Leaning Tower of Pisa — Updated Information

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SYNOPSIS : The paper is aimed at giving information on the present situation of the Leaning Tower of Pisa and on the activities undertaken for its safeguard by the International Committee appointed in May 1990 by the Italian Government.
After a brief review of the subsoil conditions of the structural features and of the observed movements of the Tower, the activities undertaken by the Committee are also summarized.

INTRODUCTION

This paper is aimed at presenting updated information on the Leaning Tower of Pisa. This world-famous curiosity part of the beautiful historical complex of Piazza dei Miracoli in Pisa - see Fig.1, has been subject, since its erection in 1173, to a progressive tilt reaching nowdays the alarming value of 5°28'09" ($\simeq$10%).
Such phenomenon has been thoroughly studied by the International Geotechnical Community [e.g. Ministero dei Lavori Pubblici (1971, 1979), Mitchell et al. (1977), Croce et al. (1981), Berardi et al. (1991), AGI (1991), Di Stefano and Viggiani (1992)] but in the last five years great concern has also grown over the structural integrity of the Monument.
This problem, rather than the increase of the inclination has prompted the Committee chaired by professors R. Jappelli and P. Pozzati, to close the Tower to visitors in 1989.
Following this decision, which caused great sensation, the Italian Government appointed, in October 1990, a 15-member multidisciplinary Commission charged with taking all necessary actions to safeguard the Tower.
In the first part of the paper some hints on the history of the Monuments will be given, thereafter the main issues concerning subsoil conditions, structural features and the observed movements of the Tower will be addressed.
In the last part of the paper an attempt will be made to clarify the mechanism at the base of the constant increase of tilting and an account of the work carried out so far by the present Committee will be provided.

HISTORICAL BACKGROUNDS

The Monuments of the Piazza dei Miracoli have been erected in the Middle Ages. The first one to be constructed was the Cathedral in late 1000. The design of the Tower, see Fig.2, is ascribed to the Architect and Sculptor Bonanno Pisano.
It consists of a hollow cylinder, surrounded by six loggias with columns and vaults merging from the base cylinder.
Inside the annular masonry body a helicoidal staircase leads to the bell chamber located at the top of the monument.
Its construction started in August 1173 but the works were interrupted at the middle of the fourth order, see Fig.3, apparently more for political reasons rather than for its leaning.
The construction was resumed in 1272 by the Architect Giovanni di Simone and in six years the Tower was brought almost to completion, up to the seventh cornice "ricorso", (tiers of stones of which the Tower facing is made).
It was during this construction phase that the deviation of the Tower axis began to appear, see Fig.4, reflecting the attempt by the "magistri lapidum" (the masons charged with the construction works) to correct the constant inclination which evolution can, in first approximation, be evaluated from the progressive change in thickness of the stone element of said "ricorso", moving southwards.

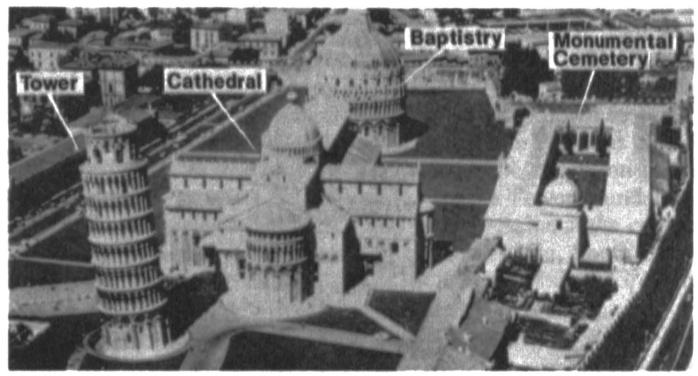


Fig. 1. Piazza dei Miracoli - Air view.

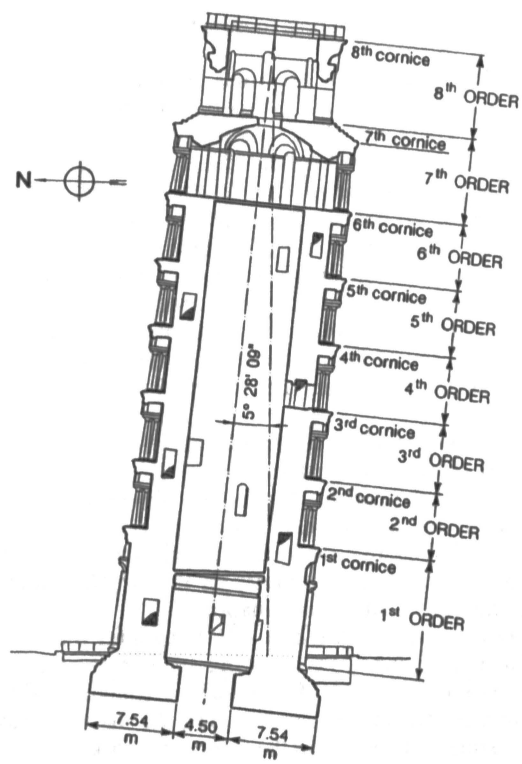


Fig. 2. Leaning tower of Pisa.

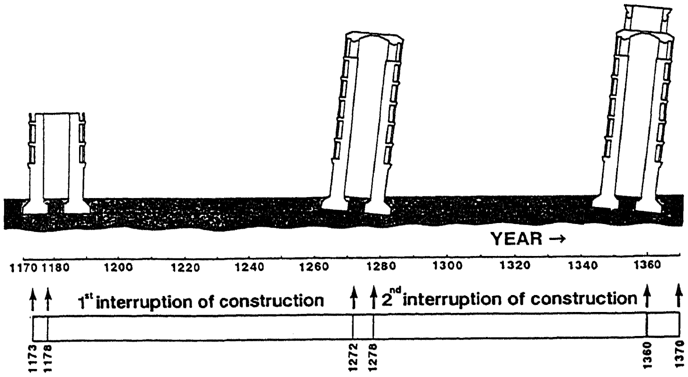


Fig. 3. Construction history.

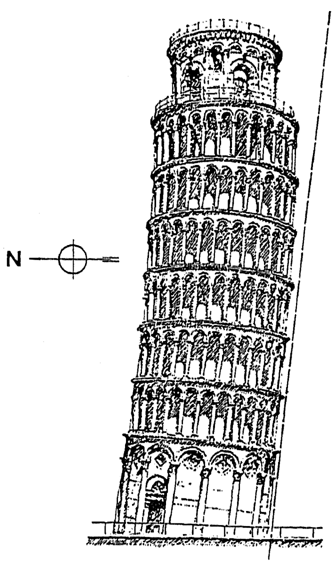


Fig. 4. Correction during construction.

It is not clear what were the rules followed by the "magistri lapidum" in the attempt at compensating for the lean during construction. There are clear indications that during the second construction stage the inclination of the Tower became evident.

In 1278 the construction of the Tower was again interrupted. Historians of the art believe that once more it was due to political problems and not to concern for its stability.

In 1292 a group of "magistri lapidum" led by Giovanni di Simone's son was appointed for measuring the inclination of the Tower by means of the plumb line method. The group may by right be considered as the precursor of the many Committees and Commissions that have ever since followed for centuries one upon the other.

Their work is historically well documented but unfortunately there is no record of the measured values.

Starting in 1360 the construction of the Tower was finally completed in 1370 by the Architect Tommaso di Andrea Pisano who added the bell chamber in such a position to testify a further attempt at correcting the geometry of the structure and at compensating for the occurring inclination.

An endeavour made by Burland (1991) to infer the evolution of the inclination of the Tower during its construction from the variation of the thickness of "ricorsi" is shown in Fig.5. This figure assumes that during construction the masons made continuous adjustments to maintain the floors at each storey horizontal. From Fig.5 and based on the data previously shown by Cambefort (1978) and Leonards (1979) it appears that the Tower has been subject, during its construction, to random inclinations in different direction. Only during the second stage of construction, the southward leaning became evident and the movement in such direction has been continuous ever since.

Considering the weight of the structure, 144.53 MN, the geometry of its foundation, see Fig.6, and the geotechnical characteristics of the subsoil, it appears that both interruptions of the construction were timely and providential events preventing the undrained bearing capacity failure of the Tower.

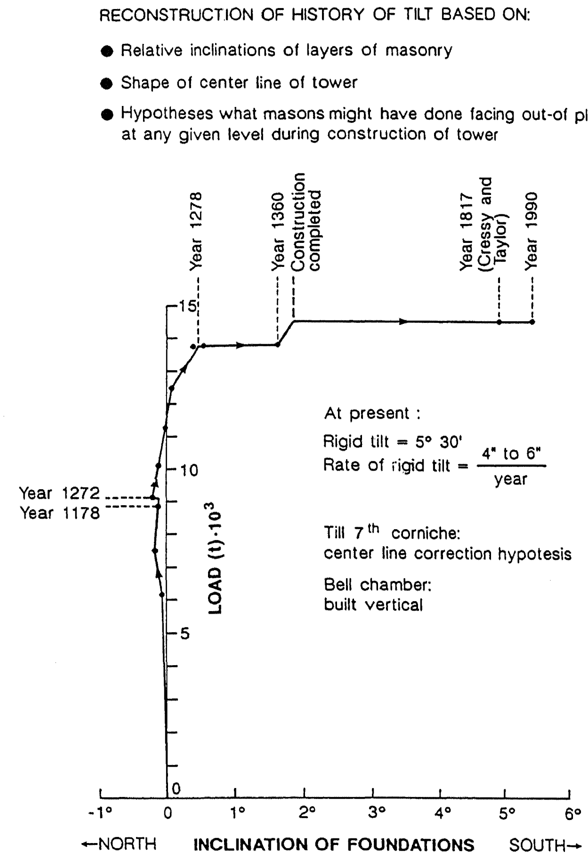


Fig. 5. History of rigid tilt *(Burland, 1991).*

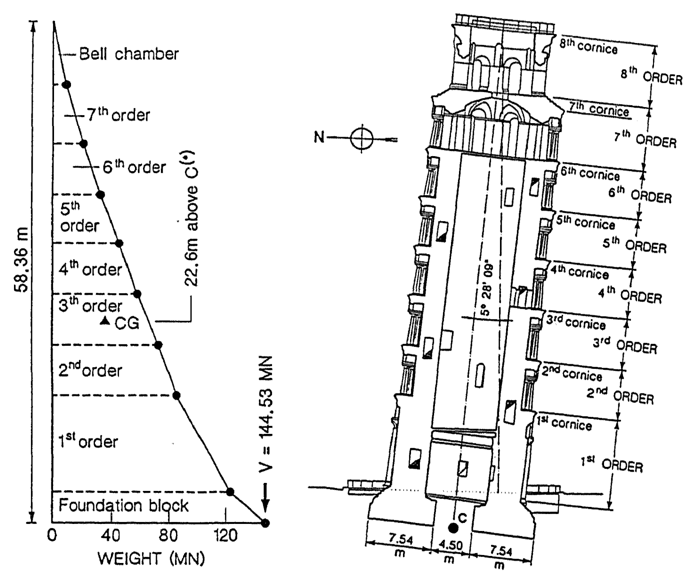


Fig. 6. Weight and geometrical characteristics of Pisa Tower.

[herefore, even if unintentional, the erection of the Tower re-
›resents an excellent example of stage construction which might leserve, with the presently available data concerning the contruction history and the soil properties, a careful re-examination ›y means of appropriate numerical analyses.

;UBSOIL CONDITIONS

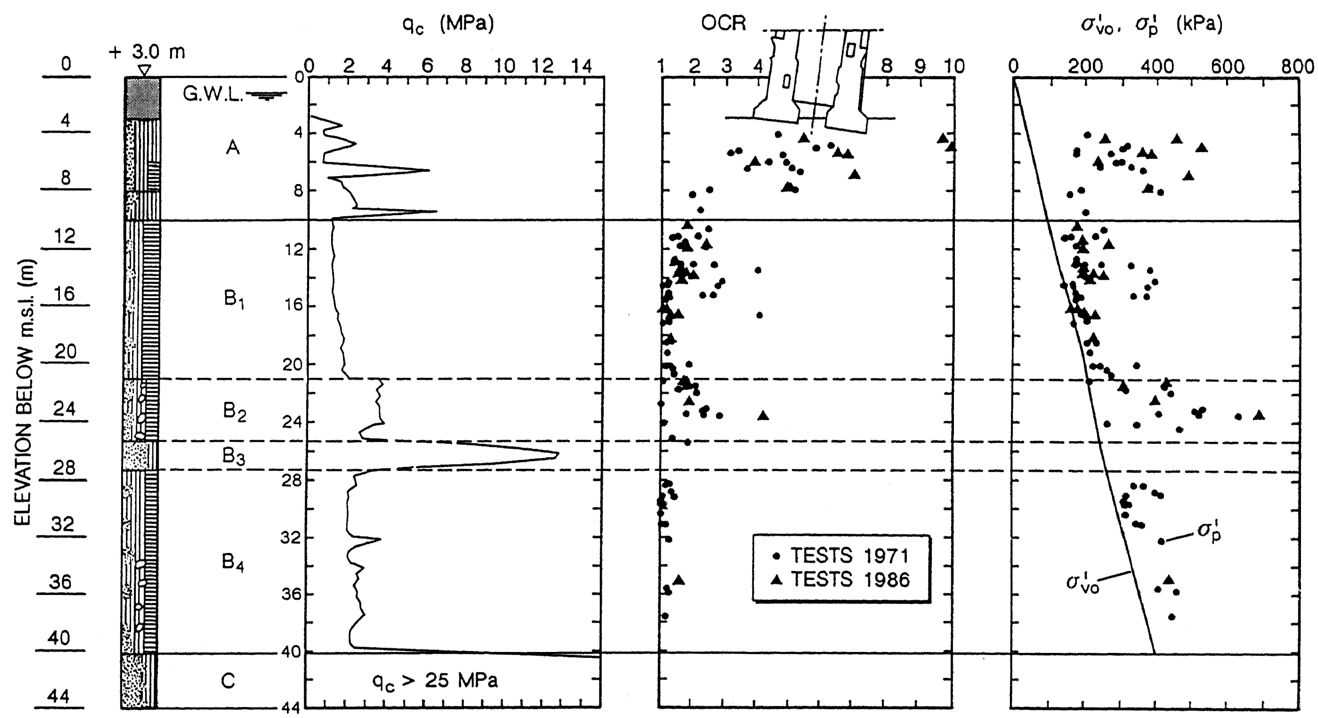
Geotechnical exploration of the soil underlying the Pisa Tower ıave been performed in different times starting from early 1913. The most comprehensive studies have been carried out by the Pol-'ani Commission and published by the MLP (1971).
;ubsequently, in late middle eighties another Committee chaired ›y professors Finzi and Sanpaolesi carried out further investigaions. The results, that have only partially been published Lancellotta and Pepe (1990), (1990a), (1991), Berardi et al. 1991), AGI (1991), Jamiolkowski (1991)], allow rather a com-›rehensive description of the subsoil profile and of the ;eotechnical characteristics of the soils under the Piazza dei Miracoli, nearby the Tower.
t might be worth pointing out that, if the soil profile directly ınder the Tower is known to a depth generally not exceeding 20 n, however, deeper borings and almost all laboratory stress-strain nd strength tests have been obtained for soils at a distance of at east 10 to 30 m from the Tower because of the concern about its tability. In these circumstances it must be kept in mind that the ;eotechnical characterization obtained from the most recent invetigations refer to the soils not influenced by the increase of tresses imposed by the weight of the Tower.
ıccordingly to the terminology adopted by the Polvani Commision, the following three main formations, belonging to the Iolocene and Pleistocene age, may be identified, see Fig.7.

Formation A; consists of slightly clayey and sandy yellow silt with interbedded lenses and layers of sand and clay. It is worth mentioning that at the bottom of this formation a layer of medium uniform grey sand is encountered. Based on the results of numerous shallow borings and static cone penetration tests (CPT) it appears that the thickness of this layer decreases moving southwards, see Fig.8.

Such formation is covered by a $\simeq$3 m thick layer of top soil containing archaeological findings whose age covers a range from the 8th century B.C. up to the 5th century A.C., extends to elev. -7.0 below m s.l..

Formation B; predominantly clayey, can be subdivided in the following four layers:
- layer B_1; from elev. approximately -7.0 to -18.0 below m.s.l., upper clay, named locally Pancone clay;
- layer B_2; from elev. -18 to -22.5, intermediate clay;
- layer B_3; from elev. -22.5 to -24.5, intermediate sand;
- layer B_4; from elev. -24.5 to -37.0, lower clay.

Formation C; slightly silty sand, which extends at least to a depth of 65 to 70 m below G.L. depth of 120 m reached during the geotechnical investigation.
At greater depth a further cohesive formation is known to occur. The boundary between the formation A and the Pancone clay is horizontal all over Piazza dei Miracoli with the exception of the bowl shaped depression encountered under the Tower foundation, see Fig.9. Referring to its centerline, the depression at the contact between formations A and B exceeds 2.5 m, while, South of the Tower the indication of a heave of $\simeq$ 0.4 m of the aforementioned contact can be also envisaged [Leonards (1979)].
Although any detailed geotechnical characterization of the soil



• Data collected by G. Calabresi (University of Rome)
▲ Lancellotta and Pepe, 1990a

Fig. 7. Soil profile and stress history of soil underlying Pisa Tower.

layers underlying the Tower is beyond the scope of the present paper, however, some of data collected up to 1988 are reported to give an overview of the properties of the foundation soils.

- Index properties of the layers belonging to formations A and B are summarized in Table 1, and Fig.10 showing the Casagrande's plasticity chart.
- Stress history of the cohesive layers i.e. the preconsolidation pressure σ'_p and the overconsolidation ratio OCR, encountered within formation A and B can be inferred from Fig.7.
  The preconsolidation mechanisms involved are probably linked to the groundwater level oscillation, secondary compression and within the formation B to some $CaCO_3$ cementation. In addition, the desiccation phenomena may have played some role within the layer of intermediate clay.
- Coefficient of earth at rest of Pancone clay in normally consolidated (NC) state K_0^{NC} results on average equal to 0.58. While, assuming that within this layer the predominant overconsolidation mechanism is the secondary compression, the best estimate of the K_o in the field within upper Pisa clay should be around of $\simeq 0.75$. This value has been obtained, referring to works by Mesri and Castro (1987) and Mesri (1989), adopting $C_{\alpha e}/C_c$ and C_c/C_r ratios equal to 0.0363 and 0.1 respectively giving an upper limit of OCR due to the secondary compression equal to 1.26, being: $C_{\alpha e}$, coefficient of secondary compression, C_c and C_r, indexes of primary compression and recompression respectively.
- Compressibility of the cohesive materials has been determined via oedometer tests. A typical example of such tests is shown in Fig.11 while, Fig.12 reports the virgin compression index λ_1 and recompression index λ_2 obtained from log(1+e) vs log σ'_v plot being e and σ'_v, void ratio and vertical effective stress respectively.
  From Fig.12 it can be noticed that both λ_1 and λ_2 can be correlated against the plasticity index PI of the tested samples.
  Fig.13 shows the results of oedometer tests performed on specimens of Pancone clay compared with the intrinsic (ICL) and sedimentation (SCL) compression lines of the same material. According to Burland (1990) the former represents the compressibility of the resedimented clay while the latter corresponds to the relationship between void index:

$$I_v = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}}$$

and log σ'_v for truly NC natural clay in situ at the end of primary consolidation by gravitational compaction, being e_{100} and e_{1000} void ratio of: $\sigma'_v = 100$ and $\sigma'_v = 1000$ kPa respectively.
The compression curves of undisturbed samples for $\sigma'_v > \sigma'_p$ are significantly steeper than ICL and SCL, and only at σ'_v one order of magnitude larger than the preconsolidation stress tend to merge into SCL. This phenomenon reflects the importance of the structure of the Pancone clay at its natural state.
The results of oedometer tests have led to the following average figures that may characterize the compressibility of the cohesive layers belonging to A and B:

Formation A:	$\lambda_1 = 0.053 \pm 0.020$;	λ_2	$= 0.0080 \pm 0.0035$
$0.68 \le e \le 0.94$	$C_{\alpha e} = 0.0037 \pm 0.0019$;	$C_{\alpha e}/C_c$	$= 0.0145$
Pancone clay:	$\lambda_1 = 0.13 \pm 0.026$;	λ_2	$= 0.013 \pm 0.0060$
$1.32 \le e \le 1.75$	$C_{\alpha e} = 0.021$	$C_{\alpha e}/C_c$	$= 0.036$

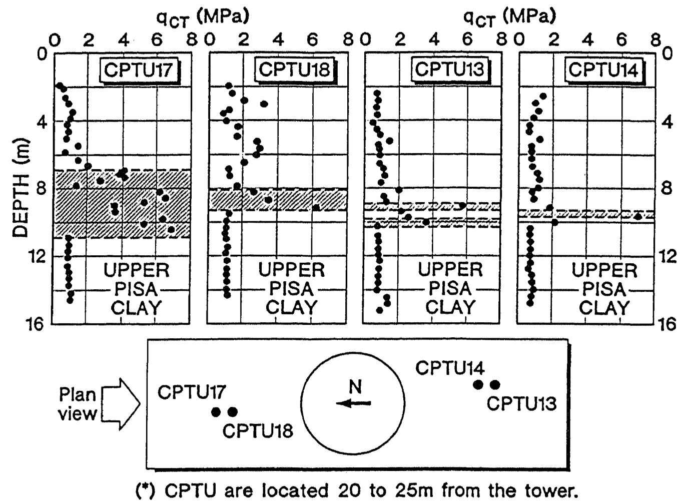


Fig. 8. Cone resistance within formation A. North-South cross-section.

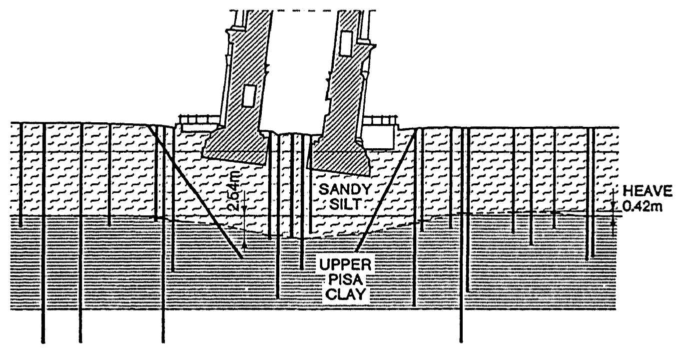


Fig. 9 Settlement and heave of surface of upper Pisa clay *(Leonards, 1979)*.

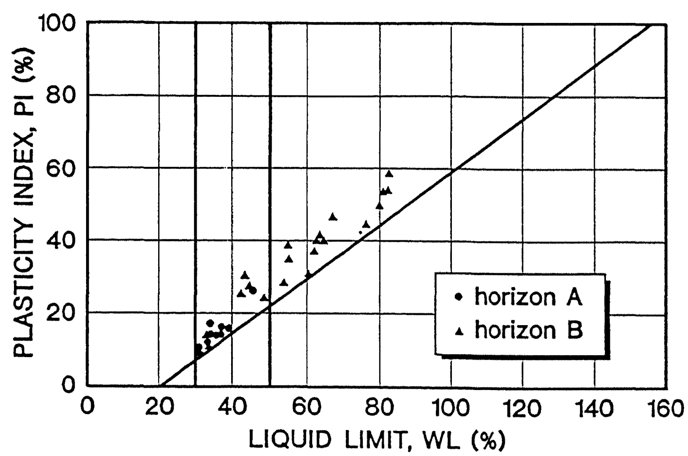


Fig. 10. Plasticity chart of soil underlying Pisa Tower.

- Undrained shear strength s_u of the Pancone clay as determined from the DSS-CK$_o$U tests led to the following empirical relationship:

$$\frac{s_u}{\sigma'_{vo}} = 0.23\,(OCR)^{0.84}$$

being:
OCR = overconsolidation ratio varying between 1 and 2.7.

- Effective stress shear strength envelope as obtained from drained-triaxial compression tests performed on both isotropically and anisotropically consolidated specimens is shown in Fig.14. Both deviator stress t' and mean plane strain effective stress s' have been normalized with respect to the σ'_p.
  The experimental results allow the following comments.
  - For specimens reconsolidated beyond the in situ σ'_p, only a small difference exists between peak and large strain strength.
  - For specimens reconsolidated to or below the in situ σ'_p, there is a substantial difference between peak and large strain strength.
  - For all specimens, the values of t at large strain form a unique straight line envelope with a zero cohesion intercept, and a value of the friction angle $\phi' = 25°$. This line corresponds to the critical state line (CSL) in the Cambridge p'-q plane.
  - The values of t' at peak, for specimens reconsolidated below σ'_p, form a curved strength envelope located above the large strain envelope, as predicted by the Hvorslev failure criterion.
  - Normalizing the peak strength envelope with respect to the equivalent pressure p'_e [Hvorslev (1937)], the Hvorslev surface is obtained; the effective cohesion intercept $c' = 0.029\ p'_e$ results a function of the water content at failure w_f. For the upper Pancone clay Lancellotta and Pepe (1990) indicated the following typical strength parameter at peak:

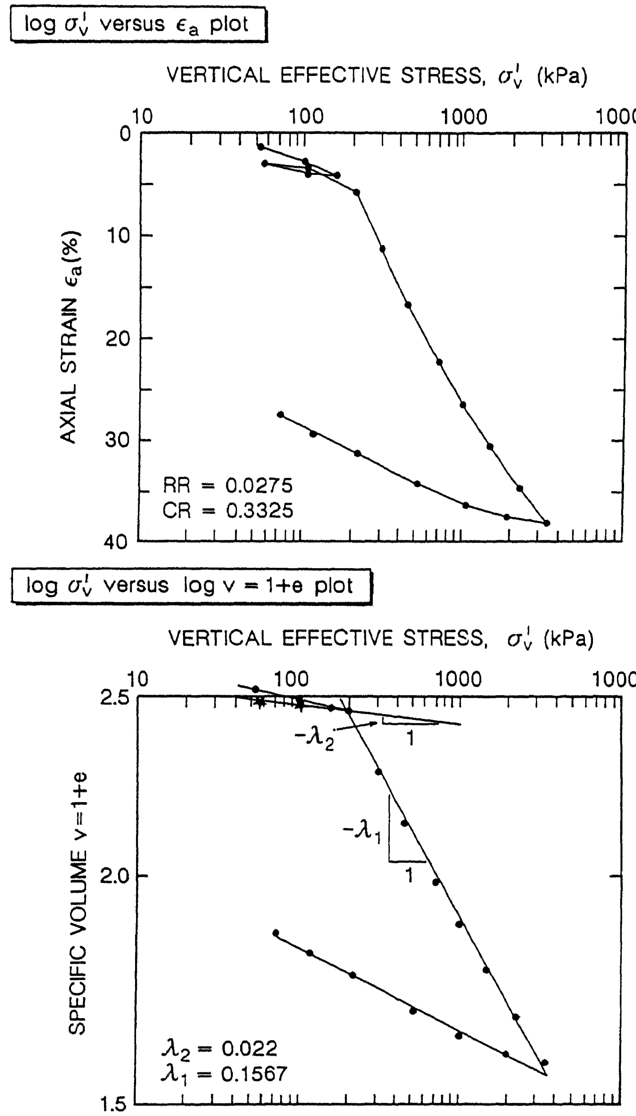
$$\phi' = 22°; \quad c' = 16 \text{ kPa}.$$

The above exposed information give a preliminary and incomplete picture of the geotechnical characteristics of the soils underlying the Leaning Tower of Pisa. A more comprehensive information will be available after the completion of the extensive in situ and laboratory tests now under execution and might result especially relevant as far as formation A is concerned, whose geotechnical characterization is at present far from being comprehensive.

FORMATION		γ (KN/m³)	LL (%)	PI (%)	Fines (%)	Gs (KN/m³)
A		18.1 to 19.0	28 to (1) 42	8 to (1) 19	22 to 100	26.4 to 26.9
B	B_1	16.4 to 17.8	53 to 61	27 to 57	> 80	27.0 to 27.3
	B_2	19.4 to 20.4	34 to 61	13 to 39	> 80	26.8 to 27.3
	B_3	18.5 to 19.4	NP	NP	3 to 50	26.2 to 26.4
	B_4	17.6 to 19.3	35 to 78	17 to 48	> 80	26.3 to 26.8
C		20.2 to 21.4	NP	NP	0 to 20	26.0 to 26.2

(1) Silt and clay layers ; NP = non plastic.

Table 1. Index of properties of soils underlying Pisa Tower.



Depth = 13.75 m ; e_0 = 1.564 ; PI = 43 % ; G_s = 26.29 kN/m³

Fig. 11. An example of compression curve in upper Pisa clay.

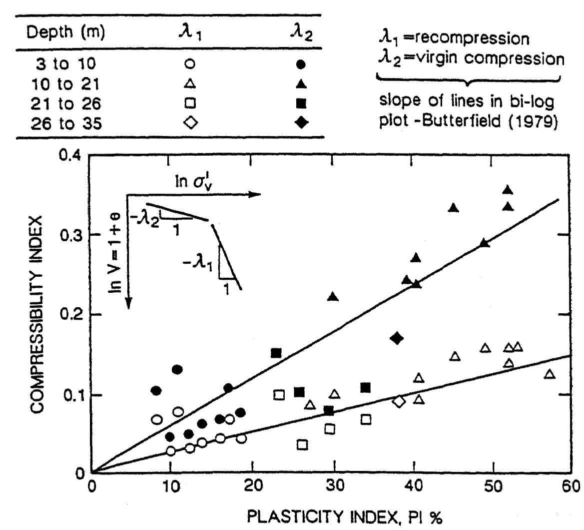


Fig. 12. Volumetric compressibility versus plasticity index in Pisa clays.

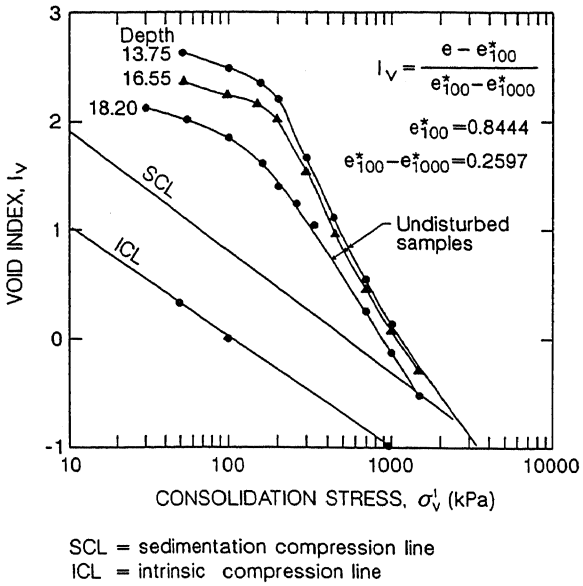


Fig. 13. Compression curves in term of void index in upper Pisa clay.

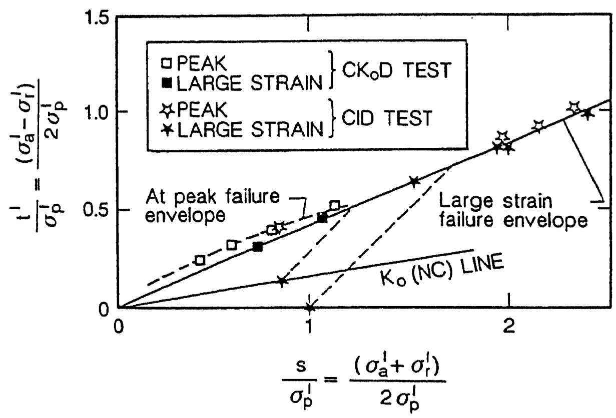


Fig. 14. Strength envelope of upper Pisa clay from drained triaxial tests.

GROUNDWATER

During the investigations carried out by the MLP Polvani Commission in 1971, some piezometers were installed at various depths in the subsoil near the Tower and later, gradually, in a larger area (Fig.15); a better picture of the piezometric conditions prevailing in the subsoil is given by Croce et al., 1981.

In the formation A there is a phreatic water table at an elevation of 1.5 to 2 m above m.s.l.

The situation at a greater depth may be described referring to Fig.16 that shows the levels in the piezometers 1, 2 and 3 of the borehole 112, installed respectively in the layers B_1 (Pancone), B_3 (intermediate sand) and in the formation C (lower sand). The trend reflected in Fig.16 is representative of a number of piezometers existing all over the Square within a radius of 200 m around the Tower.

The piezometric level within formation C (piezometer 112/3) is, on average, at elev. -2 m, that is about 4 m below the phreatic water table; furthermore, it is subject to a cyclic fluctuation of plus or minus 2 m, within a one year's period.

This trend is caused by pumping from deep wells all over the Pisa plain; the fluctuations, with a maximum in January and a minimum in August, reflect the seasonal differences in the quantity of water extracted.

It may be seen in Fig.16 that the mean piezometric level in the lower sand of formation C progressively decreased from elev. -2 m in 1967 to elev. -6 m in 1973, returning to the previous values in 1978.

In the upper clay of the Pancone (piezometer 112/1) the level is almost constant at elev. +1.7 m, while in the intermediate sand (piezometer 112/2) a smooth, long term variation occurs, which seems to follow, at a greatly reduced scale, the mean trend of the level in the lower sand of formation C. The effects of the pumping from deep wells on the behaviour of the Tower will be discussed in the following.

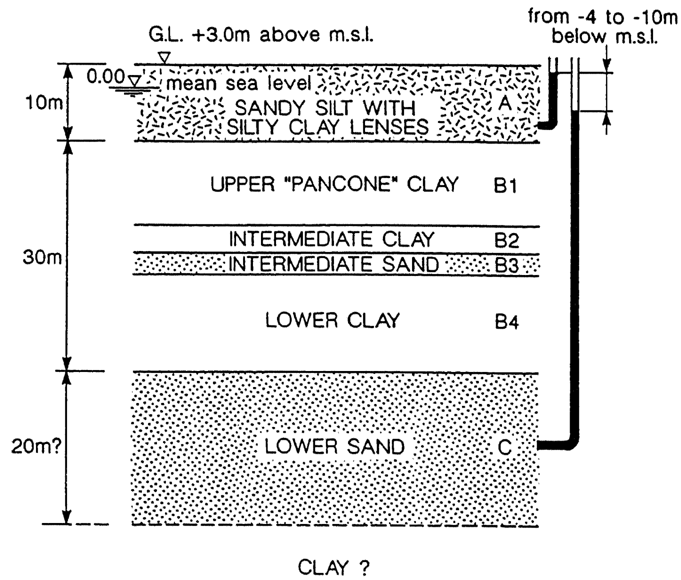
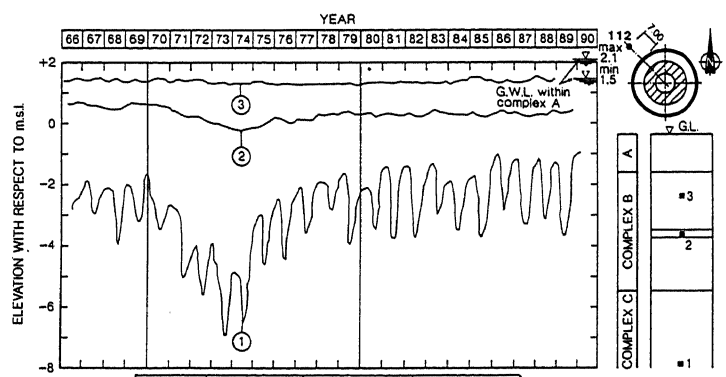


Fig. 15. Subsoil profile and piezometric levels of Piazza dei Miracoli.



Location	Piezometer n.	Depth below ground surface (m)	Elevation m.s.l.
112	1	59.60	-56.70
	2	26.00	-23.10
	3	15.60	-12.70

Fig. 16. Piezometric levels since 1966.

STRUCTURAL FEATURES

As shown in Fig.6 the Leaning Tower of Pisa consists of a hollow masonry cylinder, surrounded by six loggias with the bell chamber on the top.

The masonry cylinder is a typical example of the so called "infill masonry" structure composed of internal and external facings made of San Giuliano marble and of a rubble infill cemented with the San Giuliano mortar, see Fig.17. A helicoidal staircase allowing the visitors to climb up to the top of the Tower is located inside the annular of the hollow cylinder.

The following characteristics of the Tower describe its loading and geometrical characteristics:

- total weight: $N=144.53$ MN; average foundation pressure: $q=497$ kPa;
- total height: $h=58.36$ m; total height above G.L.; $h' \simeq 55$ m;
- distance from the centre of gravity to the foundation plane $h_{cg}=22.6$ m;
- annular foundation, inner diameter; $d_i=4.5$ m, outer diameter $d_o=19.58$ m;
- area of the annular foundation: $A \simeq 285$ m^2, present inclination: $\alpha=5°28'09"$;
- present eccentricity of N; $e \simeq 2.3$ m.

Relevant mechanical properties of the two components of the Tower cross section of Fig.17 are summarized in Table 2. To complete the picture of the characteristics of the Tower structure a typical cross section of the annular foundation is displayed in Fig.18.

Even a preliminary analysis of the Tower structure led to the conclusion that the most dangerous cross-section corresponds to the contact between the first loggia and the base segment where, in addition to the effect of tilt, and the weakening effect of the void represented by the staircases, the diameter of the hollow cylinder suddenly decreases.

At this location on the South side, a compressive stress as high as 7.6 MPa has been measured by flat jacks in the external marble facing. An overall picture of the state of stress in the Tower section under discussion attempted by Leonhardt (1991) is shown Fig.19.

In these circumstances considering:

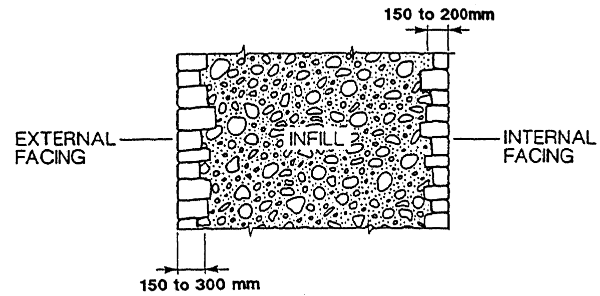
- the high compressive stresses in the external facing on the South side;
- the almost no bond strength between rubble infill and facings;
- the presence of voids and inhomogeneities in the rubble infill ascertained by non-destructive geophysical tests, i.e.; sonic tomography, georadar and infrared termography;
- the heavy loaded external facing laying directly on the infill masonry because of the change of the cross-section of the hollow cylinder at the level of first cornice;
- the deviation of the compressive stress trajectories from the vertical direction in the Tower shaft due to the presence of the staircase and imperfections of the bed joints (Fig.20) leading to the appearance of the horizontal force components as evidentiated in Figs. 18 and 19.

This situation has generated serious concern over the structural safety of the Monument and has led in 1989, to the decision by the Commission established by the MLP and chaired by Jappelli and Pozzati to close the Tower to the visitors.

The envisaged risk is of a failure due to the local buckling in compression of the external facing of the masonry in the most severely stressed section at the South side of the Tower at the level of the first cornice.

This kind of mechanisms has been responsible for the sudden catastrophic collapses of the Bell Tower in San Marco square in Venice in 1902, and, more recently in 1989, of the Bell Tower of the Cathedral of Pavia, both Towers were made of infill masonry with bricks facings.

Due to the fragility of such structures the local buckling in compression of the facings led to their almost instantaneous collapse with no warnings.



FACINGS: • VERY HARD MARBLE OF SAN GIULIANO.

INFILL: • ROCK FRAGMENTS AND STONES CEMENTED WITH SAN GIULIANO MORTAR, FREQUENT VOIDS OF DIFFERENT SIZE.

Fig. 17. Cross-section of Pisa Tower masonry.

	σ_c (MPa)	σ_t (MPa)	E (MPa)
SAN GIULIANO MARBLE FACING	110 to 190	4 to 8	70000 to 90000
INFILL MASONRY	4 to 8	0.3 to 1.3	5000 to 7500

THICKNESS OF FACINGS { OUTSIDE ≃ 200mm, INSIDE ≃ 150mm

σ_c = COMPRESSION STRENGHT
σ_t = TENSILE STRENGHT
E = ELASTICITY MODULUS

Table 2. Mechanical properties of Pisa Tower masonry.

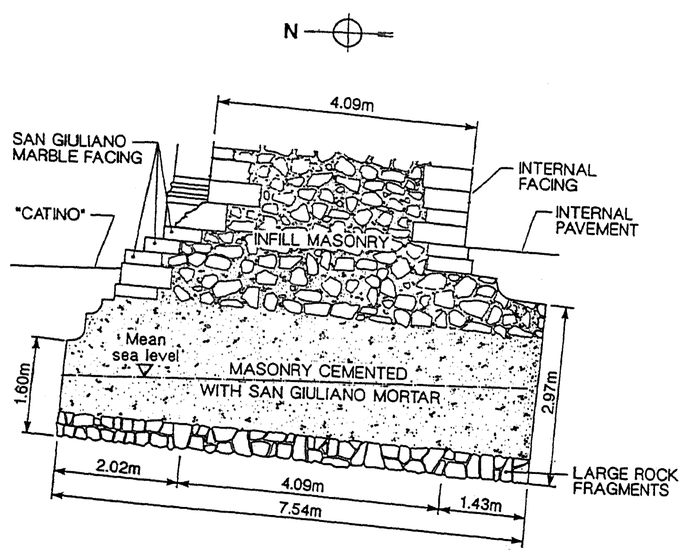


Fig. 18. Cross-section of annular foundation.

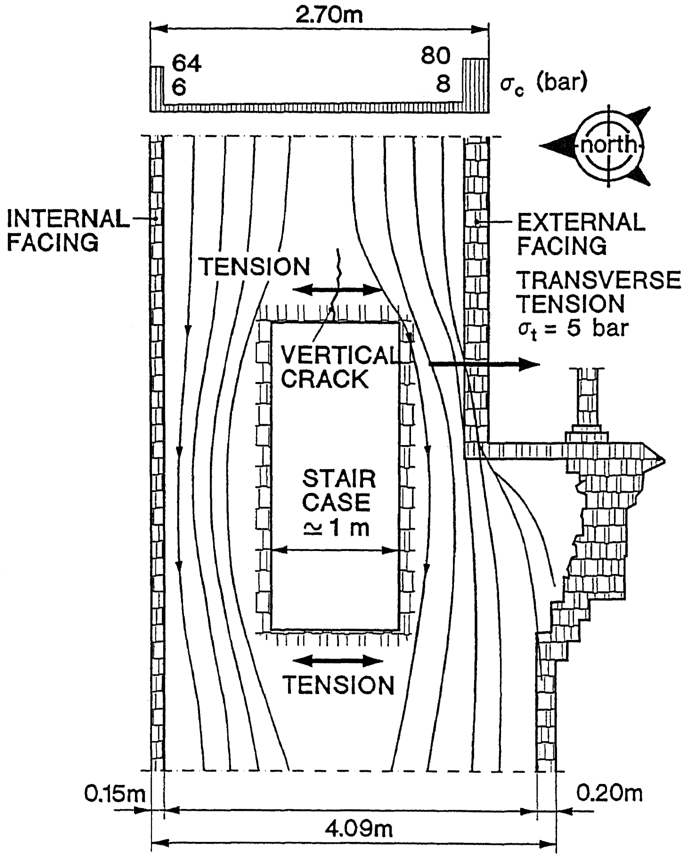


Fig. 19. Cross-section of Pisa Tower at first cornice. Stress trajectories on South side.

MOVEMENTS OF THE TOWER

The systematic monitoring of the Tower started in 1911 adopting the so called geodetic method of the tilt measurements. It consists in measuring, from a fixed station in Piazza dei Miracoli, the horizontal distance between the South edges of 7th and the 1st cornices. Such measurements are usually performed twice a year, and incorporate the rigid tilt of the foundation as well as the variation of the geometry of the Tower axis, influenced by the environmental conditions, i.e. temperature changes and wind effects.

In 1934 two additional monitoring devices have been installed:

- Genio Civile (GC) Bubble Level installed in the instrumentation room located at the level of 1st cornice, see Fig.21.
  It allows to measure over a span of 4.5 m the tilt in two orthogonal planes N-S and E-W. The measurements are taken once a week; they are only moderately affected by wind action and temperature changes.
- Girometti-Bonecchi Pendulum Inclinometer, 30 m long fixed to the internal wall of the Tower at the elevation of the 6th cornice (Fig.21) swings 1.5 m above the floor of the instrumentation room.
  The measurements are made continuously obtaining simultaneously the displacements of the Tower in the same two orthogonal planes mentioned in connection with the GC-level. The sensitivity of the instrument is ≃0.01 seconds but the readings are strongly affected by the wind effect and temperature changes.
  As an example, Fig.22 shows the movements of the Tower obtained by means of the GB-Pendulum testifying the response of the Tower to the changes in the environmental conditions which occur during 24th.

In 1965 high precision levelling of fifteen bench marks (Fig.20) located on the foundation plinth has been initiated. Due to the lack of deep datum point all measured settlements are relative as they are referred to a bench mark located at the cast door of the Baptistery.

Because of their position the bench marks under consideration are practically not affected by temperature changes and therefore better than the previously mentioned measurements, are suitable to reflect the evolution of the rigid tilt of the Tower foundation.

Fig.23 gives the increase of rigid tilt resulting from the high precision levelling of the 15 bench marks located on the foundation plinth. Comparing the evolution of the tilt versus time taken as the difference in the relative settlements of the points 4 and 11 and that resulting from the multiple linear regression of all the 15 bench marks giving the plane of the rigid tilt, it results that the Tower foundation behaves as infinitely rigid.

An overall picture of the Tower tilt in North-South plane since 1911 is shown in Fig.24. It is based on geodetic and GC-Level measurements that, if examined on a long term basis, lead to comparable and reliable results.

A long-term trend of a steady increase of the Tower inclination emerges from this figure. It is remarkable that such trend shows three major perturbations: one occurred suddenly in 1935, the second one began in the mid sixties and went on gradually for about ten years and the third one occurred in 1985.

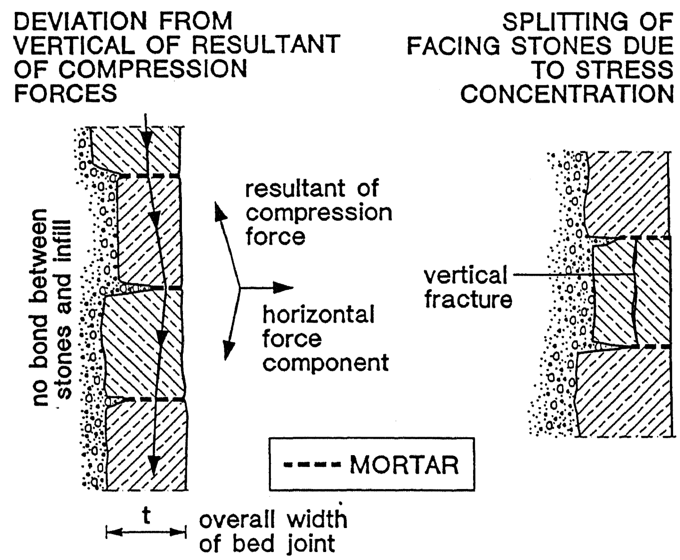


Fig. 20. Pisa Tower marble stone facing. Imperfections of bed joints.

The first perturbation occurred after a cement grouting into the base of the Tower and the soil surrounding the catino, which was aimed at sealing the water inflow.

The second perturbation was first observed during the site investigations carried out by the Polvani Commission, see Croce et al. (1981), and originated serious concern. It became evident that the increase in the rate of rigid tilt was connected to the exceptionally pronounced drawdown of the piezometric level in the sand aquifer, formation C, which occurred from 1970 to 1974. The lowering of the watertable produced an increase of the tilt of about 30 seconds of arc in the North-South direction and of about 20 seconds of arc in the East-West one. Following these observations, some wells in the vicinity the Tower were closed and a partial recovery of the piezometric level was obtained in 1975 and 1976. Soon afterwards a significant decrease in the rate of tilt was remarked.

The third perturbation occurred after the boring performed in the Northern edge of the foundation in 1985. The increase of tilt was about 7 seconds of arc in the north-south direction.

In order to get a picture of the progress of the inclination of the Tower that does not include the effects of the above mentioned events and of the environmental changes, Burland (1990) has attempted to subtract from the GC-Level measurements and from the high precision topographical levelling data, the effects of perturbations. The obtained results reported in Fig.25 show a definite trend of a slowly increasing rate of tilt; the implications of such findings with regard to a future overturning instability of the Tower are evident.

It has only recently [Croce et al. (1981)] been determined that the subsidence which the whole Pisa plain is subject to may affect the movements of the Tower because of the local phenomena occurring in the Piazza dei Miracoli. Despite the lack of the deep datum point one can argue that the differential subsidence occurring in the Square might contribute to the present rate of tilting of the Tower.

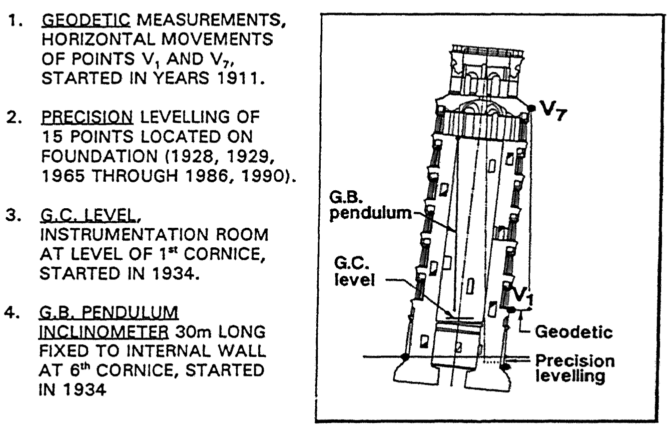


Fig. 21. Measurements of tilt.

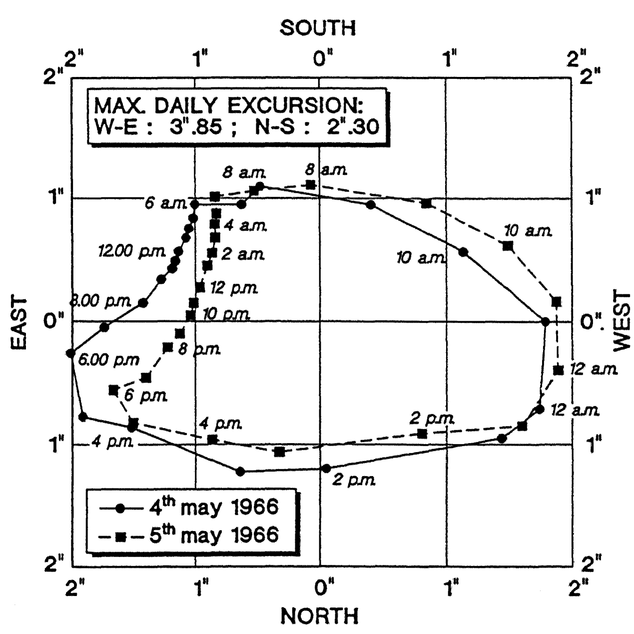


Fig. 22. Daily movements of Pisa Tower due to change of temperature.

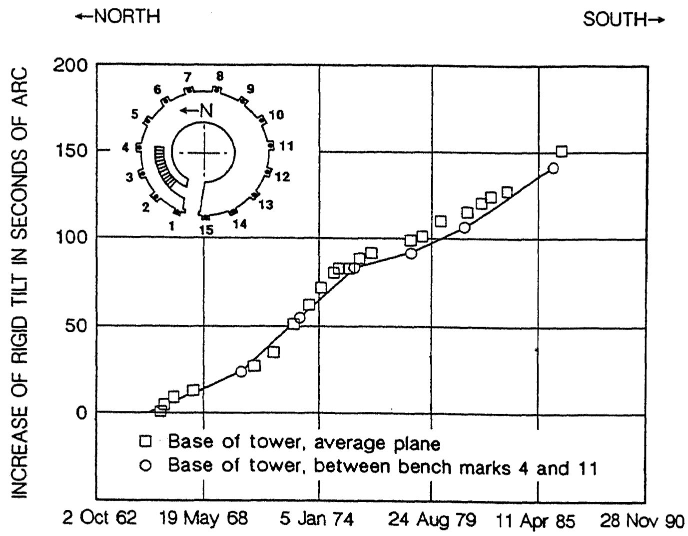


Fig. 23. Increase of rigid tilt of Pisa Tower. Years 1962 through 1990.

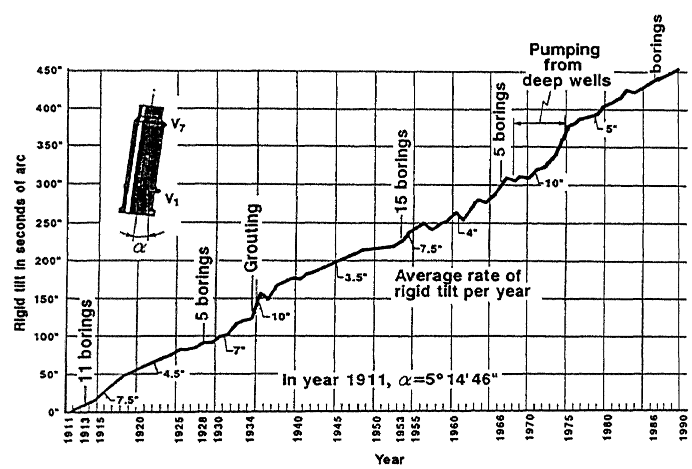


Fig. 24. Rigid tilt of leaning Tower of Pisa.

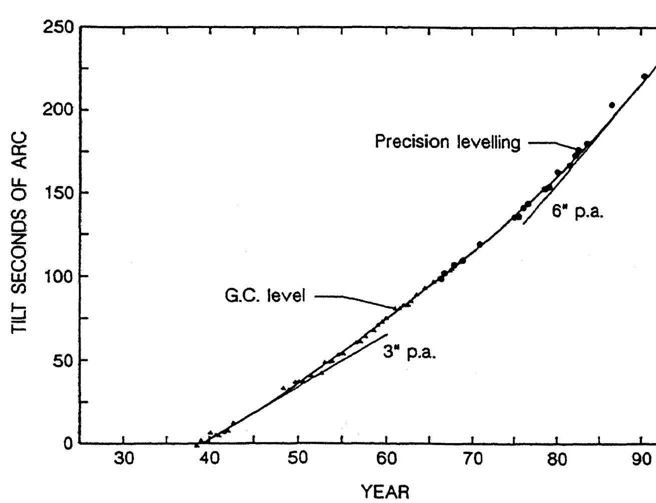


Fig. 25. Net tilt of tower foundation *(Burland, 1990a)*.

CLOSING REMARKS

In the preceding sections of this paper geotechnical and structural aspects of the leaning Tower Pisa as well its movements have been summarized. These latter aspects are recalled in Fig.26 and Table 3. Fig.26 reports the evaluation of the Tower settlement using a simple elastic perfectly plastic soil model and geotechnical data that have been gathered since 1989 by Giunta (1988) and Costanzo (1989). The obtained results are in reasonable agreement with those postulated by Leonards (1979) on the basis of the shape of the settlement bowl encountered at the contact between formations A and B, see Fig.9. Table 3 shows the evolution of the Tower inclination and of the related overturning moment with time; the displayed data have been high qualitative till 1758, while since the measurements performed by Taylor and Cresy (1829) they reflect in a quantitative manner the evolution of the Tower tilt.

As shown in Fig.25 the inclination of the Tower is growing and the increase is at present around 5 to 6 second of arc' per annum excluding perturbations due to the environment.

This behaviour conforms to that of the phenomenon of self-driving instability which only recently has recalled the attention of Abghari (1987), Hambly (1990), Cheney et al. (1991) and Lancellotta (1992) in relation to the stability of tall structures seated on soft compressible soils.

This phenomenon named also leaning instability, is in some way similar to that from structural mechanics dealing with the instability of columns having an initial bent. In case of tall structure on compressible support the geometrical imperfection is represented by an initial tilt which at least in first approximation doesn't engage the resisting moment of soil reaction. In case of Pisa Tower the mechanism which might have triggered the initial tilt (α_o) i.e. the leaning instability should be linked to the inclination which occured suddenly during the second construction stage, see Fig.5 and to the subsequent differential settlements, both phenomena related in some manner to the pronounced spatial variability of the mechanical properties of formation A, e.g. Fig.8.

In contrast with the ordinary foundations, whose safety factor against overturning stability is controlled by the vertical load (N) and its eccentricity (e), in case of tall structures subject to leaning instability, the height of the centre of gravity of the structure with respect to the foundation plane (h_{cg}) becomes also one of the controlling factors.

With reference to the above mentioned mechanism of the leaning instability and assuming:

- the initial tilt of the Tower southwards at the end of the second stage of construction $\alpha_o = 40'$
- a non linear relation between the rotation of the Tower α and the resisting moment of the soil M_R which has been postulated of hyperbolic form.

Lancellotta (1992) finds the safety factor against the overturning of the Tower equal to 1.09. This value represents the ratio of the critical vertical load N_{cr} to that due to the height of the Tower. The N_{cr} can be regarded as an equivalent of the critical load for a column with an initial geometrical imperfection α_o.

The above mentioned value not taking into account the influence of creep on α_o should be considered as an upper limit of the possible values of safety factors.

In view of what above stated, the present Committee appointed to safeguard the leaning Tower of Pisa, considering:

- the high but non quantifiable risk of a structural collapse which is increasing with the increase of inclination,
- the very low safety factor against overturning instability evidentiated by progressive increase of rigid tilt at increasing rate,
- the absolute need to avoid that the geotechnical and structural stabilization works become too intrusive or lead to a heavy visual impact and in order to preserve at any cost the artistic and historical value of the monument and of the whole Piazza dei Miracoli,
- the need, to carry out a series of multidisciplinary studies involving; archaeology, history of medieval arts, architecture geotechnical and structural engineering whose completion requires at least two years,

has resolved upon the strategy hereunder outlined:

- to design and to implement the temporary, and completely reversible, local reinforcement of the most critical cross-section of the structure at the level of the 1st cornice in order to improve the structural safety of the monument. This accomplishment has already been put un place, see Fig.27. It consists in post-tensioned cables aimed at preventing local buckling in compression of the marble stones forming the external facing;
- to improve the foundation stability against overturning by placing 6 MN lead counterweight of the north rim of the Tower base as shown in Fig.28. This temporary and reversible intervention will be implemented very gradually in the next future, keeping Tower movements and the possible changes of the pore water pressure in the foundation soil under constant monitoring.

  The highly controlled application of the counterweight will hopefully lead to a reduction of the inclination of the Tower by few minutes, producing a situation analogous to that existing 30 to 50 years ago. In addition, the application of the lead counterweight will represent a valuable full scale test of the response of the Tower to the effect of small scale stabilizing moment;
- in order to mitigate the possible influence of the subsidence of Piazza dei Miracoli on the present rate of tilt of the Tower, it was resolved to close a number of water wells in the area within 1 km of the Tower. Such decision, although causing some social problem to the municipality, has recently been approved by the Mayor of Pisa and will be enforced in the near future.

YEAR	WEIGHT (MN)	MOMENT (MNm)	TILT
1178	94.80	-	-
1272-1278	137.28	55.1	0° 06' 11"
1285	137.28	598.80	1° 06' 44"
1360-1370	144.53	977.00	1° 36' 39"
1550	144.53	284.72	4° 41' 07"
1758	144.53	293.54	4° 49' 50"
1817	144.53	310.16	5° 06' 11"
1911	144.53	318.98	5° 14' 46"
1990	144.53	332.56	5° 28' 09"

Table 3. Weight, overturning moment and rigid tilt versus time.

After the above mentioned actions which slightly improve the safety of the monument, the Committee has started examining the feasibility of different possible solutions to stop or even to reduce, by no more than one degree, the Tower tilting.

Of the different possible approaches, the one leading to a controlled settlement of the ground at the soil-structure interface on the North side of the Tower is being considered by the Committee. If feasible, it will allow, without touching the monument, to stop the increase of inclination and with a reduction of the tilt of order of 30' to 60' to modify positively the state of stress in the critical sections of the structure.

In order to achieve this goal two alternative solutions are taken into account:

- to cause the reduction of the volume in the top most part of the Pancone clay by means of a properly devised electrosmotic consolidation treatment, see for example Mitchell (1991);
- to induce the settlement under the North part of the Tower foundation by means of the under excavation technique adopted with success in the last few years in Mexico City to reduce the differential settlement under a number of buildings damaged by the 1985 earthquake, see Tamez et al. (1992). This method is going to be employed in the next future to reduce the extremely large differential settlements to which has been subject the XVI century Metropolitan Cathedral of Mexico City.

While the Committee starts trial fields and numerical modelling to ascertain the feasibility of the above mentioned intervention methods, the solution with the ground anchors, shown in Fig.29, is also being developed. It consists in the use of ten ground anchors designed for the working load of 1 MN and connected to the Tower by means of a prestressed concrete ring that is evidentiated in the same figure.

The above mentioned intervention can be envisaged in relation to one of the following scenarios:

- during implementation of electrosmosis or under excavation it might result necessary to apply to the North rim of the Tower base a load in addition to the lead counterweight in order to assure the contact between settling soil and the foundation;
- the feasibility of the two above mentioned methods of controlled subsidence will not be demonstrated. In this case the solution with the ground anchors after possible positive response of the Tower to the application of the lead counterweight might become an alternative measure for the permanent stabilization of the Tower foundation.

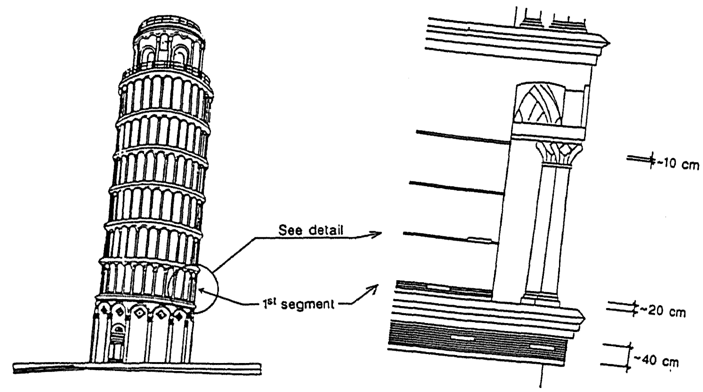


Fig. 27. Temporary structural strengthening (light circumferential prestressing).

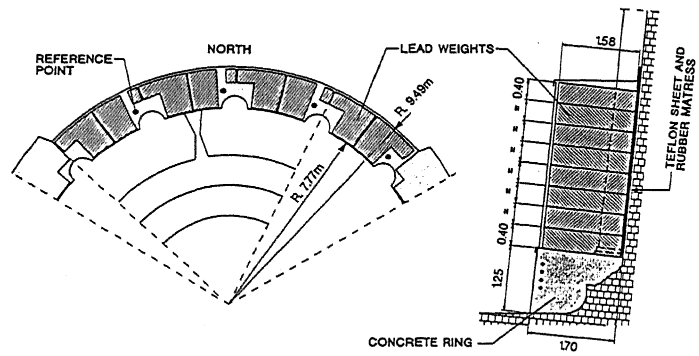


Fig. 28. Counterweight on North edge of foundation position of lead weights.

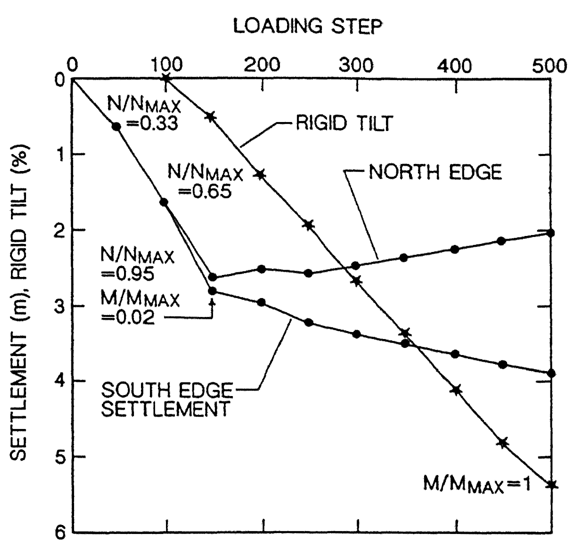


Fig. 26. Numerical* modelling of movements of Pisa Tower (Lancellotta and Pepe, 1991).

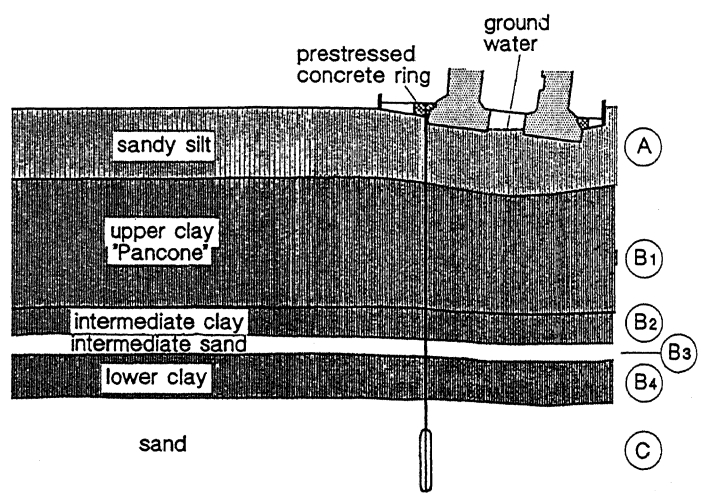


Fig. 29. Temporary stabilization scheme of foundation.

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