VABLJENO PREDAVANJE

Prof. Michele JAMIOLKOWSKI
Technical University of Turin

POŠEVNI STOLP V PISI

THE LEANING TOWER OF PISA

The Leaning Tower of Pisa

Prof. M.B. Jamiolkowski
Technical University of Turin, Italy

1st Šuklje's Day
Slovenian Geotechnical Society
Ljubljana, Oct. 12, 2000

INTRODUCTION

This paper describes the current condition of the Leaning Tower of Pisa, and up to date information.

A brief summary history of the Monument will introduce the information concerning the subsoil condition and its structural features, followed by the presentation of the monitored data documenting the progressive increase of the Tower inclination.

On the basis of the above information, a phenomenological outline motivating the reasons for the continuous increase of the Tower's inclination over time, since the completion of its construction, is subsequently presented. At this point it will be possible to attempt to formulate some considerations about the margin of safety relative to the risk of the Tower falling over.

Finally, a brief update on the state of knowledge concerning the Monument, an equally concise description of the stabilisation works on the Tower foundation, as well as the project to reinforce its structure undertaken by a 14-member International Multi-Disciplinary Commission appointed by Italian Government in the middle of 1990, will be presented.

HISTORICAL BACKGROUND

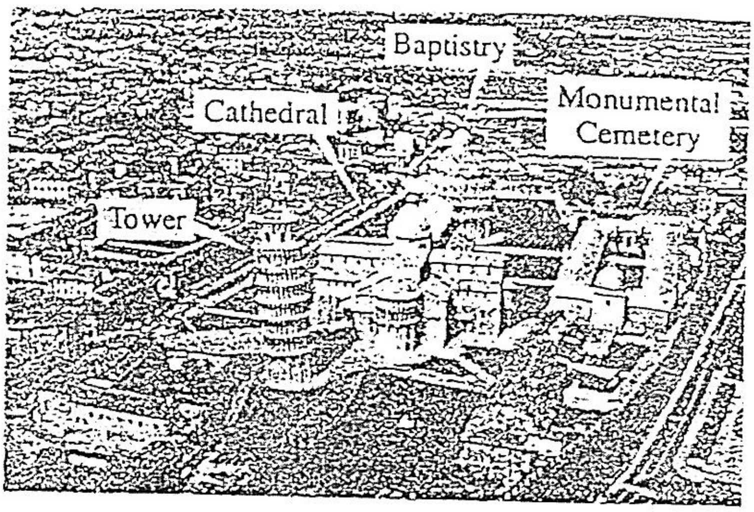


Fig.1 - Piazza dei Miracolli - air view

The Monuments of Piazza dei Miracoli, see Fig. 1, including the Tower, are: the Cathedral, the Baptisery and the Monumental Cemetery, which were all erected during the Middle Ages.

In fact, construction of the Cathedral, the first monument to be erected, began in late 1000.

The design of the tower is ascribed to the Architect and Sculptor Bonanno Pisano.

The tower consists, see fig. 2, of a hollow masonry 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

$V \cong 142$ MN, $M \cong 327$ MNm, $e \cong 2.3$ m *

(*) Situation in year 1990

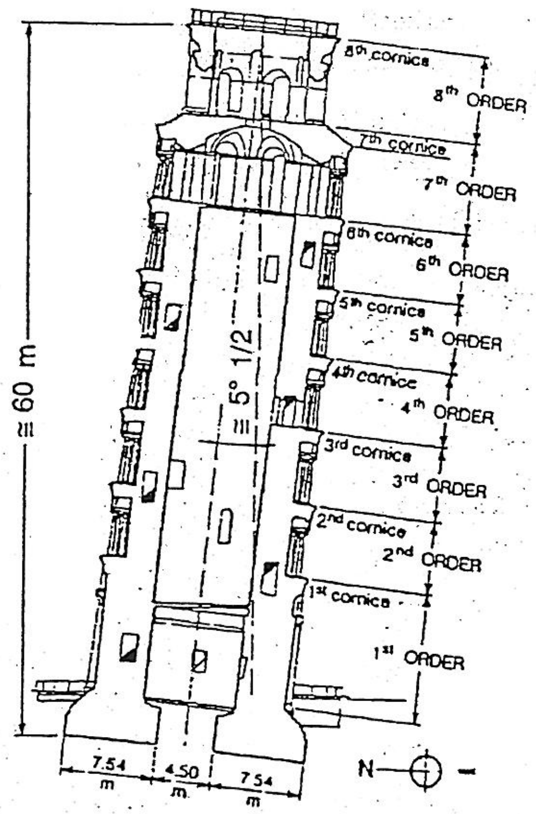


Figure 2 – Leaning Tower of Pisa – Cross-section

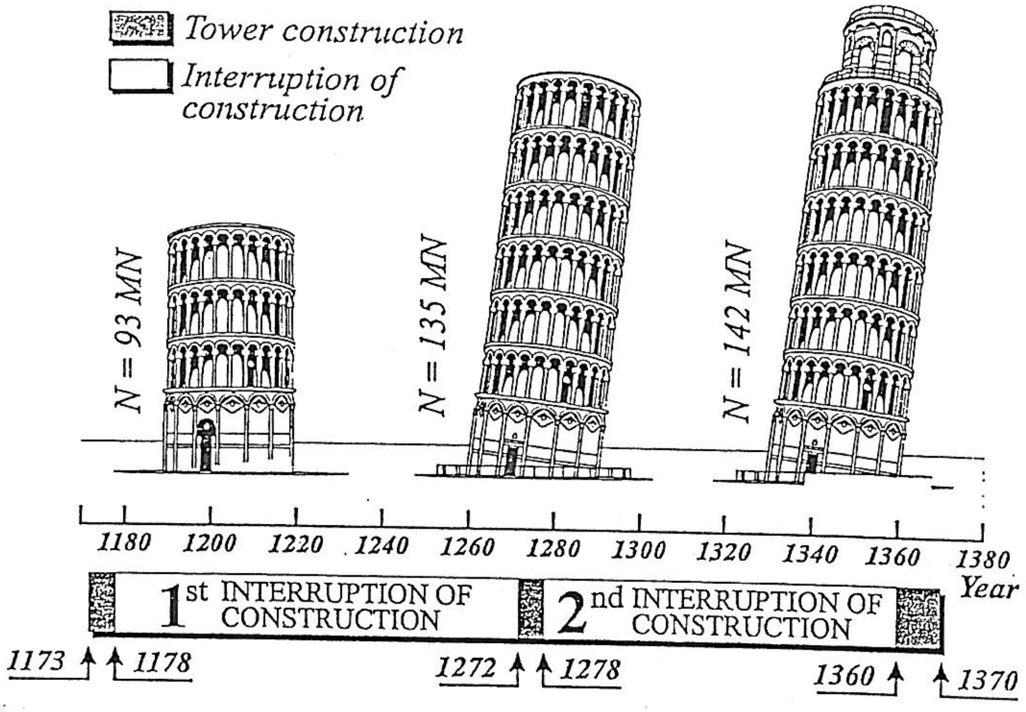


Fig. 3 - Construction History

but after five years the works were interrupted at the middle of the fourth order as shown in Fig.3. The construction was resumed in 1272 under the lead of the Architect Giovanni Di Simone who brought the Tower almost to completion, up to the seventh cornice (Fig.3) in six years.

The construction of the Tower was finally completed when Architect Tommaso di Andrea Pisano added the bell chamber between the years 1360 and 1370.

It was during the second construction phase that the curvature in the axis of the Tower began to appear, see Fig.4, reflecting the attempt of the masons, charged with the construction works, to compensate against the on going manifestation of tilting.

This compensation was attempted by a progressive change in thickness of properly hand cut stone blocks of each "ricorso"

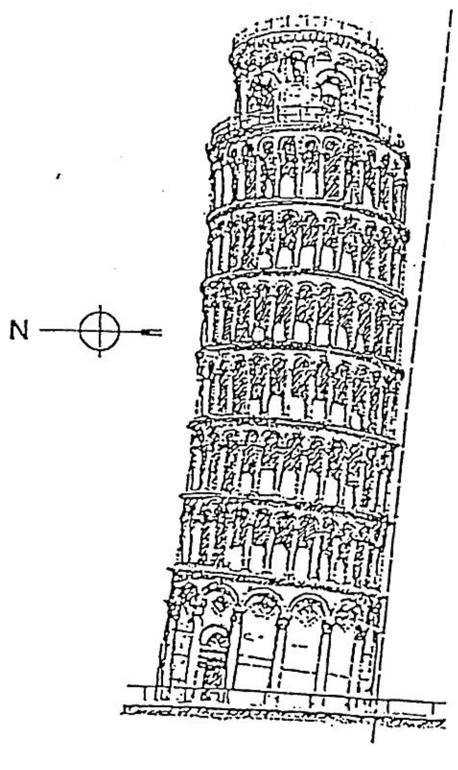


Fig. 4 - Correction made during construction

(tiers of stones of which the Monument facing is made) while moving from North, southwards. By measuring the thickness of blocks within each "ricorso", the evolution of inclination during the construction period can be in first approximation inferred.

The position in which the bell chamber was added, by Tommaso di Andrea Pisano, testifies a further attempt to correct the geometry of the structure and to compensate for the occurring inclination.

Our timeline of the Tower's history is based on the variation of the thickness of "ricorsi" and on other historical evidence such as:

- the fresco by Antonio Veneziano of 1384 showing the funeral of Saint Ranieri;
- the work life of Arnolfo by Vasari 1550;
- the measurements of the tilt performed in 1818 with the plump line by two English Architects E. Cresy and G.L. Taylor;
- measurements similar to those mentioned above carried out by the French Rouhault De Fleury in 1859. There is no record of an inclination measurement but only mention of an appreciably larger inclination than that recorded by the two English Architects.

The increased rate of inclination after the Cresy and Taylor measurements is usually attributed to the works by Architect Della Gherardesca who, in 1838, excavated an annular ditch around the Tower called "catino" as shown in Fig.5. The aim of the catino was to uncover the basis of the columns, originally from the upper portion of the foundation plinth, which sank into the ground as a consequence of settlement. Given that the bottom of the catino is below the groundwater table, it has been necessary, since 1838 to continuously dewater it triggering an increase in the Tower tilt rate.

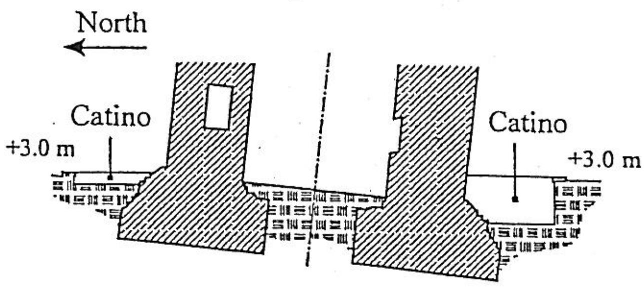


Fig. 5 - Catino Cross-section

Only in 1935 [MPW (1971)] when the Ministry of Public Works under the supervision of Eng. Girometti performed the cement grouting in the Tower plinth and implemented a new waterproofed catino structure, the dewatering was stopped.

The reconstruction of the history of the Monument tilt shown in Fig.6 has also been possible because of the geodetic measurements of the inclination, started in 1911. It must be pointed out

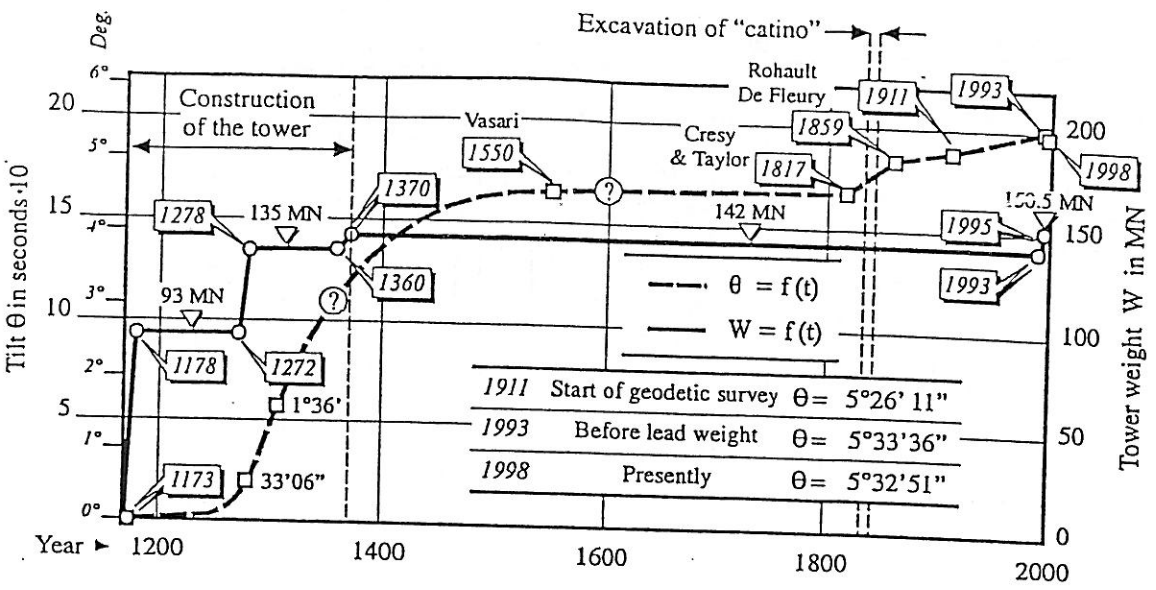


Fig. 6 - Evolution of Rigid Tilt with Time

however, that all information dated prior to the start of systematic modern monitoring concerning the inclination, should be considered as approximate, highly qualitative and, to some extent, subjective.

SUBSOIL CONDITIONS

Many geotechnical investigations have been performed at different times around the Tower. The most relevant and comprehensive among them are described in detail in:

- Three volumes published by the Ministry of Public Works Commission MPW (1971) whose data are summarized and updated in the work by Croce et al. (1981).
- Works by Jamiolkowski (1988), Berardi et al. (1991), Lancellotta and Pepe (1990, 1990a), which report the results of soil investigation carried out in the mid eighties by the Design Group, appointed by the Ministry of Public Works chaired by Finzi and Sanpaolesi.
- The investigation carried out in the years 1991 through 1993 by the International Committee presently charged for the project on safeguarding the Monument. The results of this investigation have been only partially published and the relevant results can be found in works by: Calabresi et al. (1993), Lancellotta et al. (1994), Costanzo (1994) and Costanzo et al. (1994).

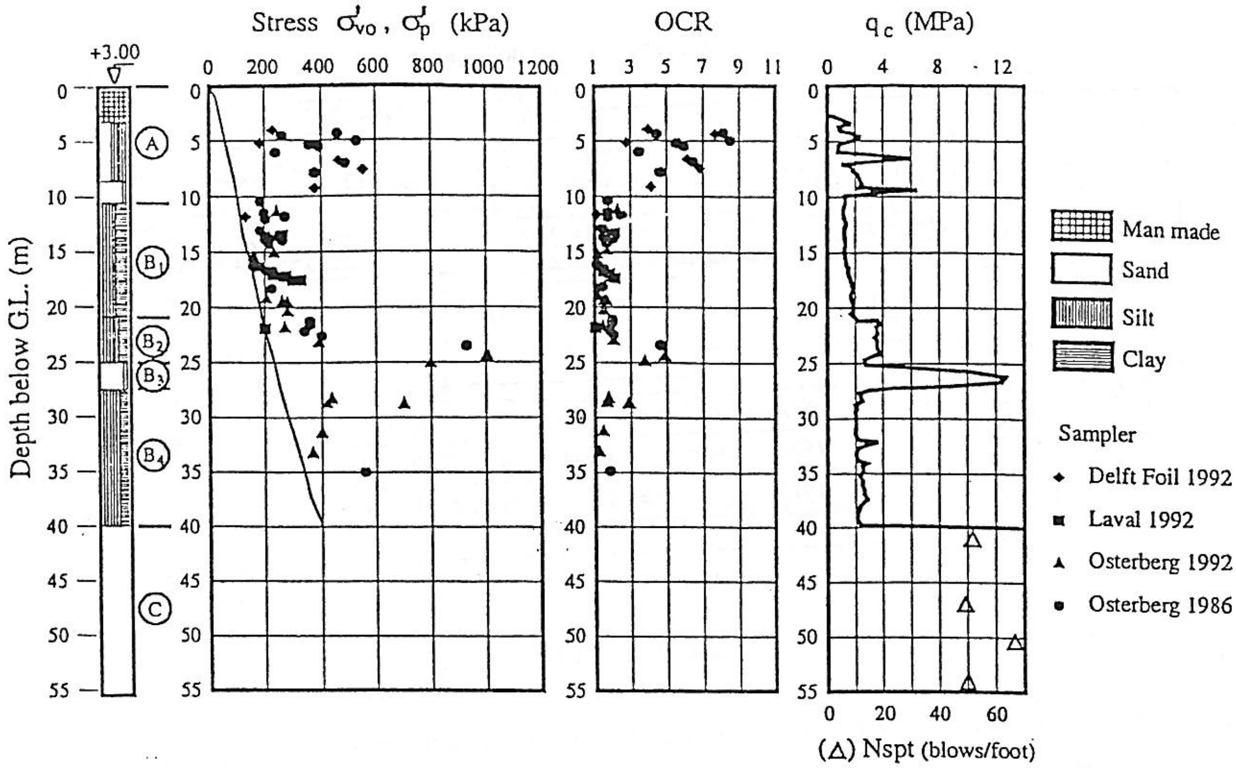


Fig. 7 - Subsoil Conditions

Based on all the above mentioned geotechnical investigations, it is possible to determine the following soil profile([1]), see also Fig.7, starting from the ground surface at an elevation of approximately +3.0 a.m.s.l.,

- Horizon A: ≅10 m thick, consists of interbedded silt, clay and sand layers as well as lenses covered by ≅3 m thick layer of man-made ground.

  This Horizon can be subdivided in the following layers:

  - Layer A_o; from elev. +3.0 to ±0.0, man-made ground containing numerous archeological remainings dated from the 3th Century B.C. to the 6th Century A.C.
  - Layer A_1; from elev. ±0.0 to –3.0, yellow silty sand and sandy silt.
  - Layer A_2; from elev. –3.0 to –5.0, yellow clayey silt.
  - Layer A_3; from elev. –5.0 to –7.0, uniform medium grey sand.

  Recent borings and piezocone (CPTU) tests performed in the vicinity of the Tower suggest that moving from the South perimeter of the Tower catino northwards, the Layer A_2 becomes increasingly sandy. Overall, a comparison of the cone resistance (q_c) yielded by CPTU's reveals that resistance of Horizon A is markedly lower at South when compared to

([1]) according to the designations of main Horizons adopted in MPW (1971).

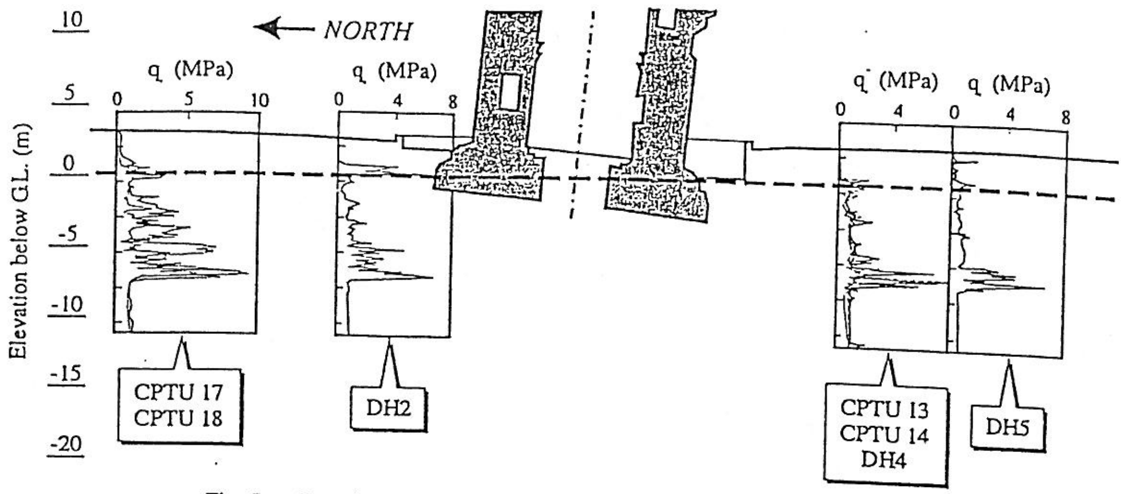


Fig. 8a - Cone Resistance in horizon A, North-South Cross-section.

the North side, see Fig.8a. The CPTU's also showed that the q_c profiles on the East side yielded an average lower cone resistance than on the West side, see Fig.8b. The above mentioned trends are confirmed by the exam of penetration pore pressure [Pepe (1995)], resulting from CPTU's.

Furthermore, it is worthwhile reporting the results of five seismic CPTU's performed in the close vicinity of the Monument, see Fig.9. In addition to the profile of shear wave velocity V_s, the figure shows the trend of q_c vs. depth which confirms what emerges from Figs.8a and 8b.

– Horizon B, ≅30 m consists of clay with an interbedded layer of sand. Within this Horizon B the following four layers can be recognised:
  - Layer B_1; from elev. –7.0 to –18.0, upper clay, locally named Pancone clay.
  - Layer B_2; from elev. –18.0 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.

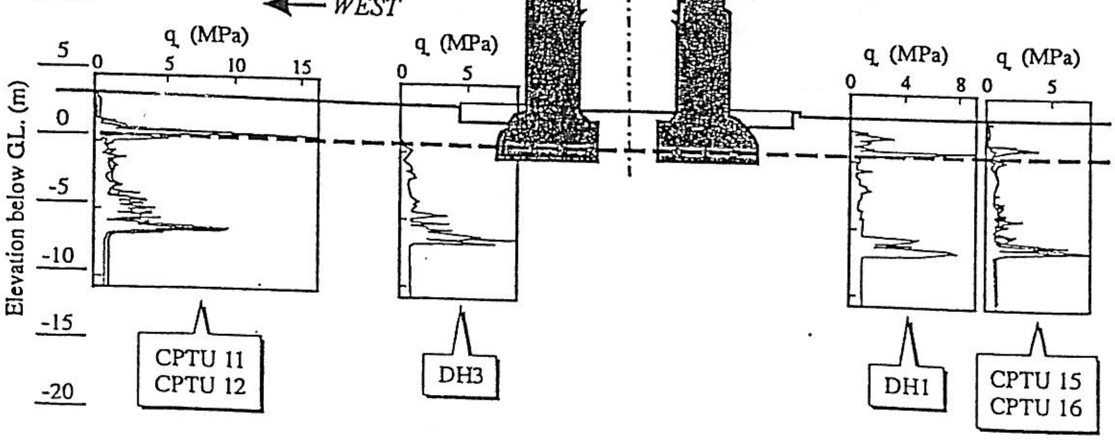


Fig. 8b - Cone Resistancein horizon A, West-East Cross-section.

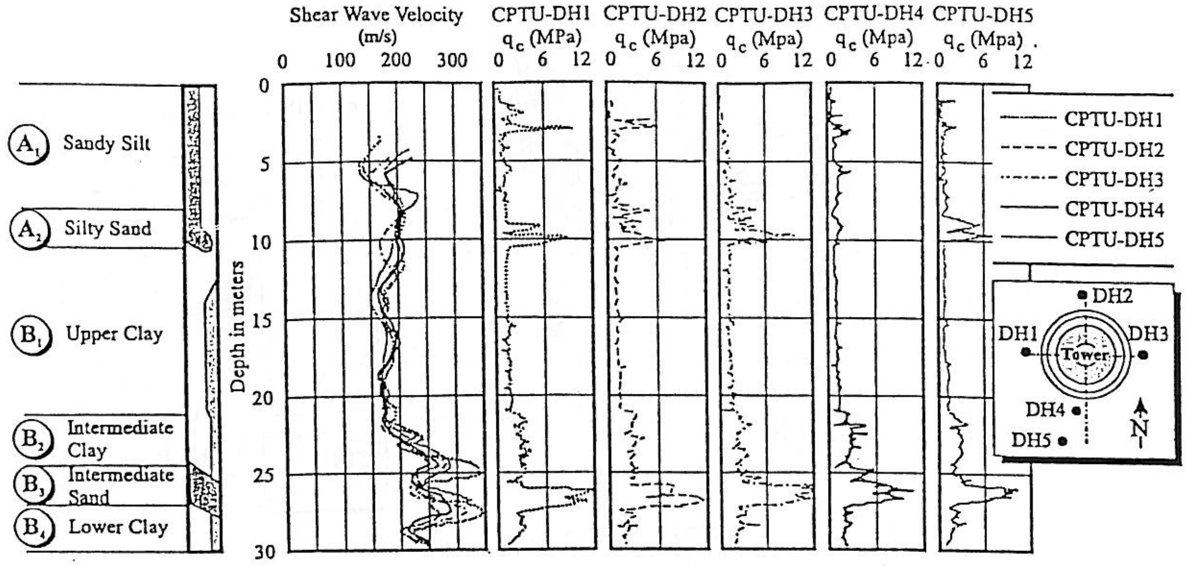


Fig 9 - Results of Seismic Cone Penetration Test

The highly comprehensive literature review of soil investigation data, produced by Calabresi et al. (1993) has allowed a further subdivision of each layer of Horizon B into a number of sub-layers. However, it is beyond the scope of the present paper to elaborate on these findings.

- Horizon C, has been recently investigated to the depth of 120 m (elev. –117 b.m.s.l.). Three distinct layers have been found.
  - Layer C_1; from elev. –37.0 to -65.0; medium to coarse grey sand rich of fossils and shells in some spots, containing randomly distributed and quite rare lenses of peat.
  - Layer C_2; from elev. –65.0 to –75.0; greenish clayey and silty sand.
  - Layer C_3; from elev. –75.0 down to the maximum explored depth, grey changing to a shade of green in lower sand.

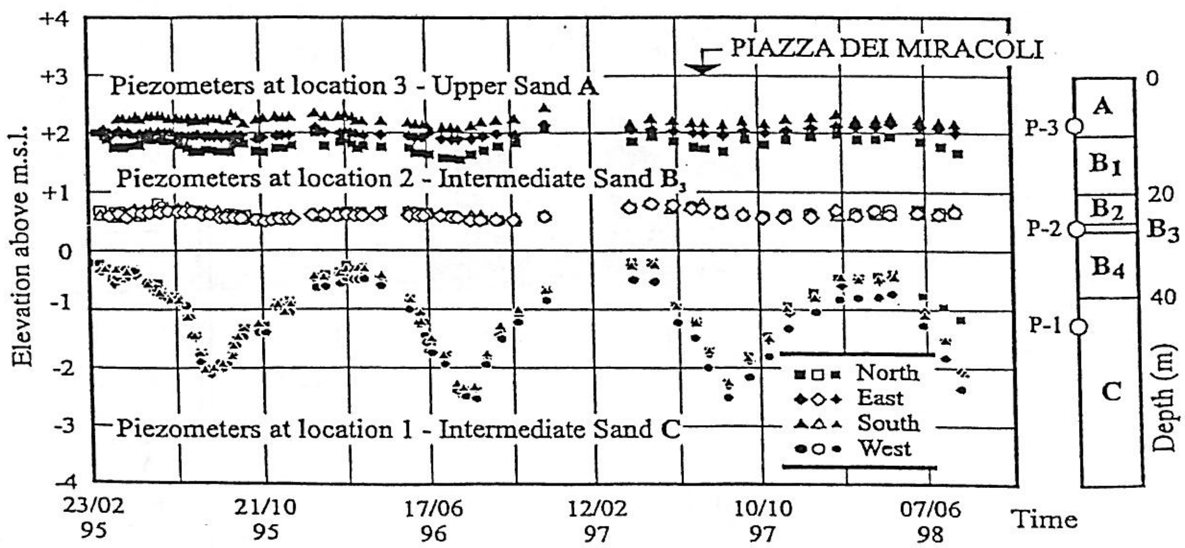


Fig . 10 - Ground water level in sand layers beneath Piazza dei miracoli

Figs.10 is representative of the groundwater conditions. Three different piezometric levels exist in the Horizon A layer B_3 and Horizon C. The latter has presently a mean piezometric level of elev. -1.5 b.m.s.l. circa with an annual fluctuation of 2 m. The phreatic water level within Horizon A has an average seasonal variation of elev. between +1.5 and +2.0 a.m.s.l.

The piezometric level in the intermediate sand layer B_3 is approximately located at the elevation of +0.70 a.m.s.l. and is subject to a minor seasonal fluctuation, ≅0.10 to 0.20 m, which at reduced scale and with some time lag, mimics the one observed in Horizon C.

The above outline of the groundwater scheme indicates that the pumping from Horizon C, which began approximately in the 1950's, triggered the consolidation of the clay layers belonging to Horizon B, causing the subsidence of the whole Pisa plane.

This phenomenon, now greatly attenuated, had become quite severe in the early seventies when the mean piezometric level in the Horizon C decreased to elev. –6.0 b.m.s.l. causing an acceleration of the Tower tilt due to the differential subsidence over the Piazza dei Miracoli. For greater details see Croce et al. (1981). This resulted in the closure of a number of wells in the vicinity of the square and led to a substantial attenuation of the phenomenon in the early eighties. Further information regarding this aspect of the problem can be found in the work by Schiffmann (1995).

Although detailing the geotechnical characterization of the soil underlying the Tower is beyond the scope of this paper, a concise summary of the index and stress-strain-strength properties will follow. However, to obtain a more extensive insight into this aspect of the problem, the MPW (1971), Lancellotta and Pepe (1990, 1990a), Calabresi et al. (1993), Lancellotta et al. (1994) and Costanzo et al. (1994) should be consulted. The mean values and the standard deviations of the index properties can be inferred from Tables 1 and 2, where:

γ = bulk density; G_s = specific gravity; W_n = natural water content;

LL = Liquid Limit; PI = Plasticity Index.

Table 1 - Grading of main soil layers

Horizon	Layer	Sand Fraction %	Silt Fraction %	Clay Fraction %
A	A_3	31.7 ± 4.7	61.1 ± 12.3	13.0 ± 4.9
	A_4	74.6 ± 16.5	17.9 ± 14.9	4.2 ± 3.3
B	B_1	< 5	42.4 ± 13.3	58.0 ± 13.0
	B_2	6.0 ± 4.2	51.1 ± 15.7	38.9 ± 13.7
	B_3	77.0 ± 8.1	19.8 ± 14.6	8.4 ± 3.1
	B_4	< 5	52.9 ± 17.0	43.1 ± 17.2
C	C	82.5 ± 14.7	7.0 ± 6.2	5.5 ± 4.2

Table 2 - Index Properties of main soil layers.

Horizon	Layer	γ (kN/m³)	G_s (-)	W_n (%)	LL (%)	PI (%)
A	A_3	19.42 ± 2.03	2.71 ± 0.03	31.6 ± 4.2	35.2 ± 4.7	13.2 ± 3.6
	A_4	18.35 ± 0.61	2.68 ± 0.03	33.6 ± 3.8	-	-
B	B_1	16.64 ± 1.05	2.78 ± 0.03	52.6 ± 7.9	70.8 ± 13.6	42.1 ± 12.5
	B_2	19.91 ± 0.50	2.73 ± 0.03	25.8 ± 3.3	51.6 ± 11.7	28.1 ± 11.2
	B_3	18.95 ± 0.45	2.69 ± 0.01	30.2 ± 3.3	-	-
	B_4	19.00 ± 1.00	2.74 ± 0.04	36.1 ± 9.2	55.9 ± 14.8	32.3 ± 13.2
C	C	20.80 ± 0.06	2.66 ± 0.01	18.7 ± 2.4	-	-

Based on the information concerning the piezometric levels and with reference to the values of γ determined in laboratory, the variation of the effective overburden stress (σ'_{vo}) with depth shown in Fig.7 has been established.

The value of σ'_{vo} in combination with preconsolidation pressure σ'_p, as determined by oedometer tests using the Casagrande (1936) procedure, led to the overconsolidation ratio values (OCR) showed in the same figure. The overconsolidation mechanism involved in the case of Pisa subsoil is generally ascribed to aging, due to secondary compression, groundwater fluctuations as well as possibly to a minor removal of the overburden not exceeding 50 to 60 kPa. In addition, in the case of Horizon A and Layer B_2, temporary emersion and related desiccation could have affected the OCR values, see also Calabresi et al. (1993).

The coefficient of earth pressure at rest (K_o), for Pancone Clay, in a normally consolidated (NC) state, ranges between 0.58 and 0.63.

The best estimate of the K_o in the field, considering the above outlined overconsolidation mechanisms and taking into consideration works by Mesri and Castro (1987), Mesri (1989), Hayat (1992) and Mesri et al. (1997) should be around 0.73 to 0.75. The writer does not have the information necessary to estimate the field K_o in other clay layers belonging to Horizon B.

The mechanical properties stated in the following information provide the reader with a general picture of the subsoil conditions:

- The compressibility of the clay layer has been investigated mostly by means of oedometer tests. As an example, Fig.11 shows the results of incremental loading oedometer tests performed on three high quality undisturbed samples retrieved from Pancone clay. The results are plotted in the plane log σ'_v vs. void index (I_v), the latter defined [Burland (1990)] as follows:

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

e = current void ratio of tested specimen

e^*_{100} = void ratio at σ'_v=100 kPa determined reconstituted specimen starting from $LL \leq W_n \leq 1.5\ LL$

e^*_{1000} = as above but referring to σ'_v = 1000 kPa

C^*_c = compression index of reconstituted clay.

Figure 11 also locates the positions of Sedimentation (SCL) and Intrinsic (ICL) Compression Lines. These represent compressional characteristics of natural NC sedimentary and reconstituted clay respectively.

The compression curves of undisturbed sample at $\sigma'_v > \sigma'_p$ are significantly steeper than SCl and ICL, and only at σ'_v, one order of magnitude higher than σ'_p they merge into SCL. This fact highlights the importance of the structure of the Pancone clay at its natural state.

Data, analogous to that obtained for Layers A_3, B_2 and B_4 may be found in the work by Lancellotta et al. (1994). The results

Table 3 - Compressibility indexes from oedometer tests.

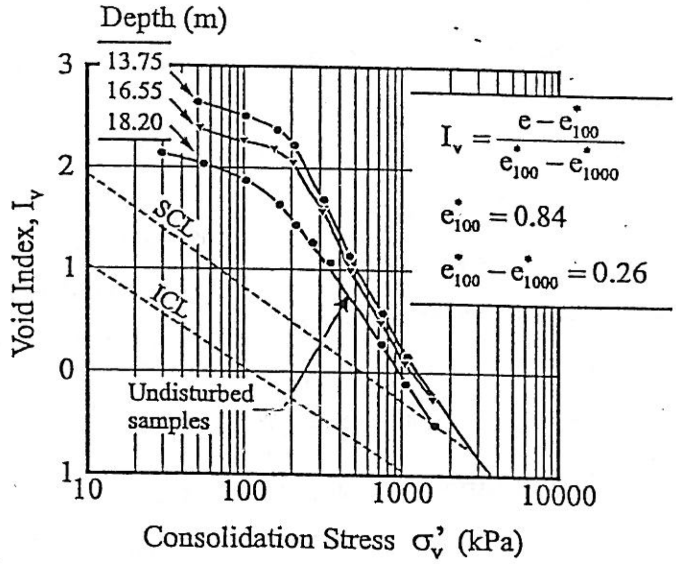
Horizon	Layer	C_{c1}	C_{c2}	$\frac{C_{c1}}{C_{c2}}$	C_s	OCR range	$\frac{C_{\alpha e}}{C_{c1}}$
A	A_3	0.243	0.243	1	0.023	2.4 ÷ 4.1	0.011
B	B_1	0.909	0.640	1.42	0.072	1.3 ÷ 2.0	0.035
	B_2	0.266	0.266	1	0.030	2	0.030
	B_4	0.280	0.280	1	0.057	1.3	0.023

C_{c1} = Primary compression index immediately beyond σ'_p

C_{c2} = Primary compression index at $\sigma'_v >> \sigma'_p$

C_s = Swelling index

$C_{\alpha e}$ = Secondary compression index immediately beyond σ'_p



SCL = sedimentation compression line
ICL = intrinsic compression line

Fig. 11 - Compression curves of upper Pisa clay in term of Void Index.

Table 4 - Drained shear strength from TX-CID compression tests.

Horizon	Layer	φ' (°)	c' kPa
A	A_3	31	0 to 20
	A_4	33	0
B	B_1	22	6 to 20
	B_2	28	12 to 30
	B_3	34	0
	B_4	27	0 to 5

(TX = triaxial test; CID = consolidated drained test)

collected by these authors led to the following values of C_c/C_c^* ratio for the tested clays: 1.4 for B_1, 1.0 for B_4, B_2 and A_3. Table 3 summarizes the characteristics of the different clay layers tested.

- The representative drained peak shear strength characteristics φ' and c' of different soil layers encountered under Piazza dei Miracoli are reported in Table 4. Those of clayey layers have been inferred from drained triaxial compression (TX-CD) tests performed on high quality undisturbed samples while those of sands have been estimated on the basis of q_c and standard penetration resistance N_{SPT}.

  The angle of friction at critical state ϕ'_{cs} has been determined only for clay of layer B_1, performing TX-CD tests on reconstituted material.
  These yielded values ranging between 24° and 25°.

- The undrained shear strength (s_u) of clay layers has been determined from K_o-consolidated undrained triaxial compression tests (TX-CK_oU) and K_o consolidated undrained direct simple shear tests (DSS-CK_oU). The tests for specimens reconsolidated under stresses representing the best estimate of those existing in situ, on average yielded the values of normalized s_u as reported in Table 5.

Table 5 - Normalized undrained shear strenght of upper Pisa clay

s_u/σ'_{vo}	*TEST*
$0.23\,(OCR)^{0.84}$	DSS - CK_0U
$0.29\,(OCR)^{0.84}$	TX - CK_0U

OCR = overconsolidation ratio
DSS = direct simple shear
TX = triaxial test
CK_0U = consolidated in K_0 - condition undrained

- The initial soil stiffness G_o, at a strain less than the linear threshold strain, has been inferred from V_s measurements performed during seismic-CPTU and from laboratory tests on high quality undisturbed samples reconsolidated to the best estimate of existing in situ stresses. Two kinds of laboratory apparatuses were employed; fixed-free resonant column apparatus and a special oedometer instrumented with pressure transducers measuring horizontal stress and bender elements allowing to generate and receive seismic body waves. A comparison of

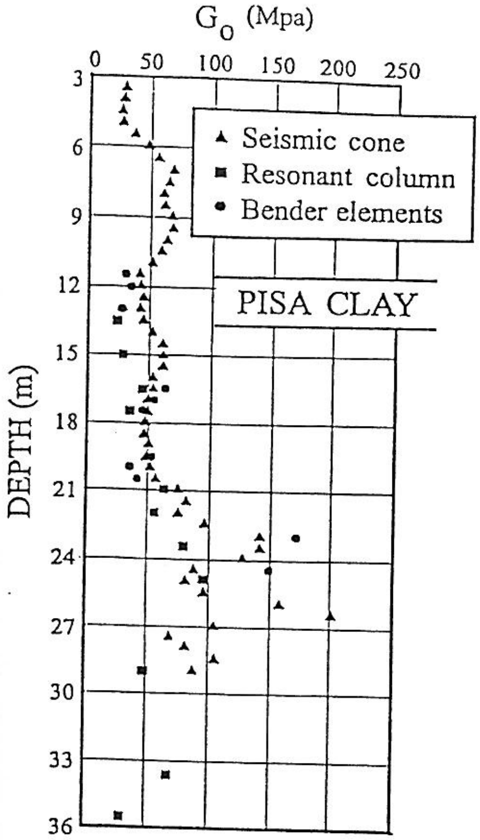
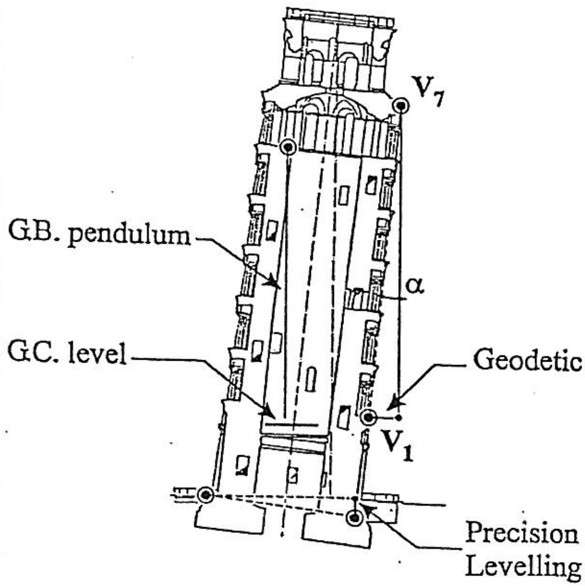


Fig . 12 - Maximum Shear Modulus from in-situ and laboratory tests

the results of in situ and laboratory tests, in terms of G_o are reported in Fig.12. Additional information concerning these tests may be found in the work by Jamiolkowski et al. (1994).

MOVEMENTS OF THE TOWER

The systematic monitoring of the Tower started in 1911 adopting the so called geodetic method which measures the degree of tilt. It consists in measuring, from a fixed station in Piazza dei Miracoli, the horizontal distance between the South edges of the 7th and the 1st cornices. Such measurements were usually performed twice a year, and incorporated 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.



- Geodetic measurements, horizontal movements of points V1 and V7, started in 1911.
- Precision levelling of 15 points located on foundation (1928, 1929, 1965 through 1986, 1990).
- G.C. level, instrumentation room at level of 1st cornice, started in 1934.
- G.B. Pendulum Inclinometer 30m long, fixed to internal wall at 6th cornice, started in 1934.

Fig. 13 - Measurements of Tower Tilt in years 1911 through 1992

In 1934 two additional monitoring devices were installed:

- Genio Civile (GC) Bubble Level installed in the instrumentation room located at the level of 1st cornice, see Fig.13.
  It allows to measure, over a span of 4.5 m, the tilt on two orthogonal planes N-S and E-W. The measurements, till 1992, were taken once a week and they were only moderately affected by wind action and temperature changes.
- Girometti-Bonecchi Pendulum Inclinometer, 30 m long. It was fixed to the internal wall of the Tower at the elevation of the 6th cornice (Fig.13). It swings 1.5 m above the instrumentation room floor.
  The continuous measurements reveal the displacements of the Tower on the same two orthogonal planes simultaneously to those relevant to 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.

In 1965 the high precision levelling of fifteen bench marks (Fig.13) located on the foundation plinth was initiated. Due to the lack of deep datum point, all settlement measurements are relative because they are based on a bench mark located on the cast iron door of the Baptistery. Given the position of the bench marks in consideration, as well as the insignificant affect of temperature changes, these measurements are more reliable than others and suitable to reflect the evolution of the rigid tilt of the Tower foundation.

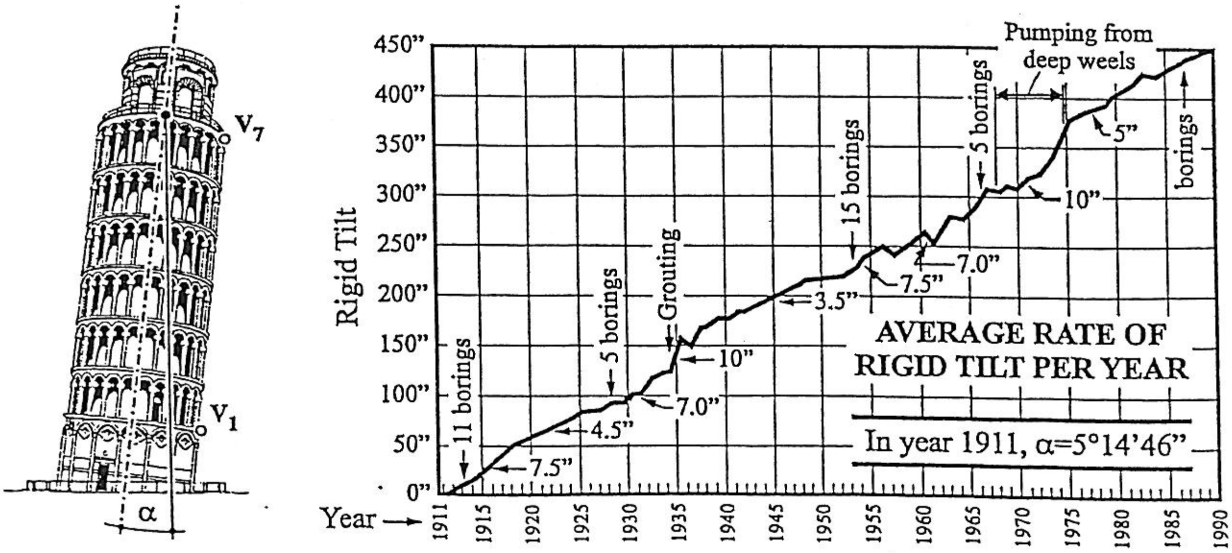


Fig. 14 - Rigid Tilt of Leaning Tower of Pisa

An overall picture of the Tower tilt on the North-South plane since 1911 is shown in Fig.14. It is based on geodetic and GC-Level measurements which lead to comparable and reliable results if examined on a long term basis.

A long-term trend of a steady increase in the Tower inclination emerges from this figure. This trend shows three major perturbations: the first occurred suddenly in 1935, a second one began in the mid sixties and continued gradually over the following ten years, and the third one occurred in 1985.

The first relevant perturbation occurred in the mid thirties (≅30") during the works aimed at redoing the catino and the cement grouting into the base of the Tower. During these works, before sealing the water proof joint between the plinth and the catino, quite intensive dewatering was put into operation.

The second perturbation was first observed during the site investigation carried out by the Polvani Commission, see Croce et al. (1981), and originated serious concerns. 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 between 1970 and 1974. The lowering of the watertable caused an increase of the tilt of approximately 40 seconds of arc in the North-South direction and of about 20 seconds of arc in the East-West direction. Following these observations, a number of wells in the vicinity of the Tower were closed allowing a partial recovery of the piezometric level reached in 1975 and 1976.

Soon afterwards a significant decrease in the rate of tilt was recorded.

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 graph the rate of the Tower inclination, which does not include the consequences of the mentioned events and of the environmental changes, Burland (1990a) 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.15, show a slow but steady increase in the rate of tilt

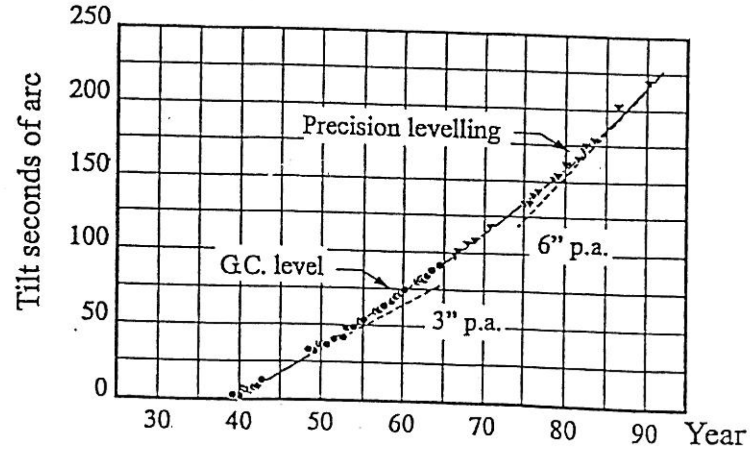


Fig. 15 - Net Tilt of Tower Plinth in years 1938 through 1990

which implies the future overturning instability of the Tower.

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

In the early nineties, prior to the stabilization works on the Tower and the consolidation of its masonry, a new monitoring system, having a high degree of redundancy, was implemented to continuously control in real time the movements of the Tower. Details may be found in works by Burland and Viggiani (1994) and Burland (1995).

This system consists in:

- Eight internal bench marks, 101 trough 109, see Fig.16, installed at the ground floor level in the entrance to the Tower.
- These survey points are linked to the previously mentioned fifteen external bench marks, 901 through 915 in Fig.16, located externally on the Tower plinth.
- Twenty-four bench marks, 1 through 24, see Fig.16, used to monitor the movements of Piazza dei Miracoli by means of precision levelling.
- Deep datum point, DD1 in Fig.16, the most important point of reference for all levellings, reveals the

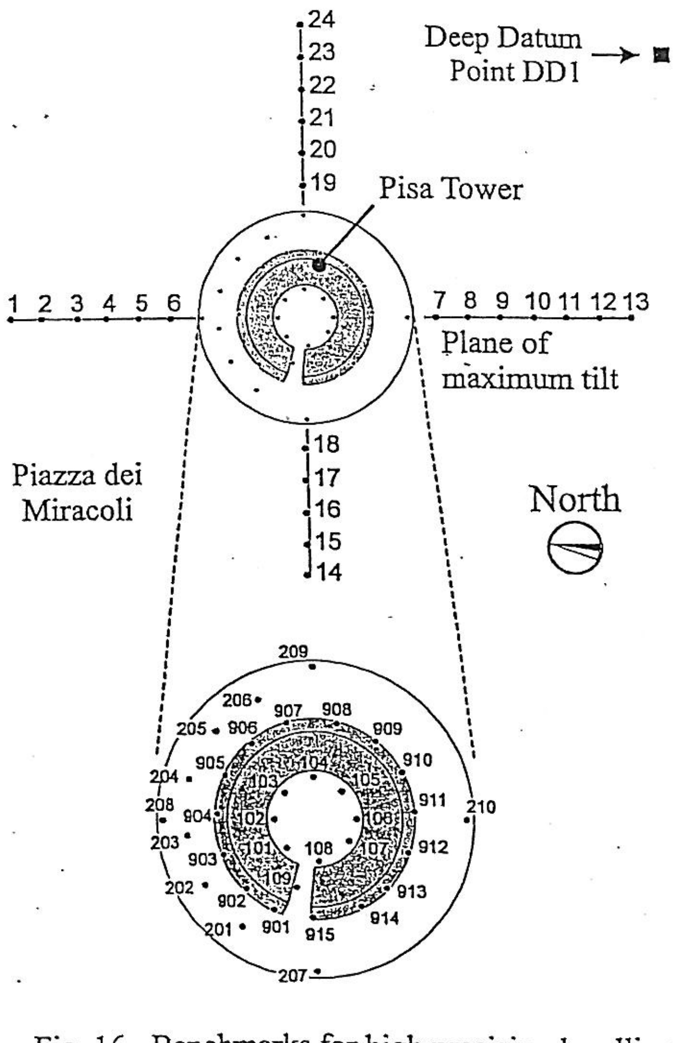
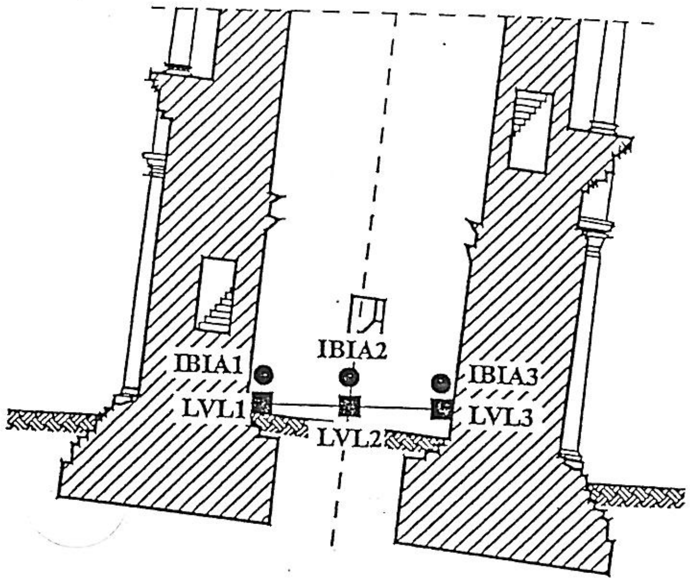


Fig. 16 - Benchmarks for high precision levelling.

absolute movements of the Tower and the ground surrounding it.

- Biaxial electrolytic inclinometers, IBIA in Fig.17, are located on the ground floor in the entrance to the Tower. The inclinometers and the automatic hydraulic livellometers, shown in the same figure, allow for the continuous measurement of change in monument tilt over a short term.

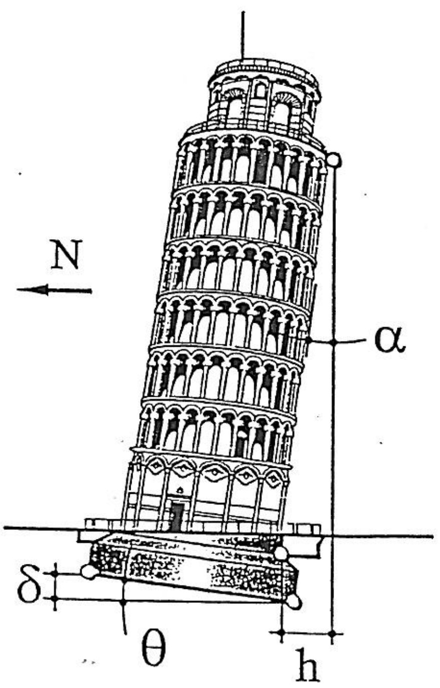


■ LIVELOMETER (LVL) ● BIAXIAL INCLINOMETER (IBIA)

Fig 17 - Measurements of Rigid Tilt at Tower Base

The description of additional instrumentation, also installed to monitor the movements of the Tower above the plinth and its masonry, is beyond the scope of this paper.

For the convenience of the reader and in relation to the monitoring exposed in the section dealing with the stabilizing measures, Fig.18 shows the reciprocal relationships between the inclination of the monuments (α) and its overhanging (h) as well as that between the plinth tilt (θ) and the relative settlement of its South edge (δ).



Inclination and overhanging in may 1993 before application of counterweight:

$\boxed{\theta = 5° \, 33' \, 36''}$

$\boxed{h = 4.47 \text{ m}}$

θ = inclination

δ = relative settlement of South edge with respect to North edge;
$\delta \, (\theta = 1'') \cong 0.095$ mm

h = overhanging referred to 7th "cornice";
$h \, (\alpha = 1'') \cong 0.22$ mm

$\theta = \alpha + 11'25''$

N.B. $1° = 60' = 3600''$; $\theta_{1998} = 19971''$

Fig. 18 - Inclination of Pisa Tower Terms of Reference.

LEANING INSTABILITY

The Tower began to lean Southwards during the second construction phase when the masonry weight exceeded 65% of the monument (Fig.6). This phenomenon has continued at a rate of 5 to 6 seconds per annum, a constant rate for the past few decades without taking into consideration environmental perturbations.

The constant rate of inclination and the relevant increase of the Tower tilt has raised much concern and controversy. Most importantly, it has always been debated the triggering factor for the phenomenon causing the continuing rotation at constant load since the end of the XIV Century as well as the present margin of safety in light of the risk of the Tower falling over.

The general consensus over the last decade [Hambly (1985, 1990), Lancellotta (1993, 1993a), Desideri and Viggiani (1994), Veneziano et al. (1995), Pepe (1995), Desideri et al. (1997)] has been that the behaviour of the Tower, since the end of construction, can be attributed to the phenomenon of the instability of equilibrium. A phenomenon similar to one relevant in the structural mechanics to initially bent slender structures, threatens the stability of tall, heavy top, structures seated on compressible soil. This kind of behaviour, also called leaning instability, is entirely controlled by the soil-structure interaction phenomena. In the case of the Pisa Tower it was triggered by the initial geometrical imperfection occurred during the second construction stage when the Tower started to lean Southwards. This can be explained in view of the fact that the resisting moment caused by a pronounced compressibility and non-linearity of soil support was unable to counteract the overturning moment generated by the ongoing tilt. The self driving mechanism was put into operation causing a steady increase of the Tower tilt, to the present, due to a progressive growth of the driving moment generated by the second order effects.

The reasons which have triggered the above depicted phenomenon of the leaning instability [Abghari (1987), Cheney et al. (1991)], are not completely understood. A number of hypotheses have been postulated by very authoritative authors:

- Differential compressibility and consolidation rate of the soft high plasticity clay layers belonging to Horizon B [Terzaghi (1960)].
- Spatial soil variability combined with differences in compressibility characteristics within Horizon A, together with local failure and consequent confined plastic flow developed in the upper part of Pancone Clay [Mitchell et al. (1977)].

- Leonards (1979) opted in favor of plastic yield of the soft Pancone clay leading to local shear failure.

- Non-homogeneity of the compressibility and permeability of soils in Horizon C has been postulated by Croce et al. (1981).

In addition, the incipient elastic instability has been suggested by Hambly (1985) as the possible cause for the initial rotation.

In essence, the mechanisms that have caused the initial geometrical imperfection triggering the leaning instability, continue to be uncertain. The writer believes that a combination of more than one of the events envisaged above have contributed to the rise of the initial inclination.

The leaning instability problem has been studied by many authors making reference to one and two degrees of freedom mechanical models shown in Fig.19. For more details see works by: Como (1965), Hambly (1985, 1990), Cheney et al. (1991), Lancellotta (1993, 1993a), Desideri and Viggiani (1994), Veneziano et al (1995), Desideri et al. (1997), Lancellotta and Pepe (1998) and others. Pepe (1995) examined these models from a theoretical point of view and presented the results of physical modelling of the Pisa Tower in the centrifuge which corroborate at phenomenological level the idea that the monument is threatened by the instability of equilibrium.

Even if a detailed discussion of the above studies is beyond the scope of this work, it may beneficial to the readers highlighting the following points:

- As pointed out by Lancellotta (1993, 1993a) and Veneziano et al. (1995) the one degree of freedom scheme, Fig.19 when coupled with a realistic model of soil restraint, offers a simple but rational approach for evaluating the present margin of safety and its evolution

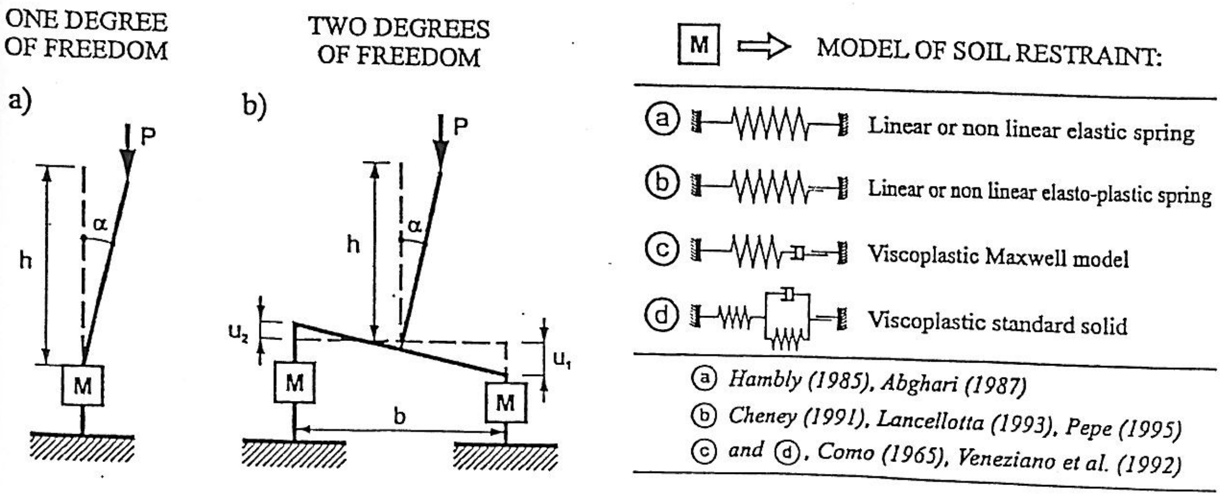


Fig. 19 - Leaning Instability Models.

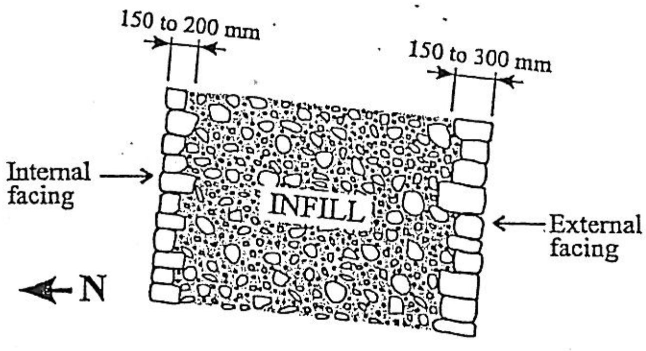
with time.

- The two degrees of freedom model [Pepe (1995), Lancellotta and Pepe (1998)] in addition to what stated above, makes it possible to investigate the effect of some of the stabilization measures that have been considered for a possible implementation on the Tower.
- In order to reproduce, in a realistic manner, the leaning instability phenomenon, the model of soil restraint referred to drained conditions should incorporate at least the following features: non-linearity of moment-rotation relationship, hypothesis about asymptotic value of resisting moment, influence of initial geometrical imperfection and of soil viscosity, variation of overturning moment with time due to secondary order effects.
- All attempts to evaluate the present factor of safety of the Tower against overturning, based on realistic soil models in which the viscous effects have been implicitly [Lancellotta (1993), Pepe (1995)], or explicitly [Veneziano et al. (1995)] considered, led invariably to very low values ranging between 1.1 and 1.2. Veneziano et al. (1995) using two different reological models, positively calibrated against historical rotation measurements, reached the conclusion that "it appears that instability of the foundation is at least several decades away. However, a non-negligible risk that Tower collapse will occur in 40 to 50 years with the risk to be around $2 \cdot 10^{-2}$ and $3 \cdot 10^{-3}$ respectively".

STRUCTURAL FEATURES

As shown in Fig.2 the Leaning Tower of Pisa consists of a hollow masonry cylinder, surrounded by six loggias with the bell chamber on the top.

The Tower 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.20. A helicoidal staircase allowing the visitors to climb up to the top of the Tower is located inside the annulus of the hollow cylinder.



Hollow cylinder, inner and outer surfaces faced with highly competent San Giuliano marble. Space between these facings is filled with rubble and mortar.

Fig. 20 - Cross-Section of Tower Masonry

The following are the essential characteristics of the Tower:

- total weight : $N \cong 142$ MN; average foundation pressure: $q \cong 497$ kPa;
- total height : $H = 58.36$ m; height above G.L.; $\cong 55$ m;
- distance from the centre of gravity to the foundation plane $h_g \cong 22.6$ m;
- annular foundation, inner diameter; $D_i \cong 4.5$ m, outer diameter $D_o \cong 19.6$ m;
- area of the annular foundation: $A \cong 285$ m^2;
- present inclination : $\alpha = 5° 28' 09''$;
- present eccentricity of N; $e \cong 2.3$ m.

Relevant mechanical properties of the two components of the Tower masonry are summarized in Table 6. 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 close to 8.0 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, 1997) is shown in Fig.21.

Table 6 - Mechanical properties of Pisa Tower Masonry

	σ_c (MPa)	σ_t (MPa)	E (Mpa)
San Giuliano Marble Facing	110 -190	4 - 8	70.000 - 90.000
Infill Masonry	4 -8	0,3 - 1,3	5.000 - 7.500

Thickness of facings: outside $\approx$200 mm ; Inside $\approx$150 mm

σ_c = Compression strength

σ_t = Tensile strength

E = Elasticity Modulus

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, infrared and radar tomographies;
- 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 leading to the appearance of the horizontal force components as evidendiated in Figs.21 and 22.

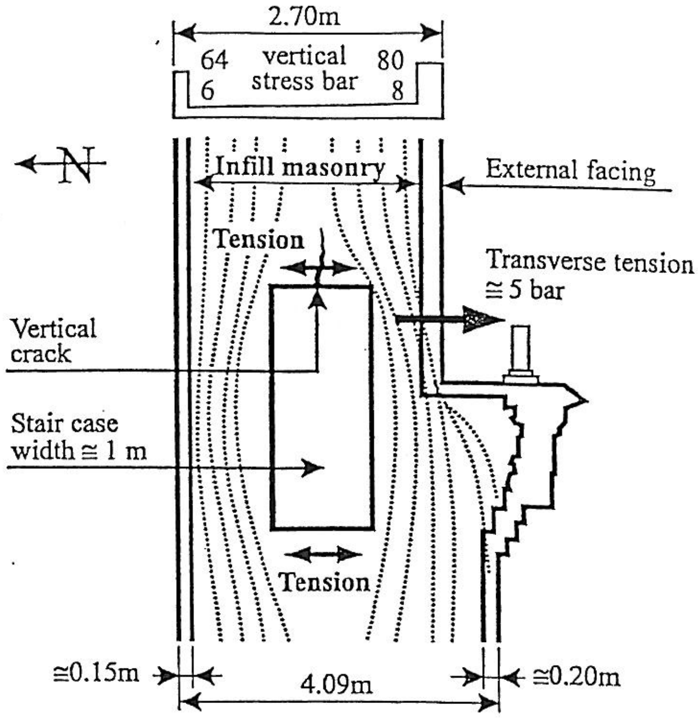


Fig. 21 - Cross-Section of Tower at first Cornice

The serious concern over the structural safety of the Monument led in 1989 to the decision by the Commission established by the MPW 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.

SPLITTING OF FACING
STONES DUE TO
STRESS CONCENTRATION

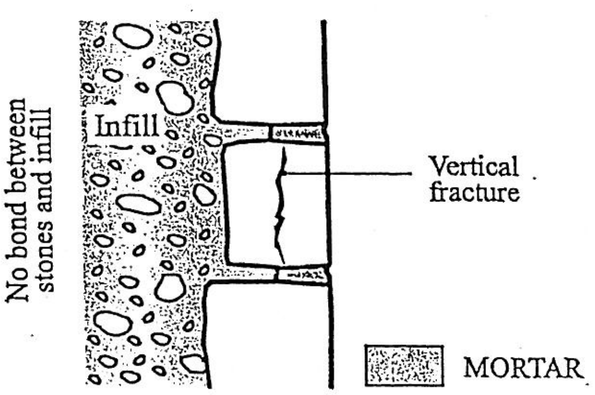
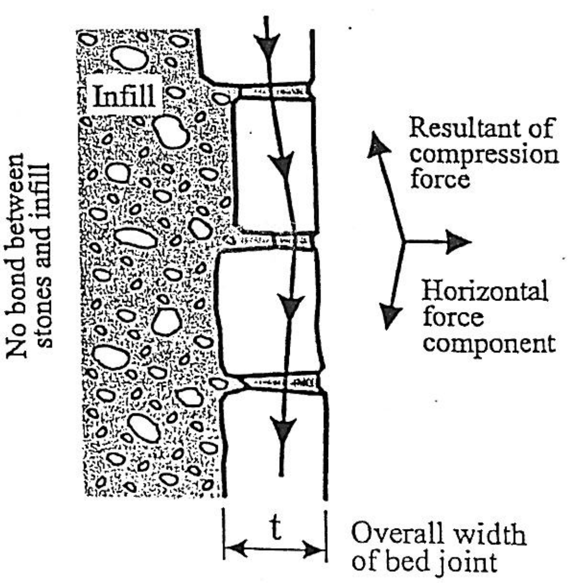


Fig. 22 - Marble Stone Facing
Imperfection of bed joints.

DEVIATION FROM VERTICAL
OF RESULTANT OF
COMPRESSION FORCE



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.

STABILIZATION WORKS

In the previous part of the paper it has been evidentiated that the leaning Tower of Pisa is endangered by two phenomena, i.e. instability of equilibrium and risk of fragile structural collapse of the masonry.
The two phenomena are obviously interdependent. The increasing inclination not only reduces the safety margin of the Monument with respect to the overturning but also causes a further increase of the stresses in the most critical section of the masonry, enhancing the risk of structural collapse.
In 1989, the MPW Commission chaired by Jappelli and Pozzati pointed out the risk of structural collapse which proved to be realistic when the XIII Century Civic Tower of Pavia [Macchi (1993)] collapsed without any warning. This event led to the closure of the Pisa Tower to the visitors in January of 1990, and triggered the appointment, by the Italian Prime Minister, of an International Committee for the safeguard and the stabilization of the leaning Tower of Pisa.
The Committee, the seventeenth in the long history of the monument [Luchesi (1995)] and the sixteenth in the modern times, has been charged to; stabilize the foundation, strengthen the structure and plan the architectural restoration and started its operations in September 1990.

The activities of the Committee can be grouped as follows:

- Numerous experimental investigations and studies dealing with a broad spectrum of problems(2), reflecting the multidisciplinary nature of the Committee and aimed at the most comprehensive learning of all the relevant features of the monument and its environment.
- The design and implementation, in a short time, of the temporary and fully reversible interventions to increase slightly the stability of the Tower foundation and to reduce the risk

(2) archeology, history of construction, strength of materials, numerical modelling of structure and foundation soils, in situ and laboratory tests, new monitoring system, methods of structural reinforcement, approach to architectural restoration, etc.

of structural collapse. This decision was taken in view of the awareness that the selection, the design and the realization of the permanent stabilization and consolidation works would require a long time.

– The studies by means of numerical and physical models as well using field trials, guiding in the selection and design of the final interventions.

  This task, especially the stabilization of the Tower with regards to the leaning instability, poses serious limitations on the selection of the appropriate solution due to the following circumstances:

  - The unanimous decision of the Committee to adopt a solution fully respecting the artistic and cultural value of the monument.

    It was given preference to the intervention able to stop and reduce the tilt of the Tower plinth acting only on the subsoil without touching the monument.

  - Given the extremely reduced safety margin of the Tower with respect to falling over, any invasive interventions like underpinning, enlargement of the plinth, etc. would represent a serious risk of collapse in the transitory phase during the execution of works.

  In these circumstances two possible solutions for stabilizing the foundation have been envisaged, both aimed at inducing differential settlement of the North edge of the plinth with respect to the South.

A brief description of the temporary stabilizing measures as well the studies and the design of the final intervention aimed at stopping-reducing the inclination of the Tower will be given in the next sections.

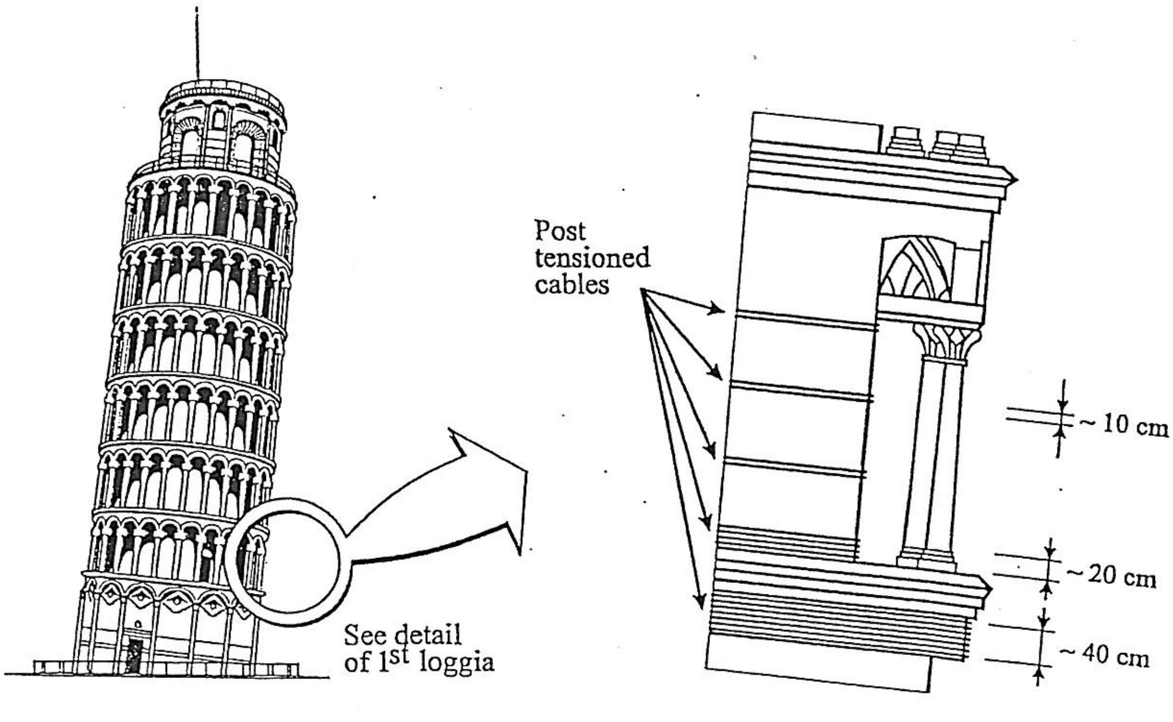


Fig. 23 - Temporary structural strengthening

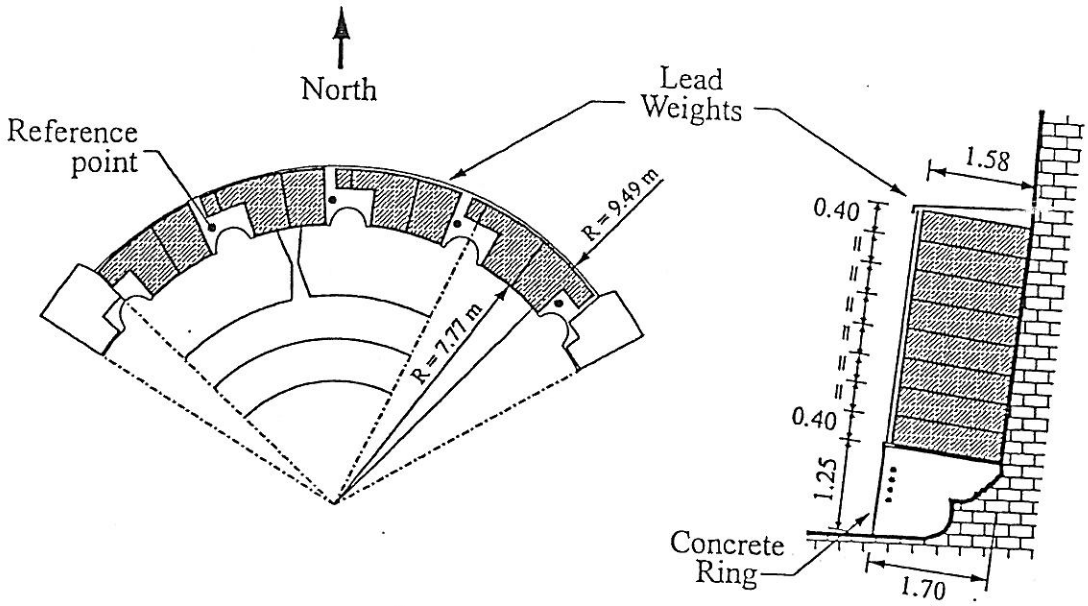


Fig. 24 - Counterweight on North edge of Tower plinth

The temporary, and completely reversible, intervention aimed and improving the structural safety of the most critical cross-section of the masonry at the level of the first loggia has been completed in 1992. It consist of 18 lightly post-tensioned tendons located in the places shown in Fig.23, their function is to prevent local buckling in compression of the marble stones forming the external facing.

The steady motion of the Tower, increasing its inclination by 5" to 6" per annum, led to the decision to implement a second temporary and fully reversible intervention aimed at reducing the rate or even stopping the progressive increase of inclination. This intervention consisted in

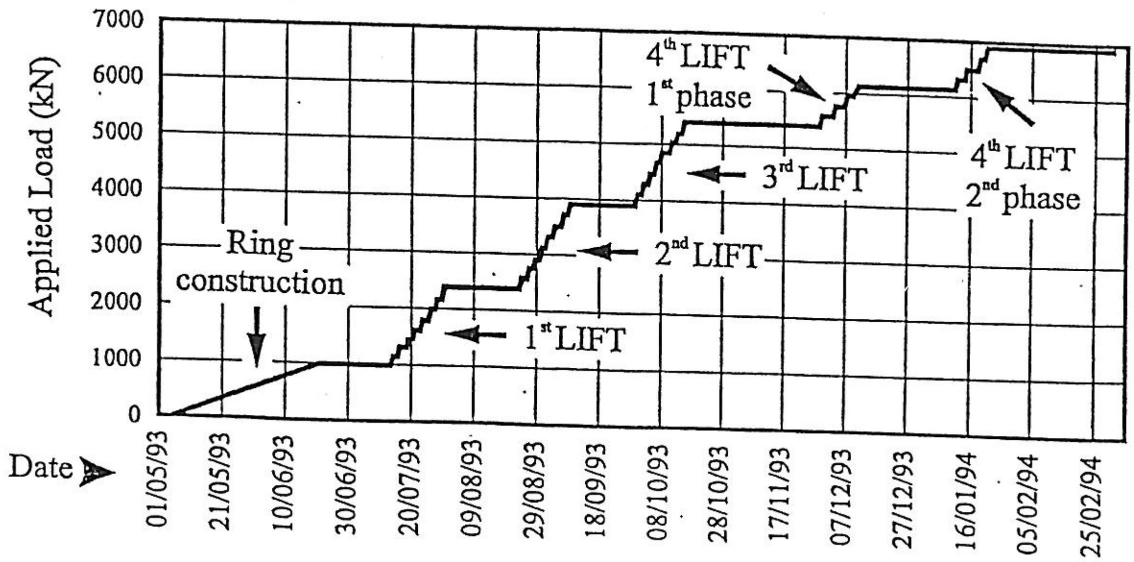


Fig. 25 - Counterweight Loading Sequence

placing 6 MN of lead ingots on the North edge of the plinth as shown in Fig.24. The lead ingots have been placed gradually (Fig.25) on the prestressed concrete ring shown in Fig.24 generating a stabilizing moment of 45 MNm. The counterweight placed in the period between May 1993 and January 1994, see Fig.26 has determined a very positive response of the monument, which, for the first time in its history, inverted the direction of the movement reducing slightly the inclination.

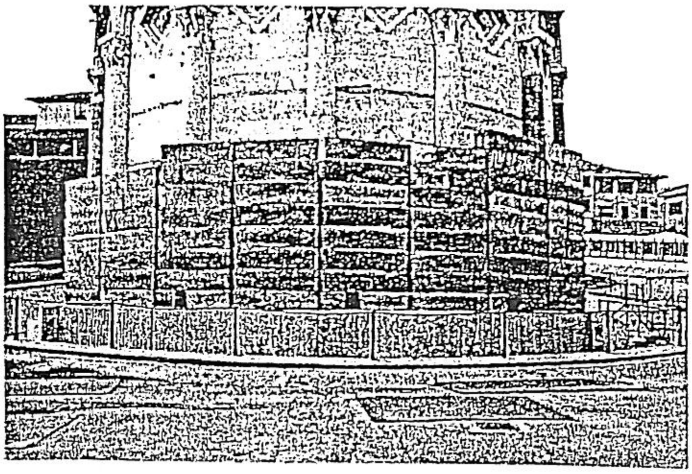


Fig . 26 - The Counterweight placed in the period between May 1993 and January 1994

The effects of the Tower tilt monitoring during the application of the lead ingots is reported in Fig.27. It results that during the loading stage the monument reduced its inclination by 34" which grew up to 54" during the following six months.

In view of the positive response of the Tower to the counterweight, but considering its visual impact, it was decided to replace the lead ingots by ten deep anchors having each a working load of 1000 kN, see Fig.28. This intervention was conceived as an intermediate measure

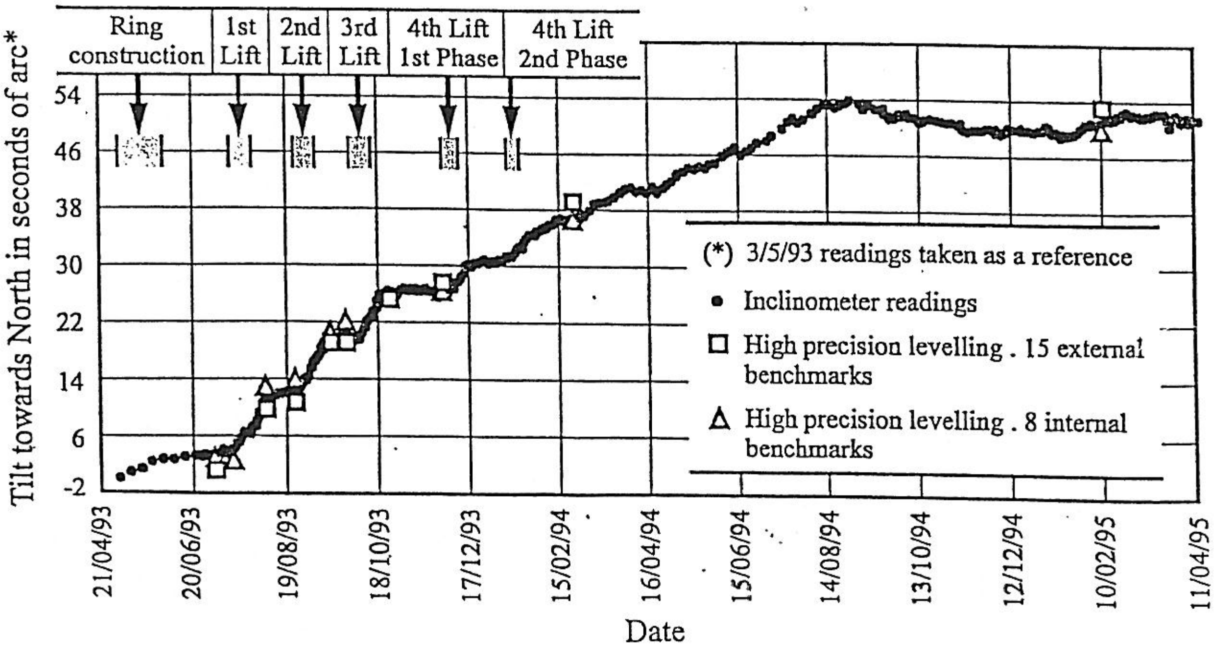


Fig. 27 - Tilt towards North as result of counterweight application.

between the temporary and the final one and presented the following advantages:

- Double the stabilizing moment with an increase of vertical load of only two third of that due to lead ingots.
- Create at the North edge of the plinth, one-directional rotational constraint able to counteract to some extent any tendency of the Tower to tilt southwards.

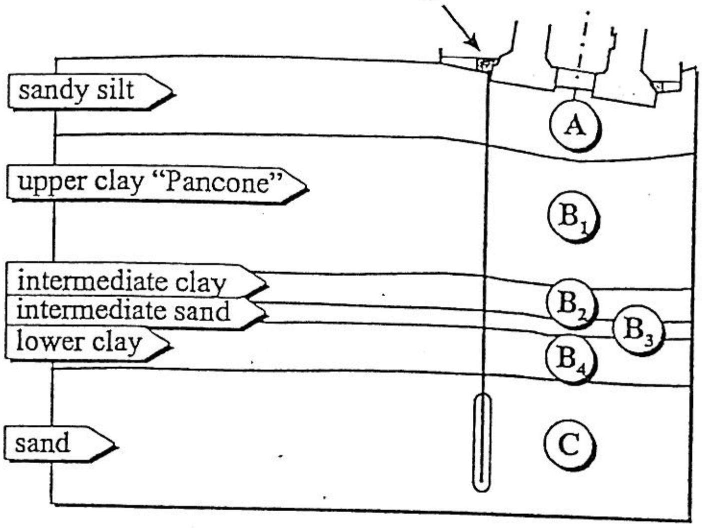


Fig. 28 - Ten Anchors solution

The implementation of this solution required the construction of a second prestressed concrete ring below that supporting the lead ingots therefore hidden beneath the catino. This, in turn, required an excavation below the perched G.W.L. ranging from 0.3 m at North to 2.0 m, South of the catino.

The design of the ten anchors solution has been developed based on the information gathered by the previous commissions, considering the catino statically independent from the Tower plinth. The only known connection was the water-proofing joint located in proximity to the foundation perimeter.

Unfortunately, during the implementation of this solution it was discovered that in the past there had been two attempts to enlarge the Tower foundation:

- The first, probably due to Della Gherardesca, who during the construction of the catino, placed around the Tower plinth, at 0.7 to 0.8 m, a thick layer of mortar conglomerate having the same width of the catino.
- The second one was implemented by the local authority for public works which in mid thirties had redone the catino. During this intervention involving the cement grouting of the Tower plinth, the under-catino conglomerate was connected to the foundation by means of steel tubes 70 mm and approximately 700 to 800 mm long. Information about this work was never reported in the official documents and was unknown to the professionals dealing with Tower till the summer of 1995.

In view of the above, the hypothesis that the catino is statically independent from the Tower is become no more truthful, see Fig.29. Moreover, considering that since mid thirties, the South edge of the Tower plinth has settled 20 to 25 mm more than the North one, it is likely that some limited load has been shared since then from the monument to the South part of catino.

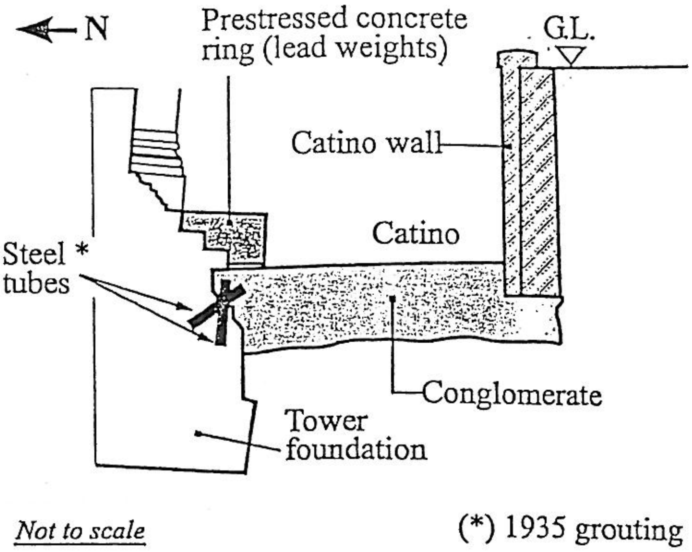


Fig. 29 - South section of Catino - Actual configuration

In fact, during the first attempt to remove in small segments the South part of the catino to build the prestressed concrete ring for the ten anchors, the Tower started to tilt towards South with a rate of 3" to 4" per day with serious concern for its stability. The phenomenon which occurred in September 1995 was counteracted by applying additional 2700 kN [Fig.30] of the lead ingots on the North edge of the plinth. Ever since, the Tower has been motionless as far as its inclination is concerned, see Fig.31. Subsequently, the design of the ten anchors solution has been modified so that to avoid any modification of the South part of catino. Whether this intervention will be completed or not, has not yet been decided by the Committee. The decision with this respect will depend on the results of the under excavation intervention described in the following.

Since 1993, the Committee has undertaken the studies aimed at finding a solution to reduce the inclination of half of degree, acting only on the foundation soils without touching the Tower. Two possible interventions, able to induce ≅200 mm of settlement of North edge of the plinth with respect to South one, have been taken into consideration. The

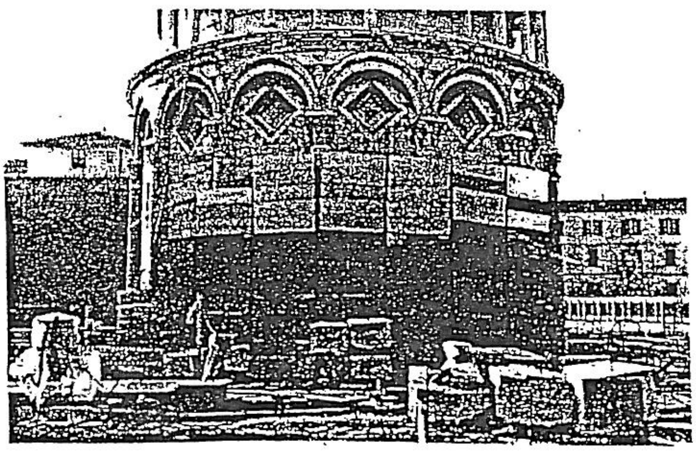


Fig. 30 - The Counterweight with additional 2700 KN added in September 1995

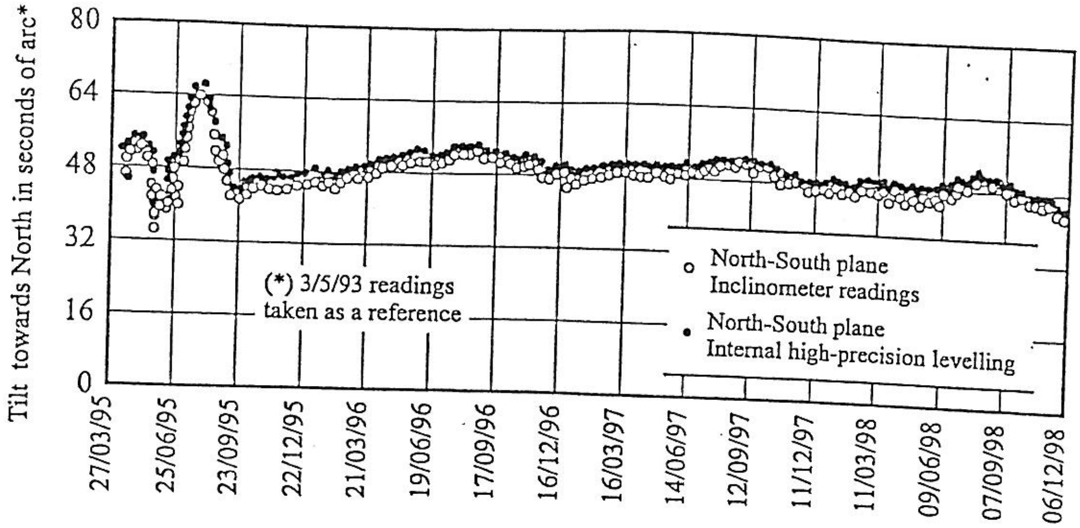


Fig.31 - Tilt of Pisa Tower since may of 1995

electro-osmosis aimed at reducing the water content hence inducing a volume change in the most upper part of Pancone clay and the gradual extraction of the soil from the lower part of Horizon A, as postulated many years ago by the Italian civil engineer Terracina (1962), see Fig.32. The method which has recently been successfully employed to mitigate the impact of very large differential settlements suffered by the Metropolitan Cathedral of Mexico City [Tamez et al. (1992, 1997)].

The large scale field trial test performed on the Piazza dei Miracoli evidentiated the non feasibility of the electro-osmosis, thus all efforts concentrated on investigating the possibility to apply the ground extraction, thereafter named underexcavation. In order to ascertain its feasibility, numerical analyses, physical modelling both in terrestrial gravity field and in centrifuge, as well large scale trial field have been performed. The latter was not only useful as far as the verification of the feasibility of the underexcavation was concerned, but allowed also to test and finalize the technological aspects of the intervention.

- Reduction of contact pressure on South side
- Reduction of present inclination (~ 10%) by 1% would suffice.
- Simplest manner, removal of soil under North side by series of borings.
- Regulating number position and diameter of borings, desired reduction of tower inclination can be achieved

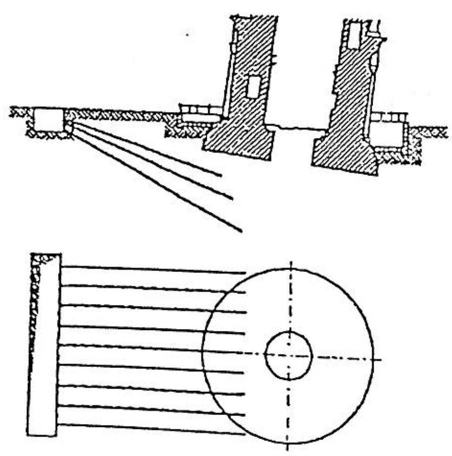


Fig. 32 - Underexcavation for correcting inclination of Pisa Tower (Terracina 1962)

In order to perform the trial field, a 7 m in diameter circular reinforced concrete footing was built on the Piazza far from the Tower, see Fig.33, and was loaded eccentrically with the concrete blocks. Both the footing and the underlying soil were heavily instrumented to monitor settlements, rotations, contact pressure and the induced excess pore pressure during the experiment. After a waiting period of a few months, allowing the completion of consolidation settlements, the ground extraction commenced by means of inclined borings having ≅150 mm in diameter as schematically shown in Fig.33. The under excavation was performed extracting gradually the soil from Horizon A by means of a procedure, shown in Fig.34, which made it possible to reduce the inclination of the trial plinth by almost 1000" of arc, as documented in Fig.35.

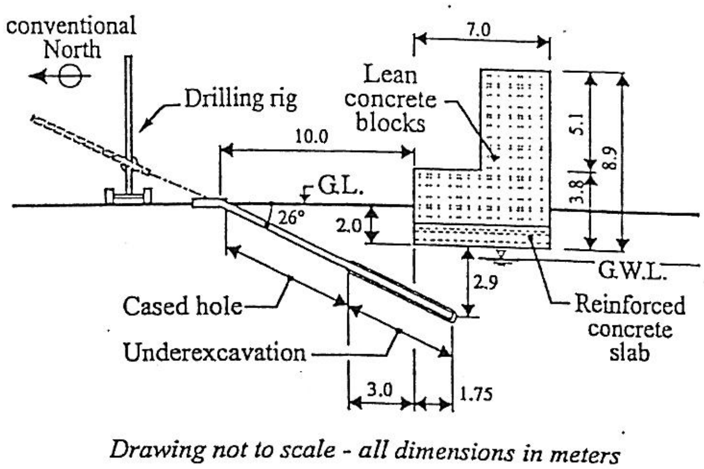


Fig. 33 - Underexcavation field trial

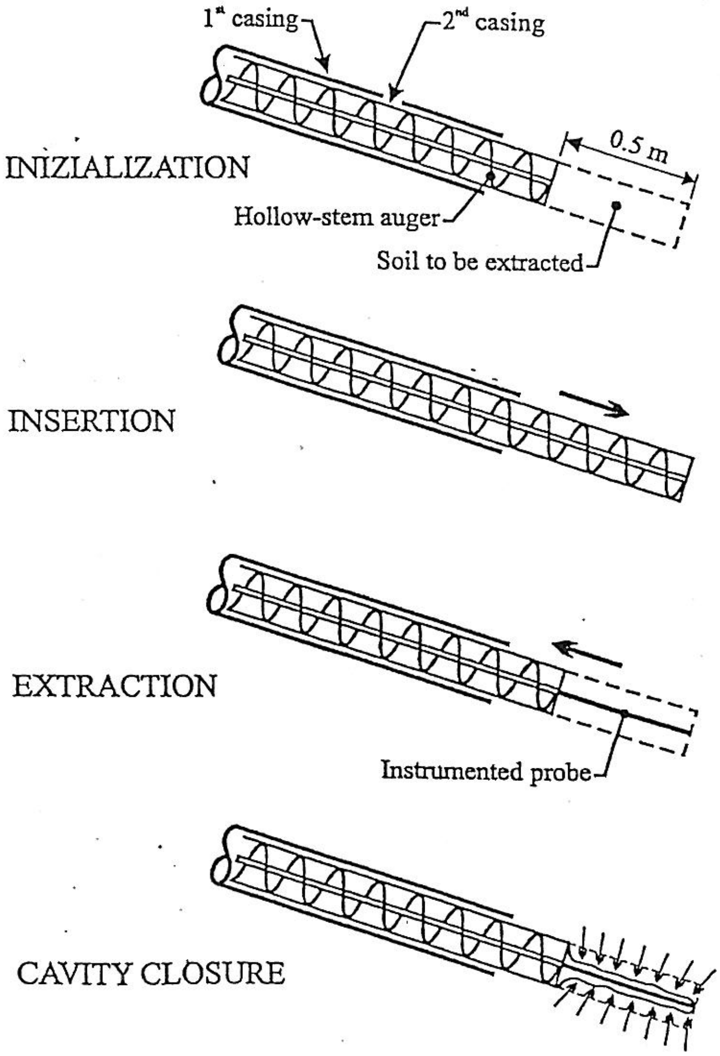


Fig. 34 - Soil extraction process

During this experiment, the following important lessons were learned:

- A critical penetration exists under the plinth. If the extraction hole exceeds it a

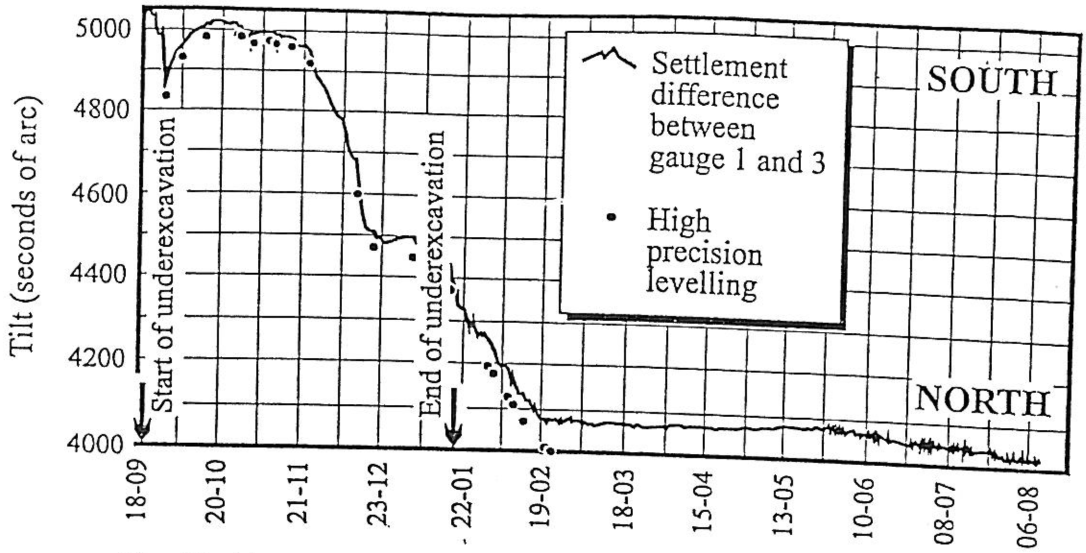


Fig. 35 - Underexcavation field trial - Tilt of plinth in North-South plane.

rotation of the foundation in the opposite direction is experienced. Such an accident occurred around end of September 1995 and may detected from Fig.35.

- Using an appropriate sequence of ground extraction operations it was possible to steer the movements of the plinth both in N-S and W-E plan in the desired way.
- Soon after the completion of the underexcavation, on February 1996, the trial plinth came to rest and up to January 1999 has exhibited negligible movements.

Because of the successful validation of the underexcavation by trial field, it was decided to start this intervention under the Tower.

A preliminary ground extraction under the monument has been planned, well aware that, by no means, the trial plinth can be considered as a model reflecting completely a possible response of a Tower suffering from the leaning instability. This preliminary intervention will consist in twelve holes whose penetration under the North rim of Tower plinth will not exceed 1 m referring to the scheme shown in Fig.36.

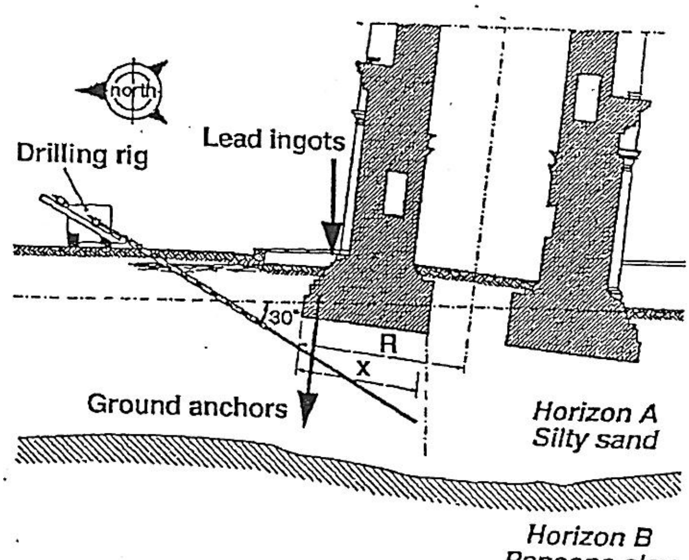


Fig. 36 - A hole for ground extraction under the Tower

Based on the response of the monument, referring to the scheme shown in Fig.36, in terms of rotations and settlements to this preliminary intervention, the conclusive decision will be taken on the use of the discussed method as a tool for the final stabilization of the Tower.

To hinder any unexpected adverse movement of the Tower that could occur during this or any other interventions aimed at final stabilization of the Tower, a safeguard structure has been implemented consisting in the cable stay shown in Fig.37..

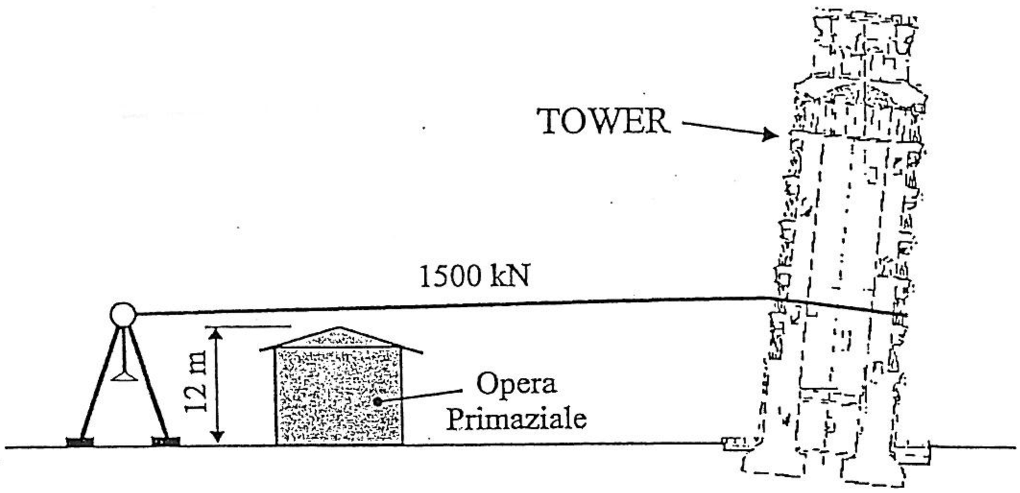


Fig. 37a - Cable Stay Structure - Cross-Section

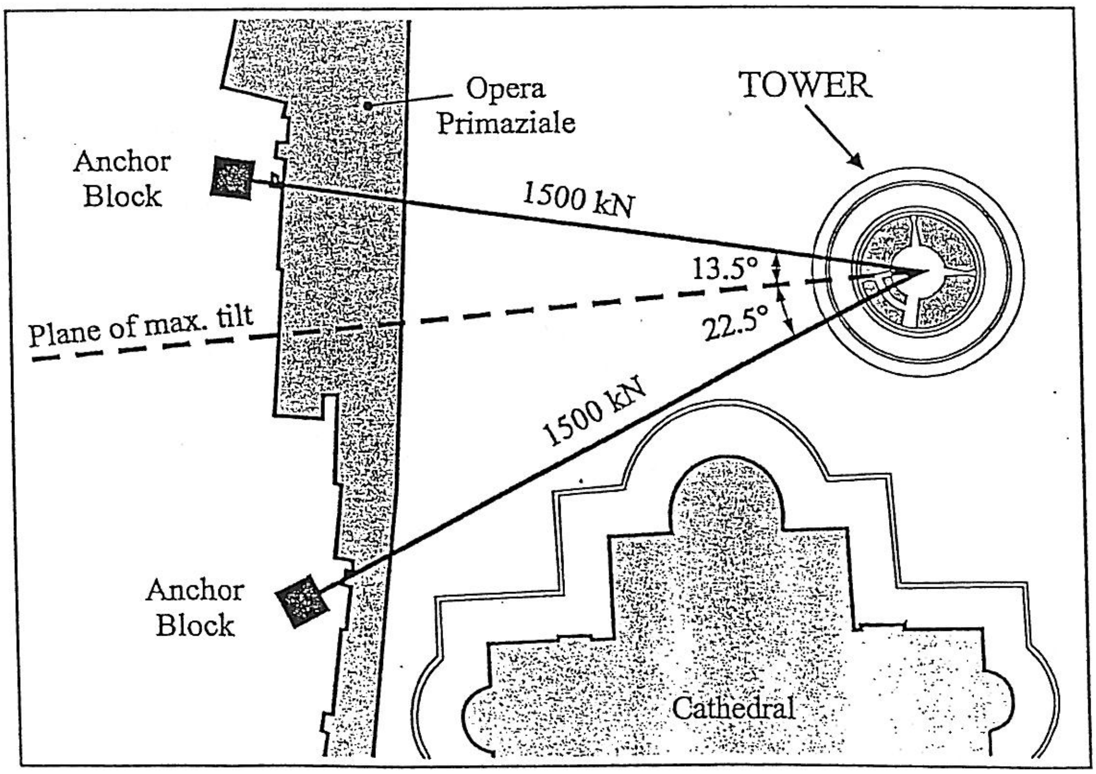


Fig. 37b - Cable Stay Structure - Plan

ABBREVIATIONS

ASCE	American Society of Civil Engineers
ECSMFE	European Conference on Soil Mechanics and Foundation Engineering
ICSMFE	International Conference on Soil Mechanics and Foundation Engineering
JGE	Journal Geotechnical Engineering
MPW	Ministry of Public Works of Italy
RIG	Rivista Italiana di Geotecnica

REFERENCES

Abghari, A. (1987). "Leaning Instability of Tall Structures". Ph.D. Thesis, University of California, Davies, Ca.

Berardi, G., Caroti, L. Giunta, G., Jamiolkowski, M. and Lancellotta, R. (1991). "Mechanical Properties of Upper Pisa Clay". Proc. X ECSMFE, Firenze.

Burland, J.B. (1990). "On the Compressibility and Shear Strength of Natural Clays". The 30th Rankine Lecture, Geotechnique, n°3.

Burland, J.B. (1990a). Personal Communication.

Burland, J.B. (1991). Personal Communication.

Burland, J.B. (1995). "Monitoring the Foundations of the Leaning Tower of Pisa". Proc. Seminar on Instrumentation in Geotechnical Engineering, Hong Kong, Institution of Engineers.

Burland, J.B. and Viggiani, C. (1994). "Osservazioni del compartimento della Torre di Pisa". RIG, n°3.

Calabresi, G., Rampello, S. and Callisto, L. (1993). "The Leaning Tower of Pisa – Geotechnical Characterization of the Tower's Subsoil within the Framework of the Critical State Theory". Report of the University of Rome La Sapienza, Report.

Casagrande, A. (1936). "The Determination of the Pre-Consolidation Load and Its Practical Significance Discussion, Proc. I ICSMFE, Harward University. Cambridge, Mass.

Cheney, J.A., Abghari, A. and Kutter, B.L. (1991). "Stability of Leaning Towers". JGE Div. ASCE, n°2.

Como, M. (1965). "Asymptotic of a Leaning Tower on Slowly Sinking Ground". Giornale del Genio Civile (in Italian).

Costanzo, D. (1994). "Mechanical behaviour of Pisa Clay". Ph. D. Thesis, Technical University of Torino, Italy.

Costanzo, D., Jamiolkowski, M., Lancellotta, R. and Pepe, C. (1994). "Leaning Tower of Pisa – Description of the Behaviour". Banquet Lecture, Settlement '94, College Station, TX.

Croce, A., Burghignoli, A., Calabresi, G., Evangelista, A. and Viggiani, C. (1981). "The Tower of Pisa and the Surrounding Square: Recent Observations". Proc. X ICSMFEE, Stockholm.

Desideri, A. and Viggiani, C. (1994). "Some Remarks on Stability of Towers". Proc. Symposium on Developments 1936-1994 in Geotechnical Engineering, Bangkok.

Desideri, A., Russo, C. and Viggiani, C. (1997). "Stability of Towers on Compressible Ground". RIG, n°1.

Hayat, T.M. (1992). "The Coefficient of Earth Pressure at Rest". Ph. D. Thesis, University of Illinois at Urbana, Champaign, Ill.

Hambly, E.C. (1985). "Soil Buckling and the Leaning Instability of Tall Structures". The Structural Engineer, No.3.

Hambly, E.C. (1990). "Overturning Instability". JGE Div. ASCE n°4.

Jamiolkowski, M. (1988). "Research Applied to Geotechncial Engineering". Proc. Instn. Civ. Engrs., Part 1, Vol.84.

Jamiolkowski, M., Lancellotta, R. and Lo Presti, D.C.F. (1994). "Remarks on the Stiffness at Small Strains of Six Italian Clays". Proc. Int. Symposium on Prefailure Deformation of Geomaterials, Sapporo, Japan.

Lancellotta, R. (1993). "The Stability of a Rigid Column with Non-Linear Restraint, Geotechnique, n°1.

Lancellotta, R. (1993a). "The Leaning Tower of Pisa – Geotechnical Theory and Model Tests". Discussion, Soil and Foundation, n.3.

Lancellotta, R. and Pepe, C. (1990). "Pisa Tower – Geotechnical Properties of the Horizon A". Research Report 2.1, Technical University of Torino.

Lancellotta, R. and Pepe, C. (1990a). "Mechanical behaviour of Pisa Clay". Research Report 2.2, Technical University of Torino.

Lancellotta, R., Costanzo, D. and Pepe, M.C. (1994). "The Leaning of Tower of Pisa – Geotechnical Characterization of the Tower Subsoil". A Summary Report Report of the Technical University of Torino.

Lancellotta, R. and Pepe, M.C. (1998). "On the Stability of Equilibrium of the Leaning Tower of Pisa". Accademia della Scienza – Atti di Scienze Fisiche n°132.

Leonards, G.A. (1979). "Foundation Performance of Tower of Pisa". Discussion. JGE Div. ASCE, GT1.

Leonhardt, F. (1991). Personal Communication.

Leonhardt, F. (1997). "The Committee to Save the Tower of Pisa: A Personal Report". Structural Engineering International, n.3.

Luchesi, G. (1995). "La Torre Pendente – Un Mistero Lungo Ottocento Anni". Edit. Bandecchi e Vivaldi, Pontedero.

Macchi, G. (1993). "Monitoring Medieval Structures in Pavia". Structural Engineering International, n°1.

Mesri, G. (1989). Personal Communication.

Mesri, G. and Castro, A. (1987). "The C_a/C_c Concept and K_o during Secondary Compression". JGE Div. ASCE, GT3.

Mesri, G., Hediew, J.E. and Shahien, M. (1997). "Geotechnical Characteristics and Compression of Pisa Clay". Proc.XIV ICSMFE, Hamburg.

Mitchell, J.K., Vivatrat, V. and Lambe, W.T. (1977). "Foundation Performance of Tower of Pisa". Journ. of JGE Div. ASCE, GT3.

MPW Ministero dei Lavori Pubblici (1979). "La stabilità della Torre di Pisa". Istituto Poligrafico e Zecca dello Stato, Roma.

MPW Ministero dei Lavori Pubblici (1971) "Ricerche e studi su la Torre pendente di Pisa ed i fenomeni connessi alle condizioni di ambiente". Edit. I.G.M., Firenze.

Pepe, M.C. (1995). "The Leaning Tower of Pisa – Theoretical and Experimental Analysis of the Stability of Equilibrium". Ph.D. Thesis – Technical Univ. of Torino (in Italian).

Schiffmann, R.L. (1995). "Settlements due to Groundwater Fluctuations in the Lower Sand". Report submitted to International Committee for Safeguard of the Leaning Tower of Pisa.

Tamez, E. Santoyo, E. and Cuaves, A. (1992). "The Metropolitan Cathedral of Mexico City and El Sagrario Church". Correcting Foundation Behaviour, Commemorative Volume Raul, J. Marsal, Edit. The Mexican Society for Soil Mechanics, in Spanish.

Tamez, E., Ovando, E. and Santoyo, E. (1997). "Underexcavation of Mexico City's Metropolitan Cathedral and Sagrario Church". Proc. XIV ICSMFE Hamburg.

Terracina, F. (1962). "Foundations of the Tower of Pisa, Geotechnique, n°4.

Terzaghi, K. (1960). "Die Ursachen der Schiefstellung des Turms von Pisa". n°1/2, 1934, pp.1-4 Reprinted in Bjerrum et al., eds., John Wiley and Sons, Inc., New York, 1960.

Veneziano, D., Van Dyck, J. And Koseki, J. (1995). "Leaning of the Tower of Pisa". Part I: Reological Models, Part II: Future Rotations and Risk of Instability". Draft of papers.

Latest edition: Facts-at-a-Glance
June 2000

The preliminary underexcavation under the Tower was started on February 8th, 1999.

The intervention consisted in a very gradual extraction of ≅7 m^3 of soil, mostly beneath the catino and only partially under the Tower's plinth. The operation was performed through 12 holes inclined by 26° with respect to the G.L. (figure 38). Only a few central holes (figure 39) penetrated 1.5 m under the North edge of the foundation.

The preliminary underexcavation was completed on June 6th 1999 leading to an extremely positive response of the Tower, see figures 40 and 41:

- At the end of the preliminary underexcavation the Tower rotated Northward at 90 arc-seconds.
- Thereafter, it continued its rotation Northward at a reduced rate until mid September 1999, achieving a reduction of inclination of 132 arc-seconds.

  Ever since and until the end of January 2000, the Tower has been motionless as far as the inclination is concerned, see figure 40.
- As to the result of the preliminary underexcavation, the Southern edge of the plinth has risen by 1.5 mm while the Northern edge has settled at 12 mm. The observed behaviour corresponds to two favorable phenomena: *a)* the point of instantaneous rotation is located within the imprint of the plinth; *b)* as a consequence, a small reduction of contact stress under the Southern edge of the foundation occurs.

In view of the positive response of the tower to the preliminary intervention, the Committee has undertaken the full underexcavation aimed at reducing the tower inclination by 1500 to 1800 seconds of arc.

This intervention was started on February the 17th of the present year and is being carried out using 41 extraction holes as shown in figure 42.

The central holes will penetrate for not more than 2.5 to 3.0 meters under the plinth, see figure 43.

On May 30th of the present year ≅ 9 m^3 have been extracted penetrating maximum 1.5 m under the plinth in the eleven central holes, extracting 1.1 m^3 of the total 9 m^3 from the stretches located beneath the tower. The remaining ground has been extracted under the catino.

The response of the tower to the final ground extraction so far performed is shown in Figs. 40 and 41.

The inclination of the tower was furtherly reduced by additional 440 second of arc, which combined with the benefits produced by the lead weights and the preliminary underexcavation, make the total reduction of inclination reached so far, of 620 seconds of arc to which corresponds to a reduction of ≅136 mm of the overhang (see figure 18) has been achieved.

This decrease of inclination brought the condition of the tower back to the last century (between 1880 and 1890), see figure 44.

Also, during the final intervention the south edge of the plinth (figure 41) has risen by 4.5 mm showing the positive behaviour that can be confirmed in the reduction of the contact stresses and in the location of the point of instantaneous rotation within the imprint of the foundation.

Up to the end of May, the underexcavation has induced, on average, a northwards rotation rate of the Monument ranging between 3 and 4 seconds of arc per day.

In view of the positive response of the tower to the ongoing intervention and considering the beneficial effects of the inclination reduction achieved so far, the Committee came to the decision of starting the gradual removal of the lead ingots (see figure 30), by removing 2 to 3 ingots a week. So far the removal of 7 ingots, ≅660 kN has not caused any perturbation to the steady northwards rotation of the tower as a result of the underexcavation.

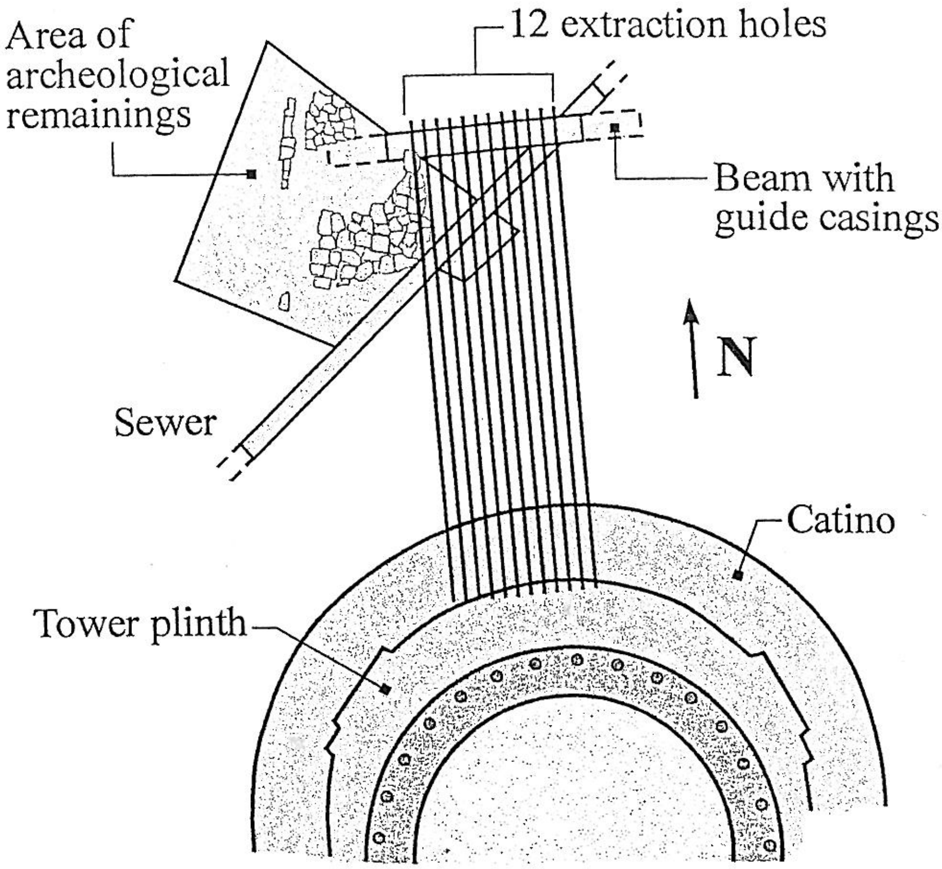


Fig. 38 - Preliminary underexcavation scheme

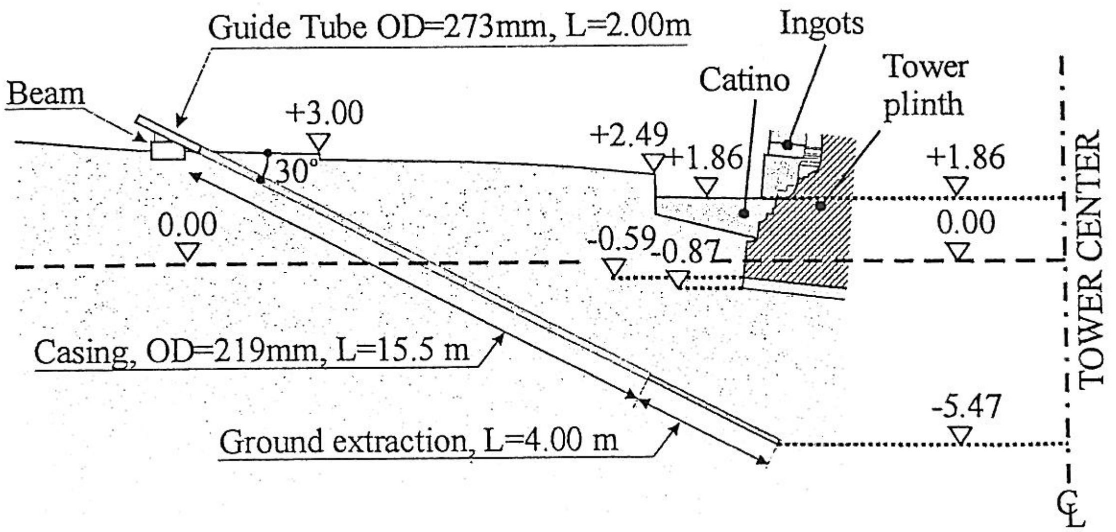


Fig. 39 - Hole for preliminary soil extraction

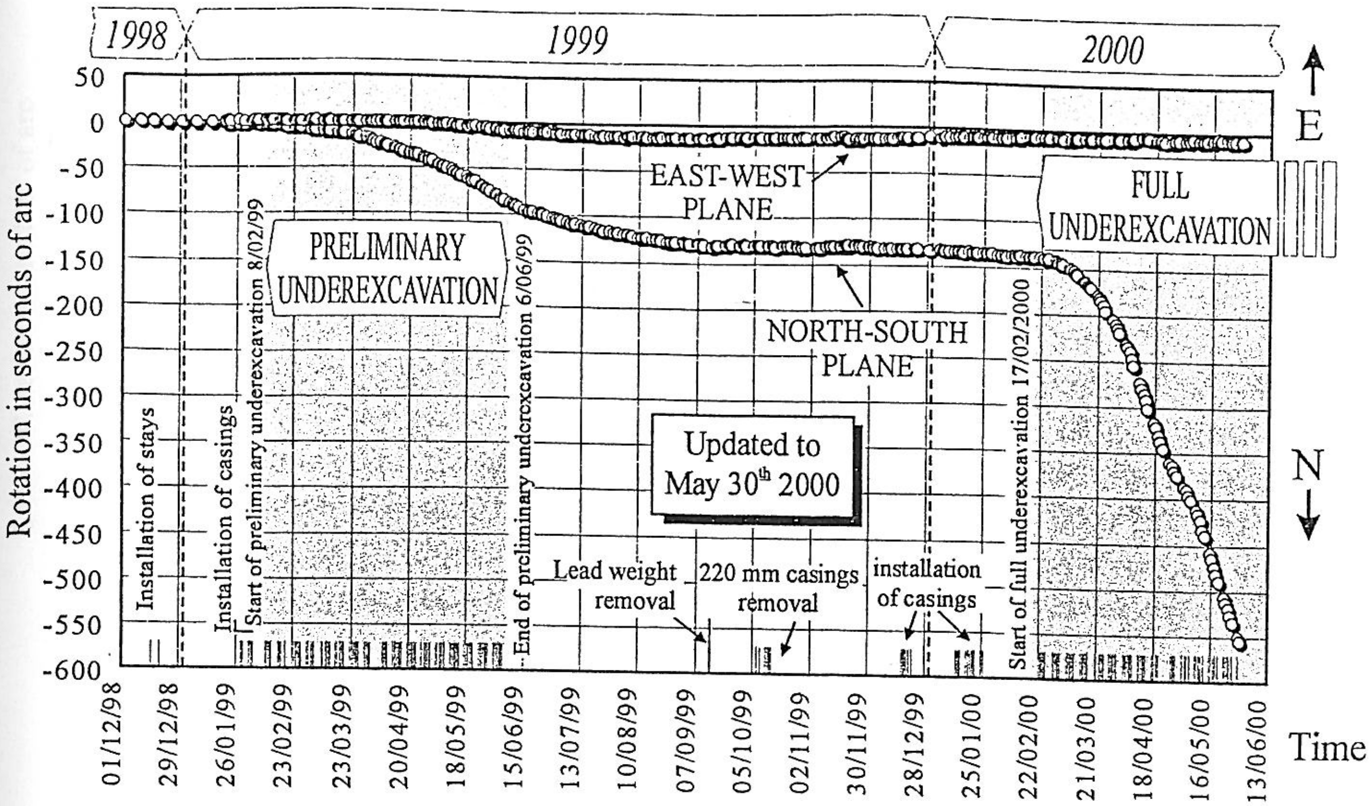


Fig.40 - Rotation of Tower plinth during underexcavation

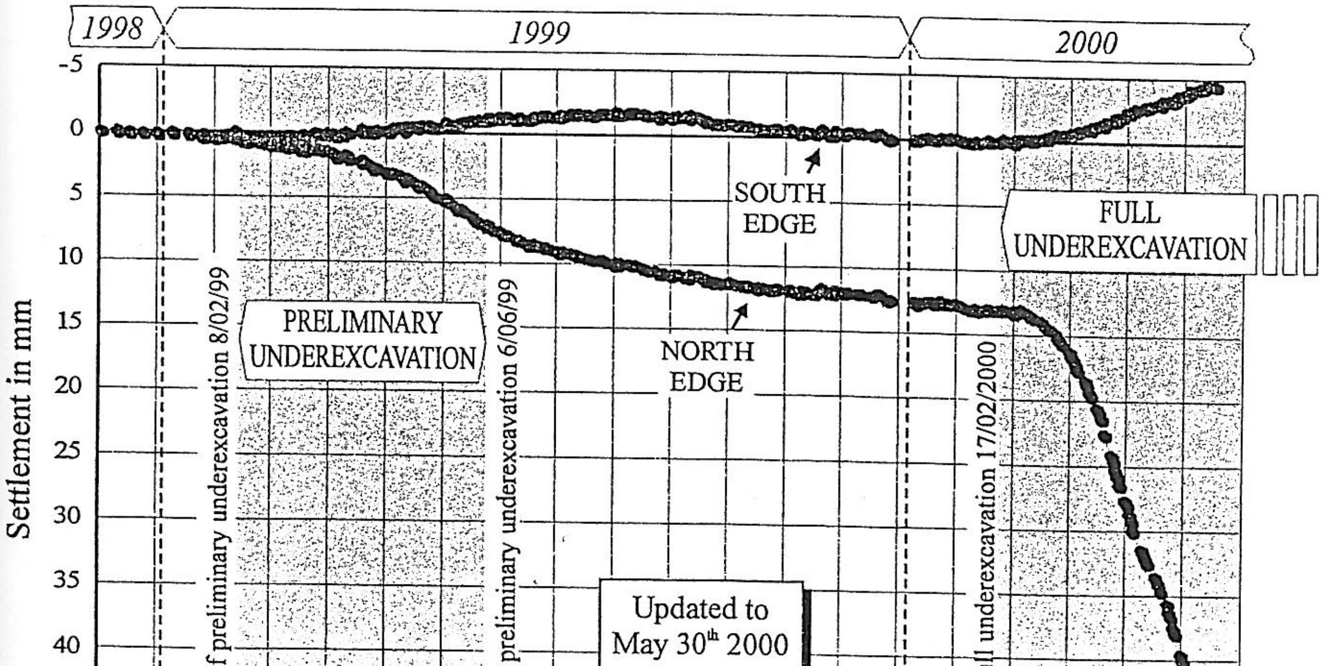


Fig. 41 - Settlement of Tower plinth during underexcavation

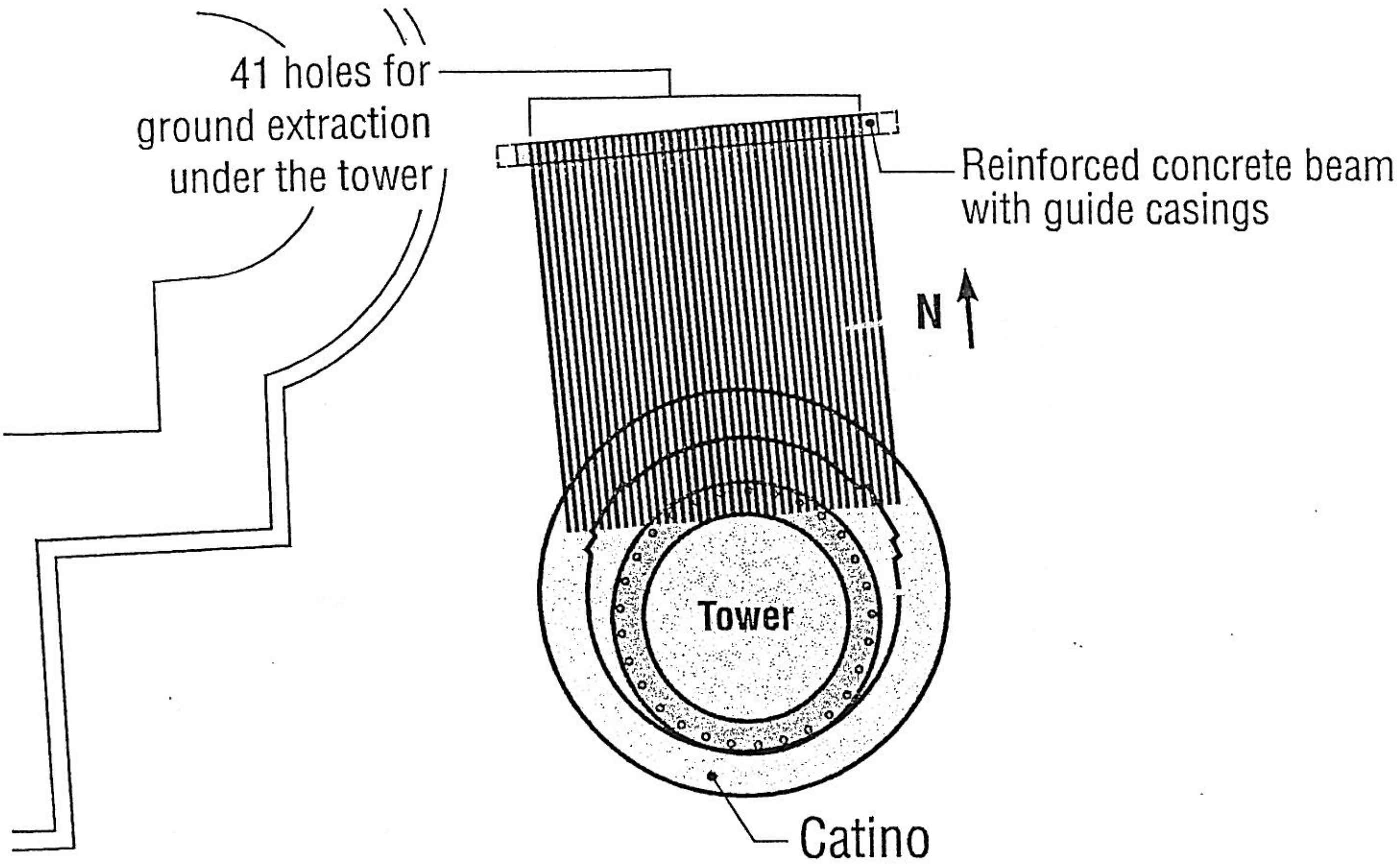


Fig. 42 - Final underexcavation scheme

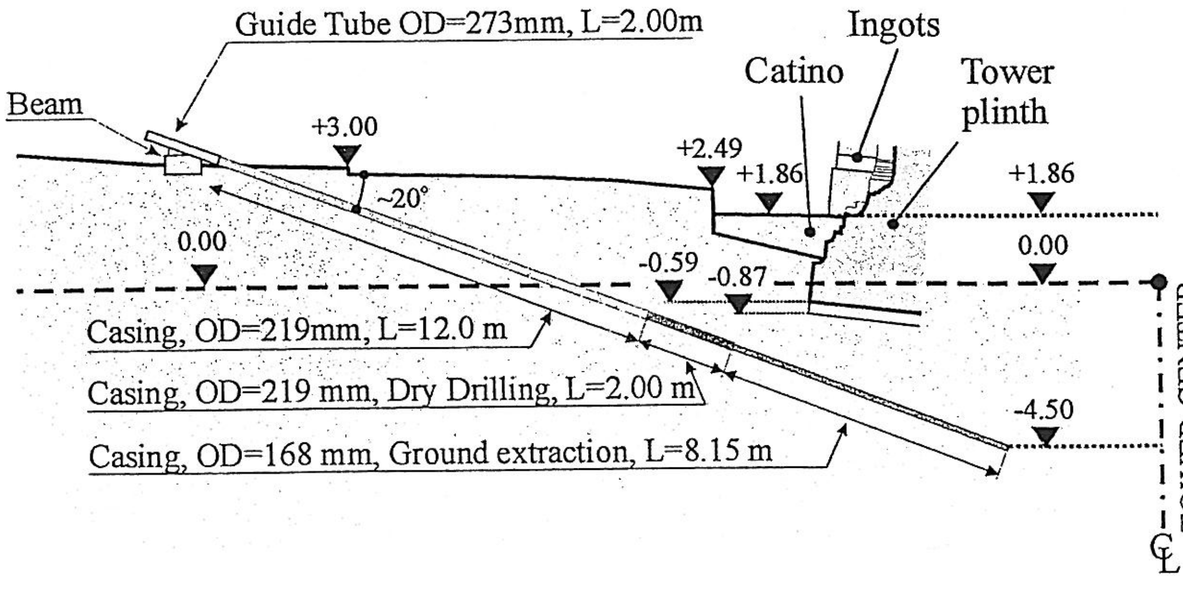


Fig. 43 - Hole for soil extraction - Full underexcavation

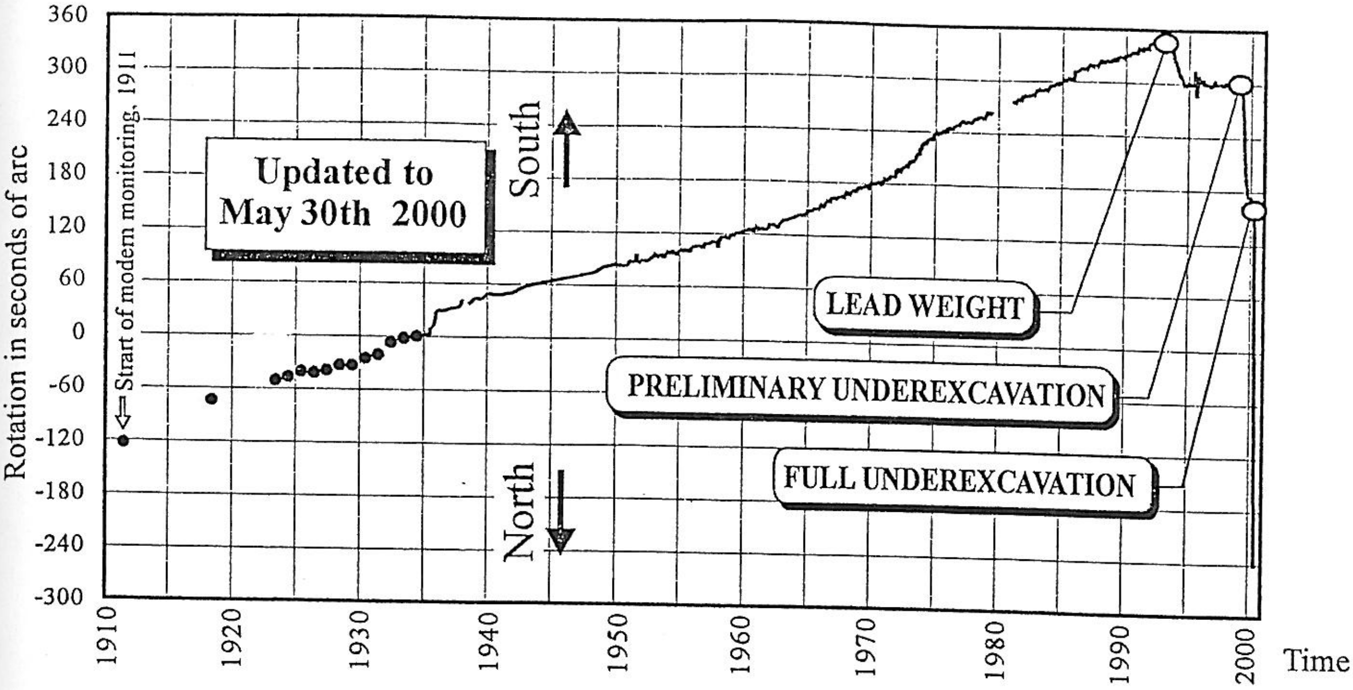


Fig. 44 - Evolution of Tower inclination with time measured with Girometti-Bonechi inclinometer

ADDENDUM: THE COMMITTEE

Sixteen government commissions have studied, measured and worried over this Italian symbol for years, until the current international committee was put in place in 1990, with a mandate finally to take action. The present International Committee for the Safeguard and Architectural Restoration of the Leaning Tower of Pisa was appointed by the Italian Prime Minister on May 1990, based on a Law voted by the Parliament.

The Committee was given the role of a special autonomous Authority for the Tower of Pisa with the task of studying, designing and implementing all the appropriate measures aimed at the geotechnical stabilisation, the structural strengthening and the architectural restoration of the Tower of Pisa. The Committee is a multidisciplinary body composed of the following thirteen Scientists:

J. Barthelemy	(Belgium)	Architect, Expert on Preservation and Restoration of Monuments
J.B. Burland	(UK)	Geotechnical Engineering
M. D'Elia	(Italy)	Expert in Preservation and Restoration of Monuments
R. Di Stefano	(Italy)	Expert in Preservation and Restoration of Monuments
R. Calzona	(Italy)	Structural Engineer
M. Cordaro	(Italy)	Expert in Preservation and Restoration of Monuments *(recently passed away, to be replaced)*
G. Creazza	(Italy)	Structural Engineer
G. Croci	(Italy)	Structural Engineer
M. Jamiolkowski	(Italy)	Geotechnical Engineer
G. Macchi	(Italy)	Structural Engineer
L. Sanpaolesi	(Italy)	Structural Engineer
S. Settis	(Italy)	Expert in Medieval Art and Archaeology
F. Veniale	(Italy)	Mineralogist, Expert in Construction Stones
C. Viggiani	(Italy)	Geotechnical Engineer

Other distinguished Experts have served the Committee in the past

M. Desideri	(Italy)	Structural Engineer (until 1995)
F. Gurieri	(Italy)	Architect (until 1992)
R. Lancellotta	(Italy)	Geotechnical Engineer (until 1996)
G.A. Leonards	(USA)	Geotechnical Engineer (passed away)
R. Lemaire	(Belgium)	Expert in Preservation and Restoration of Monuments (passed away)
F. Leonhardt	(Germany)	Structural Engineer (passed away)
A. M. Romanini	(Italy)	Expert of Medieval Art (resigned over medical grounds)

Moreover, among the many the following eminent scientists and engineers have, over the years, contributed to the studies carried out by the Committee

R. Bartelletti	(Italy)	Technical Advisor to the Site Engineer
G. Calabresi	(Italy)	Geotechnical Engineer
E. Faccioli	(Italy)	Earthquake Engineer
G. Grandori	(Italy)	Earthquake Engineer
J.K. Mitchell	(USA)	Geotechnical Engineer
D. Potts	(U.K.)	Geotechnical Engineer
G. Solari	(Italy)	Expert in Wind Engineering

Overall, the Committee meets every six weeks to take decisions as far as the execution of studies, the approval of design documents and the implementation of works are concerned. Reunions of a limited number of experts are held at regular intervals aimed at developing and preparing the documents to be approved during the plenary meetings.

For each important activity one or two member are appointed as scientific responsible. Some among the most relevant decision taken by the Committee have been steered by the following members:

Cable Stay Safeguard Structure	R. Calzona and Prof. L. Sanpaolesi
Structural Strengthening	G. Croci and Prof. G. Macchi
Under-excavation	J.B. Burland and Prof. C, Viggiani
Design and Technical Specifications of Architectural Restoration	M. D'Elia and Prof. R. Di Stefano
Data Bank, Web Site, Special Volume summarising the works of the Committee	S. Settis.

Moreover, Prof. R. Di Stefano acts as Chief Site, responsible for contractual obligation with respect to contractors operating on site and the writer chairs the Committee.

Underexcavating the Tower of Pisa: back to future

John B. BURLAND
Imperial College of Science, Technology and Medicine, London, UK

Michele B. JAMIOLKOWSKI
Politecnico di Torino, Torino, Italy

Carlo VIGGIANI
Università di Napoli Federico II, Napoli, Italy

SYNOPSIS The stabilization of the Tower of Pisa is a very difficult challenge for geotechnical engineering. The tower is founded on weak, highly compressible soils and its inclination has been increasing inexorably over the years to the point at which it is about to reach leaning instability. Any disturbance to the ground beneath the south side of the foundation is very dangerous; therefore the use of conventional geotechnical processes at the south side, such as underpinning, grouting, etc., involves unacceptable risk. The internationally accepted conventions for the conservation and preservation of valuable historic buildings, of which the Pisa Tower is one of the best known and most treasured, require that their essential character should be preserved, with their history, craftsmanship and enigmas. Thus any intrusive interventions on the tower have to be kept to an absolute minimum and permanent stabilization schemes involving propping or visible support are unacceptable and in any case could trigger the collapse of the fragile masonry.
In 1990 the Italian Government appointed an International Committee for the safeguard and stabilisation of the Tower. It was conceived as a multidisciplinary body, whose components are: experts of arts, restoration and materials; structural engineers; geotechnical engineers.
After a careful consideration of a number of possible approaches, the Committee adopted a controlled removal of small volumes of soil from beneath the north side of the foundation (underexcavation). The technique of underexcavation provides an ultra soft method of increasing the stability of the tower which is completely consistent with the requirements of architectural conservation.
The paper reports the analyses and experimental investigations carried out to explore the applicability of the procedure to the stabilisation of the leaning tower of Pisa. All the results being satisfactory, a preliminary stage of underexcavation of the tower has been carried out in 1999; the results obtained are presented and discussed.

INTRODUCTION

A cross section of the Leaning Tower of Pisa is reported in fig. 1. It is nearly 60 m high and the foundation is 19,6 m in diameter; the weight is 141.8 MN. In the early 90's the foundation was inclined southwards at about 5,4° to the horizontal. The average inclination of the axis of the tower to the vertical is somewhat less, due to its slight curvature resulting from corrections made by masons during the construction, to counteract the inclination already occurring. The seventh cornice overhangs the first one by about 4.1 m.
Construction is in the form of a hollow cylinder. The inner and outer surfaces are faced with marble and the annulus between these facings is filled with rubble and mortar within which extensive voids have been found. A spiral staircase winds up within the annulus. Fig. 1 clearly shows that the staircase forms a large opening on the south side just above the level of the first cornice, where the cross section of the masonry reduces. The high stress within this region was a major cause of concern since it could give rise to an abrupt brittle failure of the masonry.

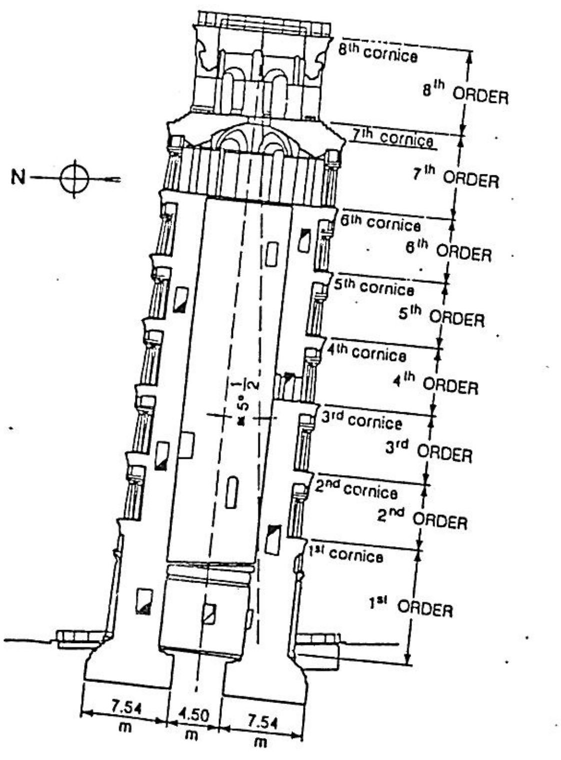


Fig. 1. Cross section through the tower of Pisa in the plane of maximum inclination (very nearly coincident with the north-south plane)

Fig. 2 shows the ground profile underlying the tower. It consists of three distinct horizons. Horizon A is about 10 m thick and primarily consists of estuarine deposits, laid down under tidal conditions. As a consequence, the soil types consist of rather variable sandy and clayey silts. At the bottom of Horizon A there is a 2 m thick medium dense fine sand layer. Based on samples descriptions and piezocone tests, the materials to the south of the tower appear to be more silty and clayey than to the north and the sand layer is locally thinner.
Horizon B consists primarily of marine clay which extends to a depth of about 40 m. It is subdivided into four distinct layers. The upper layer is soft sensitive clay locally known as the Pancone. It is underlain by an intermediate layer of stiffer clay, which in turn overlies a sand layer (the intermediate sand). The bottom layer of horizon B is a normally consolidated clay known as the lower clay. Horizon B is laterally very uniform in the vicinity of the tower.
Horizon C is a dense sand (the lower sand) which extends to considerable depth.
The water table in horizon A is between 1 m and 2 m below the ground surface. Pumping from the lower sand has resulted in downward seepage from horizon A with a pore pressure distribution with depth through horizon B which is slightly below hydrostatic.
The many borings beneath and around the tower show that the surface of the Pancone clay is dished beneath the tower, from which it can be deduced that the average settlement of the monument is approximately 3 m.
Fuller details about the tower and its subsoil, including a wide list of references, are reported by BURLAND *et al.* (1999).

In 1990 the Italian Government appointed an International Committee for the safeguard and stabilisation of the Tower. It was conceived as a multidisciplinary body, whose components are: experts of arts, restoration and materials; structural engineers; geotechnical engineers.

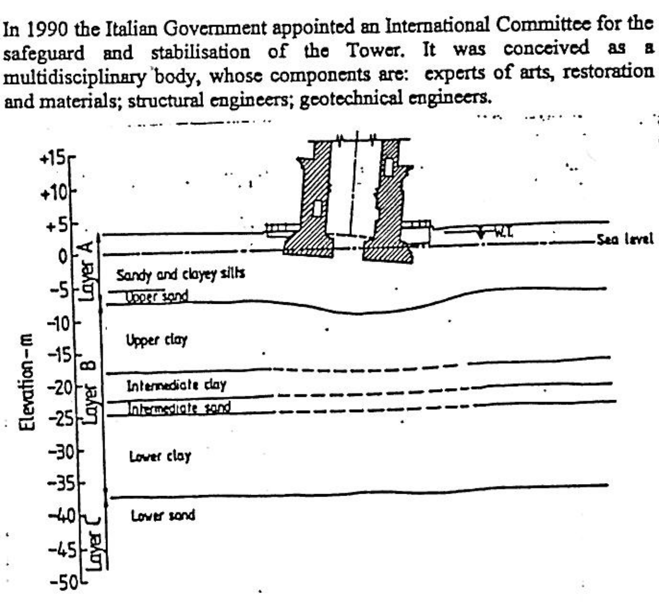


Fig. 2. Soil profile beneath the tower

The Committee recognised the need for temporary and fully reversible interventions, aimed at an improvement of the safety against the risk of structural collapse or overturning by foundation failure of the tower. The temporary measures gave to the Committee the time to complete the investigations and analyses necessary to conceive and implement the final stabilisation measures.
In the summer of 1992 the safety of the masonry was temporarily improved by applying slightly pre-stressed steel strands around the tower in the vicinity of the first cornice. At present the masonry is being consolidated by grouting and the temporary circumferential strands will soon be removed.
The observation that the northern side of the tower foundation had been steadily rising for most of this century led to the suggestion of applying a north counterweight to the tower, as a temporary measure to improve the safety against overturning by reducing the overturning moment. Accordingly, a design was developed consisting of a pre-stressed concrete ring cast around the base of the tower for supporting a number of lead ingots. This intervention was successfully implemented in 1993.
The Committee has developed a detailed understanding of the history of the inclination of the tower, and in particular of the movements it has experienced last century. These have been observed by a very comprehensive monitoring system, installed on the tower since the beginning of the 20th century and progressively enriched. The behaviour of the tower clearly indicates that its equilibrium is affected by leaning instability, a phenomenon controlled by the stiffness of the subsoil rather than by its strength (GORBUNOV POSSADOV, SEREBRIANY, 1961; HABIB, PUYO, 1970; SCHULTZE, 1973; HAMBLY, 1985; CHENEY *et al.*, 1991; LANCELLOTTA, 1993, DESIDERI, VIGGIANI, 1994; DESIDERI *et al.*, 1997). The analysis of the problem, taking into account the non elastic and non linear restraint exerted by the foundation, shows that a limited decrease of the inclination of the tower would greatly increase its safety and arrest the progress of inclination.

The Committee has been exploring a variety of approaches to permanently stabilising the tower.
The fragility of the masonry, the sensitivity of the underlying clay and the very marginal stability of the foundations impose severe restraints. Any measures involving the application of concentrated loads to the masonry or underpinning operations beneath the south side of the foundation have thus been ruled out. Moreover conservation considerations require that the impact of any stabilising measures on the formal, historical and material integrity of the monument should be kept to an absolute minimum.
After a long and heated discussion, the Committee decided to give priority to so called "very soft" solutions, aimed at reducing the inclination of the tower by up to half a degree (i.e. by about 10% the present inclination) by means of an induced settlement beneath the north side of the foundation, without even touching the structure of the tower. Besides improving the stability of the foundation, such an approach allows also a reduction of masonry overstressing, thus contributing to reducing to a minimum the work needed to consolidate the tower fabric itself.
The Committee gave careful consideration to a number of possible approaches, such as the construction of a ground pressing slab to the north of the tower, coupled to a post-tensioned concrete ring constructed around the periphery of the foundations and loaded by tensioned ground anchors, or the consolidation of the Pancone Clay by means of electro-osmosis. Eventually the choice was that of a controlled removal of small volumes of soil from beneath the north side of the foundation (underexcavation).
Underexcavation was originally proposed by TERRACINA (1962) as a method to increase the stability of the tower of Pisa. Recently the method has been successfully employed in Mexico (TAMEZ *et al.*, 1989; SANTOYO *et al.*, 1989); among many cases, an important application was aimed at mitigating the impact of the very large differential settlements which affected the Metropolitan Cathedral of Mexico City (TAMEZ *et al.*, 1995). The principle of the method is to extract a small volume of soil at a desired location, leaving a cavity. The cavity gently closes due to the overburden pressure, causing a small surface subsidence. The process is repeated at various chosen locations and very gradually the inclination of the tower is reduced.
The present paper reports the analyses and experimental investigations carried out to explore the applicability of the procedure to the stabilisation of the leaning tower of Pisa. All the results being satisfactory, a preliminary stage of underexcavation of the tower has been carried out in 1999; the results obtained are also presented and discussed.

SMALL SCALE 1g TESTS

EDMUNDS (1993) performed a number of small scale physical tests on a model tower resting on a bed of fine sand, to study the effect of underexcavation on a tower close to the collapse for leaning instability. A sketch of the experimental setup is reported in fig. 3. A model tower with a diameter of 102 mm was placed at the top of a very loose fine sand bed, and loaded through a hanger at height of 126 mm over the base. The ratio 126/102 is approximately equal to the ratio of the height of the centre of gravity of the tower of Pisa to the diameter of its foundation.
Loading the model tower produced a settlement and a rotation α. A total of 8 load tests were carried out; the load at failure varied between 120 and 190 N. Failure in all cases was by toppling with the lowest edge of the model tower's base sinking into the sand as the tower rotates toward horizontal.

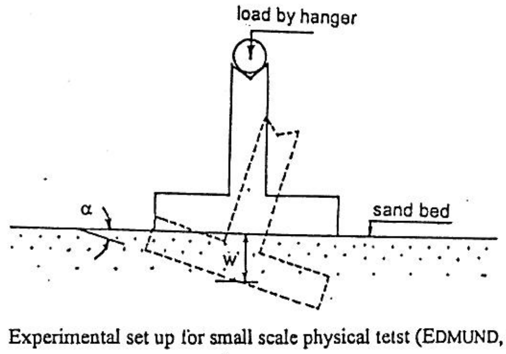


Fig. 3. Experimental set up for small scale physical tetst (EDMUND, 1993)

The individual plots of α varying with load give somewhat variable results, but when combined into one plot, as in fig. 4, a well defined envelope of results emerges. The envelope shows a pronounced change in curvature at a load of 160 to 165 N, where the inclination averages 0.9 ($\alpha \sim 5°$).

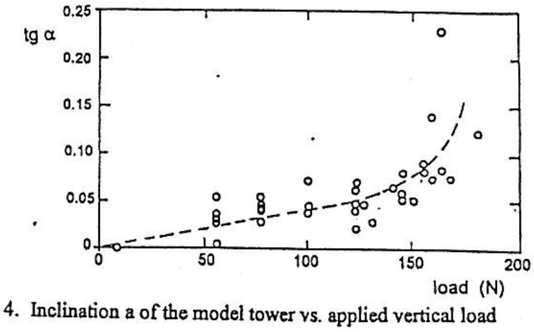


Fig. 4. Inclination a of the model tower vs. applied vertical load

After this preliminary investigation the underexcavation tests were performed starting with a load of 165 N and a rotation of 5,5°. These conditions are believed to be representative of a tower on the verge of leaning instability.
Underexcavation was performed by inserting a stainless steel tube with an outer diameter of 6 mm, and iside it an inner suction tube connected to a vacuum pump. The inner tube, with an outer diameter of 2.1 mm, removes the sand from inside the larger tube, that is thus advanced into the soil by a form of self boring, without significant disturbance of the surrounding soil. The whole probe is advanced to the desired location and then retracted, leaving a cavity that closes instantaneously.
The underexcavation tubes are held in position by external guides and penetrate the sand at an angle of 18° 26' (3/1). Five radial tubes have been adopted, covering a sector of 90° centered on the north side of the model tower.
A total of 14 underexcavation tests with different combinations of probe positions and penetration sequence have been performed. The most important indications emerging from those tests are as follows:

- underexcavation can be used to reduce the tilt of the model in a controllable manner. Reduction of tilt up to 1° have been obtained;
- the movement of the tower can be steered using probes inserted at a

range of positions around the tower;

- the results are reproducible, at least qualitatively;
- a *critical point* exists some 10 mm north of the central axis of the tower, in the ground beneath it, beyond which ground removal aggravates the tilt, but behind which underexcavation produces a decrease in tilt;
- repeated use of one probe in isolation rapidly ceases to significantly affect the tower's tilt;

Most of these indication are believed to apply qualitatively to the case of the tower of Pisa.

PRELIMINARY SIMPLIFIED ANALYSES

THILAKASIRI (1993) modelled the subsoil of the tower as a set of elastic-perfectly plastic Winkler springs, and determined the spring constants by fitting the observed behaviour of the tower during construction. The analysis confirms that the inclination of the tower started during the second stage of construction, because of leaning instability; it accurately reproduces the present situation of the tower in terms of settlement and inclination.
Underexcavation was simulated by removing a single strip of reaction stress at the soil-foundation interface. It has been found that underexcavation has a positive effect, provided it is confined north of the position of the load resultant. This is obvious by elementary static; in fact, no critical point has been predicted north of the load resultant.
The effectiveness of the operation depends on the position from which the stress is removed; the optimum position has been determined at about half radius from the northern edge of the foundation.
DESIDERI & VIGGIANI (1994) modelled the overall behaviour of the subsoil and the tower foundation by an elasto-plastic strain hardening restraint, and simulated the underexcavation intervention by a reduction in overturning moment with constant vertical force. DESIDERI *et al.* (1997) modelled the subsoil as a bed of Winkler type elastic-strain hardening plastic springs; they found that the overall behaviour of the tower and the subsoil can be described by a set of yield loci eventually merging into a failure locus, and by an associative flow rule. All these spring models do not predict the occurrence of a critical line.
COMO *et al*. (1999) simulate the subsoil as a bed of elastic-perfectly plastic Winkler springs, and assume that the effects of underexcavation may be simulated by a reduction of stiffness and strength of a part of these springs. According to such a model, the occurrence of a critical line is connected to a contraction of the yeld locus of the foundation, due to the strength reduction in the soil. They claim that no critical line can be found if an elastic model is assumed for the subsoil.

FEM ANALYSIS

Finite element analyses of the behaviour of the tower and its subsoil have been carried out using a finite element geotechnical computer program developed at Imperial College and known as ICFEP (POTTS & GENS, 1984). The constitutive model is based on Critical State concepts and is non linear elastic work hardening plastic. Fully coupled consolidation is incorporated, so that time effects due to the drainage of pore water in the soil skeleton are included.
The prime object of the analysis was to improve the understanding of the mechanisms controlling the behaviour of the Tower (BURLAND & POTTS, 1994). Accordingly, a plane strain approach was used for much of the work, and only later was three dimensional analysis used to explore certain detailed features.
The layers of the finite element mesh matched the soil sub-layering that had been established from soil exploration studies, as reported in § 2.2 above. Fig. 5a shows the adopted mesh, while fig. 5b reports the detail of the mesh in the immediate vicinity of the foundation. In Horizon B the soil is assumed to be laterally homogeneous; however a tapered layer of slightly more compressible material was incorporated into the mesh for layer A_1 as shown by the shaded element in fig. 5b. This slightly more compressible region represents a more clayey material found beneath the south side of the foundation; in applied mechanics terms this slightly more compressible tapered layer may be considered as an "imperfection". The overturning moment generated by the lateral movement of the centre of gravity of the tower was incorporated into the model as a function of the inclination of the foundation, as shown in fig. 5.
The construction history of the tower was simulated by a series of load increments applied to the foundation at suitable time intervals. The excavation of the catino in 1838 was also simulated in the analysis. Calibration of the model was carried out by adjusting the relationship between the overturning moment generated by the centre of gravity and the inclination of the foundation. A number of runs were carried out with successive adjustments being made until good agreement was obtained between the actual and the predicted present day value of the inclination.

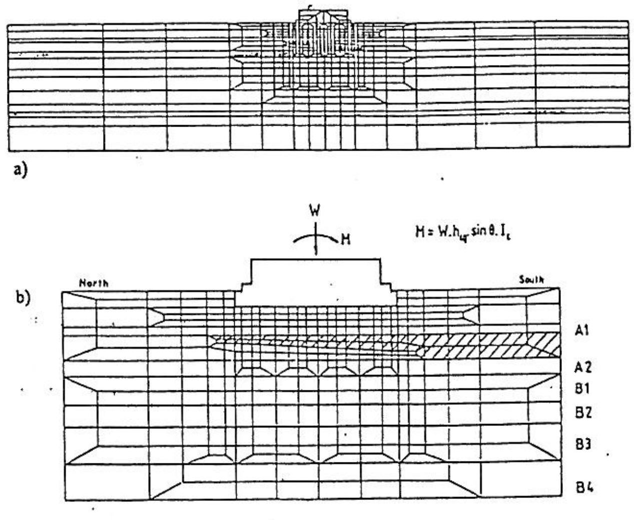


Fig. 5. Finite element model. a) General mesh. b) Mesh in the vicinity of the tower foundation

Fig. 6a shows a graph of the predicted changes in inclination of the tower against time, compared with the deduced historical values. From about 1272 onwards there is a remarkable agreement between the model and the historical inclination. Note that it is only when the bell chamber was added in 1360 that the inclination increases dramatically (fig. 6b). Also of considerable interest is the excavation of the catino in 1838 which results in a predicted rotation of about 0.75°. It should be noted that the final imposed inclination of the model tower is 5.44° which is slightly less than the present day value of 5.5°. It was found that any further increase in the final inclination of the tower model resulted in instability: a clear indication that the tower is very close to falling over.
The analysis has demonstrated that the lean of the tower results from the phenomenon of settlement instability due to the high compressibility of the Pancone Clay. The principal effect of the layer of slightly increased compressibility beneath the south side of the foundation is to determine the direction of lean, rather than its magnitude. The main limitation is that the model does not deal with creep. Nevertheless the model provides important insights into the basic mechanisms of behaviour and has proved valuable in assessing the effectiveness of various proposed stabilisation measures.
As reported before, a lead ingot counterweight was installed on the north side of the tower between May 1993 and January 1994; the observed behaviour of the tower is reported in fig. 7. On 29th February 1994, one month after completion of loading, the northward change of inclination was 33"; by the end of July it had increased to 48". On 21st February 1994 the average settlement of the tower relative to the surrounding ground was about 2.5 mm.
Fig. 8 shows a comparison of the Class A prediction and measurements of (a) the changes in inclination and (b) the average settlement of the tower during the application of the lead ingots. The computed settlement are in good agreement with the measured values; the predicted changes in inclination are about 80% of the measured values.
The movements observed during the counterweight application have been used to further refine the model. After such a refinement, that involved a small reduction of the value of G/p'_0 in horizon A, a better overall agreement between computed and observed values has been obtained (fig. 9).

The re-calibrated model has been used to simulate the extraction of soil from beneath the north side of the foundation. It should be emphasised that the finite element mesh had not been developed with a view of modelling unde.excavation; the individual elements are rather large for representing regions of extraction. Thus the purpose of the modelling was to throw light on the mechanisms of behaviour rather than attempt a somewhat illusory "precise" analysis.
The soil extraction has been simulated by reducing the volume of any chosen element of ground incrementally, so as to achieve a predetermined reduction in volume of that element.

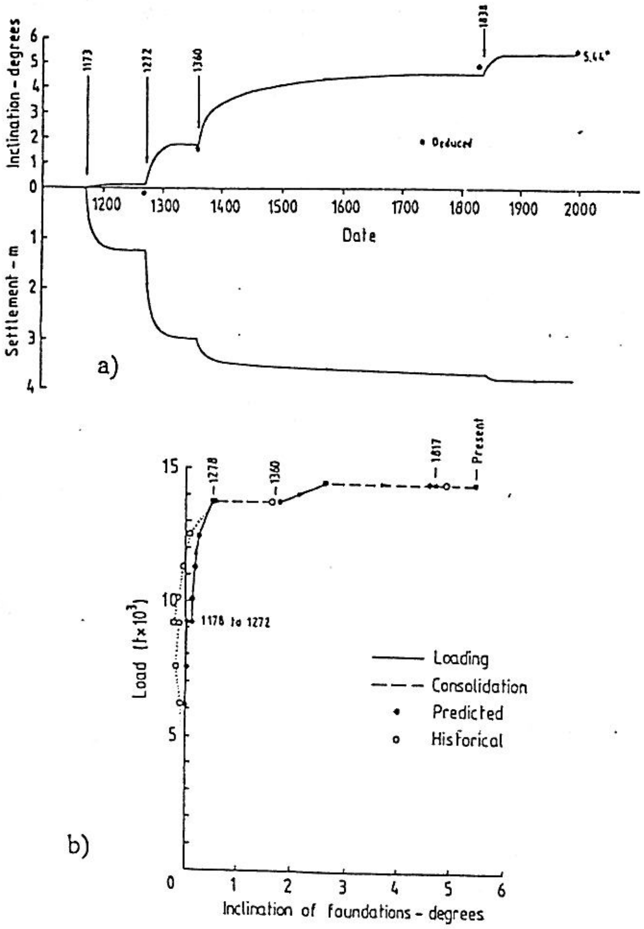


Fig. 6. Comparison between the history of the tower and the results of the finite element model. a) Inclination and settlement versus time. b) Relationship between the weight and the inclination of the tower

The first objective of the numerical analysis was to check whether the concept of a critical line, whose existence was revealed by the small scale tests by EDMUNDS (1993) was valid. Fig. 10 shows the finite element mesh in the vicinity of the tower. Elements numbered 1, 2, 3, 4 and 5 are shown, extending southwards from beneath the north edge of the foundation. Five analyses were carried out in which each of the elements was individually excavated to give full cavity closure and the response of the tower computed. For excavation of elements 1, 2 and 3 the inclination of the tower reduces, so that the response is positive. For element 4 the response is approximately neutral, with an initial slight reduction in inclination which, with further excavation, was reversed. For element 5 the inclination of the tower increased as a result of excavation.
The above analyses confirm the concept of a critical line separating a positive response from a negtive one. For the plane strain computer model the location of the critical line is towards the south end of element 4 which is at a distance of 4.8 m underneath the foundation of the tower, i.e. about one half the radius of the foundation.

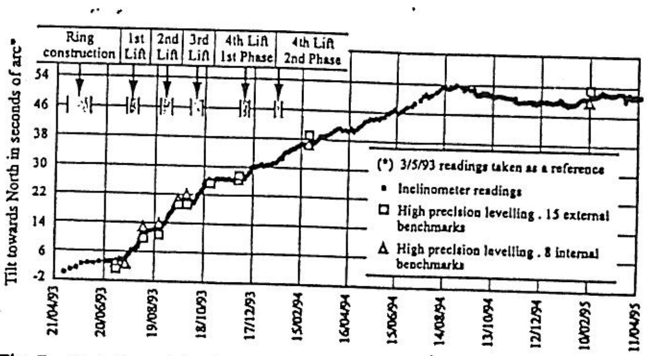


Fig. 7. Variation of the inclination of the tower in response to placement of the counterweight

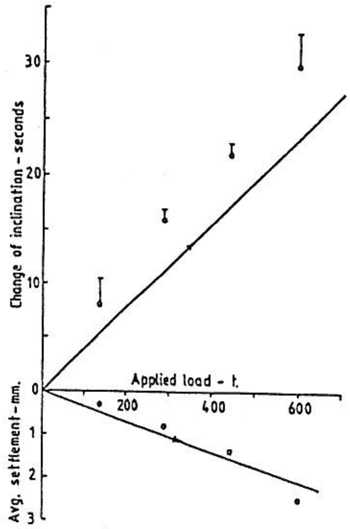


Fig. 8. Plane strain finite element prediction and observed response of the tower to the application of counterweight

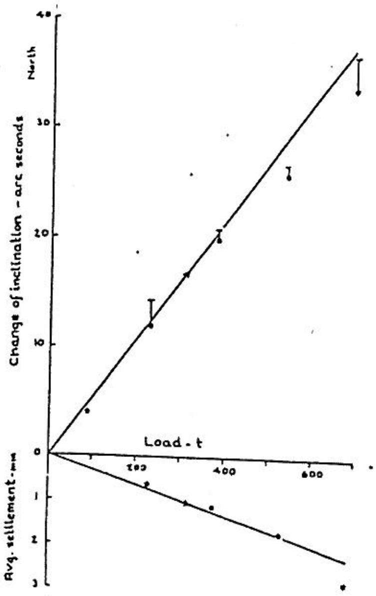


Fig. 9. Comparison of the observed response of the tower to the counterweight with the recalibrated plane strain model prediction

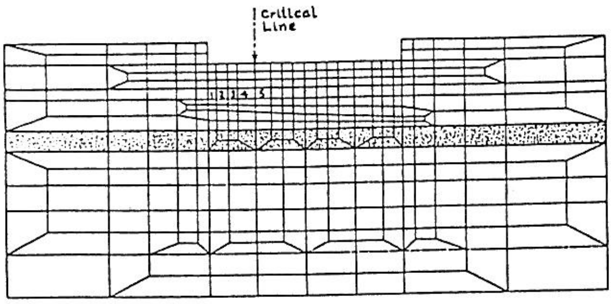


Fig. 10. Finite element mesh in vicinity of tower foundation showing elements which were individually excavated to investigate the existance of a critical line.

It was noted that, as the location of excavation moved further and further south beneath the foundation, the settlement of the south side steadily increases as a proportion of the settlement of the north side. Excavation of elements 1 and 2 give a proportion of less than one quarter.

Having demonstrated that localised soil extraction gives rise to a positive response, the next stage was to model a complete underexcavation intervention aimed at safely reducing the inclination of the tower by a significant amount.

A preliminary study was carried out of extraction using a shallow inclined drill hole, extracting soil from just beneath the foundation. Although the response of the tower in terms of decrease of inclination was favourable, the stress change beneath the foundation were large; consequently a deeper inclined extraction hole was investigated.

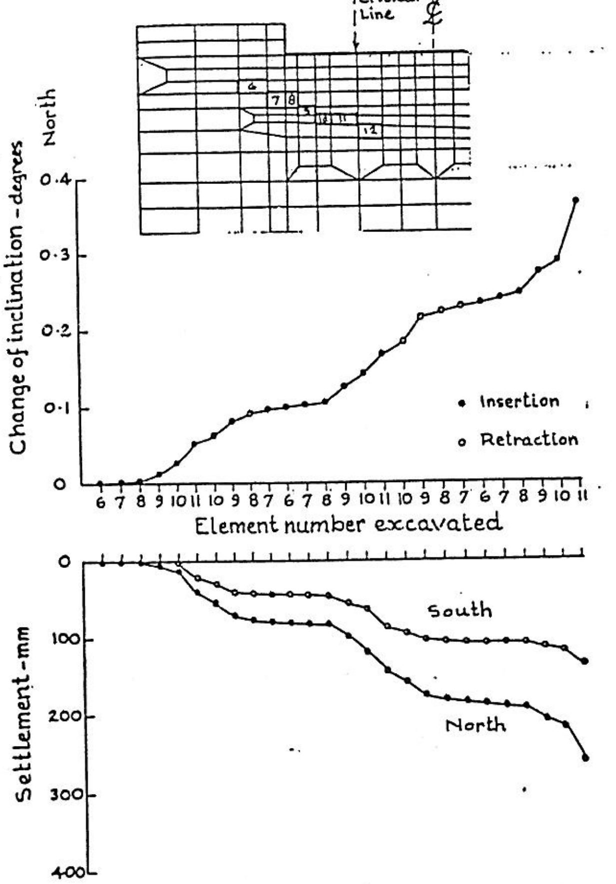


Fig. 11. Predicted response of the tower to underexcavation beneath the north side by means of an inclined drill hole. The volumetric reduction for each element was approximately 5%

The insert in fig. 11 shows the finite element mesh in the vicinity of the foundation on the north side. The elements numbered 6 to 12 were used for carrying out the intervention and are intended to model an inclined drill hole. It should be noted that element 12 lies south of the critical line established by localised soil extraction as described above. The procedure for simulating the underexcavation intervention was as follows:

- the stiffness of element 6 is reduced to zero;
- equal and opposite vertical nodal forces are applied progressively to the upper and lower faces of the element until its volume reduces by about 5%. The stiffness of the element is then restored;
- the same procedure is then applied successively to the elements 7, 8, 9, 10 and 11 thereby modelling the progressive insertion of the drill probe. For each step the inclination of the tower reduces;
- when element 12 is excavated the inclination of the tower increases, confirming that excavation south of the critical line gives a negative response. The analysis is therefore re-started after excavating element 11;
- the retraction of the drill probe is then modelled by excavating elements 10, 9, 8, 7 and 6 successively. For each step the response of the tower is positive;
- the whole process of insertion and retraction of the drill probe is then repeated. Once again excavation of element 12 gives a negative response.

The computed displacements of the tower are plotted in fig.11. The sequence of excavation of the elements is given on the horizontal axis; the upper diagram shows the change of inclination of the tower due to underexcavation; the lower diagram shows the settlement of the north and south sides of the foundation.

As underexcavation progresses from elements 6 through 11 the rate of change of northward inclination increases as do the settlements. As the drill is retracted the rate decreases. At the end of the first cycle of insertion and retraction of the drill, the inclination of the tower is decreased by 0.1°. The settlement of the south side is rather more than one half of the north side. For the second cycle a similar response is obtained but the change of inclination is somewhat larger. After the third insertion of the drill the resultant northward rotation was 0.36°. The corresponding settlements of the north and south sides of the foundation were 260 mm and 140 mm respectively.

As for the contact stress distribution, the process results in a slight reduction of stress beneath the south side. Beneath the north side, some fluctuations in contact stress take place, as it was to be expected, but the stress changes are small. In general the stress distribution after retraction of the drill are smoother than after insertion.

CENTRIFUGE TESTS

Centrifuge modelling of the tower and its subsoil have been carried out at ISMES, with the aim of exploring the present stability conditions of the tower and their possible evolution with time. The results obtained are reported and discussed by PEPE (1995). They gave further insight into the mechanisms of the instability and confirmed the elastoplastic character of the restraint exerted by the foundation and the subsoil on the motion of the tower.

In fig. 12 the properties of the foundation soils of the tower are compared with the properties obtained in the small scale model after consolidation under geostatic load in the centrifuge; the main features of the soil profile are satisfactorily reproduced in the model.

Fig. 13 reports the simulation of the construction of the tower, as obtained by one of the centrifuge tests. It may be seen that both the settlement and the rotation of the tower are in good overall agreement.

The centrifuge was also used to assess the effectiveness of underexcavation as a means to stabilise the tower. The process of soil extraction was modelled by inserting into the ground beneath the model tower flexible tubes with wires inside, prior to the commencement of the experiment. Once the model tower had come to equilibrium at an appropriate inclination under increased gravity, the wires were pulled out of the flexible tubes by an appropriate amount, while the model was in flight, causing the tubes to close simulating the closure of the cavity produced by a drill probe.

Fig. 14 reports the results of a typical experiment. The test results confirmed the existence of a critical line and showed that soil extraction north of this line always gave a positive response.

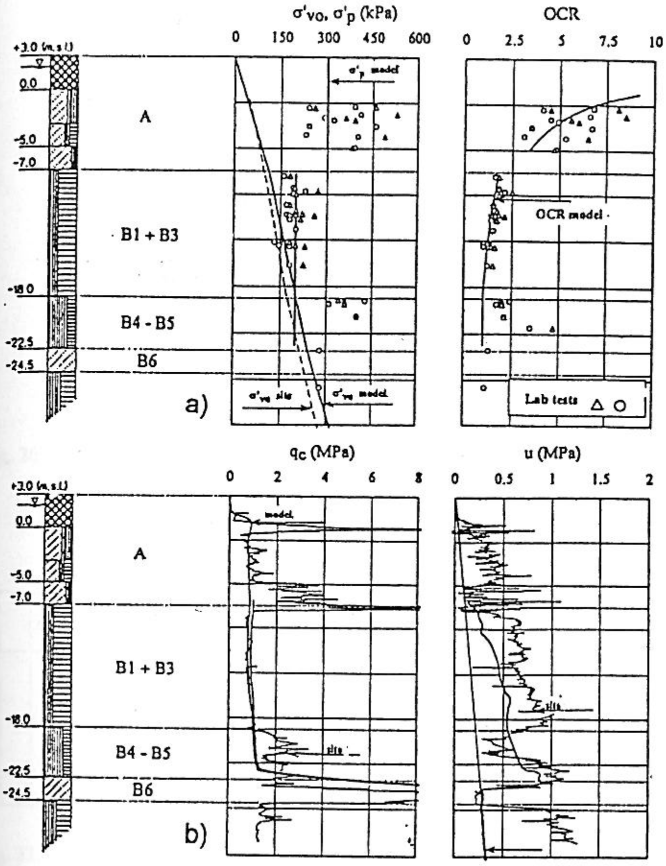


Fig. 12. Comparison between the properties of the subsoil of the tower and those of the centrifuge model. a) Overconsolidation of clay layers. b) Piezocone profiles

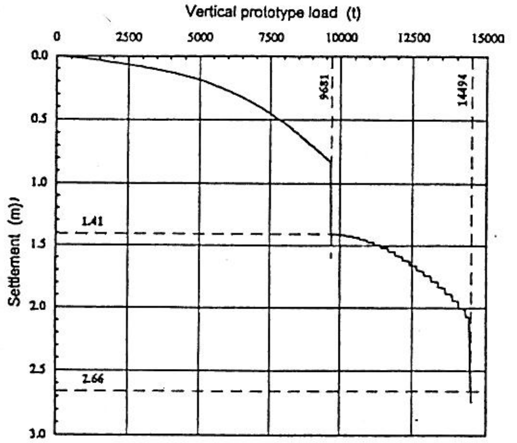


Fig. 13. Centrifuge simulation of the construction of the tower; test Y/E15

LARGE SCALE FIELD EXPERIMENT

The results of the physical and numerical modelling work on underexcavation were sufficiently encouraging to undertake a large scale development trial of the field equipment. The objectives of the trial where:

- to develop a suitable method of forming a cavity without disturbing the surrounding ground during drilling;
- to study the time involved in the cavity closure;
- to measure the changes in contact stress and pore water pressures beneath the trial footing;
- to evaluate the effectiveness of the method in changing the inclination of the trial footing;
- to explore methods of "steering" the trial footing by adjusting the drilling sequence;
- to study the time effectes between and after the operation.

For this purpose a 7 m diameter eccentrically loaded circular reinforced concrete footing was constructed in the Piazza north of Baptistry, as shown in fig. 15. Both the footing and the underlying soil were instrumented to monitor settlement, rotation, contact pressure and pore pressure. .

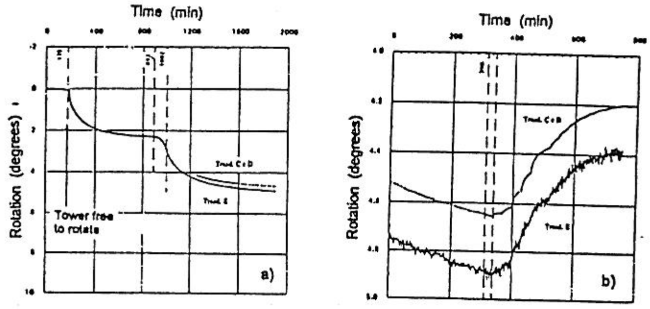


Fig. 14. Centrifuge simulation of the underexcavation. a) Construction of the tower. b) Underexcavation

After a waiting period of a few months, allowing the settlement rate to come to a steady value, the ground extraction commenced by means of inclined borings, as schematically shown in fig. 16. Drilling was carried out using a hollow stemmed continuous flight auger inside a contra-rotating casing.

The trial has been very successful. When the drill is withdrawn to form the cavity, an instrumented probe located in the hollow stem is left in place to monitor its closure (fig. 17). A cavity formed in the Horizon A material has been found to close smoothly and rapidly. Fig. 18 reports the measurements of the contact stress at the soil-foundation interface along the north - south axis, before underexcavation (19.09.95) and after a substantial rotation of the footing (01.12.95). The stress changes beneath the foundation were found to be very small. The trial footing was successfully rotated by about 0.25° and directional control was maintained even though the ground conditions were somewhat non uniform. Rotational response to soil extraction was rapid,

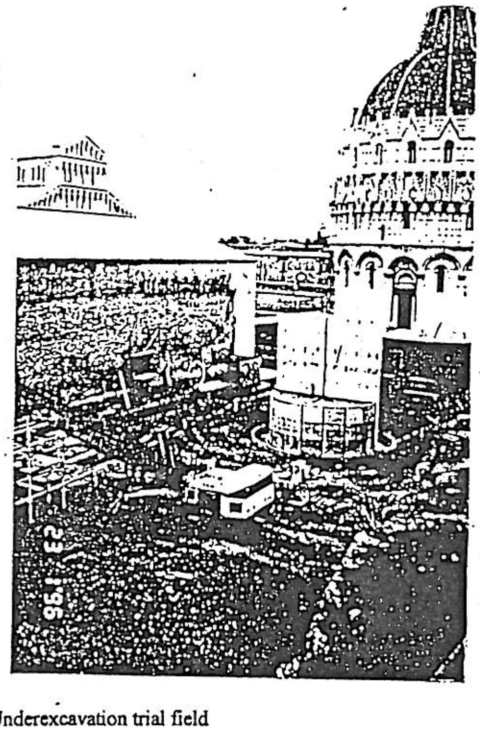
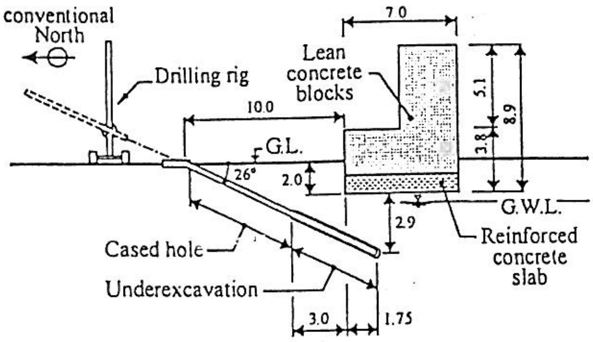


Fig. 15. Underexcavation trial field



Drawing not to scale - all dimensions in meters

Fig. 16. Underexcavation trial field; cross section

taking a few hours. At the completion of the underexcavation, on February 1996, the plinth came to rest and since then it has exhibited negligible further movements (fig. 19). Very importantly, an effective system of communication, decision taking and implementation was developed.
It is of importance to note that, early in the trial, over enthusiastic drilling resulted in soil extraction from excess penetration beneath the footing causing a counter rotation (fig. 19). Therefore the trial also confirmed the concept of a critical line.

PRELIMINARY UNDEREXCAVATION OF THE TOWER

The results of all the investigations carried out on the underexcavation were positive, but the Committee was well aware that they might be not completely representative of the possible response of a tower affected by leaning instability. Therefore it was decided to implement preliminary ground extraction beneath the tower itself, with the objective of observing its response to a limited and localised intervention. This preliminary intervention

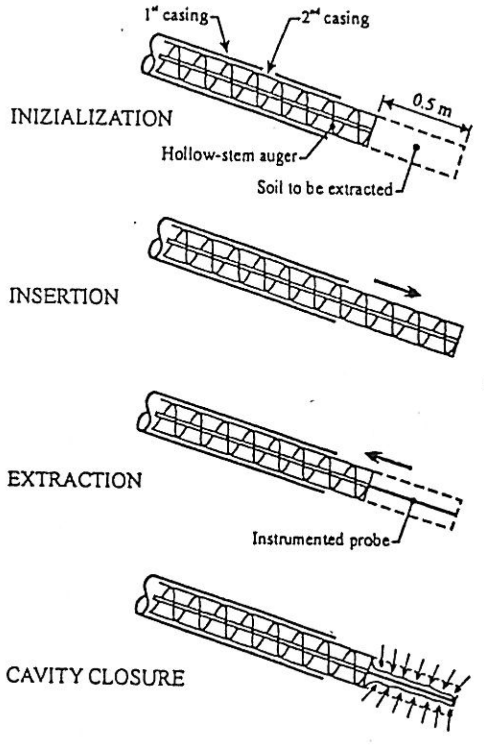


Fig. 17. Soil extraction process

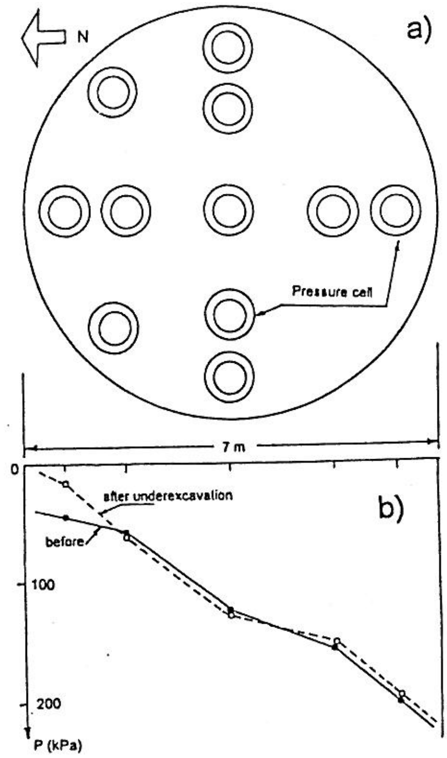


Fig. 18. Stress variation at the soil-foundation interface during the large scale underexcavation trial. a) Layout of the pressure cells. b) Pressure distribution along the north-south axis at two different dates.

consists in 12 holes (fig. 20 and 21) to extract soil from Horizon A to the north of the tower foundations, penetrating beneath the foundation not more than 1 m. The goal was to decrease the inclination of the tower by a significant amount, in order to check the feasibility of underexcavation as a means to permanently stabilise the tower, and to adjust the extraction and measurement techniques.
To protect the tower from any unexpected adverse movement during this or any other intereventions aimed at the final stabilisation of the monument, a safeguard structure was considered mandatory. The structure finally chosen consists of two sub-horizontal steel stays connected to the tower at the level of the third order and to two anchoring steel frames located behind the building of the Opera Primaziale, to the north of the tower. The scheme of the safeguard structure is reported in fig. 22; it was installed and connected to the tower in December 1998.
Each stay is capable of applying a maximum force of 1500 kN, with a safety factor equal to 2. The force may be applied by dead weights or by hydraulic jacks; the value of the applied load is continuously monitored. At present, the

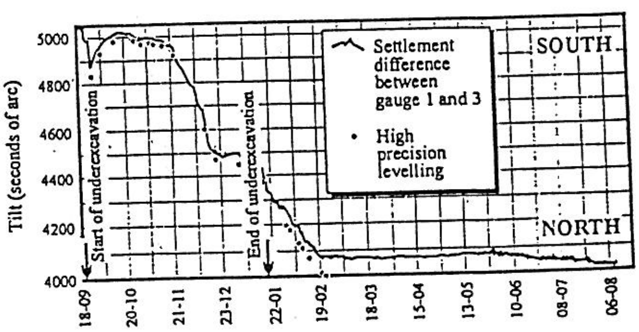


Fig. 19. Underexcavation large scale field experiment: rotation of the plinth in the north-south plane

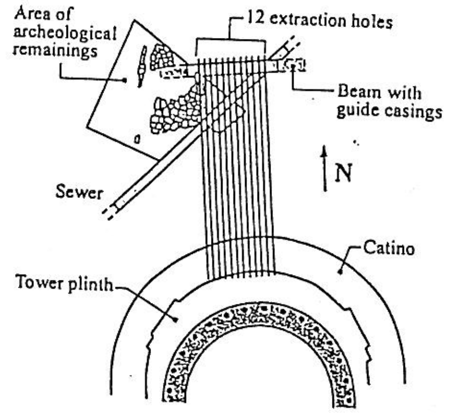


Fig. 20. Preliminary underexcavation experiment beneath the tower: layout in plan

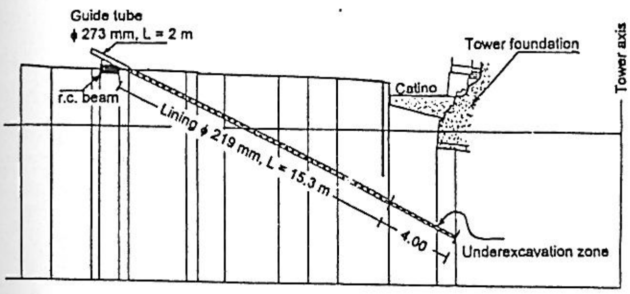


Fig. 21. Preliminary underexcavation experiment beneath the tower: cross section

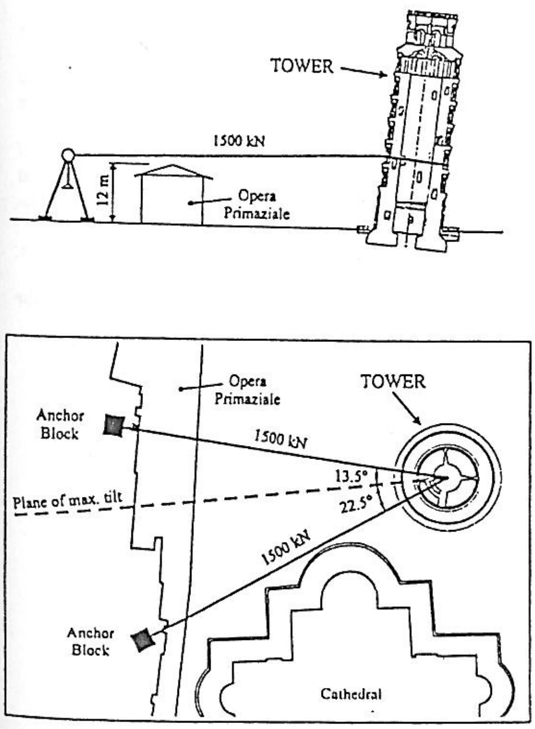


Fig. 22. Cable stay provisional structure. a) cross section. b) Layout in plan

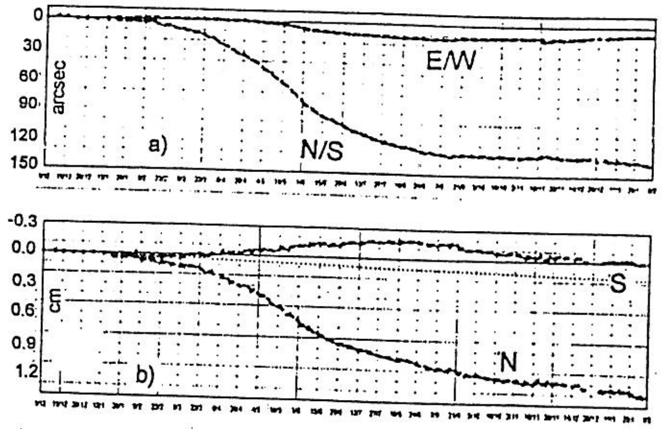


Fig. 23. Results of the preliminary underexcavation experiment. a) Variation of the inclination. b) Settlement of the north (N) and south (S) edges of the foundation

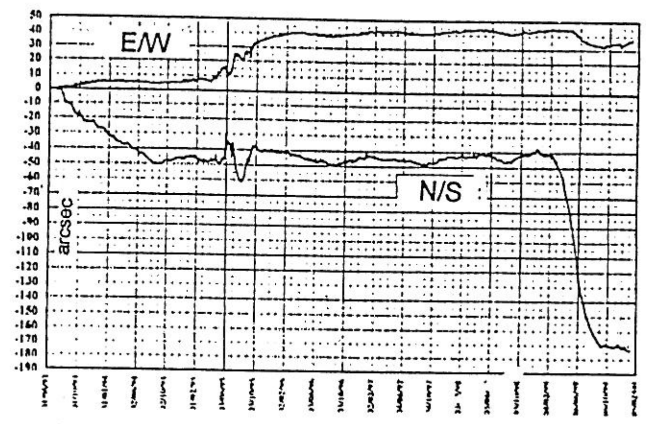


Fig. 24. Results of the preliminary underexcavation experiment in terms of variation of the inclination as measured by internal precision levelling

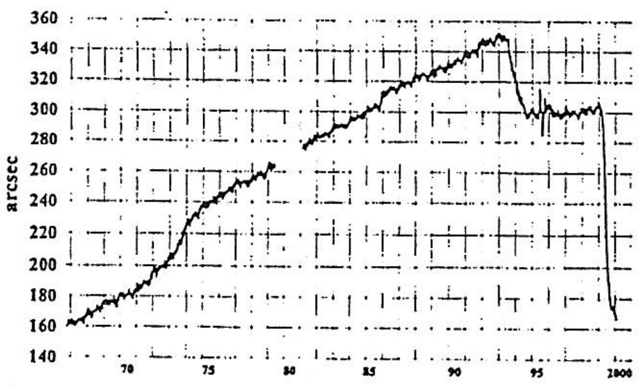


Fig. 25. Results of the preliminary underexcavation experiment in terms of variation of the inclination, as measured by the pendulum inclinometer

load applied to each stay is equal to about 72 kN, just enough to keep it in position.

The underexcavation experiment has been carried out between February and June, 1999. The results obtained are reported in fig. 23. During the underexcavation period, the tower rotated northwards at an increasing rate, as the extraction holes where drilled gradually ahead near the north boundary of the foundation and below it. At the beginning af June 1999, when the operation ceased, the northwards rotation of the tower was 90"; by mid September it had increased to 130". At that time three of the 97 lead ingots (weighing about 10 t each) acting on the north side of the tower were removed; since then the tower has exhibited negligible further movements. As a matter of fact, the preparatory operations for the final underexcavation (removal of the 12 guide casings of the preliminary undexcavation, installation of the 41 guide casings for the final underexcavation) have produced a slight further northward rotation, bringing the overall decrease of inclination in March, 2000 to 135".

The rotation in the east - west plane has been much smaller, reaching a final value of about 10" westwards, as intended.

Due to underexcavation, the north side of the tower foundation underwent an overall settlement equal to 1,3 cm; in the mean time the south side first raised up to 2 mm, and then gradually settled by the same amount, showing that the axis of rotation is located between the two points.

To put these results in perspective, the evolution of the tilt of the tower base since 1993 is reported in fig. 24. The effect of the underexcavation experiment largely overwhelms that of counterweight and the seasonal cyclic movements.

A longer time perspective is gained by the diagram in fig. 25, reporting the inclination since 1935 as measured by a pendulum inclinometer installed at that time. It may be seen that the effect of the preliminary underexcavation has been to bring the tower "back to future" by over 30 years.

CONCLUDING REMARKS

The stabilisation of the Tower of Pisa is a very difficult challenge for geotechnical engineering. The tower is founded on weak, highly compressible soils and its inclination has been increasing inexorably over the years to the point at which it is about to reach leaning instability. Any disturbance to the ground beneath the south side of the foundation is very dangerous. Therefore the use of conventional geotechnical processes at the south side, such as underpinning, grouting, etc., involves unacceptable risk.

The internationally accepted conventions for the conservation and preservation of valuable historic buildings, of which the Pisa Tower is one of the best known and most treasured, require that their essential character should be preserved, with their history, craftsmanship and enigmas. Thus any intrusive interventions on the tower have to be kept to an absolute minimum and permanent stabilisation schemes involving propping or visible support are unacceptable and in any case could trigger the collapse of the fragile masonry.

The technique of underexcavation provides an ultra soft method of increasing the stability of the tower which is completely consistent with the requirements of architectural conservation. Different physical and numerical models have been employed to predict the effects of soil removal on the stability. It is interesting to point out that some mechanisms (as, for instance, the occurrence of a crititical line beyond which the underexcavation becomes dangerous) are predicted by physical modelling and by the FEM analyses, while are missed by the simplified Winkler type models.

The preliminary underexcavation intervention, undertaken after having been satisfied by comprehensive numerical and physical modelling together with a large scale trial, has demonstrated that the tower responds very positively to soil extraction.

There is still a long journey ahead for the Tower, requiring detailed communication and control and the utmost vigilance, but indeed the first step has been taken in the permanent geotechnical stabilisation.

REFERENCES

BURLAND J.B., JAMIOLKOWSKI M, VIGGIANI C. (1999) *The restoration of the Leaning Tower: geotechnical aspects.* Workshop on the restoration of the Leaning Tower of Pisa, Pre-print Volume, 105 pp. Pisa

BURLAND J.B., POTTS D.M. (1994) *Development and application of a numerical model for the leaning tower of Pisa.* IS Pre-Failure deformation characteristics of geomaterials. Hokkaido, Japan, vol. 2, 715-738

CHENEY J.A., ABGHARI A., KUTTER B.L. (1991) *Leaning instability of tall structures.* Journ. Geot. Eng., Proc. ASCE, vol. 117, n. 2, 297-318

COMO M., IANNIRUBERTO U., IMBIMBO M. (1999) *Un modello rigido plastico per la sottoescavazione di torri pendenti.* Dpt. Ingegneria Civile, Univ. Roma Tor Vergata, Rapporto n. 52

DESIDERI A., VIGGIANI C. (1994) *Some remarks on the stability of towers.* Symp. on Development in Geotechnical Engineering, Bangkok

DESIDERI A., RUSSO G., VIGGIANI C. (1997) *La stabilità di torri su terreno deformabile.* RIG, vol. XXXI, n. 1, 5-29

EDMUNDS H. (1993) *The use of underexcavation as a means of stabilising the leaning tower of Pisa: scale model tests.* MSc Thesis, Imperial College, London

GORBUNOV-POSADOV M.I., SEREBRAJANYI R.V. (1961) *Design of structure upon elastic foundations.* Proc. V ICSMFE, Paris, vol. 1, 643-648

HABIB P., PUYO A. (1970) *Stabilité des fondations des constructions de grande hauteur.* Annales ITBTP, pp. 117-124

HAMBLY E.C. (1985) *Soil buckling and leaning instability of tall structures.* Struct. Eng., vol. 63, n. 3, pp. 78-85

JAMIOLKOWSKI M. (1997) *The leaning tower of Pisa.* XIV Manuel Rocha Lecture. Geotecnia, n. 85, March 1999

LANCELLOTTA R. (1993) *Stability of a rigid column with non linear restraint.* Géotechnique, vol. 41, n. 2, 243-256

POTTS D.M., GENS A. (1984) *The effect of the plastic potential in boundary value problems involving plane strain deformations.* Int. Journ. Anal. Numer. Methods in Geomechanics, vol. 8, 259-286

PEPE M. (1995) *La torre pendente di Pisa. Analisi teorico-sperimentale della stabilità dell'equilibrio.* PhD Thesis, Politecnico di Torino, 265 pp.

SANTOYO E., SÀNCHEZ B., PRIETO H. (1989) *Renivelaciòn de un conjunto de edificios.* Simposio sobre recimentaciones, Sociedad Mexicana de Mecanica de Suelos.

SCHULTZE E. (1973) *Der Schiefe Turm von Pisa..* Mitteilungen Technische Hochschule Aachen

TAMEZ E., SÀNCHEZ B., DE LA TORRE O. (1989) *Renivelaciòn de un edificio de 16 niveles.* Simposio sobre recimentaciones, Sociedad Mexicana de Mecanica de Suelos.

TAMEZ E., SANTOYO E., CUEVAS A., OVANDO E. (1995) *Catedral Metropolitana; diagnostico y proyecto geotecnico.* Catedral Metropolitana: correcciòn geométrica, informe tecnico. Asociaciòn Amigos de la Catedral Metropolitana de Mexico a.c., p. 39-113

TERRACINA F. (1962) *Foundation of the leaning tower of Pisa.* Géotechnique, vol. 12,

THILAKASIRI H.S. (1993) *A theoretical study of underexcavation for stabilising the tower of Pisa.* MSc Thesis, Imperial Colleg, London