XIV LIÇÃO MANUEL ROCHA, 1997

A XIV Lição Manuel Rocha intitulada "Leaning Tower of Pisa. An overview of the Problem" foi proferida pelo Prof. Michele Jamiolkowski em 24 de Outubro de 1997, na Fundação Calouste Gulbenkian.

A apresentação do Prof. Michele Jamiolkowski foi feita pelo Prof. Pedro Sêco e Pinto, Presidente da Sociedade Portuguesa de Geotecnia:

Exmª D. Teresa Rocha e Família
Exmº Sr. Eng.º Guimarães Lobato, em representação do Sr. Presidente da Fundação Gulbenkian
Exmº Sr. Prof. Santos Pereira, Vice-Presidente da Associação dos Antigos Alunos Geotécnicos da U.N.L.
Exmº Sr. Prof. Michele Jamiolkowski
Exmº Sr. Secretário de Estado de Obras Públicas
Prof. Maranha das Neves
Exmº Sr. Director do Laboratório Nacional de Engenharia Civil
Prof. Arantes e Oliveira

Caros Colegas
Minhas Senhoras e Meus Senhores

Em nome da Sociedade Portuguesa de Geotecnia, gostaria de deixar aqui uma palavra de apreço e gratidão à Fundação Calouste Gulbenkian, pelo apoio à realização destes eventos, desde a primeira lição Manuel Rocha em 1984.

Exaltar a figura e a obra de Manuel Rocha que tão alto lugar ocupa nesta imensa e ínclita galeria de cientistas de Portugal é uma tarefa ciclópica.

Considerando porém as lições que aprendi, o convívio que tive a honra de desfrutar com o Prof. Manuel Rocha, senti que as objecções do meu foro interior se iam pouco a pouco diluindo.

Venho animado daquele sentimento, feito de orgulho e humildade e de contrita consciência das minhas limitações.

Pensador genial, Manuel Rocha foi um dos talentos mais fulgurantes que iluminaram a Engenharia Geotécnica.

A tradição sempre fecunda em interpretações engenhosas, compraz-se em atribuir-lhe à testa ampla, a abundância e magnificência do estilo e a orientação da sua existência para um ideal mais vasto e mais nobre.

Uma das suas convicções é que nenhum conhecimento importante podia ser adquirido só com o ouvir da explicação. O verdadeiro método de aprender a ciência é o de se lançar à descoberta da verdade científica.

Sob o manto diáfano da fantasia e imaginação, a arte de Manuel Rocha apresentava mais uma vez, a nudez forte da verdade, ao dissecar com mestria as leis de semelhança dos modelos experimentais, em que concretiza os voos audaciosos das suas intuições de génio.

Sempre ávido em materializar a realidade complexa, na perspectiva cintilante e penetrante da sua análise, não deixou de detectar a necessidade de uma caracterização das propriedades mecânicas dos maciços rochosos, dando lugar a um progressivo movimento de esclarecimento, doutrinação, reflexão e, com o tempo de preparação técnica e científica, cujos frutos não tardariam a sazonar.

Foram os homens, de que Manuel Rocha é personagem arquétipo, animados de um ideal e dotados de uma têmpera, que por vezes a nós próprios parece incompatível com a nossas fraquezas, forjaram um conceito *sui generis* de Engenharia, que configura um novo perfil geotécnico e que constitui um paradigma para todos nós.

Encontramos assim pela força desse mesmo espírito, sentados fraternalmente neste Auditório, envolvidos neste "Karma", a celebrar a 14ª Lição Manuel Rocha, na companhia da Exmª Sr.ª D. Teresa Rocha e Família, que com a sua presença quiseram dignificar este acto.

Como seu discípulo junto-me no coro uníssono e vibrante de entusiasmo e admiração pelas excepcionais virtudes do mestre.

Nas conversas que trava com as pessoas, na dedicação e paciência com que acolhe e educa os jovens, na firmeza do seu carácter, na delicadeza e afabilidade do trato, entrevê-se o homem revestido duma missão especial.

À luz da sua obra em que o cientista e o professor se fundem numa só alma, em que a beleza e a verdade se dão amigavelmente as mãos, se justifica plenamente o aplauso, com que a posteridade e os contemporâneos acolheram a sua obra.

Passo agora a fazer a apresentação do Conferencista, que por ser tão conhecido entre nós, terá de ser necessariamente breve.

On behalf of the Portuguese Society for Geotechnique, it is for me a great honour and privilege to thank Prof. Michele Jamiolkowski from University of Torino and President of International Society for Soil Mechanics and Foundation Engineering during the tenue (1993-1997) for accepting our invitation to deliver the 14th Rocha Lecture.

I ask your permission to introduce your biography (a short version) in Portuguese.

Educação

Michele Jamiolkowski obteve o mestrado em Mecânica dos Solos e Geologia de Engenharia na Universidade Técnica de Varsóvia (Polónia) em 1959.

Frequentou cursos de pós-graduação em Engenharia Civil na Universidade Técnica de Torino, de 1960 a 1962, e cursos especiais na Universidade de Kiev, Laval (Quebeque) e MIT (Cambridge).

Actividades Académicas

De 1965 a 1969 foi assistente de investigação, de 1969 a 1978 Professor Associado e desde 1979 é Professor Catedrático de Engenharia Geotécnica da Universidade Técnica de Torino.

Desde 1982 é co-responsável do programa italiano para a outorga do grau de Doutor em Ciências.

Desde 1980 é Director do Laboratório de Mecânica de Solos da Universidade Técnica de Torino.

Foi eleito para os seguintes cargos:

- Presidente do Comité Internacional da Leaning Tower of Pisa, criado pelo Primeiro Ministro Italiano em 1990, tendo sido consultor geotécnico do grupo projectista de 1984 - 1989.
- Presidente da Sociedade Internacional de Mecânica de Solos e Engenharia de Fundações (1993 - 1997).
- Membro do Comité Executivo da Sociedade Italiana de Geotecnia durante oito anos.
- Doutor *Honoris Causa* em Engenharia Civil da Universidade Técnica de Bucareste.
- Membro da Academia Lagrangiana de Ciências de Torino
- Professor Honorário da Academia Sinica (Guangzhou)
- Responsável italiano no programa Tempus da União Europeia
- Membro correspondente estrangeiro da Academia Polaca de Ciências

Professor Visitante e Conferencista

Proferiu conferências e foi Professor Visitante das Universidades do Texas (U.S.A.), Ghent (Bélgica), Delft (Holanda), Gotenburg e Estocolmo (Suécia), Gdansk (Polónia), Purdue (U.S.A.), Oxford (U.K.), Cambridge (U.K.), Católica do Rio de Janeiro, Helsínquia (Finlândia), Instambul (Turquia), Califórnia (Berkeley e Los Angeles), Singapura, Sydney, Melbourne e Adelaide e ainda na Academia Polaca de Ciências e no Waterways Experimental Station.

Conferências Internacionais

Desde 1979 (VII Conferência Europeia de Mecânica dos Solos e Engenharia de Fundações) tem sido convidado para apresentar em quase todas as conferências Europeias e internacionais relatos gerais ou conferências especiais.

Experiência Profissional

- Foi director do laboratório de Geotecnia da Ródio em Milão de 1960 a 1964.
- Fundador da empresa de projecto e consultoria de Engenharia "Studio Geotécnico Italiano", em 1964.
- Consultor de grandes projectos de engenharia no domínio de centrais nucleares e térmicas, refinarias, portos, barragens de terra, infra-estruturas de transporte, em Itália, China, Egipto, Iraque, Irão, Argentina, Polónia, Arábia Saudita, Índia, Ceilão, Noruega, Dinamarca, Rússia, Israel, Brasil, Hong-Kong, Colômbia, Turquia e Espanha.

Publicações

É autor de mais de 170 publicações técnicas e científicas.

Prof. Michele Jamiolkowski, Mike for the friends, is a man of prodigious energy and challenging intellect and has wonderful ability to perceive and to present Nature. We are indebted for your outstanding contribution for the advancement of knowledge, in soil mechanics, laboratory and field tests, soil dynamics, soil modelling, shallow and deep foundations, and soil - structure interaction.

We are fortunate indeed that despite his tremendous responsibilities he has accepted the invitation to deliver the 14th Manuel Rocha Lecture entitled Leaning Tower of Pisa - Overview of the Problem, that will cover geotechnical structural and environmental aspects and also historical and arquitechtonical features, that will fascinate the audience.

He needs to return tomorrow, because he has to prepare a very important meeting, related with Tower of Pisa that will take place in Pisa on Monday 27th October.

It is for me a great honour and privilege to ask Prof. Michele Jamiolkowski to deliver the 14th Manuel Rocha Lecture entitled "Leaning Tower of Pisa. An overview of the Problem".

THE LEANING TOWER OF PISA*

A inclinação da torre de Pisa

MICHELE JAMIOLKOWSKI**

INTRODUCTION

1. Madame Rocha, Dr. Lobato, Mr. President of the Portuguese Geotechnical Society, dear Friends and Colleagues, I consider a great honour to be with you today and I thank you for granting me the privilege of delivering the prestigious Manuel Rocha Lecture.

Unfortunately, I have not had the opportunity to meet Prof. Rocha personally, yet I have known him through his valuable works that encompass a wide range of research interest in geotechnical and civil engineering. Also, I have had the possibility of attending some of Prof. Rocha's presentations at International Symposia and Conferences, realising and recognising how huge a contribution he has made worldwide to the knowledge of geotechnical engineering.

Throughout his career, Prof. Rocha's insight and expertise has been sought all over the world on his research application in the field of geotechnical, structural and hydraulic engineering. His intuition and the depth of his concepts testify that Prof. Rocha was extremely creative with a particular competence in seizing the physical and mechanical aspects of the behaviour of geomaterials.

Both the works and the goals achieved by Prof. Rocha are certainly such as to place him among the top ten greatest engineers of our century.
I am sure therefore, that you will understand all my concern of rising to this very special occasion of presenting the Fourteenth Manuel Rocha Lecture.

2. The aim of this paper is to present the current condition of the leaning Tower of Pisa, updated till the end of year 1998.

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.

3. 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 the Italian Government in the middle of 1990, will be presented.

---

* XIV Lição Manuel Rocha, 1997

** Professor da Universidade Técnica de Turim

4. The Monuments of Piazza dei Miracoli, see Fig. 1, including the Tower, are: the Cathedral, the Baptistery 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.

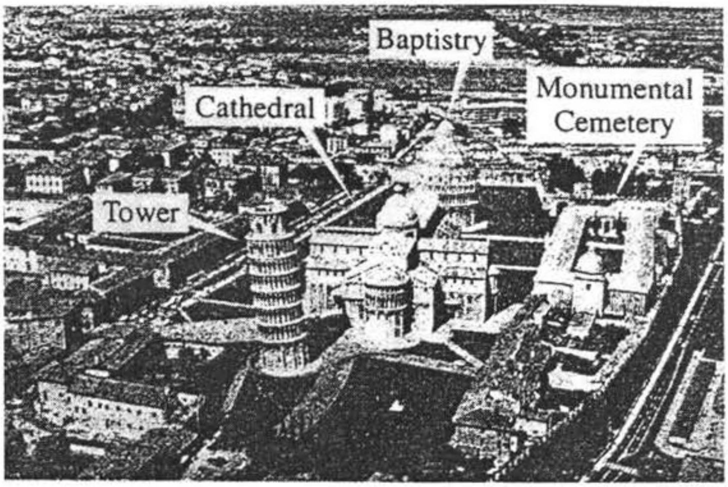


Figure 1 – Piazza dei Miracolli – Air view

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 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.

5. 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" (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.

V ≅ 142 MN, M ≅ 327 MNm, e ≅ 2.3 m *

(*) Situation in year 1990

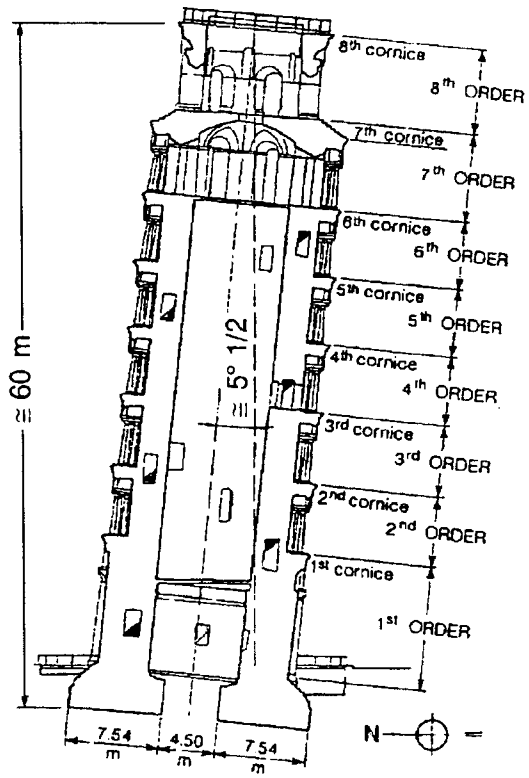


Figure 2 – Leaning Tower of Pisa – Cross-section

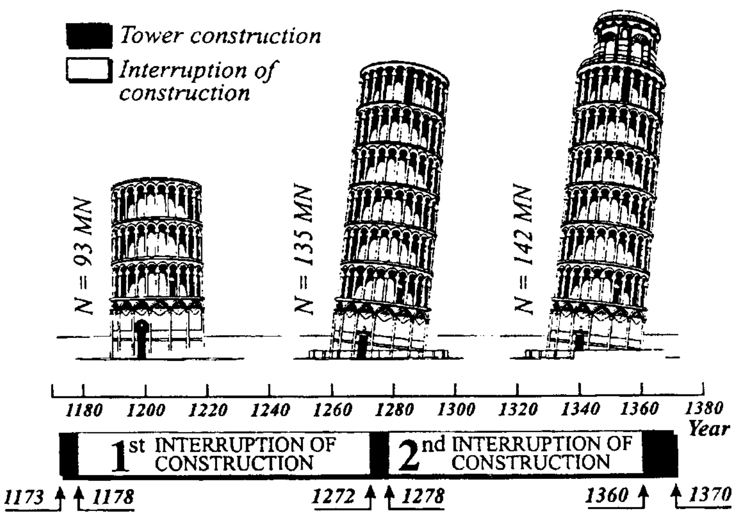


Figure 3 – Construction history

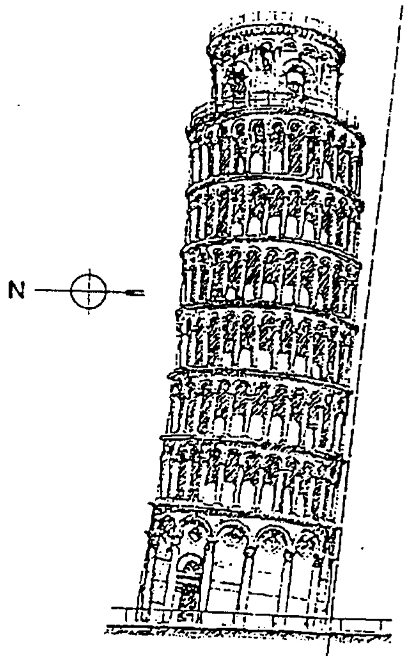


Figure 4 – Correction made during construction

6. 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.

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 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.

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

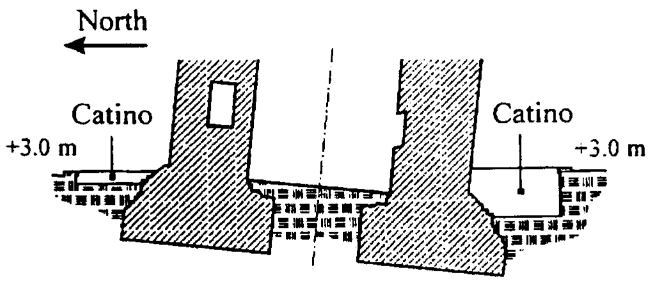


Figure 5 – Catino cross-section

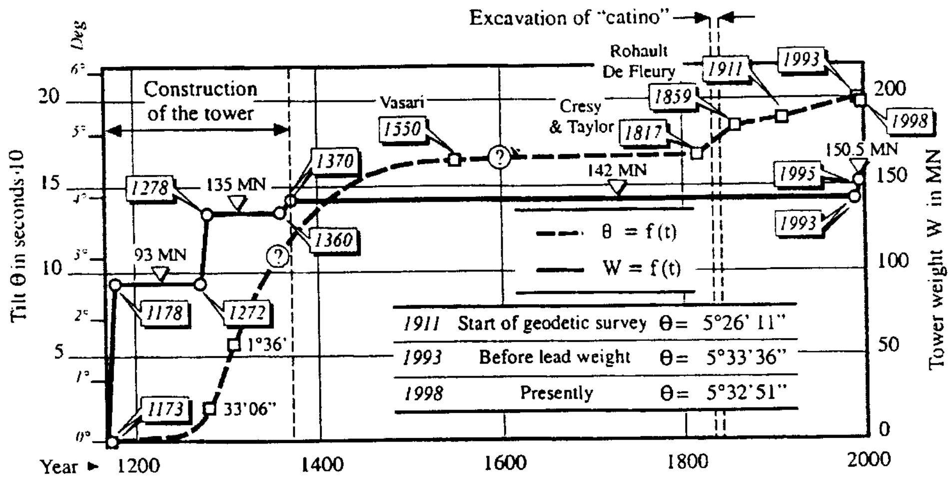


Figure 6 – Evolution of rigid tilt with time

– Three volumes published by the Ministry of Public Works Commission MPW (1971) whose data are summarised 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).

8. 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.

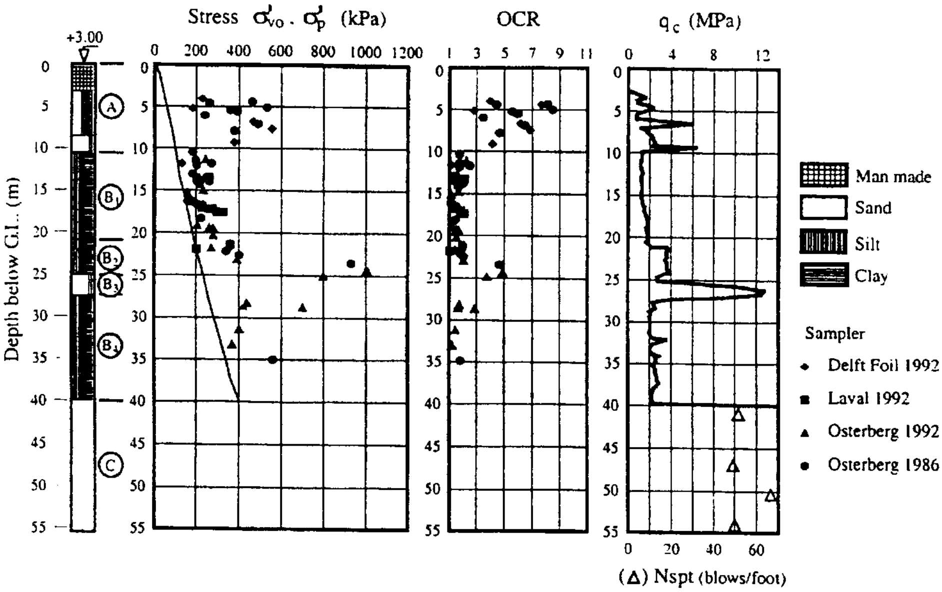


Figure 7 – Subsoil conditions

This Horizon can be subdivided in the following layers:

- Layer A_o; from elev. +3.0 to ±0.0, man-made ground containing numerous archaeological remainings dated from the 3rd 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 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:

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

- 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.

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.

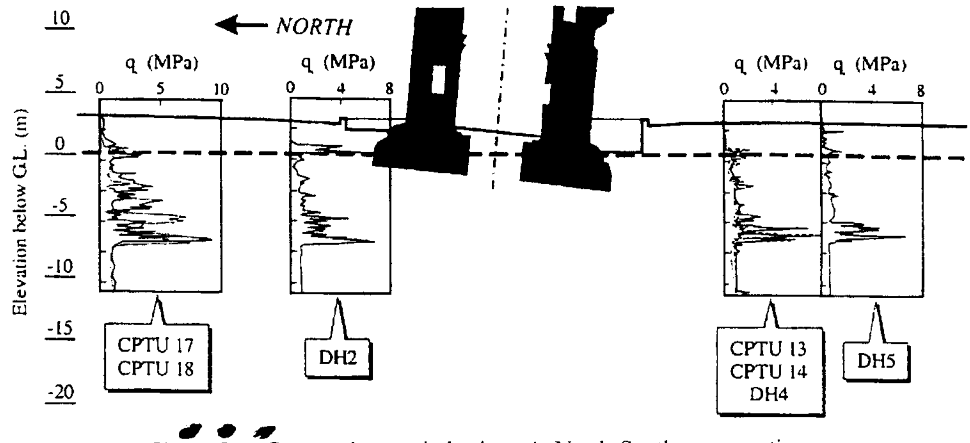


Figure 8a – Cone resistance in horizon A, North-South cross-section

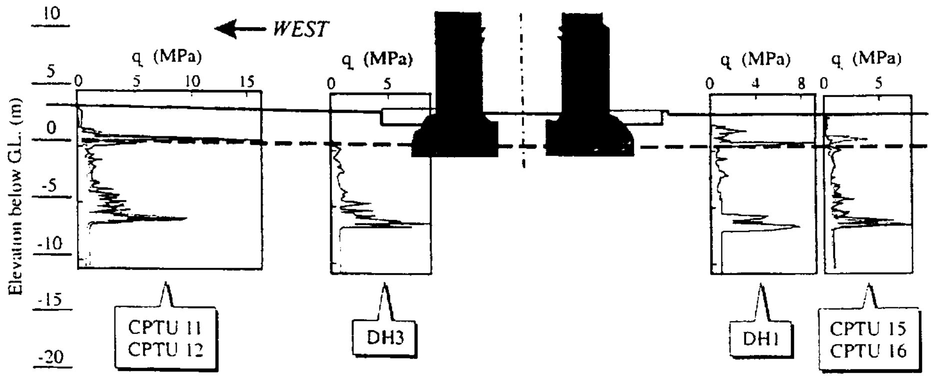


Figure 8b – Cone resistance in horizon A, West-East cross-section

- 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.

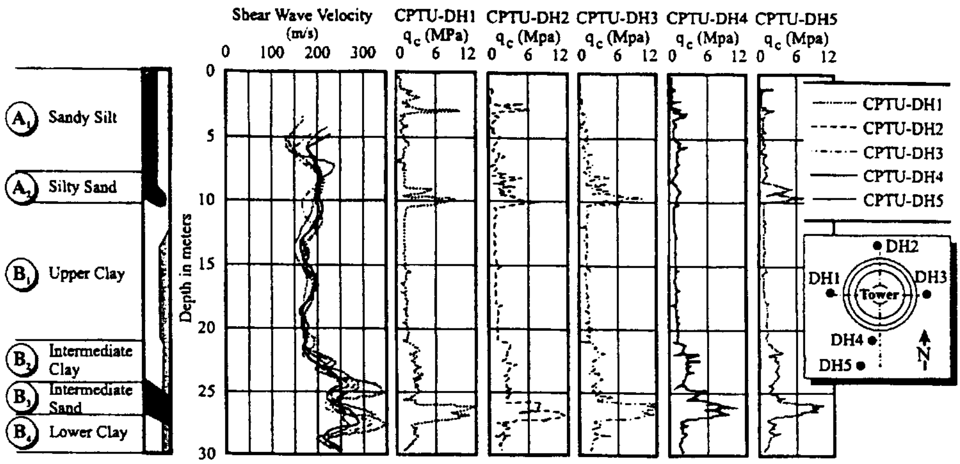


Figure 9 – Results of seismic cone penetration test

9. Fig.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).

10. Although detailing the geotechnical characterisation 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.

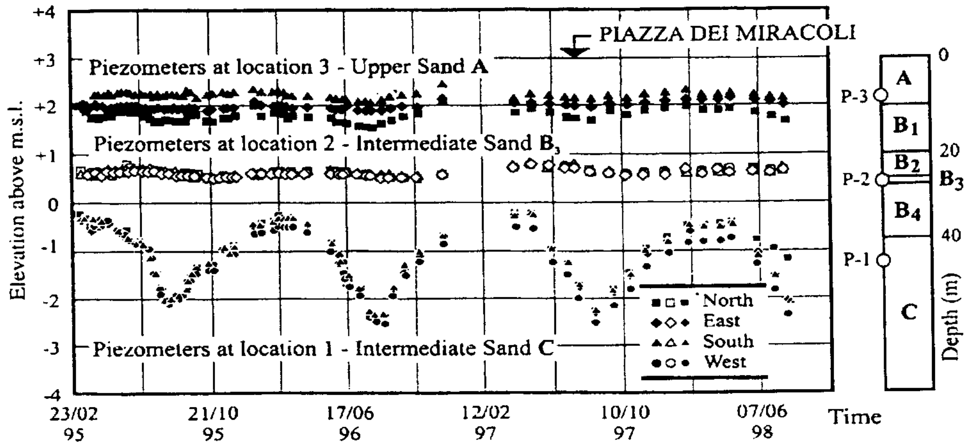


Figure 10 – Ground water level in sand layers beneath Piazza dei Miracoli

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 ageing, 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 rests (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 coefficient of earth pressure at rests (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.

11. 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:

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

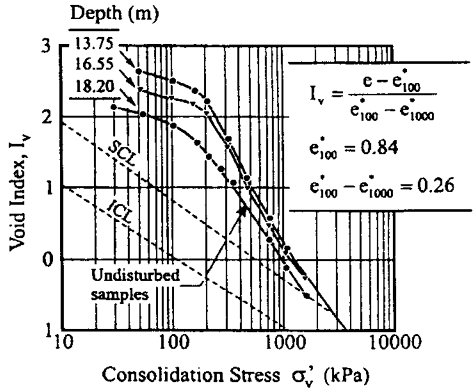
e = current void ratio of tested specimen

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.



SCL = sedimentation compression line
ICL = intrinsic compression line

Figure 11 - Compression curves of upper Pisa clay in term of void index

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

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.

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)

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 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 summarises 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.

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

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 normalised s_u as reported in Table 5.

– 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. Comparisons of the results of in situ and laboratory tests, in terms of Go 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

12. 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.

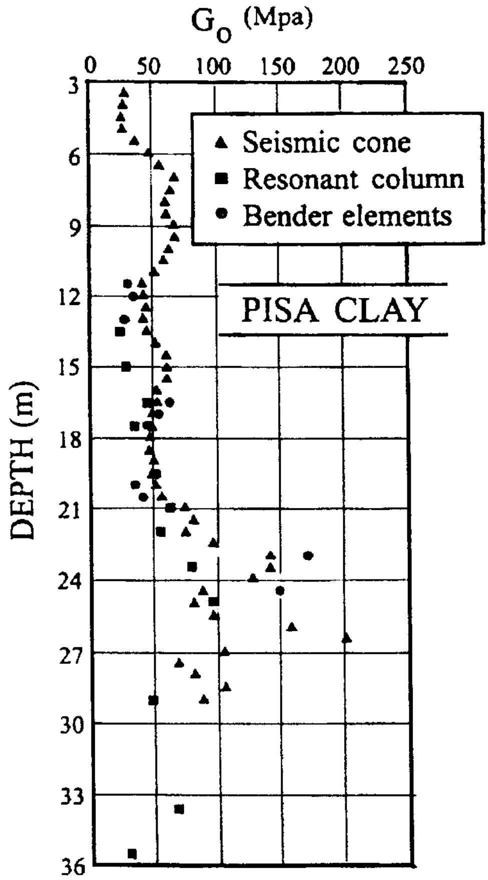


Figure 12 – Maximum shear modulus from in situ and laboratory tests

13. In 1934 two additional monitoring devices were installed:

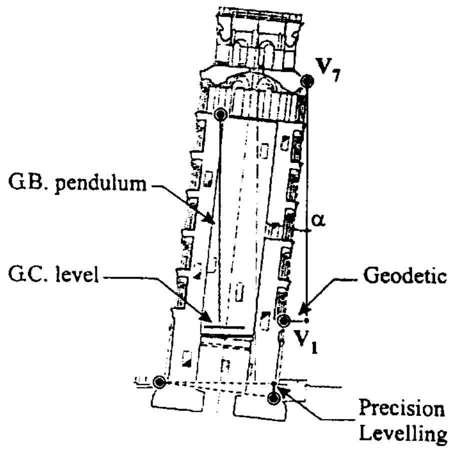
– Genio Civile (GC) Bubble Level installed in the instrumentation room located at the level of 1st cornice, see Fig. 13.

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.



- 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.

Figure 13 – Measurements of tower tilt in years 1911 through 1992

14. 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 benchmark located on the cast iron door of the Baptistery. Given the position of the benchmarks 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.

15. 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.

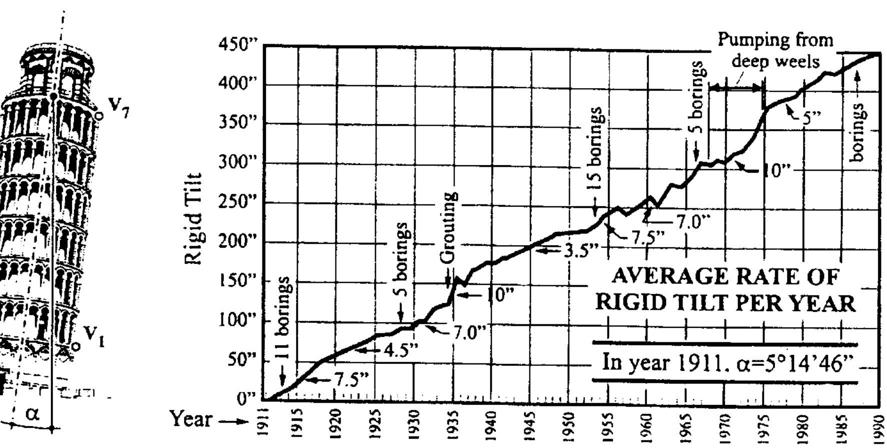


Figure 14 – Rigid tilt of leaning Tower of Pisa

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 waterproof 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, which implies the future overturning instability of the Tower.

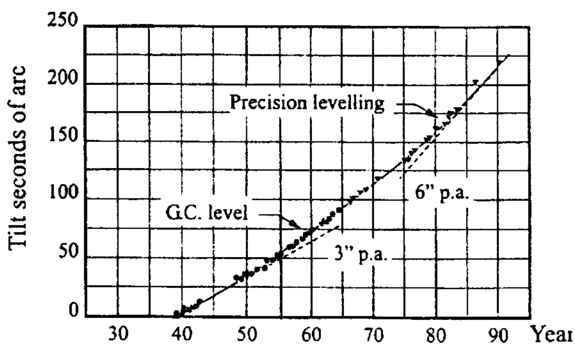


Figure 15 – Net tilt of tower plinth in years 1938 through 1990

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.

16. In the early nineties, prior to the stabilisation 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 benchmarks, 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 benchmarks, 901 through 915 in Fig. 16, located externally on the Tower plinth.

– Twenty-four benchmarks, 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 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.

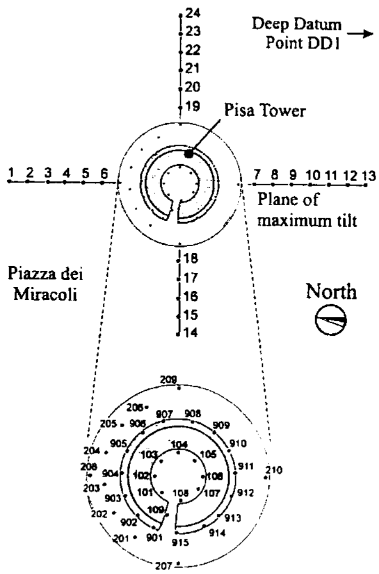


Figure 16 - Benchmarks for high precision levelling

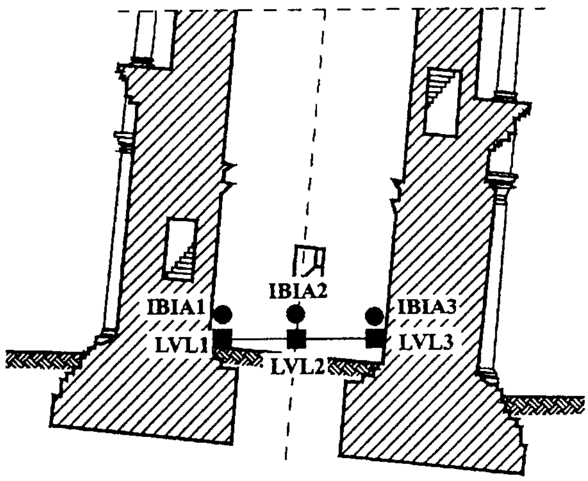
The description of additional instrumentation, also installed to monitor the movements of the Tower above the plinth and its masonry, is beyond the scope of this paper.

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).

LEANING INSTABILITY

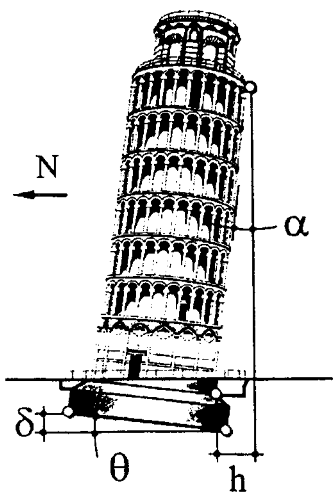
17. The Tower began to lean Southwards during the second construction phase when the masonry weight exceeded 65% of the monument (Fig. 6). This phenomenon has continued at a rate of 5 to 6 seconds per annum, a constant rate for the past few decades without taking into consideration environmental perturbations.

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.



■ LIVELOMETER (LVL) ● BIAXIAL INCLINOMETER (IBIA)

Figure 17 – Measurements of rigid tilt at tower base



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

Figure 18 – Inclination of Pisa Tower terms of reference

18. 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.

19. 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 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 stabilisation 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".

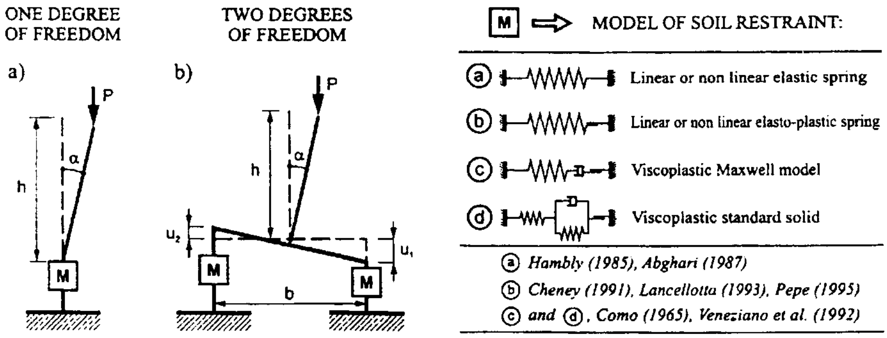


Figure 19 – Leaning instability models

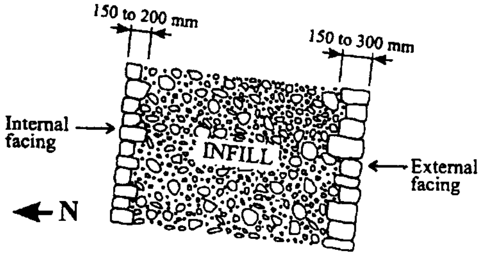
STRUCTURAL FEATURES

20. 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.

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.



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

Figure 20 – Cross-section of tower masonry

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

21. Relevant mechanical properties of the two components of the Tower masonry are summarised 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.

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.

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.

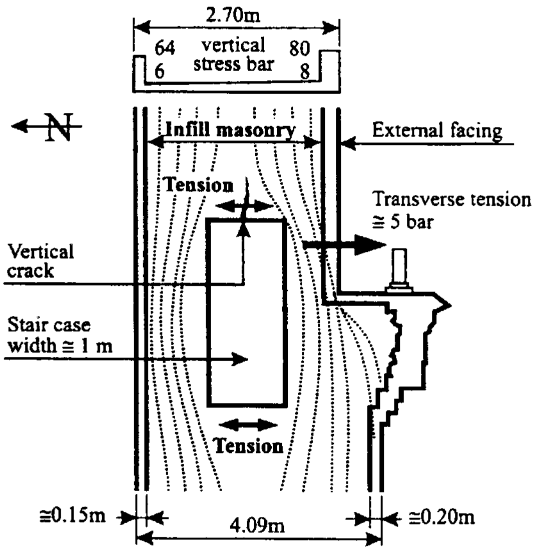


Figure 21 – Cross-section of tower at first cornice

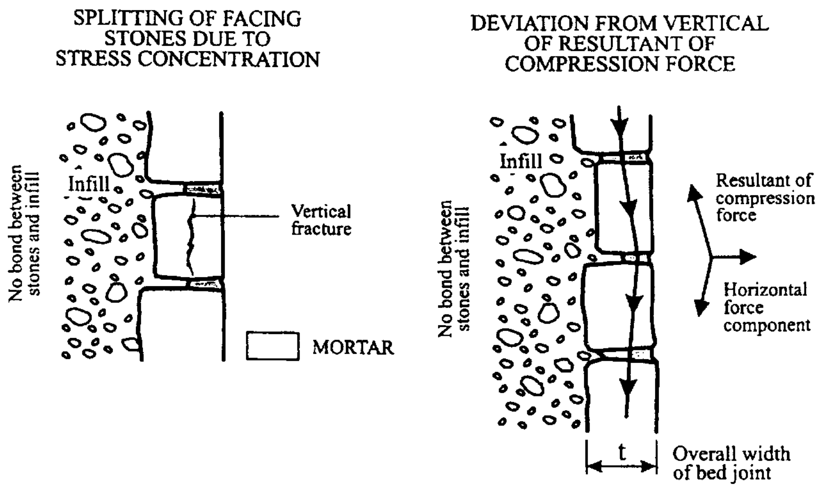


Figure 22 – Marble stone facing imperfection of bed joints

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

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

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

STABILIZATION WORKS

23. 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 stabilisation 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; stabilise the foundation, strengthen the structure and plan the architectural restoration and started its operations in September 1990.

24. 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 of structural collapse. This decision was taken in view of the awareness that the selection, the design and the realisation of the permanent stabilisation 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 stabilisation 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 stabilising 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.

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

25. 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.

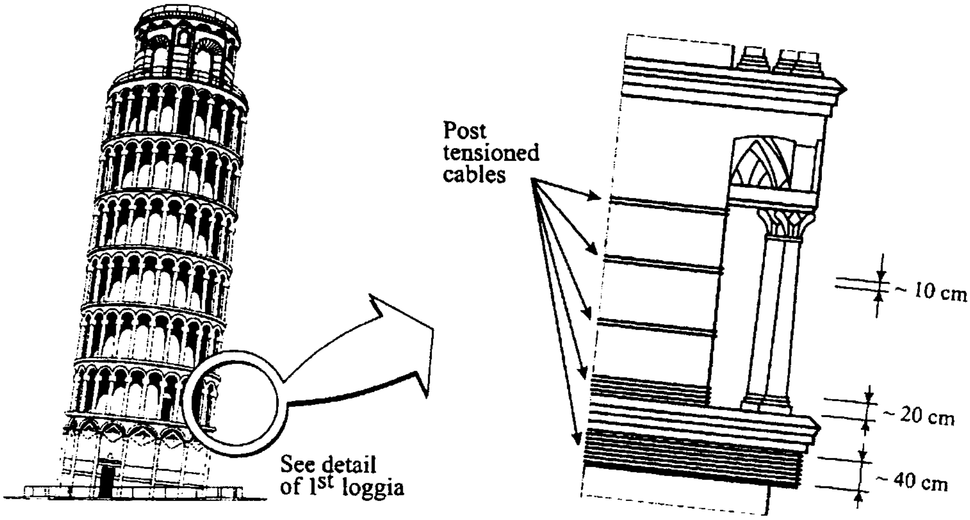


Figure 23 – Temporary structural strengthening

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 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.

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.

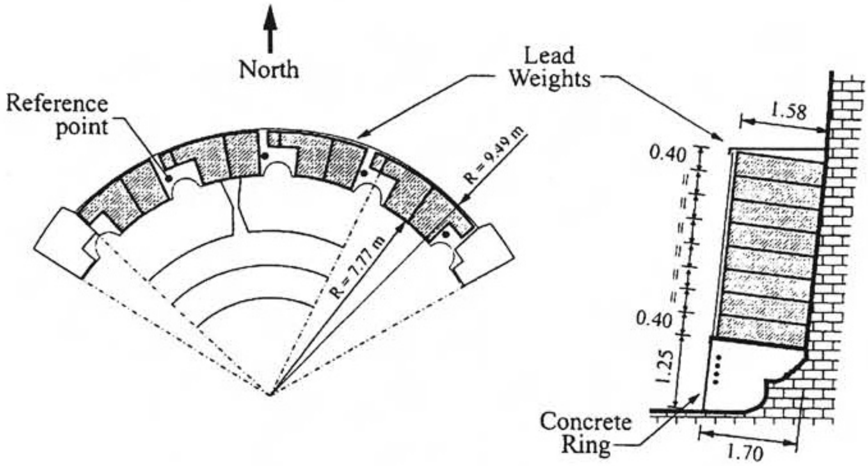


Figure 24 - Counterweight on North edge of Tower plinth

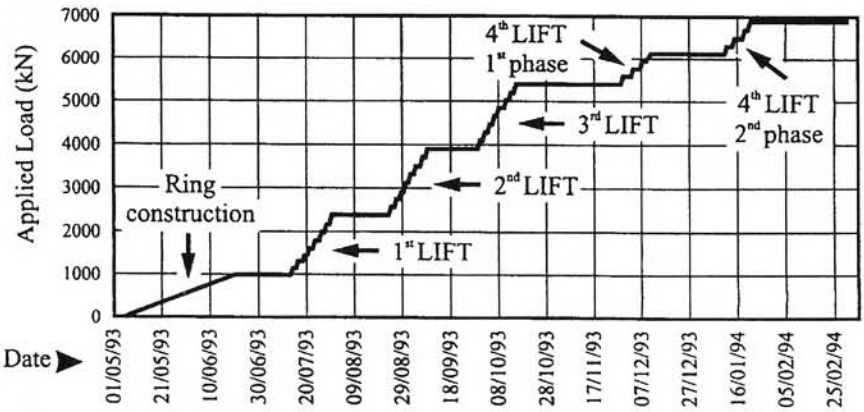


Figure 25 – Counterweight loading sequence

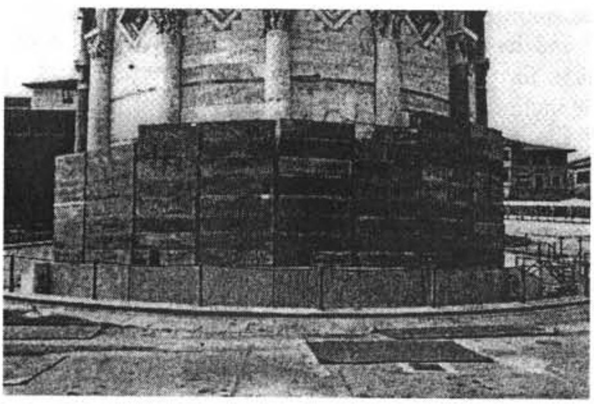


Figure 26 – The counterweight placed in the period between May 1993 and January 1994

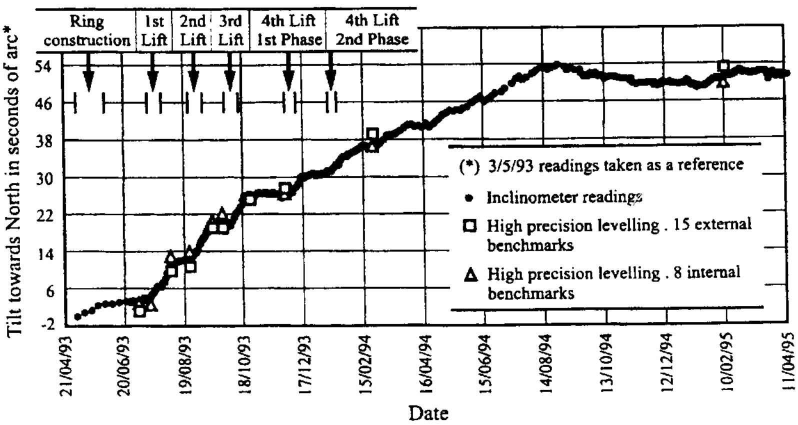


Figure 27 – Tilt towards North as result of counterweight application

26. 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 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.

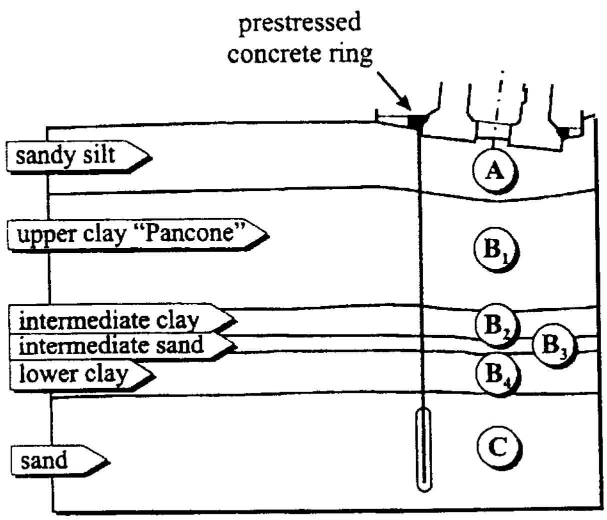


Figure 28 – Ten anchors solution

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.

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.

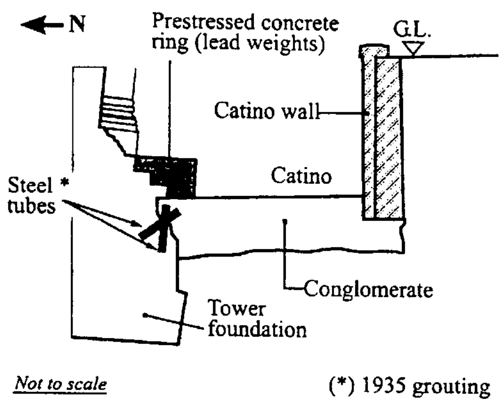


Figure 29 – South section of Catino – Actual configuration

27. 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.

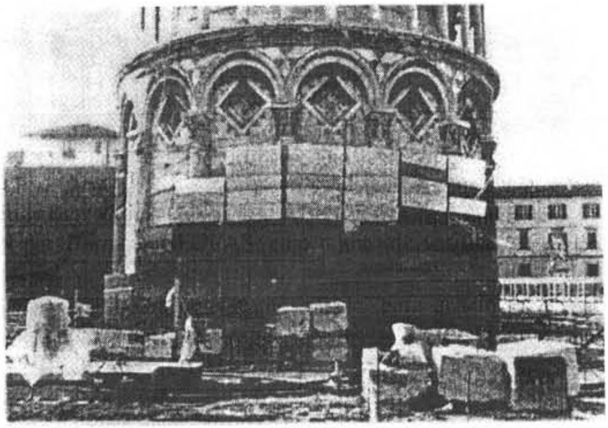


Figure 30 - The counterweight with additional 2700KN added in September 1995

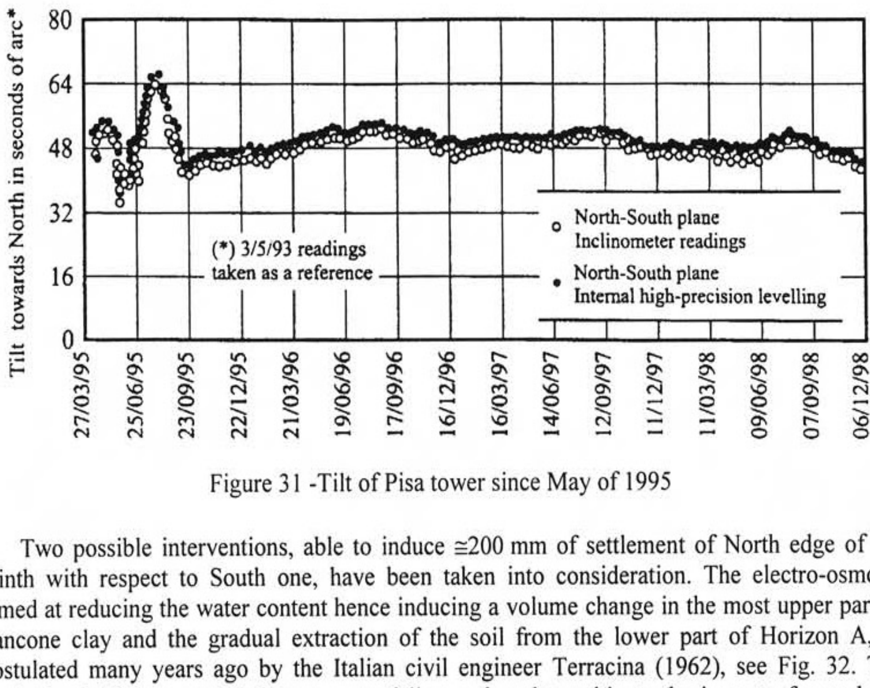


Figure 31 -Tilt of Pisa tower since May of 1995

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 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).

- 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

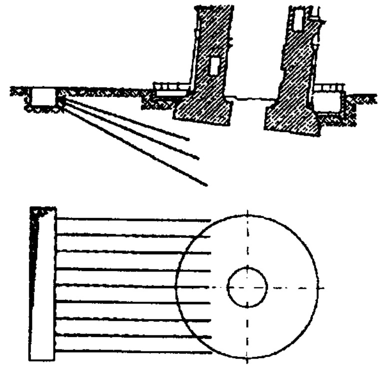


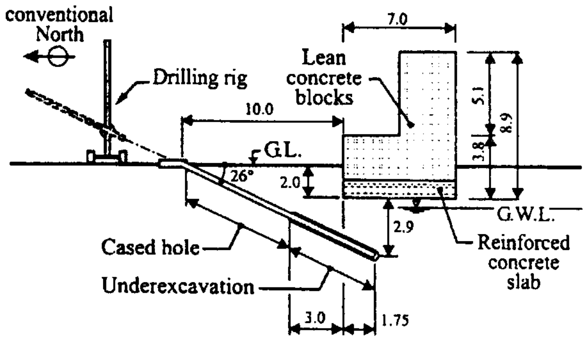
Figure 32 – Underexcavation for correcting inclination of Pisa tower (Terracina, 1962)

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 finalise the technological aspects of the intervention.

28. 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.

During this experiment, the following important lessons were learned:

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



Drawing not to scale - all dimensions in meters

Figure 33 - Underexcavation field trial

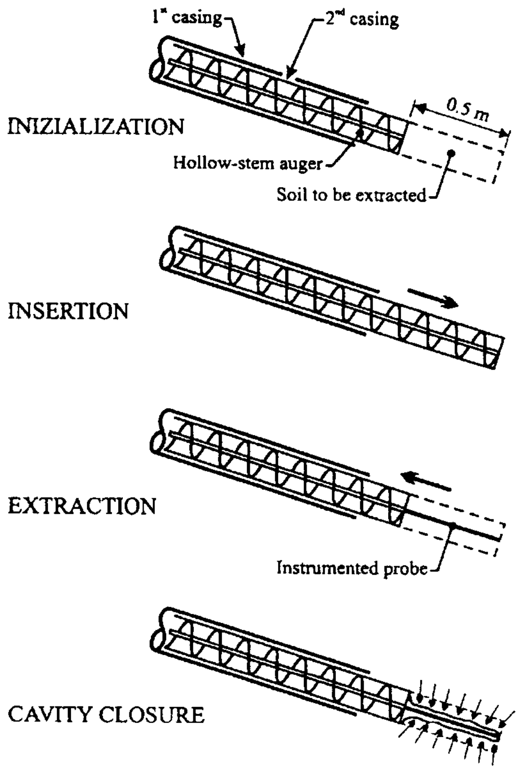


Figure 34 - Soil extraction process

29. 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. 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 stabilisation of the Tower.

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

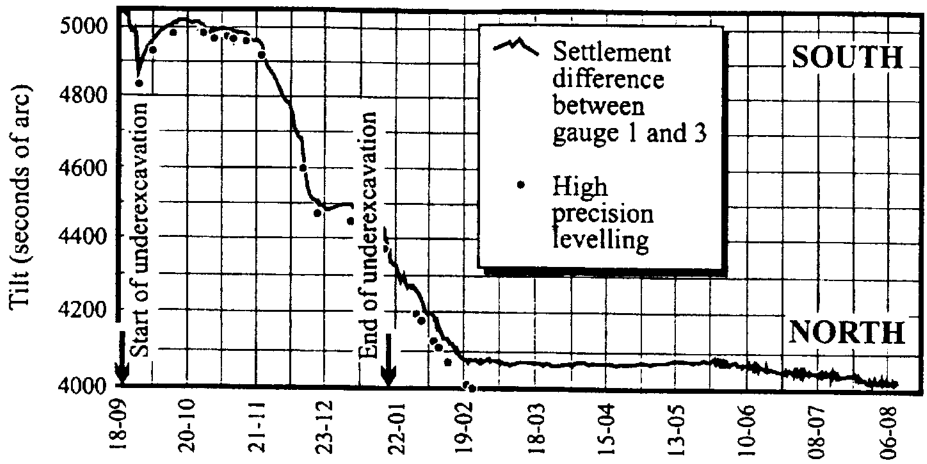


Figure 35 – Underexcavation field trial – Tilt of plinth in North-South plane

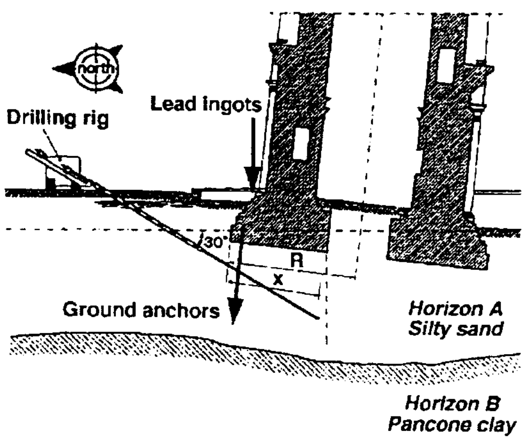


Figure 36 – A hole for ground extraction under the tower

"STOP PRESS" ADDENDUM

30. In February 1999, the preliminary underexcavation (see Paragraphs 27, 28 and 29) intervention under the Tower was started. The intervention consists in a very gradual controlled ground extraction from 12 holes shown in Fig. 38. These holes are inclined ≅26° with respect to the G.L. and penetrate under the catino and the preliminary stage only one meter under the North edge of the Tower plinth, see Fig.38.

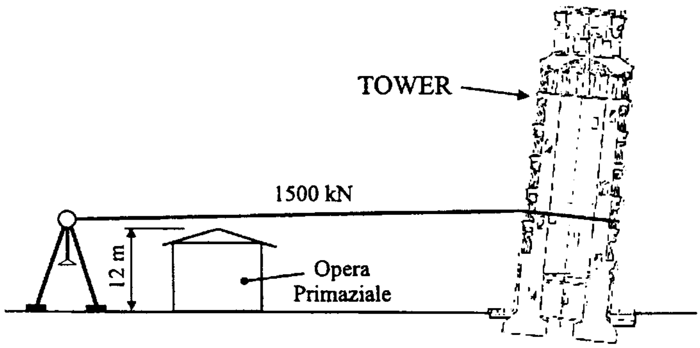


Figure 37a – Cable stay structure – Cross-section

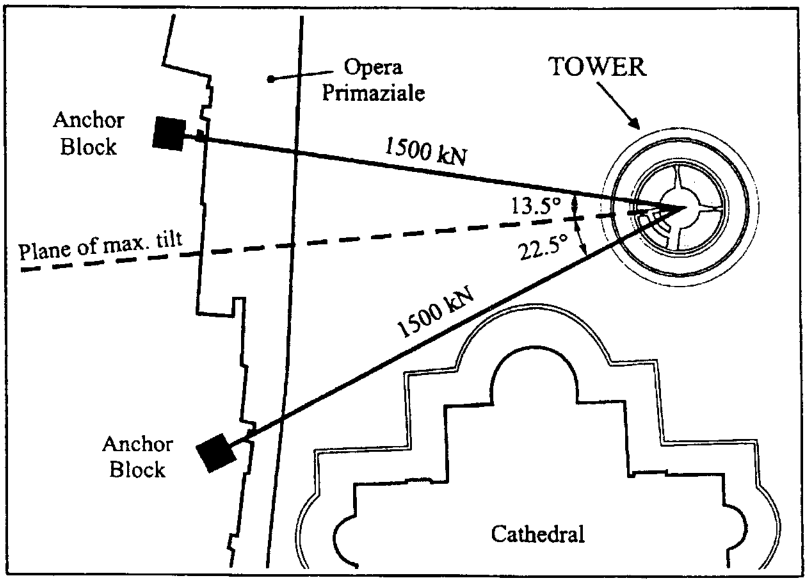


Figure 37b – Cable stay structure – Plan

The aims of the preliminary underexcavation, which will be completed at the end of May 1999, are to furtherly refine the technological aspects of the operation and to ascertain how the Tower is responding to this intervention. At the time of writing (April 23rd 1999), approximately 4 m^3 of the ground has been extracted. Since the start of the intervention, the Tower has responded positively rotating towards the North. With the instantaneous centre of rotation corresponding to the South edge of the plinth. The achieved reduction of the Tower inclination at present is 36 seconds of arc.

After the completition of the preliminary underexcavation, the Committee for the safeguard of the leaning Tower of Pisa will analyse the obtained results and if they are positive as those yielded till now, will prepare a more massive ([3]) underexcavation intervention aimed at reducing the Tower inclination by 1500 to 2000 seconds of arc. The achievement of such a goal will stop the phenomenon of leaning instability, hopefully for ever, or at least it will reduce it greatly, guaranteeing the stability of the Monument for the next 200 to 300 years.

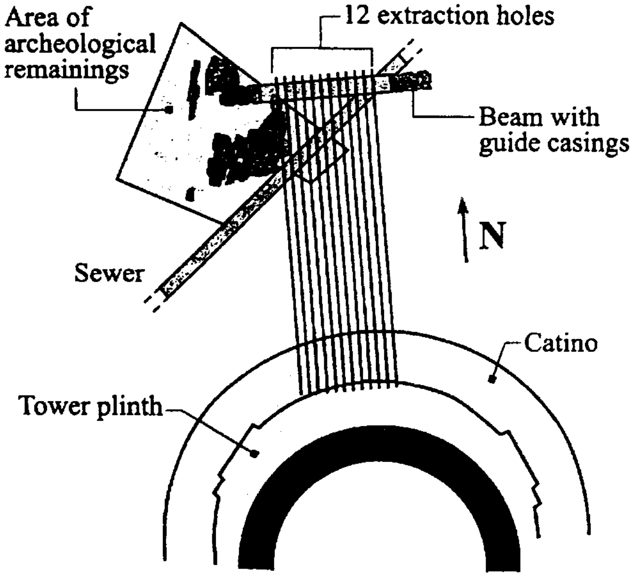


Figure 38 – Preliminary underexcavation scheme

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. - *Leaning instability of tall structures.* Ph.D. Thesis, University of California, Davies, Ca.,1987.

BERARDI, G., CAROTI, L. GIUNTA, G., JAMIOLKOWSKI, M. and LANCELLOTTA, R. - *Mechanical properties of upper Pisa clay.* Proc. X ECSMFE, Firenze, 1991.

BURLAND, J.B. - *On the compressibility and shear strength of natural clays.* The 30th Rankine Lecture, Geotechnique, no.3, 1990.

BURLAND, J.B. - Personal Communication, 1990a.

BURLAND, J.B. - Personal Communication, 1991.

([3]) employing ≅36 to 48 holes penetrating a little more deeper under the plinth.

BURLAND, J.B. - *Monitoring the foundations of the leaning Tower of Pisa.* Proc. Seminar on Instrumentation in Geotechnical Engineering, Hong Kong, Institution of Engineers, 1995.

BURLAND, J.B. and VIGGIANI, C. - *Osservazioni del compartimento della Torre di Pisa.* RIG, no.3, 1994.

CALABRESI, G., RAMPELLO, S. and CALLISTO, L. - *The leaning Tower of Pisa – Geotechnical characterisation of the Tower's subsoil within the framework of the critical state theory.* Report of the University of Rome La Sapienza, Report, 1993.

CASAGRANDE, A. - *The determination of the pre-consolidation load and its practical significance discussion.* Proc. I ICSMFE, Harward University. Cambridge, Mass, 1936.

CHENEY, J.A., ABGHARI, A. and KUTTER, B.L. - *Stability of leaning towers.* JGE Div. ASCE, no.2, 1991.

COMO, M. - *Asymptotic of a leaning tower on slowly sinking ground.* Giornale del Genio Civile (in Italian), 1965.

COSTANZO, D. - *Mechanical behaviour of Pisa clay.* Ph.D. Thesis, Technical University of Torino, Italy, 1994.

COSTANZO, D., JAMIOLKOWSKI, M., LANCELLOTTA, R. and PEPE, C. - *Leaning Tower of Pisa – Description of the behaviour.* Banquet Lecture, Settlement '94, College Station, TX, 1994.

CROCE, A., BURGHIGNOLI, A., CALABRESI, G., EVANGELISTA, A. and VIGGIANI, C. - *The Tower of Pisa and the surrounding square: Recent observations.* Proc. X ICSMFEE, Stockholm, 1981.

DESIDERI, A. and VIGGIANI, C. - *Some remarks on stability of towers.* Proc. Symposium on Developments 1936-1994 in Geotechnical Engineering, Bangkok, 1994.

DESIDERI, A., RUSSO, C. and VIGGIANI, C. - *Stability of Towers on Compressible Ground.* RIG, no.1, 1997.

HAYAT, T.M. - *The coefficient of earth pressure at rest.* Ph.D. Thesis, University of Illinois at Urbana, Champaign, Ill, 1992.

HAMBLY, E.C. - *Soil buckling and the leaning instability of tall structures.* The Structural Engineer, no.3, 1985.

HAMBLY, E.C. - *Overturning instability.* JGE Div. ASCE, no.4, 1990.

JAMIOLKOWSKI, M. - *Research applied to geotechncial engineering.* Proc. Instn. Civ. Engrs., Part 1, Vol.84, 1988.

JAMIOLKOWSKI, M., LANCELLOTTA, R. and Lo Presti, D.C.F. - *Remarks on the stiffness at small strains of six Italian clays.* Proc. Int. Symposium on Prefailure Deformation of Geomaterials, Sapporo, Japan, 1994.

LANCELLOTTA, R. - *The stability of a rigid column with non-linear restraint.* Geotechnique, no.1, 1993.

LANCELLOTTA, R. - *The leaning Tower of Pisa – Geotechnical theory and model tests.* Discussion, Soil and Foundation, no.3, 1993a.

LANCELLOTTA, R. and PEPE, C. - *Pisa Tower – Geotechnical properties of the horizon A.* Research Report 2.1, Technical University of Torino, 1990.

LANCELLOTTA, R. and PEPE, C. - *Mechanical behaviour of Pisa clay.* Research Report 2.2, Technical University of Torino, 1990a.

LANCELLOTTA, R., COSTANZO, D. and PEPE, M.C. - *The leaning of Tower of Pisa – Geotechnical characterisation of the tower subsoil.* A Summary Report of the Technical University of Torino, 1994.

LANCELLOTTA, R. and PEPE, M.C. - *On the stability of equilibrium of the leaning Tower of Pisa.* Accademia della Scienza – Atti di Scienze Fisiche no.132, 1998.

LEONARDS, G.A. - *Foundation performance of Tower of Pisa.* Discussion. JGE Div. ASCE, GT1, 1979.

LEONHARDT, F. - Personal Communication, 1991.

LEONHARDT, F. - *The committee to save the Tower of Pisa: A personal report.* Structural Engineering International, no.3, 1997.

LUCHESI, G. - *La torre pendente – Un mistero lungo ottocento anni.* Edit. Bandecchi e Vivaldi, Pontedero, 1995.

MACCHI, G. - *Monitoring medieval structures in Pavia.* Structural Engineering International, no.1, 1993.

MESRI, G. - Personal Communication, 1989.

MESRI, G. and CASTRO, A. - *The C_a/C_c concept and K_o during secondary compression.* JGE Div. ASCE, GT3, 1989.

MESRI, G., HEDIEW, J.E. and SHAHIEN, M. - *Geotechnical characteristics and compression of Pisa clay.* Proc.XIV ICSMFE, Hamburg, 1997.

MITCHELL, J.K., VIVATRAT, V. and LAMBE, W.T. - *Foundation performance of Tower of Pisa.* Journ. of JGE Div. ASCE, GT3, 1977.

MPW Ministero dei Lavori Pubblici - *La stabilità della torre di Pisa.* Istituto Poligrafico e Zecca dello Stato, Roma, 1979.

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

PEPE, M.C. - *The leaning Tower of Pisa – Theoretical and experimental analysis of the stability of equilibrium.* Ph.D. Thesis – Technical Univ. of Torino (in Italian), 1995.

SCHIFFMANN, R.L. - *Settlements due to groundwater fluctuations in the lower sand.* Report submitted to International Committee for Safeguard of the Leaning Tower of Pisa,1995.

TAMEZ, E. SANTOYO, E. and CUAVES, A. - *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 1992.

TAMEZ, E., OVANDO, E. and SANTOYO, E. - *Underexcavation of Mexico City's Metropolitan cathedral and Sagrario church.* Proc. XIV ICSMFE, Hamburg, 1997.

TERRACINA, F. - *Foundations of the Tower of Pisa.* Geotechnique, no.4, 1962.

TERZAGHI, K. - *Die Ursachen der Schiefstellung des Turms von Pisa.* No.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. - *Leaning of the Tower of Pisa. Part I: Reological models; Part II: Future rotations and risk of instability.* Draft of papers, 1995.

14ª LIÇÃO MANUEL ROCHA

VOTO DE AGRADECIMENTO

Foi com enorme prazer que aceitei o convite da organização da 14ª Lição Manuel Rocha para pronunciar o voto de agradecimento ao Professor Michele Jamiolkowsky pela brilhante conferência que acaba de realizar.

Vão desculpar-me que o faça em inglês já que este agradecimento é fundamentalmente dirigido ao nosso ilustre convidado.

Professor Michele Jamiolkowsky,

The lesson that you have just presented to us was brilliant and could only be given by someone so familiar with the most complex aspects of the geotechnical area.

The foundation problems of the leaning tower of Pisa are a Kind of symbolic challenge for the geotechnical profession. From a distant past, is uncountable the number of those who tried to stop the dangerous movement of the tower. All of them tried but without success. In certain cases, an increase of the inclination was the result of the attempt. The invitation of the Italian Government to you, to be the Chairman of that last international committee created to solve a problem where the foregoing committees always failed, is a clear recognition of your outstanding skill on these matters.

As we were told through this conference, the tower movement was finally stopped an a rational explanation for this behaviour exists. All the geotechnical community must acknowledge this result.

The mixture of the history of an ancient structure with the more complex geotechnical methods and techniques resulted in a splendid lecture that all of us would keep in memory.

It is then with the very great pleasure that I propose a vote of thanks to Professor Michele Jamiolkowsky for a most memorable 14th Manuel Rocha Lecture.

E. Maranha das Neves