delta a(t_K)\sum_{i=1}^{19} K_i A_i x_i + \Delta b(t_K)\sum_{i=1}^{19} K_i A_i x_i^2 - \sum_{i=1}^{19} \Delta y_i^c(t_K) K_i A_i x_i &= \Delta M(t_K). \end{aligned} \tag{1}$$

where

$$\Delta y_i^c(t_K) = \sigma_i(t_{K-1})\beta_i\left[\log t_K - \log(t_{K-1})\right],$$

is the increase in displacement produced by the dash-pot.

The stiffness of the non-linear springs has been continuously updated according to the stress level, as shown in Fig. 2a, cutting off the elements in tension. Similarly, the external moment $\Delta M(t_K)$ on the right-hand side of eqs. (1) increases with increasing tilt (second order effect), while both the axial load and the external moment could vary if some stabilising measures were applied. Convergence has been reached, at each time step, through an iterative procedure.

The range of non-linear spring and dash-pot parameters has been deduced from the extensive geo-technical tests performed on the soil near the tower [1]; final calibration has been obtained by comparing the slope history provided by the model and the measurements taken on the tower since 1550 [2].

In Fig. 3, the tilt vs. time diagram obtained from the model is compared with the available measurements. Agreement seems to be very satisfactory; in particular the simultaneous effects of the reduction in the loaded area and second order effects produces a strong increase in the tilt rate, in keeping with the observations made since 1970.

The model makes it possible to predict the future behaviour of the tower and to assess its response to external actions. If no stabilising measures are taken, the tower will collapse between 2030 and 2040, due to divergence of equilibrium.

A simulation of a very soft stabilising intervention, consisting in the application of a vertical force of 2123 kN at a distance of + 6.83 m from the centroid of the foundation, is presented in Fig. 4. This would achieve a very small instantaneous reduction in tilt, but its delayed effect would be considerable, as a number of springs and dash-pots would return from the plastic to the elastic field, leading to a substantial stabilisation of the tower.

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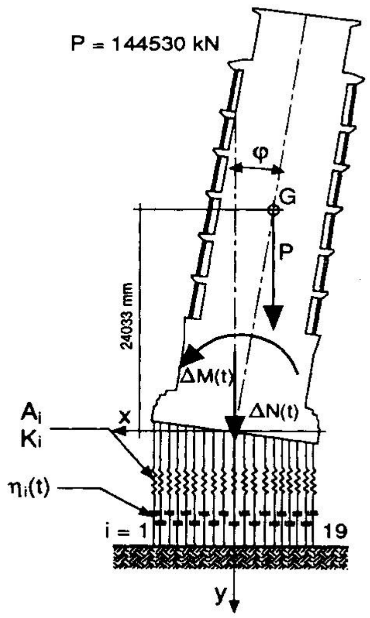

Fig. 1 - Soil-structure model.

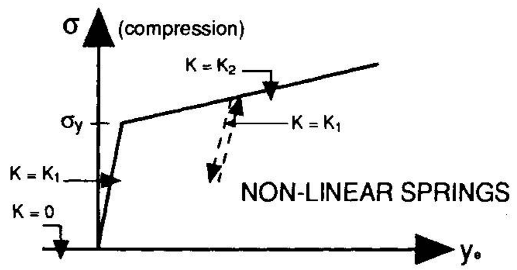

Fig. 2a - Stress/displacement relationship in non linear springs.

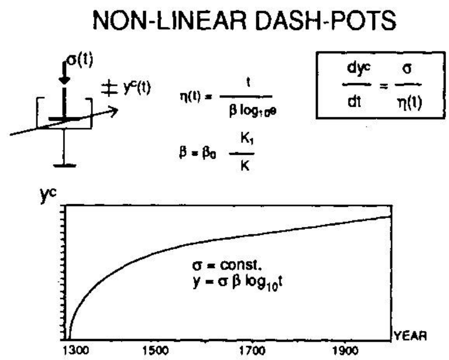

Fig. 2b - Stress/time/displacement relationship in non-linear dash-pots.

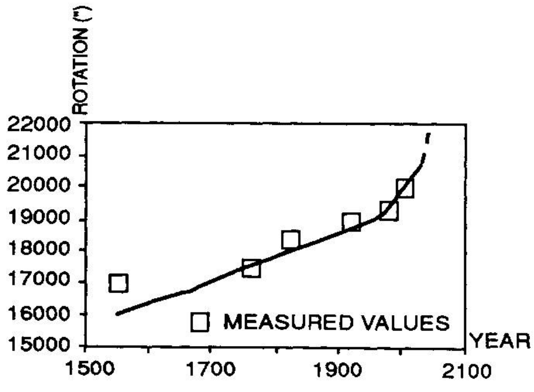

Fig. 3 - Rotation/time diagram and comparison with measured values.

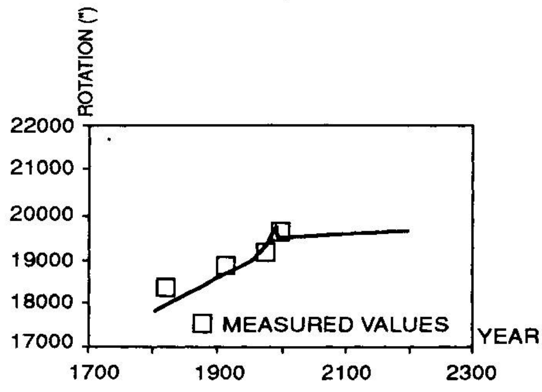

Fig. 4 - Instantaneous and delayed effects of the stabilisation measures.