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FOUNDATIONS

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L.G.W. Verhoef

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- Problems and Possibilities -

FOUNDATIONS

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INTRODUCTION ON FOUNDATIONS

What do we understand by the term 'foundations'? This general idea may embody the literal 'groundwork' that provides support for a building and may possibly include the entire structural works that serve to prevent subsidence. Remarkably, in this definition the word 'groundwork' incorporates both the 'ground' itself and the structural 'works'. Ground itself is a general word for the usually layered formations containing elements such as sand, clay peat and silt deposited beside each other or are intermingled, but each having its own characteristics. The ground is the basis upon which or in which the object must be placed in such a way that settlement in the form of controlled deformations is anticipated and specified magnitudes are not exceeded. To obtain any certainty about this, it is necessary to have insight into how the ground will deform under increasing loading or, for example a fall in the groundwater table. This process has been perfected over the centuries and in principle, it has developed on the basis of the measured or perceived resistance when an element is being driven into the ground. It has also become clear that the ground itself will settle under the influence of loading processes, and that it can move beside fixed objects, thereby relaxing the loading. Often the foundation is supposed to bear the structure. Indeed, if the foundation is not adequate, settlement will occur, followed by cracking in the rising masonry wall. The cracks tell the observer directly where the problems in the foundation are located, but they show that the masonry above ground is also part of the foundation. Later changes to the part of the building that is above ground affect the deformation behaviour in the foundation. Thus, the building itself is part of the structure that is needed to support the building.

In principle, depending on the penetration of frost into the ground, a shallow foundation may be laid. If the upper layers cannot take up the loading of the structure sufficiently free of settlement, deeper layers must be sought which can bear the loads without subsidence. With the increase in the necessary depth, a sequence is initiated starting with the deeper construction of the masonry in which, to save material relieving arches are sometimes made. When even deeper foundations are needed, first pits are dug, then masonry is added, and with still greater settlement, wooden piles are used.

![Shallow foundation](image1)

![Foundation on pits](image2)

![Amsterdam foundation system](image3)
One of the earliest uses of wooden piles in Dutch ground is found beneath a late Roman bridge over the River Maas at Cuijk. From dendrological investigations it appears that the first bridge was built in the period of 330-350 AD, while the restoration of the bridge took place at the end of the fourth century AD. The piling was very exactly executed according to a geometric pattern. The oak piles were fitted with wrought iron points to facilitate the piling work. A framework of wooden beams was laid on the piles, on top of which comes the masonry. From the traces on the masonry, this bridge appears to have been constructed from recycled material from other Roman buildings. (Source: Drs. B. Goudzwaard, National Service of Archeological Heritage) In the following centuries there was very little change in the system of using wooden piles with a wooden frame under the masonry

Roman pile with wrought iron point
Photo: Peter Bersch, Cuijk

The setting of the Roman piles
National service of archeological heritage
(Logesterion, A. Beerens, B. Goudswaard, R. Kroes, Stichting RAAP Amsterdam)

Over the centuries, buildings are always subjected to changes. Parts are demolished, added on or built up and very often new buildings are constructed adjacent to existing buildings. All this changes the loading regime on the subsoil, resulting in different types of settlement. In areas where a very solid foundation is present this does not have much effect. In countries such as the Netherlands, as also in other countries where the uppermost strata are very cohesive and sensitive to settlement, this was and still is severe in the buildings above. If it is detected that a foundation is not satisfactory, an attempt is often made to improve the situation at a later date. For this, the foundation must be exposed, which leads to great damage in and around the building and hindrance to its functioning. Moreover, the cost of

Not satisfactory foundation
Exposing the foundation. The cost of re-strengthening is considerable
restrengthening is considerable. Even so, demolition and reconstruction is increasingly less of an option. This is partly because of the great increase in the price of property, in consequence of which the cost of restoration becomes relatively lower. However, it is also due to the development of new techniques that strive to provide supplementary bearing capacity.

Ideas about foundations have been so extended that the adaptation of foundations should now be part of basic knowledge. It is too easy, and therefore expensive, to design an entirely new foundation under an building and behave as if the existing elements have no significant load bearing capacity. From the point of view of cost, and also for environmental considerations foundations should be adapted rather than replaced.

As an example: Piles which are not placed deep enough in the deep sand layer (which can bear the load of the structure, free of settlement) were jacked deeper after the use of the building and after noticing too much deformations. Two piles together are jacked deeper, with the existing building as a counter weight.

The jack holds the weight of the piles under the existing foundation beam. After cutting the iron of the two piles, the new moulded concrete beam and the two concrete piles are jacked deeper.

Aspects that are involved in the adaptation of foundations include:
- the assessment of the existing load bearing capacity of the foundation. Can the loading (solicitation) be increased without the reliability of the foundations being put at risk?
- the assessment of the deformation capacity of the structure above the foundation. To what degree is settlement acceptable?
- how do we know where the base of the foundation is located? Can we determine this without major excavations?
- have specific techniques been developed, or can we develop techniques, for example, that will reduce the effects of settlement near piles or will reduce or stop the general settling of the ground?
- is it possible to correct deformations at a later date?
- if we want to introduce supplementary load bearing capacity how can we calculate what proportion goes to the new elements and what goes to the original foundation elements?
- The measures that are not related to the adaptation, but simply to the maintenance of the structure need to be known as a 'state-of-the-art' and available for use.
A new foundation. Collaboration between existing and new piles.
Left: The newly added piles carry the building
Right: The newly added piles work together with the existing piles under the wall

Structural designer: Leo G.W. Verhoef

All these questions will be considered separately and together during this international congress. The achievement of the objective of preserving our urban heritage is highly dependent on foundations being able to fulfill their function for a very long time. It is better still if they can be adapted to the changes in the building above them to permit the building to enter a new functional phase of life, whether modified or not. As sub-theme for the congress, we have chosen ‘Problems and Possibilities’. What are the problems and how can we tackle them in an economically acceptable manner. This must become clear today and serve as a guideline for the future preservation of buildings.

Delft, 1999-10-06

Leo G.W. Verhoef
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Mission

Founded in 1971 COST, is an intergovernmental framework for European Co-operation in the field of Scientific and Technical Research, allowing the co-ordination of nationally funded research on a European level. COST Actions cover basic and pre-competitive research as well as activities of public utility.

The goal of COST is to ensure that Europe holds a strong position in the field of scientific and technical research for peaceful purposes, by increasing European co-operation and interaction in this field.

COST has clearly shown its strength in non-competitive research, in pre-normative co-operation and in solving environmental and cross-border problems and problems of public utility. It has been successfully used to maximise European synergy and added value in research co-operation and it is a useful tool to further European integration, especially concerning Eastern and Central European countries.

Ease of access for institutions from non-member countries also makes COST a very interesting and successful tool for tackling topics of a truly global nature.

To emphasize that the initiative came from the scientists and technical experts themselves and from those with a direct interest in furthering international collaboration, the founding fathers of COST opted for

a flexible and pragmatic approach. COST activities have in the past paved the way for Community activities and its flexibility allows COST Actions to be used as a testing and exploratory field for emerging topics.

The member countries participate on an “à la carte” principle and activities are launched on a “bottom-up” approach. One of its main features is its built-in flexibility. This concept clearly meets a growing demand and in addition, it complements the Community programmes.
COST has a geographical scope beyond the EU and most of the Central and Eastern European countries are members. COST also welcomes the participation of interested institutions from non-COST-member states without any geographical restriction.

COST has developed into one of the largest frameworks for research co-operation in Europe and is a valuable mechanism co-ordinating national research activities in Europe. Today it has almost 200 Actions and involves nearly 30,000 scientists from 32 European member countries and more than 50 participating institutions from 11 non-member countries.

**COST Status 1999**

In total, institutions from 43 countries participate in COST under different forms:

**32 member states:**
- Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, The Netherlands, Norway, Poland, Portugal, Rumania, Slovakia, Slovenia, Spain, Sweden, Switzerland, Turkey, United Kingdom.

**11 states with participating Institutions:**
- COST has a geographical scope beyond the EU. Institutions from non-COST countries may join COST Actions and at present, there are institutions from the observer states and the following states: Australia (4), Canada (5), Egypt (1), India (1), Israel (11), Japan (3), Kazakhstan (2), New Zealand (1), Russia (22), Ukraine (3), USA (7).
COST actions

COST is based on Actions. These are networks of co-ordinated national research projects in fields which are of interest to a minimum number of participants (at least 5) from different member states. The Actions are defined by a Memorandum of Understanding (MoU) signed by the Governments of the COST states wishing to participate in the Action.

The duration of an Action is generally 5 years. Running Actions, their participating countries (signatories) and their duration are shown in the table.

Number of actions:
The success of COST can be best seen by the ever increasing number of COST actions (see below). Together with the new actions under preparation the number of COST actions will reach a total close to 200 during 1999.

Level of participation:
The participation of the various countries in COST actions is shown below. In general the participation over the member countries appears to be evenly distribute with no dominationg contry. No significant change in this distribution has taken place over the last 5 years.

Scientific domains
COST covers a wide range of scientific and technological domains. The present 17 domains and their part of running actions in 1999 are shown below.

Yearly evolution of the Running and Starting Actions

Funding
COST represents an estimated volume of national funding of more than 1.5 billion Euro per year. This funding is basically used to cover co-ordination expenditures such as contributions to workshops/conferences, travel costs for meetings, contributions to publications and short term scientific missions of researches to visit other laboratories.

An average of 50 000 to 60 000 Euro is available per action depending on size and activity of the action. The EU co-ordination expenditure represents in the average 0.5% of the overall national funding which shows that COST gives excellent value for money.
COST Organisation Structure

Committee of Scientific Officials (CSO)
The CSO is the decision making and highest body in COST. It is composed of COST member states representatives, one of whom in each case acts as COST National Co-ordinator (see below). Technical Committees (TC) are responsible for a particular sector under the authority of the CSO and has mainly to prepare proposals for research projects. A TC is also responsible for technical preparation work and overseeing the proper implementation of the projects as well in an advisory capacity in the co-ordination and evaluation of the ongoing projects. Management Committees (MC) are in charge of the implementation, supervision and co-ordination of a COST Action. A MC is formed by not more than two representatives of each Signatory Country.
COST National Coordinator (CNC)
One member of the CSO from each member state acts as national coordinator (CNC) for COST projects. The CNC provides the liaison between the scientists and institutions in his country and the Council COST secretariat.
The CNC has specifically:
- Ensure that the national funding is available;
- Appoint or officially forward the names of the national delegates to the technical committees (TC) and management committees (MC);
- Forward proposals from other countries to experts in his own country;
- Assess at national level all projects undertaken within the COST framework.

Secretariat
The secretariat is provided by the Secretariat General to the Council of the European Union (secretariat of the CSO) and of the European Commission (scientific and administration matters).

COST Urban Civil Engineering
- Technical Committee on Urban Civil Engineering
- Finished Actions as of August 2000
  C1: “Control of the semi-rigid behaviour of civil engineering structural connections”
  C2: “Large scale infrastructures and quality of urban shape”
  C3: “Diagnosis of Urban Infrastructure”
  C4: “Management and information application development in Urban Civil Engineering”
- Ongoing COST UCE Actions
  C5: “Urban heritage - Building maintenance”
  C6: “Town and infrastructure planning for safety and urban quality for pedestrians”
  C7: “Soil-Structure interaction in Urban Civil Engineering”
  C8: “Best practice in sustainable urban infrastructure”
  C9: “Processes to reach urban quality”
PROBLEMS WITH OLD FOUNDATIONS

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Summary
Many old settlements can be found in lowlands near inland waters where the ground consists of young limnic sediments. These include firm soils like sand and gravel, but also clayey and organic soils. On firm soils above the groundwater table, shallow foundations of stones were built. In soft soils under the groundwater table, different methods of foundation constructions with timber piles and sleepers were used from Roman times until the beginning of the 20th century. Long piles embedded into a firm layer have been used since the 16th or 18th century. In former times, only short piles, which were meant to compact the ground, were used.

The bearing capacity of old foundation constructions was normally used to a higher degree than would be admissible following modern codes for new buildings. Different damage mechanisms can occur causing continuous settling even until today. Damage caused by terminated settlement can also be found. Part of the mechanisms derives from the interaction of the ground and the foundation construction. The biological decay of the timber construction and older wood in the ground by fungi and bacteria is a distinctive and serious cause of damages. The damage mechanism being decisive in assessing the necessity and kind of measures to be taken on the foundation, in every case an attempt should be made to find this mechanism. The significant methods to investigate the ground and the old foundations - if differing from those for new buildings - and some considerations about the diagnosis of the damage are described. The calculation of the rate of settlement resulting only from creeping of the soil is a successful means for the assessment of old foundations, but the diagnosis can still be difficult. The criteria relating to the necessity of measures for strengthening or the replacement of old foundations that have proved worthwhile are described. It may not be necessary to apply measures to the foundation, even if damage due to settlement or continuing settling is obvious.

The modern geotechnical methods of repair, strengthening, and replacement of old foundation constructions are discussed critically. Four categories of methods are available: conventional underpinning, constructions with micro-piles, jet-grouting and ground improvement by injection. All methods have pros and cons; no method is suitable for all cases or always superior.

Introduction
There are three cases, in which we have to deal with foundation problems in existing buildings in old cities:
- the building shows damage due to settlement of the foundation, which should be repaired
- another use of the building is intended, implying higher loads on the foundation and the ground (when doing this, the requirements concerning the serviceability and bearing capacity prescribed by the actual codes have to be met)
- a new building requiring a deep excavation will be erected next to the old building.

In all these cases, the necessity of taking measures in the ground and the foundation of the old building has to be checked. The ground, the existing foundation and the upper construction have to be investigated and judged. The design of the old foundations, and their deficiencies should be known before a decision is taken. There are several different modern techniques of underpinning, deep foundations and soil improvement the most suitable of which has to be chosen for each single case. The peculiarity of
these problems is that the foundation constructions of old buildings (until the beginning of the 20th century) are completely different from the contemporary ones in this age of reinforced concrete, powerful machinery and modern codes.

The earlier foundation constructions consist of different materials, especially of timber and natural stone masonry. Their stiffness and strength are considerably lower, and they make much more use of the strength of the soil than it is allowed in modern foundations. They are subjected to different mechanisms of damage, e.g. biological decay. An important question is whether the old foundation must meet the same standards as new buildings in respect to serviceability and bearing capacity. Additionally the demand of maintaining the original material and significance must be considered. Not every modern method of underpinning technique is feasible for each type of old foundation construction; e.g. timber piles and masonry made of natural stones constitute difficult obstacles for some methods.

When dealing with old foundations many specific problems and difficulties have to be solved, as will be explained in the following. One of the aims of this survey is to contribute to the maintenance of as many parts of the historical foundations as possible.

Locations of old settlements
Old settlements are preferably situated in lowlands of rivers or bigger lakes. Living nearby a water body had many advantages in former times. In mountainous areas, e.g. in Southern-Germany, valley bottoms were preferably chosen for settlements. Still, in such areas, settlements on mountain ridges and high up terraces can be found too. Thus, the ground of many old settlements consists of sediments of rivers and lakes. These include firm soils like cobblestones, gravel and sand, but also many mineral soft soils like limnic clay, valley loam and organogeneous or organic soils like lake marl, gyttja, and peat. The groundwater table follows the level of the inland water. In the Northern German coast region and in the Netherlands, the ground on which of old settlements were built consists of the typical coast deposits like sand, sea clay and peat, very often in layer sequence.

In old settlements, on top of the natural ground we can find layers of civilization rubble, often approximately 3m thick. It contains the rubble and remaining foundations of old buildings and domestic, handicraft and animal wastes. Some of these constituents are organic and can continue to decompose above the groundwater table. In some places, the civilization layer is classified as an historical monument.

The soft sediments along waters that are typical of the ground of old settlements have been the direct or indirect causes of most of the problems with foundations of the buildings up to the present. The ground continues to creep, and the old foundation constructions, which had been designed for the soft ground lose their strength or even decay. Serious problems arise when a new building is being erected neighbouring an old building on soft limnic clay.

On valley slopes and mountain ridges, the ground is normally firm enough to bear the foundations. However, the top layer in which the building is founded can move as a whole mass, due to the geology, e.g. by creeping of a clay slope and by karst development.

Historical foundation constructions

Foundations made of masonry only
On soils above the groundwater table which were considered sufficiently firm, shallow foundations of natural stone masonry were built, until the reinforced concrete became accepted. The lower layers of the foundation masonry were made without mortar. The necessary width of the strip foundation was achieved
by steps with an average angle of 70 - 80° (figure 1a). Owing to the necessary height of this stair-like masonry, the width of these foundations was limited.

The base of the foundation was normally put on natural ground, never on a young fill, but sometimes on old fills or the remains of earlier buildings. These old fills or remains under the foundations, can cause permanent settling continuing until the present, especially when containing organic parts.

In order to widen the base of the foundation, between the strip foundations, reversed vaults were sometimes built against the ground (figure 1b). However, the horizontal anchoring of these walls is often missing so bottom pressure cannot be transmitted. Walls were sometimes built on single deep pillar foundations bridged by arches of masonry figure 1c). But in some cases, the bottom of old foundations is placed only few centimetres below the cellar floor (figure 1d).

Until the beginning of the 20th century, the relation between the bearing capacity of a strip or pad foundation and its width and especially its depth were unknown. The increase of the bearing capacity with depth that has been observed was explained by an increase in the strength of the soil with depth [3]. There were only rules-of-thumb based on experience for choosing the dimensions of a foundation. These rules lead to much smaller strip and pad foundations than required by contemporary calculation models and safety demands. When another use of the building is intended, the required calculation of the bearing capacity of the small foundations causes problems, even if they have been carrying their loads for centuries. A generally accepted safety concept for old foundations is badly needed.

Timber foundations

Since classical Roman times, on soft soils below the groundwater table even heavy stone buildings were founded on timber constructions of piles and sleepers onto which the stone foundations were built. These constructions, which date back to the stone ages, did not develop much until the 16th century. Indeed, they were partially built in the old manner until the beginning of the 20th century when they were finally replaced by reinforced concrete constructions.

In medieval times piles were usually shorter than 2m, with a maximum length of 4m, and only 10 to 30 cm in diameter. These short piles were driven into the ground for short distances and did not normally reach a firm layer. Let us call them “short piles”. Long piles of greater diameter, which were intended to penetrate a firm layer and were driven in a regular arrangement for larger distance were already occasionally used already in classical Roman times. Their use was general practice in the Netherlands from the 16th century and in Germany from the 18th century. Only these “long piles” correspond to our modern geotechnical definition of a pile.

For the timber parts of foundations, all kinds of available wood, with long, straight trunks that were hard enough, were used. For the sleepers, oak and elm were preferred in medieval times, but other deciduous trees that were less resistant to rot were also used, in Germany for example, mainly beach and alder. Alder was common in dike constructions. Because of their straight growth, the widespread coniferous trees such as spruce, fir, pine and larch were preferred for piles; but short piles were made also from deciduous trees. For long piles and sleepers, pinewood, which is very resistant under water, was commonly used in the 19th century. Larch, which would have been even better, was not available in sufficient quantity.

(1) natural stones without mortar
(2) natural stone masonry with mortar
(3) reversed vaults
(4) anchor (often missing)
Four types of timber foundation constructions can be distinguished [2]:

Pure timber raft foundations

The medieval type of timber raft foundation consists of trunks, e.g. diameter 0.3 to 0.4 m and length 8 m. These were put on the ground close to each other in the longitudinal direction of the wall and fixed by short sleepers lying on top or below in the cross direction all 1 or 2 m in length (figure 2a). In the corners of the building, the longitudinal sleepers were put crosswise. The joints were staggered. The space between the sleepers was filled with gravel, building rubble or loamy soil. Upon this fill the strip or pad foundation a foundation only of stones was built, i.e. the lower layers of natural stones without mortar, those above with mortar. The cross section of the strip or pad foundation was like a foundation without sleepers. The timber raft usually was not wider than the strip foundation. In exceptional cases, the timber raft covered the whole area of a building.

The later version of timber raft foundation, which developed from the 16th century on and became standard in the 19th century, was constructed with rectangular beams of 20 to 30 cm height and approximately the same width. The longitudinal sleepers were no longer laid close to each other but at a distance of 0.4 to 0.7 m, e.g. one sleeper under each edge and a third one under the middle of the strip
foundation (figure 2b). The short sleepers in the cross direction had notches in which to fix the longitudinal sleepers from above or below. After the filling of the spaces between the beams with soil, 5 to 8 cm thick boards were nailed in cross direction onto the longitudinal beams upon which the strip foundation was built. The joints of the longitudinal beams were made tension-proof by timber connections and iron straps.

Those timber rafts were mainly used simple buildings until the beginning of the 20th century, with increasingly skilful and accurate craftsmanship. The intention was always to produce a stable base for the foundation masonry, a better distribution of the loads, and to stiffen the construction of the building at the level of the foundation. To increase the bearing capacity sometimes a wooden sheeting wall was installed next to the sleeper frame (figure 2c).

(1) cross sleepers
(2) longitudinal sleepers
(3) natural stone masonry without mortar
(4) natural stone masonry with mortar
(5) thick boards
(6) fill
(7) masonry
(8) wooden sheeting wall

Figure 2: Timber raft foundations
(a) Medieval type
(b) method of the 19th century
(c) with a lateral sheeting wall

Foundations with short piles
Short piles were applied alone or in combination with a timber raft. The classical Roman and the medieval types are made of short piles with a length of 0.5 to 3m (mostly 1.5m) and a diameter of 10 to
20 cm. These driven in close to each other into the soft ground (figure 3). Generally, the points of the piles did not reach a firm layer. Until the beginning of the 20th century when these piles were used, it was always intended to compact the upper ground and to produce a stable base for the masonry. Those piles are called compaction piles (end of the 19th century [3], in German also “Spick”-Pfähle = larding piles [18]). Today, we think that the close arrangement produces a stiffened compound construction of soil and piles bearing like a deformable strip or pad foundation.

The strip or pad foundation was built in the usual manner either right upon the surface strengthened by the short piles (figure 3a) or upon a timber raft of the kind described put upon the piles (Figure 3b). In the second case, the piles were arranged to cover the whole area of the strip or pad foundation. In some cases, they were concentrated in the corner areas of the building and also under the pillars, with the raft bridging the distance between the pile groups. The piles of one group were driven in from inside to outside pushing a lot of soil outside. This method resulted in the densest arrangement of piles and the highest demand for wood (the piles took approximately 50 p.c. of the total area [4]). In the 19th and 20th centuries, the first piles were driven in a loose and regular arrangement of approximately 1m x 1m; thereafter further piles were driven between them [1]. So, the best compaction of the soil was achieved with fewer piles.

A second and refined type of the combined short pile-timber raft foundation has existed since the 13th century and has become the normal version since the 16th century. A timbered frame of sleepers, tension-proof in the cross direction, was fixed upon single short piles, which were slightly longer and thicker than the normal compaction piles. The upper sides of the cross and the longitudinal sleepers were on the same level. The interior of the frame was completely filled with compaction piles (figure 4). The horizontal support of the piles by the timber frame causes an increase in the inner and outer bearing capacity of the block of earth and piles. This construction was also used for heavy foundations with wide bases, e.g. towers and the pillars of bridges. A shortcoming, which was finally realized, was the considerable weakening of the longitudinal sleepers by the deep notches for the flush connections with the cross sleepers.

Combined short pile-timber raft foundations were preferably used for important buildings, timber rafts without piles for rather simple ones.

(1) short piles
(2) longitudinal sleepers
(3) natural stone masonry without mortar
(4) masonry with mortar
Figure 3: Short pile foundations, medieval types
(a) strip foundation built right upon the short piles
(b) with timber raft upon the short piles
(1) short piles
(2) longitudinal beams
(3) cross beams
(4) compaction piles

Figure 4: Short pile-timber raft foundation, later type (since the 16th century)
(a) Cross section
(b) ground-plan
(c) and (d) connections of the beams

Foundations with long piles
For the long piles, which had been made since the 16th century in the Netherlands and since the 18th century in Germany, whole trunks of trees of 20 to 40 cm diameters had been driven in. The length depended on the stratification of the ground and was restricted by the availability and the handling of the trunks and the free height of the pile driver. Allegedly, the maximum length of one trunk driven in Amsterdam in the 16th century was 20m, requiring an enormous pile-driving plant. Generally, trunks 5 to 7m long and not exceeding 12 m were used. To make longer piles two or more shorter trunks were connected one on top of the other during driving in. By this method, piles up to a length of 22m were installed in Potsdam in the 18th century.

There were two types of pile drivers: The pull driver and the winch driver [1], [3]. Both worked with a dropping hammer, the so-called pile hammer, which was drawn up on scaffolding with a rope or a chain and then dropped onto the pile head. In case of the pull driver, 10 to 50 workers pulled rhythmically on the rope, the pile hammer weighed 150 to 600 kg and the dropping height was 1.2m. Two to three hundred blows per hour could be achieved. In the case of the winch driver, the pile hammer was moved with a winch driven by workers and later by steam-power. The pile hammer weighed 600 to 800 kg and the height of drop was 2 to 8m. Only 20 to 30 blows per hour were achieved when this was driven by
workers. Winch drivers were produced industrially in the 19th century, e.g. by Menck & Hambrock [3]. From the penetration length per number of blows, it was concluded whether the pile had reached a sufficient depth or bearing capacity.

When driving in these long and thick piles, a certain distance had to be maintained between the piles to avoid hindrance due to the displacement and compaction of the soil. Normally a distance of 1.0 to 1.25 m was realized [1]. The group effect produced by dense arrangement of the piles, which was the characteristic feature of the medieval short pile foundations, was avoided in the long-pile foundations to reduce the driving resistance and to increase the bearing capacity of the single pile. This is the essential distinction between the medieval short pile and the modern long-pile foundations.

After the pile had been driven to the required depth, its head was cut with a special saw under the water table and a tenon was worked out for the connection with the pile head beam. Onto the pile heads prepared in this manner a timber raft constructed of longitudinal and cross beams was placed (figure 5). The spaces between the longitudinal beams were filled up and a covering of thick boards was nailed in cross direction, above which the strip foundation was made in the usual way. In some regions, only thick boards were placed crosswise in several layers onto the pile heads.

In the ideal case the long piles were driven through the soft layers deeply enough into a sufficiently firm layer. For example in Amsterdam piles were driven into the so-called first sand layer to a depth of 10 to 12 m in the 17th century. Such piles are called "standing piles". The settlement of standing piles ceases and it can be assumed that the piles take the total load from the strip foundation and transmit it into the firm layer. However, in many cases the pile length was not sufficient to reach a firm layer, which could be attributed to the expenses or limited feasibility. Then, the piles ended in or above a compressible layer, yielding the so-called "floating" pile foundations. The floating piles continued to settle and can go on settling due to creeping of the soil, even until today. Floating piles were frequently not capable of permanently bearing the total load of the foundation. Because of the con-

Figure 5: Long-pile foundation, 19th century
continuing settlement, a part of the load is no longer carried by the piles but is transmitted directly from the strip foundation into the soil between the pile heads. The floating pile foundations act as combined pile-shallow foundations which are comparable with the modern so-called combined pile-raft foundations.

If a pile was intended to behave as a standing pile, it was installed with the thicker end downward in order to reduce the driving resistance due the skin friction in the soft layers. On the other hand, floating piles were driven in with the thinner end downward to mobilize the highest possible skin resistance. Most probably, the latter case occurred more frequently.

Several formulas were developed in the 19th century to estimate the bearing capacity of long piles. The formulas take into consideration the penetration resistance during the driving and the weight and falling height of the pile hammer on the basis of experience [3]. Several times, these formulas overestimated the bearing capacity [3]. According to a suggestion of the famous French bridge builder Perronet (18th century), the admissible load of timber piles was assumed to be as follows:

<table>
<thead>
<tr>
<th>Diameter (cm)</th>
<th>Admissible load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 - 24</td>
<td>250</td>
</tr>
<tr>
<td>31</td>
<td>500</td>
</tr>
</tbody>
</table>

This assumption is also rather optimistic and at best is valid for perfectly standing piles. In recent years loading tests on piles from the 19th century at the castle of Schwerin and at the Reichstag in Berlin showed limit loads of only 100 to 200 kN. These observations confirm that many old long-pile foundations behave as combined pile-shallow foundations.

**Damage mechanisms of old foundations**

In old buildings damage may already exist or still occur due to settlement that may be caused by many different mechanisms. In order to decide whether the damage requires measures in the ground, on the foundation or only on the upper construction, the causes of the settlement should be known. The measure to be taken also depends on the cause of the settlement. In the following, the most important mechanisms are described.

Mech. 1 - Base failure of a shallow foundation or sinking of a pile foundation because of overload: Many old foundations are heavily loaded in respect to their bearing capacity. However, base failure rarely occurs with loads and the geometry remaining unchanged, because the strength of the soil is increased by consolidation. In some buildings base failure had already occurred during construction and ceased due to stabilizing effects. Base failure may also have taken place because of later overloading after conversion or excessive storage loads. The risks are the same with alterations in the use of a building and in the building being performed today. However, “knock out” calculation of building by making assumptions that are too much on the safe side should be avoided.

Mech. 2 - Primary consolidation: This is the gradual consolidation and settlement of the soil under squeezing out of pore water after an increase of load. Because of the lamination of the layers, the primary consolidation of the silty, clayey and organic limnic sediments goes on much more rapidly than in case of an ideal homogeneous layer (e.g. one year instead of 50 years). Therefore, the primary consolidation settlement of old buildings has been terminated for a long time, in general. However, an old building may still show damage caused by primary consolidation that no longer increases today. Considerable differential settlement may arise from the primary settlement these depending not only on the soil properties and the thickness of layers but also on the load of the building. Depending on the viscos-
ity of the soil and the age of the building, additional loads due to building alterations exceeding a certain percentage of the former load (e.g. more than 5 to 10 p.c.) will induce additional primary settlement.

Mech. 3 - Secondary consolidation and settlement, creeping of the soil: The secondary consolidation and the corresponding settlement are caused by the viscosity of the soil. They continue after the end of primary consolidation for infinite time, but the rate decreases with \(1/t\), \(\sim 1/t\), wherein \(t\) is the time of the loading which caused the preceding primary consolidation. The amount of secondary settlement increases linearly with \(\log t\). All soils behave more or less viscously, but only clayey and organic soils show considerable secondary settlement. Because the rate of secondary settlement is independent of the stress, provided that the stress is constant in time, this settlement of a building is usually distributed more uniformly than the primary ones. All continuing settlement of old buildings on clayey or organic soils is caused at least partially by secondary consolidation. If the current settlement of a building is all secondary, this demonstrates that the foundation is in a safe equilibrium state. The strength of soil increases during secondary consolidation. By decreasing the load, the secondary settlement can be stopped instantly and delayed for a long time [11]. On the basis of compression tests, the rate of secondary consolidation can be calculated to compare these results with the measured rate of settlement. However, it must considered that the real rate of secondary settlement of a thick layer may exceed the calculated value by a factor of 2 to 4 owing to several effects.

For new buildings in general, it is mainly the primary settlement that is estimated and taken into consideration.

Mech. 4 - Biological decay of the wood of the foundation construction: Two processes have to be distinguished: decay caused by fungi and caused by bacteria.

**Decay by fungi**

This process develops only under aerobic conditions and therefore only above the groundwater table. Even a thin airtight soil layer may suffice to delay the growth of fungi. The entire organic substance of the wood is decomposed with time, i.e. only a negligible part of the material remains. Depending on the air supply, the decay may proceed quickly during a few years or slowly during many years. All woods become infested, but the loss of strength proceeds differently because of the differing cell structures. For this reason timber foundations were generally placed below the lowest groundwater level, which may of course have fallen later. Sometimes, a shallow foundation was built on ground, which contains the wooden remains of a preceding building over the groundwater table [5]. Growth of fungi generally results in total loss of the timber construction.

**Bacterial decay**

These processes develop under anaerobic conditions and therefore even below the groundwater table. The decay by bacteria goes on much more slowly than by fungi. Therefore, timber piles below the groundwater table remain solid for centuries. Conifers, especially pine and larch are more resistant than the wood of deciduous trees because only the secondary cell walls of conifers containing much cellulose are attacked whereas the primary cell walls and medium lamellas containing much lignin withstand. Therefore, the volume of the wood remains nearly constant, and the strength in the longitudinal direction decreases only moderately [16]. The damage of deciduous woods by bacteria, especially of beach and alder, is more serious because of the differing structure of the cells. The rate of the decay depends on the supply of nutrients.

The gradual loss of the strength of the timber in the foundation due to bacteria causes slow settlement like the creep settlement of the soil after a long time. From our experience, this rate of settlement of
standing piles made of pine amounts to 0.1 to 0.3 mm/a [5]. The effect of loss of strength is most serious on beams heavily loaded in cross direction, e.g. on the sleepers at the supporting surface on the pile. These spots may be crushed gradually.

The decomposition of the wood by fungi and bacteria can be seen in samples under a microscope.

The extent of the settlement due to biological decay of the timber foundation depends also on the details of construction of the foundation. The most extensive effect comes from rotting sleepers or boards covering the whole bottom surface of the foundation. If the strip foundation is standing with only of its base surface upon timber, and with the other part directly on the ground. For example, in the case of single piles or a timber raft with long distances between the sleepers, the soil is capable of bridging the weakened wooden parts as long as the wood is capable of sustaining the lateral earth pressure. The behaviour of the foundation changes from a combined pile-shallow foundation to a pure shallow foundation.

Mech. 5 - Biological decay of organic material in the ground: Just like the timber construction other organic material in the ground may also be decomposed by fungi when it is above the groundwater table and by bacteria when it is below the groundwater table. This material may be the remains of a former settlement being part of the civilization layer or a natural organic layer. The process takes place especially when the groundwater level has fallen.

Mech. 6 - Loss of cohesion or settling of sensitive or cemented soils due to saturation: This failure mechanism occurs owing to a rise of the groundwater table or a discharge of rain water. For example, the cementation of loess may be dissolved by rainwater from a gutter. Considerable local settlement may result.

Mech. 7 - Shrinking by drying of a clayey ground below the foundation caused by the suction of the roots of trees: Buildings founded insufficiently deep may settle due to this influence.

Mech. 8 - Undermining by rats under shallow foundations: The bottom surface of shallow foundations of old buildings is sometimes placed less than 0.1m below the floor of the cellar (figure 1d). If the soil does not contain gravel and the floor is not covered by a pavement, it may happen that rats burrow under the foundation causing considerable settlement.

Mech. 9 - Dissolving of gypsum from a deep layer as a result of karst development: The results are continuous slow settlement and displacements of the upper layer in which the buildings are founded.

Mech. 10 - Creeping of a clay slope: The result of these continuous movements in an inclined direction are settlement, stretching and shearing causing damage to buildings up to uselessness or collapse.

Mech. 11 - Vibrations due to traffic or heavy building machinery: In grounds consisting of loose sand or soft limnic sediments below the groundwater table, vibrations may effect compaction or settlement by temporary and incremental liquefaction and consolidation, respectively.

Mech. 12 - Execution of a neighbouring deep excavation for a new building: There are many mechanisms and influences of a deep excavation, which may cause settlement and displacements on neighbouring buildings: Imperfections and unavoidable displacements during production of drilled piles or another type of retaining wall, decrease of the overburden pressure by excavation, insufficient support of the retaining wall and the bottom of the excavation, and ground-water lowering. Displacements and settlement due to an excavation are partly unavoidable in some cases, which may raise problems of liability.
Geotechnical investigations and diagnosis of the cause of damage

The geotechnical investigations serve to achieve the data for the diagnosis of the causes of the damage and to prevent further damage. Because the execution of the whole series of possible investigations would be an exaggerated response and not necessary in all cases, a hypothesis concerning the causes of damage should be established before starting. Previous knowledge concerning the geology, the history of the building and the description of the damage and its evolution should be considered for this hypothesis.

In many cases, the following investigations are necessary and sufficient:

**Soil investigation to determine the stratification, the types of soils and the groundwater level**

This can be done in the usual manner used for a new building. It is very important to detect creeping and decomposing natural and manmade organic layers in the ground and to take samples.

If the damage includes continuous settlement, the secondary consolidation behaviour of the creeping and compressible soils has to be investigated by special one-dimensional compression tests. For new building projects, it should be sufficient to observe the primary consolidation and the first decade of secondary consolidation only (duration of load stage 24 hours in the incrementally loading test). For the assessment of old settling buildings, at least one load stage should be maintained for several days.

If the outer bearing capacity of a shallow foundation is a problem, careful direct or triaxial shear tests on the upper soils should be done. A bearing capacity calculation on the basis of the usual conservative estimates of shear strength parameters often leads to the inapplicable result that the foundation should have collapsed long ago or is close to its limiting state.

**Measurements of settlement**

They are the most important and most reliable basis for the proof of the existence and extent of continuous settlement and the explanation of its causes. Repeated measurements of the heights at defined times by the geodetic method are suitable and should be recommended. They should be done once per year, preferably in late autumn. The settlement should be recorded in 0.1mm steps. An observation time of at least 3 to 5 years is normally necessary because the unavoidable deviation is in the same scale as the settlement per year. On the basis of such series of measurements, at best a mean value of the rate of settlement can indeed be determined, but not its dependence on time.

Additionally, the measurement of the height of a long and originally straight masonry layer or ledge is useful and recommendable. By this method, the amount and distribution of the relative settlement can be determined. Plaster marks or electronic measurements of the width of cracks also indicate movements but their results are unsuitable for the comparison with settlement calculations.

**Investigation of the type of construction, size and condition of the foundation**

The type of construction of an old foundation is often unknown because of lack of original documents. In this case, the determination of the type and size of the foundation is difficult. The most comprehensive information is obtained by test pits. In the test pits, the condition of a timber foundation can be investigated and samples can be taken from them using a simple wood drill. These samples should be examined with a microscope to determine the extent of decomposition; but test pits are expensive and could cause damage. Diagonal core borings with small diameter through the strip foundation up to the ground are cheaper, save the substance and are sufficient in many cases. With those borings, even a timber foundation can be detected sometimes.
How to detect a timber foundation with physical, not invasive methods is an unsolved problem, until now.

If the firm head of a timber pile can be uncovered and is accessible vertically or in an inclined direction from above, the pile length can be determined saving the substance with a hammer blow method [12]. The method may be recommended as basis to estimate the bearing capacity of a long pile foundation.

Sometimes it is difficult in spite of the above mentioned investigations to determine clearly the causes of settlement. The following very simple method of diagnosis has proven to be reliable in many cases [5]: Calculate on the basis of one-dimensional compression tests or the given material properties of the soils, the thickness of the layers and the age of the building, the rates of secondary settlement \( s_{vc} \), taking into consideration the above mentioned possible deviation of this calculation (for sand and gravel \( s_{vc} = 0 \) can be assumed). Compare the results of this calculation with the measured rates of settlement of the building. If the result is clearly \( s > s_{vc} > s_{vc} \) there are additional causes of settlement besides secondary consolidation. If the bottom of strip foundation is above the lowest ground water table, biological decay of a timber foundation or other organic material has to be taken into consideration. By this method, timber and other organic remains of previous buildings can be detected under foundations and proven subsequently by borings. The described method of diagnosis is reliable, provided that vibrations have no influence and changes of geometry due to the settlements do not increase the load of the foundations (counter-example: leaning towers).

The proof that in a special case vibrations by traffic caused the settlement - or the opposite proof - is difficult and mentioned here as a problem that has not been solved in a satisfactory way.

**In which cases does strengthening or replacement of the old foundation have to be recommended?**

After the diagnosis of the causes of settlement, it has to be decided whether the existing foundation will continue to be sufficient or should be repaired, strengthened or replaced by a new construction. General rules for this decision cannot be given. The question must be considered and decided in each individual case. Both, the geotechnical and the construction engineer should elaborate the decision in teamwork. The following criteria have been proven valid in many cases:

Case 1 - The load of the foundation will increase and exceed the admissible value because of a projected building alteration: In this case, the foundation has to be strengthened or replaced to ensure sufficient bearing capacity. The modified foundation has to be calculated as if for a new building because its response to the increased loads cannot be predicted from the behaviour of the old foundation subjected to the former loads. In some cases the incorporation of new separate pillars or walls with separate new foundations next to the old ones has to be recommended in order to maintain the old construction.

Case 2 - There is damage in the building caused by terminated primary settlement: In this case, strengthening or replacement of the old foundation is not required and should not be recommended. It is sufficient to repair the upper construction.

Case 3 - Continuing settlement is being completely evident by secondary consolidation or general creeping of the soil: In this case, if possible the foundation should be maintained change because of the following three facts being true. The bearing capacity is sufficient, the future development can be predicted and the rate of settlement will decrease in the long run. The construction engineer should indeed estimate whether the actual rate of settlement after the repair of the building will be tolerable with
respect to the serviceability for an adequate time (tolerable rate of settlement e.g. 0.2 to 2 mm/a depending on the construction of the building). Consequently, the last decision concerning the necessity for measures on the foundation has to be taken by the construction engineer in agreement with the owner.

Case 4 - There is continuing settlement, at least a part of which is caused by biological decay of a timber foundation above the groundwater table: In this case the foundation has to be repaired or replaced by a new construction. This is because in most cases, the additional settlement, which has to be expected, would be incompatible for the building and the rate of settlement cannot be predicted and may even increase arbitrarily. This recommendation is also valid when the settlement is caused by biological decay of wood or other organic remains of previous buildings above the groundwater table.

Case 5 - Continuing slow settlement is being caused by gradual loss of strength of a timber foundation below the groundwater table due to bacteria (and only additionally by secondary consolidation): Although the rate of settlement will not decrease with time, conclusions analogous to case 3 should be drawn.

Case 6 - One of the mechanisms, 6 or 7, is acting: An attempt should be made to remove the causes of the settlement. Only if this is not possible (e.g. large trees under protection of the countryside) the foundation should be made deeper.

Case 7 - Undermining by rats: In this case, the bottom of the shallow foundation has to be put deeper or the soil has to be hardened by cementation.

Case 8 - One of the geological mechanisms, 9 or 10, is acting: Measures on the foundation only in most of the cases are, not sufficient or not suitable to improve the situation. We have to restrict ourselves to repair and stiffen the upper construction of the building. In the case of mechanism 10, we can attempt to stabilize the creeping slope, if not too expensive, by geotechnical measures as drainage, anchors or pile dowels. The mechanism 9 usually cannot be eliminated, so the building has to continue to be subjected to this influence [17].

Case 9 - Continuing settlement caused by vibrations is evident: If the vibrations originate in neighbouring construction works, an attempt should be made to reduce them. This may be possible by taking certain additional measures like damping layers or more cautious operation of the machinery. Vibrations due to traffic usually cannot be eliminated by practicable measures. We have to restrict the measures to the affected building. The necessity for such measures should be decided analogously to case 3.

Case 10 - The old building is being affected by a deep excavation for a new building: The range of an excavation in soft ground amounts up to 5 times the depth. Following the principle of liability, the measures to protect the old building should be taken mainly on the support of the excavation, whereas the measures on the old building should be restricted to the careful observation of the deformations by measurements.
Repair, strengthening and replacement of old foundations

If an old foundation affected by one of the described mechanisms has to be strengthened or replaced, the measure used has to be the vertical bridging of a rotten construction or a soft or loose soil layer by a solid construction or strengthened soil up to a sufficiently firm soil layer. In the modern techniques of deep foundations and underpinning, there are several methods to achieve that. All methods have pros and cons and are not applicable in all cases. No method is the best in all cases. In spite of the variety of the methods, it may be difficult to find the suitable method for a particular case. This suggests the need for compromises. The methods may be classified into the following four groups:

**Group 1 - Conventional underpinning**

The old foundation is undermined in separate small sections and the bad material is removed. In the shafts, which have to be supported during construction, a new foundation up to the firm ground or the stable part of the old timber piles is made (figure 6). The method is often applied when an excavation for a new building has to be executed next to an existing building, but it shows several shortcomings in repairing old foundations:

* The old wall, which has to be underpinned, is often not capable of bridging the open shafts owing to existing cracks. Anyway, the method is not applicable for pillars.
* The natural stone masonry of the old foundation could disintegrate above the shafts and therefore it has to be broken down too.
* Below the groundwater level, the method is only applicable with difficulties.
* Settlement during construction is inevitable even under the best conditions.

A common method in the Netherlands is to saw out the rotten wooden pile heads and to replace them by new short pile pieces of steel tube and concrete.

![Scheme for the repair of an old foundation by means of conventional underpinning](image)

**Group 2 - Construction with drilled micro-piles**

The piles have a diameter of 15 to 30 cm and can be produced even from low and narrow rooms. They have either a central carrying element of thick steel (30 - 63 mm), a thick-walled steel tube, or they are reinforced with thin concrete steels. The admissible load amounts up to 900 kN. The piles must be embedded several meters deep in a firm layer. In Germany, micro-piles are regulated by DIN 4128.

One of the problems with micro-piles is the connection of the piles with the wall or the pillar respec-
tively. Two methods of construction are available:
* The piles are drilled in an inclined direction through the wall and the strip or pad foundation (the so-called root piles [13], Figure 7). The load must be transmitted from the wall into the pile by friction and adhesion. Therefore, the wall must generally be reinforced by steel needles or anchors in cross direction. An additional and critical shortcoming is the necessity for inclined drilling through the natural stone masonry and the timber foundation, a very difficult or even impracticable work.
* The piles are drilled vertically next to the old foundation and linked on the foot of the wall or pillar by bordering beams made of reinforced concrete (figure 8). The bordering beams are connected with the masonry by indentation and anchors (figure 8a) or by cross beams (figure 8b). This connection is only practicable indeed with a brick wall, whereas it could be problematic with a natural stone wall. The connection can also be achieved by means of a continuous plate of framework or reinforced concrete built in throughout in the level of the cellar floor (figure 8c).

![Figure 7: Underpinning with inclined "root-piles"](image_url)

(a) micro-piles
(b) thickening on the pile head
(c) reinforcement with steel needles
(h) available high for the pile drilling machine

(a) different situations

(1) natural stone masonry
(2) brickwork
(3) micro-pile
(4) bordering beam  
(5) anchors  
(6) cross beams  
(7) plate or frame of reinforced concrete  
(8) console  
(9) cross beam  

Figure 8: Constructions to connect underpinning micro-piles with the building  
(a) with bordering beams and anchors  
(b) with bordering and cross beams  
(c) with a continuous plate or frame of reinforced concrete  

Micro-piles, which have to bridge a rotten timber foundation or other organic material, must be designed to carry the total load on the foundation because the decomposition will continue. If the piles have to restrict secondary settlement, it is sufficient to design them for only part of the total load, e.g. 30 to 50 p.c., because the settlement is interrupted for a long time by a stress decrease [11]. If the settlement has to be terminated immediately, the piles should be preloaded against the bordering beams using a hydraulic press.

Micro-piles also can be used to relieve old overloaded timber piles. Here the problem arises that the micro-piles become stiff after a settlement of 5 to 10 mm when subjected to their design load, whereas the old timber piles will continue to settle slowly, because of loss of strength due to by bacteria and other influences. The differential settlement arising in this way between the old and the new piles has to be considered in the design of the new piles and in the reconstruction of the upper building. For example, the embedding section of the piles in the ground can be designed to behave as softly as possible by using a shorter embedding length or a thickened foot.

**Group 3 - Underpinning or ground improvement by jet-grouting**

With the jet-grouting method [14], the ground is disrupted in a cylindrical volume and mixed with cement grout by high speed jets of air, water and grout (trifluid system) or only grout (monofluid system) coming out of radial holes in the injection chamber on the point of the drill rod. Thus, cylindrical bodies of cemented soil are produced in the ground with a diameter of usually 0.6 to 1.5 m and a guaranteed one-dimensional compressive strength of 3 to 10 N/mm². By combining such bodies, an underpinning construction can be formed (figure 9). The method is suitable for all mineral soils since it is not aggressive against cement and in exceptional cases, for organic soils too.

(1) pre-excavation  
(2) jet-grouting wall  
(3) anchoring  
(4) impervious layer made by jet-grouting  
(5) final bottom of excavation  
(6) protruding part  
(7) pipe  
(8) creeping soft layer  
(9) jet-grouting  
(10) firm layer
Figure 9: Application of the jet-grouting method
(a) underpinning wall due to a deep excavation
(b) underpinning of a foundation to bridge a compressible layer below the bottom
(c) bridging of a creeping layer between firm layers

This method was often applied for underpinning of foundations next to a deep excavation (figure 9a [8]) and for replacement of overloaded old foundations (figure 9b). The creep settlement of a deep soft layer embedded between two stiff layers can be stopped by bridging the soft layer with separate jet-grouting columns, without the columns directly supporting the strip foundation (figure 9c). While the rate of settlement decreases gradually, the forces of the columns increase until approximately 30 p.c. of the total overburden load is transmitted by the columns, depending on the viscosity of the soft layer [10, eq. 7] and the stiffness of the columns.

The execution of the jet-grouting method presents several problems, some of them are considered in the admittance paper or described in other publications [9]. In the application to old foundations, additional difficulties and restrictions have to be considered, two of which are mentioned here:

* The method is not suitable between still solid timber piles and sleepers because the jet cannot cut up solid wood. The solid timber parts produce “shadows” into the jet so that the ground behind them cannot be reached by the grout. Only thoroughly rotten wood is cut up to small pieces, which are carried out by the surplus suspension.

* The cylindrical hollow space in the ground eroded by the jet of fluid is permanently supported by the hydrostatic pressure of the heavy suspension of water, soil and cement filling the space and the bore hole up to the surface. This hydrostatic pressure is sufficient to balance the active earth pressure of the ground under plane strain conditions, subjected to a surface load not exceeding a certain value. Therefore, the method can be executed with very small deformations only, provided that the loads are not too high. However, below heavy buildings and foundations subjected to high loads, the hydrostatic pressure of the mud alone is not sufficient to balance the plane strain active earth pressure. Therefore, horizontal supporting vaults must be developed in the ground around the cylindrical space. The development of those vaults requires movements of the soil to the mud filled area, causing settlement of the building. So, in this case, the method cannot be executed without settlement. To minimize the settlement, the diameter of the jet-grout bodies must be limited and a long enough distance between newly made bodies must be chosen.

If the conditions for the application of the jet-grouting are favourable and the measurement is carefully prepared and executed, it is very suitable to deepen foundations. Compared to micro-piles, it is an
advantage that a special construction for the connection with the wall is not necessary. Jet-grouting is superior to micro-piles mainly for very heavy buildings with a bottom pressure higher than approximately 500 kN/m², e.g. towers, because the necessary number of piles could not be placed within the bottom surface.

**Group 4 - Ground improvement by injection of a fluid or suspension**

Usually a cemented grout is used as suspension. The suspension is injected at low pressure into the pore space or cracks without changing the structure and porosity of the soil. Only the cracks can be generated and widened by the injection.

Three different effects may be intended:
* Strengthening by cementation
* compaction by sideways displacement
* rising by generation and widening of horizontal cracks [15] (so-called soil fracturing)

In some cases the ground below a building can be improved in such a way that settlement is stopped or avoided. For example, the cementation of loose sand to avoid settlement due to saturation with water (Mech. 6), vibrations or increasing of the loads. However, because of several limitations of the application, the method is not suitable for most of the damage mechanisms mentioned above:

* Cement grout penetrates only the pores of coarse soils like sand and gravel; i.e. only these soils can be strengthened by cementation, but they are rarely the cause of settlement. Silt, clay and organic soils that are often the cause of settlement, cannot be cemented.
* Compaction by sideways displacement can only be executed without risk in certain unsaturated soils. In soft and saturated silty and clayey soils raising, together with an increase of pore water pressure, occurs first. This is followed by primary consolidation with settlement (because of the same effect, it has to be advised against pressing or driving piles into such soils without a borehole). The volume increase by the injected material could be not sufficient to compensate the volume decrease by the induced primary consolidation.
* The process of continuing settling cannot be stopped by raising the ground through injecting grout into horizontal cracks, since the cause of the settlement is not removed (except for inclined towers). In addition, in general, it is better to repair the upper construction in the deformed state than to reverse the settlement before the repair.
* For strengthening or the replacement of parts of a rotten timber foundation or other rotten organic substance, injection of cement grout is normally not suitable because the grout does not penetrate this material. Only if the soil between the wooden parts is coarse enough (e.g. sand or gravel), if this soil is ble to bridge the wooden parts, and if the lower parts of the piles are no longer needed to carry the loads, can the foundation be improved by this method.

The improvement of saturated soft clay or silt by injection is still an unsolved geotechnical problem.

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THE ASSESSMENT AND BUILDING RESPONSE TO TUNNELING, RISK OF DAMAGE AND STRUCTURAL MONITORING

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Introduction
In recent years, protecting and maintaining our urban heritage against the impact of major construction projects has become an essential part of proposals for new developments. The construction of tunnels in urban areas, for example, has long-term environmental benefits along with some significant environmental drawbacks. Many of these are temporary (e.g. construction traffic, noise, vibration and dust) and result from the necessary construction activities. Longer-term impacts would include land and building acquisition, traffic and ventilation noise and vibration levels, and other impacts such as pollution, ground water changes and effects on ecology.

This paper is concerned with the ground movements that will result from tunnelling and deep excavations and how they affect overlying structures and services. Rational procedures for assessing the risks of damage are necessary, both for engineering design and for planning and consultation. Coupled with such assessments is the requirement for effective protective measures which can be deployed when predicted levels of damage are judged to be unacceptable.

The approach adopted for assessing the risks of subsidence damage for the London Underground Jubilee Line Extension (JLE), which involved tunnelling under densely developed areas of central London, is summarised. The approach draws on the results of a number of studies including the prediction of ground movements, the description and classification of damage and limiting distortions of brickwork and masonry walls. A new methodology developed at Imperial College is also described where the influence of the building stiffness is taken into account.

There are still many areas of uncertainty in the appraisal of structural response to tunnelling, concerning (i) subsidence trough; (ii) soil-structure interaction; (iii) protective measures; (iv) remedial measures; (v) damage and (vi) time effects. A collaborative government-industry sponsored research project (known as the LINK project) has been undertaken to study these areas of uncertainty. About thirty study buildings were selected along the route of the JLE that represent different structural forms and foundation types and that were influenced by different tunnelling and excavation methods in London Clay or the more granular deposits of the Lambeth Group (formerly known as the Woolwich and Reading Beds) and Thanet Sand.

Brief details of the project are given and the monitoring that was carried out during the study is discussed. In order to investigate the responses of the ground and buildings, simple and reliable real-time methods of monitoring movement are required. These are also vital to the proper control of construction works and protective measures.

Some of the monitoring results from two of the case study buildings are presented to illustrate the prediction techniques described and the observed building behaviour.
Ground movement due to tunneling and excavation

The construction of tunnels or surface excavations will inevitably be accompanied by movement of the ground around them. At the ground surface these movements manifest in what is called a ‘settlement trough’. Figure 1 shows diagrammatically the surface settlement trough above an advancing tunnel. For ‘greenfield sites’, the shape of this trough transverse to the axis of the tunnel approximates closely to a normal Gaussian distribution curve - an idealisation which has considerable mathematical advantages.

![Diagram of surface settlement trough above an advancing tunnel](image)

Figure 1: Surface settlement trough above an advancing tunnel.

Settlement due to tunnelling

Figure 2a shows such an idealised transverse settlement trough. Attewell et al. (1986) and Rankin (1988) have summarised the current widely used empirical approach to the prediction of immediate surface and near surface ground displacements. The settlement $S$ is given by:

$$S = S_{\text{max}} \exp\left(\frac{-x^2}{2i^2}\right)$$

expression (O'Reilly and New, 1982):

$$i = K z_o$$

where $S_{\text{max}}$ is the maximum settlement and $i$ is the value of $x$ at the point of inflection. It has been found that, for most purposes, $i$ can be related to the depth of the tunnel axis $z_o$ by the linear

$K$ depends on the soil type. It varies from 0.2 to 0.3 for

although the value of $K$ for surface settlements is approximately constant for various depths of tunnel in the same ground, Mair et al. (1993) have shown that its value increases with depth for subsurface settlements.
Figure 2: (a) Ground movements from a normal Gaussian distribution and (b) point sink assumption.

The immediate settlements caused by tunnelling are usually characterised by the 'volume loss' \( V_L \) which is the volume of the surface settlement trough per unit length \( V_s \), expressed as a percentage of the notional excavated volume of the tunnel.

Integration of Equation (1) gives:

\[
V_s = \sqrt{2\pi} \ i \ S_{\text{max}}
\]

so that:

\[
V_L = \frac{3.192 i \ S_{\text{max}}}{D^2}
\]

Where \( D \) is the diameter of the tunnel. Combining Equations (1), (2) and (4) gives the surface settlement \( S \) at any distance \( x \) from the centre-line:

\[
S = \left( \frac{0.313 V_L D^2}{K z_o} \right) \exp \left( -\frac{x^2}{2 K^2 z_o^2} \right)
\]
A cumulative probability function is used to generate a profile of longitudinal settlement ahead of and behind the advancing tunnel heading (Attewell and Woodman, 1982). Combining the longitudinal and transverse equations can lead to predictions of the complete picture of surface settlement above a tunnel excavation. The new design method only considers the transverse settlement profile, orthogonal to the direction of tunnel drive.

**Horizontal displacements due to tunnelling**

Building damage can also result from horizontal tensile strain, and therefore predictions of horizontal movement are required. Unlike settlements, there are few case histories where horizontal movements have been measured. The data that do exist show reasonable agreement with the assumption of O'Reilly and New (1982) that the resultant vectors of ground movement are directed towards the tunnel axis (see Figure 2b). It follows that the horizontal displacement \( u \) can be related to the settlement \( S \) by the expression:

\[
\frac{u}{z_0} = \frac{S}{x}
\]

Equation (6) is easily differentiated to give the horizontal strain \( \varepsilon_h \) at any location on the ground surface.

Figure 2a also shows the relation between the settlement trough, the horizontal displacements and the horizontal strains occurring at ground level. In the region \( i > x > -i \), the horizontal strains are compressive. At the points of inflection the horizontal displacements are a maximum and \( \varepsilon_h = 0 \). For \( i < x < -i \), the horizontal strains are tensile.

**Assessment of surface displacements due to tunnelling**

The above empirical equations provide a simple means for estimating the near surface displacements due to tunnelling, assuming 'green field' conditions i.e. ignoring the presence of any building or structure.

A key parameter in this assessment is the volume loss \( V_L \). This results from a variety of effects which include movement of ground into the face of the tunnel and radial movement towards the tunnel axis due to reductions in supporting pressures. The magnitude of \( V_L \) is critically dependent on the type of ground, the ground water conditions, the tunnelling method, the length of time in providing positive support and the quality of supervision and control. The selection of an appropriate value of \( V_L \) for design requires experience and is greatly aided by well documented case histories in similar conditions.

A number of other assumptions are involved in the prediction of ground displacements due to tunnelling. For example, in ground containing layers of clay and granular soils there is uncertainty about the value of the trough width parameter \( K \). When two or more tunnels are to be constructed in close proximity, the assumption is usually made that the estimated ground movements for each tunnel acting independently can be superimposed. In some circumstances this assumption may be unconservative and allowance needs to be made for this.

It is clear from the above that, even for 'green field' conditions, precise prediction of ground movements due to tunnelling is not realistic. However, it is possible to make reasonable estimates of the likely range of movements provided tunnelling is carried out under the control of suitably qualified and experienced engineers.
Ground movements due to deep excavations

Ground movements around deep excavations are critically dependent both on the ground conditions (e.g. stratigraphy, groundwater conditions, deformation and strength properties) and the method of construction (e.g. sequence of excavation, sequence of propping, rigidity of retaining wall and supports). In general open excavations and those supported by cantilever retaining walls give rise to larger ground movements than strutted excavations and those constructed by ‘top-down’ methods. In the urban situation the latter are clearly to be preferred if building damage is to be minimised.

The calculation of ground movements is not straightforward because of the complexity of the problem, and much experience is required to make any sensible use of complex analyses. It is therefore essential that optimum use is made of previous experience and case histories in similar conditions. Peck (1969) presented a comprehensive survey of vertical movements around deep excavations which was updated by Clough and O’Rourke (1990). Burland et al. (1979) summarised the results of over ten years of research into the movements of ground around deep excavations in London Clay. The Norwegian Geotechnical Institute have published a number of case histories of excavations in soft clay in the Oslo area (e.g. Karlsrud and Myrvoll, 1976). For well supported excavations in stiff clays, Peck’s settlement envelopes are generally very conservative with settlements rarely exceeding 0.15% but movements can extend to 3 or 4 times the excavation depth behind the basement wall. Horizontal movements are generally of similar magnitude and distribution to vertical movements, but may be much larger for open and cantilever excavations in stiff clays.

Advanced methods of numerical analysis, based on the finite element method, are widely used for prediction of ground movements around deep excavations. Such analyses can simulate the construction process, modelling the various stages of excavation and support conditions. However, comparison with field observations, shows that successful prediction requires high quality soil samples with the measurement of small-strain stiffness properties using local strain transducers mounted on the sides of the samples (Jardine et al., 1984).

As for tunnelling, it is essential that deep excavation work be carried out under the close supervision of an experienced engineer. Unless positive support is provided rapidly and groundwater is properly controlled, large unexpected ground movements can develop.

Definitions of ground and foundation movement

Burland and Wroth (1974) suggested nine definitions for the deformation of foundations. Some of these building deformation parameters are regularly used to assess potential building damage (see figure 3).

(i) Settlement: denoted by $S$, is a downwards displacement of the ground. If the displacement is upwards it is termed heave and denoted by $S_h$. Differential settlement is denoted $dS$.

(ii) Rotation or slope: denoted by $q$, is used to describe the change in gradient of the straight line defining two reference points embedded in the foundation or the ground.

(iii) Horizontal strain ($e_h$): a change of length $dL$ over a length $L$ gives rise to a tensile strain $e_{ht} = dL/L$. A shortening of $-dL$ over a length $L$ gives rise to a compressive strain $e_{hc} = -dL/L$.

(iv) Deflection ratio (DR): a sagging ratio, $DR_s$, or hogging ratio, $DR_h$, is determined as $D/L$; where $D$ is the relative deflection (relative sag or hog), and $L$ is the length of structure subjected to that deflection. The relative deflection is the maximum displacement relative to the straight line connecting two reference points a distance $L$ apart. Relative sag produces upward concavity (as at B in figure 3) for which $D$ is positive. Relative hog produces downward concavity (as at E in Figure 3) for which $D$ is negative. The sign convention for deflection ratio is as for relative deflection.
The above definitions only apply to in-plane deformations and no attempt has been made to define three-dimensional behaviour.

**Classification of damage**

**Introduction**

Assessment of degree of building damage can be a highly subjective, and often emotive, matter. It may be conditioned by a number of factors such as local experience, the attitude of insurers, the cautious approach of a professional engineer or surveyor who might be concerned about litigation, market value and 'saleability' of the property etc. In the absence of objective guidelines based on experience, extreme attitudes and unrealistic expectations towards building performance can develop. It is worth stressing that most buildings experience a certain amount of cracking, often unrelated to foundation movement, which can be dealt with during routine maintenance and decoration.

![Figure 3: Building movement parameters (after Burland and Wroth, 1974).](image)

Clearly, if an assessment of risk of damage due to ground movement is to be made, the classification of damage is a key issue. In the U.K. the development of an objective system of classifying damage is proving to be very beneficial in creating realistic attitudes towards building damage and also in providing logical and objective criteria for designing for movement in buildings and other structures. This classification system will now be described.
Categories of damage

Three broad categories of building damage can be considered that affect: (i) visual appearance or 'aesthetics', (ii) serviceability or function and (iii) stability. As foundation movements increase, damage to a building will progress successively from (i) through to (iii).

It is only a short step from the above three broad categories of damage to the more detailed classification given in Table 1. This defines six categories of damage, numbered 0 to 5 in increasing severity. Normally categories 0, 1 and 2 relate to 'aesthetic' damage, 3 and 4 relate to 'serviceability' damage and 5 represents damage affecting 'stability'. It was first put forward by Burland et al. (1977) who drew on the work of Jennings and Kerrich (1962), the U.K. National Coal Board (1975) and MacLeod and Littlejohn (1974). Since then it has been adopted with only slight modifications by BRE (1981, 1990), Institution of Structural Engineers (1978, 1989 and 1994) and the Institution of Civil Engineers (Freeman et al., 1994).

The system of classification in Table 1 is based on 'ease of repair' of the visible damage. Thus, in order to classify visible damage it is necessary, when carrying out the survey, to assess what type of work would be required to repair the damage both externally and internally. The following important points should be noted.

(a) The classification relates only to the visible damage at a given time and not to its cause or possible progression which are separate issues.
(b) The strong temptation to classify the damage solely on crack width must be resisted. It is the ease of repair which is the key factor in determining the category of damage.
(c) The classification was developed for brickwork or blockwork and stone masonry. It could be adapted for other forms of cladding. It is not intended to apply to reinforced concrete structural elements.
(d) More stringent criteria may be necessary where damage may lead to corrosion, penetration or leakage of harmful liquids and gases or structural failure.

Besides defining numerical categories of damage, Table 1 also lists the 'normal degree of severity' associated with each category. These descriptions of severity relate to standard domestic and office buildings and serve as a guide to building owners and occupiers. In special circumstances, such as for a building with valuable or sensitive finishes, this ranking of severity of damage may not be appropriate.
<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Description of typical damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>Hairline cracks less than about 0.1mm</td>
</tr>
<tr>
<td>1</td>
<td>Very Slight</td>
<td>Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1mm.</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths up to 5mm.</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths are 5 to 15mm or several &gt; 3mm.</td>
</tr>
<tr>
<td>4</td>
<td>Severe</td>
<td>Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15 to 25mm but also depends on the number of cracks.</td>
</tr>
<tr>
<td>5</td>
<td>Very Severe</td>
<td>This requires a major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are greater than 25mm but depends on the number of cracks.</td>
</tr>
</tbody>
</table>

Note: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it.

Table 1: Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry.

The division between categories 2 and 3 damage
The dividing line between categories 2 and 3 damage is particularly important. Studies of many case records shows that damage up to category 2 can result from a variety of causes, either from within the structure itself (e.g. shrinkage or thermal effects) or associated with the ground. Identification of the cause of damage is usually very difficult and frequently it results from a combination of causes. If the damage exceeds category 2 the cause is usually much easier to identify and it is frequently associated with ground movement. Thus the division between categories 2 and 3 damage represents an important threshold which will be referred to later.
Concept of limiting tensile strain

Onset of visible cracking

Cracking in masonry walls and finishes usually, but not always, results from tensile strain. Following the work of Polshin and Tokar (1957), Burland and Wroth (1974) investigated the idea that tensile strain might be a fundamental parameter in determining the onset of cracking. A study of the results from numerous large scale tests on masonry panels and walls carried out at the U.K. Building Research Establishment showed that, for a given material, the onset of visible cracking is associated with a reasonably well defined value of average tensile strain which is not sensitive to the mode of deformation. They defined this as the critical tensile strain \( e_{\text{crit}} \) which is measured over a gauge length of a metre or more.

Burland and Wroth made the following important observations:

(a) The average values of \( e_{\text{crit}} \) at which visible cracking occurs are very similar for a variety of types of brickwork and blockwork and are in the range 0.05% to 0.1%.

(b) For reinforced concrete beams the onset of visible cracking occurs at lower values of tensile strain in the range 0.03% to 0.05%.

(c) The above values of \( e_{\text{crit}} \) are much larger than the local tensile strains corresponding to tensile failure.

(d) The onset of visible cracking does not necessarily represent a limit of serviceability. Provided the cracking is controlled, it may be acceptable to allow deformations well beyond the initiation of visible cracking.

Burland and Wroth (1974) showed how the concept of critical tensile strain could be used in conjunction with simple elastic beams to develop deflection criteria for the onset of visible damage.

Limiting tensile strain - a serviceability parameter

Burland et al. (1977) replaced the concept of ‘critical tensile strain’ with that of ‘limiting tensile strain’ \( e_{\text{lim}} \). The importance of this development is that \( e_{\text{lim}} \) can be used as a serviceability parameter which can be varied to take account of differing materials and serviceability limit states.

Boscardin and Cording (1989) developed this concept of differing levels of tensile strain. Seventeen case records of damage due to excavation induced subsidence were analysed. A variety of building types were involved and they showed that the categories of damage given in table 1 could be broadly related to ranges of \( e_{\text{lim}} \). These ranges are tabulated in table 2. This table is important as it provides the link between estimated building deformations and the possible severity of damage.

Strains in simple rectangular beams

Burland and Wroth (1974) and Burland et al. (1977) used the concept of limiting tensile strain to study the onset of cracking in simple weightless elastic beams undergoing sagging and hogging modes of deformation. This simple approach gives considerable insight into the mechanisms controlling cracking. Moreover, it was shown that the criteria for initial cracking of simple beams are in very good agreement with the case records of damaged and undamaged buildings undergoing settlement. Therefore, in many circumstances, it is both reasonable and instructive to represent the facade of a building by means of a simple rectangular beam with the height from foundation level to the eves representing the depth of the beam.
<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Limiting tensile strain ($E_{lim}$) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>0 - 0.05</td>
</tr>
<tr>
<td>1</td>
<td>Very slight</td>
<td>0.05 - 0.075</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>0.075 - 0.15</td>
</tr>
<tr>
<td>3</td>
<td>Moderate*</td>
<td>0.15 - 0.3</td>
</tr>
<tr>
<td>4 to 5</td>
<td>Severe to very severe</td>
<td>&gt; 0.3</td>
</tr>
</tbody>
</table>

*Note: Boscardin and Cording (1989) describe the damage corresponding to $E_{lim}$ in the range 0.15 - 0.3% as ‘moderate to severe’. However, none of the cases quoted by them exhibit severe damage for this range of strains. There is therefore no evidence to suggest that tensile strains up to 0.3% will result in severe damage.

Table 2: Relationship between category of damage and limiting tensile strain ($E_{lim}$) (after Boscardin and Cording 1989)

Sagging and hogging

The approach adopted by Burland and Wroth (1974) is illustrated in Figure 4 where the building is represented by a rectangular beam of length $L$ and height $H$. The problem is to calculate the tensile strains in the beam for a given deflected shape of the building foundations and hence obtain the sagging ratio $\Delta/L$ at which cracking is initiated. It is immediately obvious that little can be said about the distribution of strains within the beam unless we know its mode of deformation. Two extreme modes are bending only about a neutral axis at the centre (Figure 4b) and shearing only (Figure 4c). In the case of bending only, the maximum tensile strain occurs in the bottom fibre and that is where cracking will initiate as shown. In the case of shear only, the maximum tensile strains are inclined at 45° giving rise to diagonal cracking. In general both modes of deformation will occur simultaneously and it is necessary to calculate both bending and diagonal tensile strains to ascertain which type is limiting.

The expression for the total mid-span deflection $\Delta$ of a centrally loaded beam having both bending and shear stiffness is given by Timoshenko (1957) as:

$$\Delta = \frac{PL^3}{48EI} \left(1 + \frac{18EI}{L^2HG}\right)$$

where $E$ is Young’s modulus, $G$ is the shear modulus, $I$ is the second moment of area and $P$ is the point load.
Equation (7) can be re-written in terms of the deflection ratio $D/L$ and the maximum extreme fibre strain $e_{b_{\text{max}}}$ (i.e. maximum bending strain) as follows:

$$\frac{\Delta}{L} = \frac{L}{12t} + \frac{3L E}{2tL H G} e_{b_{\text{max}}}$$

where $t$ is the distance of the neutral axis from the edge of the beam in tension. Similarly for the maximum diagonal strain $e_{d_{\text{max}}}$, Equation (7) becomes:

$$\frac{\Delta}{L} = \left(1 + \frac{H L^2 G}{18 E} \right) e_{d_{\text{max}}}$$

Similar expressions are obtained for the case of a uniformly distributed load with the diagonal strains calculated at the quarter points. Therefore the maximum tensile strains are much more sensitive to the value of $D/L$ than to the distribution of loading.

By setting $e_{\text{max}} = e_{\text{lim}}$, Equations (8) and (9) define the limiting values of $D/L$ for the deflection of simple beams. It is evident that, for a given value of $e_{\text{lim}}$, the limiting value of $D/L$ (whichever is the lowest in Equations (8) and (9)) depends on $L/H$, $E/G$ and the position of the neutral axis. For example, Burland...
and Wroth showed that hogging with the neutral axis at the bottom edge is much more damaging than sagging with the neutral axis in the middle—a result that is well borne out in practice. Figure 5 shows the limiting relationship between \( D/L \) normalised by \( e_{\text{lim}} \) and \( L/H \) for an isotropic beam \( (E/G = 2.6) \) undergoing hogging with its neutral axis at the bottom edge. For values of \( L/H < 1.5 \) the diagonal strains from Equation (9) dominate whereas for \( L/H > 1.5 \) the bending strains dominate.

Figure 5: Relationship between \( (D/K)/e_{\text{lim}} \) and \( L/H \) for rectangular beams undergoing hogging with the neutral axis at the bottom edge.

### The influence of horizontal strain

It was shown in Section 2 that ground surface movements associated with tunnelling and excavation not only involve sagging and hogging profiles but significant horizontal strains as well. Boscardin and Cording (1989) included horizontal tensile strain \( E_h \) in the above analysis using simple superposition, i.e. it is assumed that the deflected beam is subjected to uniform extension over its full depth. The resultant extreme fibre strain \( E_{br} \) is

\[
E_{br} = E_{b\text{max}} + E_h
\]

given by:

In the shearing region, the resultant diagonal tensile strain \( E_{dr} \) can be evaluated using a Mohr's circle of strain. The value of \( E_{dr} \) is then given by:

\[
E_{dr} = E_h \left( \frac{1-v}{2} \right) + \sqrt{E_h^2 \left( \frac{1+v}{2} \right)^2 + E_{d\text{max}}^2}
\]

where \( v \) is Poisson's ratio. The maximum tensile strain is the greater of \( E_{br} \) and \( E_{dr} \). Thus, for a beam of length \( L \) and height \( H \), it is a straightforward matter to calculate the maximum value of tensile strain \( E_{\text{max}} \) for a given value of \( \Delta/L \) and \( E_h \) in terms of \( t, E/G \) and \( v \). This value of \( E_{\text{max}} \) can then be used in conjunction with Table 2 to assess the potential associated damage.
Use of simplified charts
As an alternative to calculating the tensile strains as explained above, simplified charts can be used, such as the one presented by Boscardin and Cording (1989) for cases of L/H = 1. This chart has the following limitations:
(a) it only relates to L/H = 1;
(b) maximum bending strains ?bmax are ignored;
(c) horizontal strain is plotted against angular distortion ?, the evaluation of which is not always straightforward;
(d) b was assumed to be proportional to D/L whereas the relationship is in fact very sensitive to the load distribution.

An equivalent chart, given directly in terms of D/L and e_h by Burland (1995), is shown in figure 6 for the case of L/H = 1 for a hogging mode, it is directly comparable with the Boscardin and Cording diagram. Other charts would be needed for different ratios of L/H.

![Figure 6: Relationship of damage category to deflection ratio and horizontal tensile strain for hogging (L/H = 1).](image)

Evaluation of risk of damage to building due to subidence
The various concepts discussed in the previous sections can be combined to develop a rational approach to the assessment of risk of damage to buildings due to tunnelling and excavation. The following broadly describes the approach that was adopted during the planning and enquiry stages of the recently constructed Jubilee Line Extension underground railway.
Level of risk
The term ‘the level of risk’, or simply ‘the risk’, of damage refers to the possible degree of damage as defined in Table 1. Most buildings are considered to be at ‘low risk’ if the predicted degree of damage falls into the first three categories 0 to 2 (i.e. negligible to slight). At these degrees of damage structural integrity is not at risk and damage can be readily and economically repaired. It will be recalled from Section 4.3 that the threshold between categories 2 and 3 damage is a particularly important one. A major objective of design and construction is to maintain the level of risk below this threshold for all buildings. It should be noted that special consideration has to be given to buildings judged to be of particular sensitivity such as those in poor condition, containing sensitive equipment or of particular historical or architectural significance.

Because of the large number of buildings involved, the method of assessing risk is a staged process as follows: preliminary assessment; second stage assessment; detailed evaluation. These three stages will now be described briefly.

Preliminary assessment
So as to avoid a large number of complex and unnecessary calculations, a very simple and conservative approach is adopted for the preliminary assessment. It is based on a consideration of both maximum slope and maximum settlement of the ground surface at the location of each building. According to Rankin (1988), a building experiencing a maximum rotation (or slope) $\theta$ of 1/500 and a settlement of less than 10mm has negligible risk of any damage. By drawing contours of ground surface settlement along the route of the proposed tunnel and its associated excavations it is possible to eliminate all buildings having negligible risk. This approach is conservative because it uses ground surface, rather than foundation level, displacements. Also it neglects any interaction between the stiffness of the buildings and the ground. For particularly sensitive buildings it may be necessary to adopt more stringent slope and settlement criteria.

Second stage assessment
The preliminary assessment described above is based on the slope and settlement of the ground surface and provides a conservative initial basis for identifying those buildings along the route requiring further study. The second stage assessment makes use of the work described in the previous sections of this paper. In this approach the facade of a building is represented by a simple beam whose foundations are assumed to follow the displacements of the ground in accordance with the ‘greenfield site’ assumption mentioned in Section 2. The maximum resultant tensile strains are calculated from the pairs of Equations (8) & (10) and (9) & (11). The corresponding potential category of damage, or level of risk, is then obtained from table 2.

The above approach, though considerably more detailed than the preliminary assessment, is usually still very conservative. Thus the derived categories of damage refer only to possible degrees of damage. In the majority of cases the likely actual damage will be less than the assessed category. The reason for this is that, in calculating the tensile strains, the building is assumed to have no stiffness so that it conforms to the ‘greenfield site’ subsidence trough. In practice, however, the inherent stiffness of the building will be such that its foundations will interact with the supporting ground and tend to reduce both the deflection ratio and the horizontal strains. This subject is discussed in Section 8.

Detailed evaluation
Detailed evaluation is carried out on those buildings that, as a result of the second stage assessment, are classified as being at risk of category 3 damage or greater (see table 1). The approach is a refinement of the second stage assessment in which the particular features of the building and the tunnelling and/or
excavation scheme are considered in detail. Because each case is different and has to be treated on its own merits it is not possible to lay down detailed guidelines and procedures. Factors that are taken more closely into account include:

Tunnelling and excavation: The sequence and method of tunnel and excavation construction should be given detailed consideration with a view to reducing volume loss and minimising ground movements as far as is practical.

Structural continuity: Buildings possessing structural continuity such as those of steel and concrete frame construction are less likely to suffer damage than those without structural continuity such as load bearing masonry and brick buildings.

Foundations: Buildings on continuous foundations such as strip footings and rafts are less prone to damaging differential movements (both vertical and horizontal) than those on separate individual foundations or where there is a mixture of foundations (e.g. piles and spread footings).

Orientation of the building: Buildings oriented at a significant skew to the axis of a tunnel may be subjected to warping or twisting effects. These may be accentuated if the tunnel axis passes close to the corner of the building.

Soil/structure interaction: The predicted ‘greenfield’ displacements will be modified by the stiffness of the building. The detailed analysis of this problem is exceedingly complex and resort is usually made to simplified procedures some of which are described in the report published by the Institution of Structural Engineers (1989). The beneficial effects of building stiffness can be considerable, as demonstrated by some recent measurements on the Mansion House, in the City of London, during tunnelling beneath and nearby (Frischman et al., 1994).

Previous movements: The building may have experienced movements due to a variety of causes such as construction settlement, ground water lowering and nearby previous construction activity. It is important that these effects be assessed as they may reduce the tolerance of the building to future movements.

As many factors are not amenable to precise calculations, the final assessment of possible degree of damage requires engineering judgement based on informed interpretation of available information and empirical guidelines. The inherently conservative assumptions used in the second stage assessment mean that the detailed evaluation will usually result in a reduction in the possible degree of damage. Following the detailed evaluation, consideration is given as to whether protective measures need to be adopted. These will usually only be required for buildings remaining in damage categories 3 or higher (see table 1).

**Incorporating building stiffness into settlement predictions**

As discussed in the preceding sections, the prediction of ground movements due to tunnelling is normally made using empirical relationships which are based on previous measurements obtained at several greenfield sites. The presence of a building is assumed to have no effect on the settlement prediction and any damage parameters are calculated using the predicted greenfield movements. This is clearly an over-simplification of reality as the stiffness of the building affects its deformation.

To improve on this situation a new design approach has been developed at Imperial College. This involves the use of two sets of design curves which are used to modify the damage parameters calculated in the conventional manner. These design curves are based on the results of an extensive parametric
study involving finite element analyses in which the width of the structure, its bending and axial stiffness, its position relative to the tunnel and the depth of the tunnel were considered. The design curves give a guide as to the likely modification to the greenfield settlement trough caused by a surface structure. They can be used to give initial estimates of consequent building damage. A description of the new design approach is given.

A modified secondary evaluation of building performance

Potts and Addenbrooke (1997) defined soil/structure relative stiffness parameters. The relative bending stiffness, \( \rho^* \) (with units \( m^{-1} \)), and relative axial stiffness, \( \alpha^* \) are defined as in Equation (12):

\[
\rho^* = \frac{EI}{E_s(L/2)^2}
\]

\[
\alpha^* = \frac{E_A}{E_sL}
\]

where \( L/2 \) represents half the length of the structure, \( EI \) is the building's bending stiffness, \( EA \) its axial stiffness, and \( E_s \) is a representative soil stiffness. \( E_s \) is the secant stiffness at 0.1% axial strain from a triaxial compression test of a sample taken from a depth \( Z/2 \).

Over 100 finite element analyses of tunnel excavation beneath a beam (representing an elastic surface structure) were completed. Figure 7 shows the geometry. \( D, Z, \) and eccentricity, \( e \), were all varied, as were the bending and axial relative stiffness, through variations in \( L/2, EI, \) and \( EA \). An eccentricity ratio is defined as \( e/(L/2) \).

Figure 7: Geometry for finite element analysis.
Each analysis gave a sagging deflection ratio and compressive horizontal strain, and/or a hogging ratio and tensile strain for the building. Division by the numerically determined greenfield values relevant to the building size and position gave a quantifiable measure of the building’s modifying effect, M, for each of the four deformation parameters. M therefore equals the structural deformation divided by the greenfield deformation.

It was determined that in the likely range of true building stiffness the relative bending stiffness controlled the degree of modification to the deflection ratio (i.e. vertical settlement profile); and the relative axial stiffness controlled the degree of modification to the horizontal strain (i.e. the horizontal displacement). Potts and Addenbrooke therefore plotted the modification factors for deflection ratio, M_{DR} and M_{DR} against \( r^* \) for each e/B; and the modification factors for horizontal strain, M_{hc} and M_{hc} against a* for each e/B. Empirical design lines were fitted through the data, and these are reproduced in figure 8.

When entering into a secondary evaluation, the profile of vertical settlement and that of horizontal ground displacement are used to give the greenfield building damage parameters, denoted DR^g and ε_h^g. Depending on whether the structure spans the point of inflection of the greenfield settlement trough, up to four values of greenfield damage criteria are established at this stage: hogging and tensile strain, as well as sagging and compressive strain.

The soil/structure relative bending and axial stiffness are determined for the structure in question. As a first estimate the engineer could consider the contribution to stiffness of the foundation alone, before considering the independent or coupled contributions to bending and axial stiffness of slabs, beams,
columns and load-bearing walls. For each combination of relative stiffness and eccentricity ratio, modification factors can be taken from the design curves of figure 8.

The greenfield values of deflection ratio and horizontal strain are multiplied by the respective modification

\[
DR_{sag} = M^{DReq} \cdot DR_{sag}^g
\]
\[
DR_{hog} = M^{DReq} \cdot DR_{hog}^g
\]

factors to obtain those likely to be imposed on the structure:

\[
-M_{hc} = M^{-M_{hc}} \cdot M_{hc}^g
\]
\[
-M_{ht} = M^{-M_{ht}} \cdot M_{ht}^g
\]

The combinations of sagging deflection ratio and compressive strain, and hogging deflection ratio and tensile strain can then be input into damage category charts such as that shown in Figure 6 to quantify the likely damage to the surface structure.

Following this modified secondary evaluation fewer buildings along a tunnel’s route will enter the detailed stage of assessment which is more costly. It is therefore beneficial to avoid it.

It is the case however that even with the modified secondary evaluation, the damage category chart which defines likely degree of damage is still based on elastic beam theory. The engineer does not gain an understanding of where cracking may first occur, or how a cracked building might subsequently behave. It is also the case that the beams analysed by Potts and Addenbrooke (1997) were elastic, and did not represent cracked behaviour of a masonry building. A numerical model capable of modelling masonry is therefore the next step towards analysis of the entire non-linear soil/structure interaction problem (see Addenbrooke and Potts, 1998).

In the event that design calculations indicate the possibility of building damage, more advanced numerical analyses are often performed. Case studies are presented in Section 11 where the observed building responses to the construction of the JLE are compared with predictions made using the new method incorporating building stiffness and using numerical analysis for two of the study LINK research project buildings.

**Projective measures**

The range of possible protective measures is summarised briefly as follows.

Tunnelling: Before considering near surface measures, consideration should be given to measures that can be applied from within the tunnel to reduce the volume loss. There are a variety of such measures such as increasing support at or near the face, reducing the time to provide such support, the use of forepoling, soil nailing in the tunnel face or the use of a pilot tunnel. These approaches tackle the root cause of the problem and may prove much less costly and disruptive than near surface measures.

If, for a particular building, tunnelling protective measures are considered either not technically effective or too expensive, then it will be necessary to consider protective measures applied near the surface.
or to the building itself. However it must be emphasised that such measures are generally disruptive and can have a significant environmental impact. The main forms of protective measures currently available fall into the following six broad groups:

1. Strengthening of the ground by means of grout injection (cement or chemical) or by ground freezing. It is normally undertaken in granular water-bearing soils. Its primary purpose is to provide a layer of increased stiffness below foundation level or to prevent loss of ground at the tunnel face during excavation.

2. Strengthening of the building in order that it may safely sustain the additional stresses or accommodate deformations induced by ground movements. Such measures include the use of tie rods and temporary or permanent propping.

3. Structural jacking to compensate for settlement.

4. Underpinning by introducing an alternative foundation system which eliminates or minimizes differential movements caused by tunnelling.

5. Installation of a physical barrier between the building foundation and the tunnel. Such a barrier is not structurally connected to the building’s foundation and therefore does not provide direct load transfer. The intention is to modify the shape of the settlement trough and minimise ground displacements adjacent to and beneath the building.

6. Compensation grouting which consists in the controlled injection of grout between the tunnel and the building foundations in response to observations of ground and building movements during tunnelling. As its name implies, the purpose is to compensate for ground loss. The technique requires detailed instrumentation to monitor the movements of the ground and the building. The technique has recently been used with success at Waterloo Station for the construction of a new 8m diameter tunnel passing within a few metres beneath two sensitive masonry structures: the Victory Arch and the Waterloo and City Line tunnels (Harris et al., 1994).

It cannot be emphasised too strongly that all of the above measures are expensive and disruptive and should not be regarded as a substitute for good quality tunnelling practice aimed at minimising settlement.

The link project and adopted building monitoring techniques

This section briefly describes the principal methods of measurement that were made as part of the LINK research project.

An outline of the project and the reasons for its inception are given in the introduction in Section 1, further details of the project, its aims, funding and management are given by Burland et al. (1996). The project involved selecting about thirty study buildings along the route of the Jubilee Line Extension (JLE) project that represent different structural forms and foundation types and that were influenced by different tunnelling and excavation methods in London Clay or the more granular deposits of the Lambeth Group and Thanet Sand. Two greenfield control sites were also identified to provide information on the ground response when no building is present. Details of the instrumentation and monitoring techniques used on the greenfield sites are given by Standing et al. (1996).

Field research monitoring requires simple, robust and reliable measuring techniques, additionally the activity may take place over a number of years and involve many operators.

Features of the main monitoring techniques used by the research team to assess building behaviour are now given. Comparisons are made between different techniques and accuracy of measurement is dis-
cussed. Observations on two of the study buildings are presented in the next section to illustrate the value of monitoring and to provide examples of the data.

**Precise levelling and taping**

Precise levelling and taping were carried out using purpose made miniaturised survey stations based on a design by the Building Research Establishment (Cheney (1973) and BRE Digest 386) - see Figure 9. The BRE surveying station consists of two components - a stainless steel socket which is grouted into the facade of the building and a removable levelling plug. When not in use the socket is sealed from dirt by a protective perspex bung. The face of the bung is flush with the facade of the building and takes on the colour of the finish so that it is barely visible from a few metres distance. This aids in the aesthetic acceptability of such survey stations.

A very important feature of the BRE levelling station is that it has been designed to ensure that the levelling plug will position with repeatable accuracy - this does not appear to be widely appreciated by many surveyors. The thread is a loose fit and is used only to pull the plug into the socket. The plug is located radially to an accuracy of 0.03mm by a precisely machined spigot and socket turned concentrically with the threads on the levelling plug and socket. As the plug is finally tightened into the socket the mating surfaces pull the two components coaxial. It is only necessary to tighten the plug firmly by hand.

![Figure 9: Details of BRE surveying stations.](image)

At the start of the LINK project difficulties were experienced in drilling 38 mm diameter holes in hard granite cladding and reinforced concrete. A miniaturised version of the BRE survey station was designed requiring a 25 mm diameter hole which is even less obtrusive. This has proved very satisfactory and hundreds of these units have been installed along the route of the JLE. By following best practice in precision levelling it has been found that accuracies of about ±0.1 mm can be achieved regularly.
The levelling plug may be replaced by a taping plug similar to one described by Burland and Moore (1973). Taping measurements provide information on changes in horizontal distance between survey stations. Thus horizontal strains, which are often used in the assessment of potential building damage, can be determined. Recent developments in the design of tape extensometers have resulted in improved resolution and accuracy of measurement. In these systems the tension of the steel tape is monitored electronically and the length of the tape given on a digital readout (Standing, 1999). Experience with the LINK project indicates that an accuracy of about ±0.1 mm can be achieved.

**Precise geodetic surveying (facade monitoring)**

The technique involves taking a series of measurements to designated points on the facade with a high precision total station, enabling building movements in a three-dimensional cartesian coordinate system to be determined.

The targets installed on building facades are retro-reflective prisms mounted on a plastic laminate. They are placed to give good coverage of the building, remote from potential vandalism. The targets can be affixed to the fabric of the building using an epoxy resin for permanent siting or a silicon based sealant for temporary location. Target positions are usually accessed via windows, balconies or roof of the building.

Most modern electronic instruments can measure angles and distances to a resolution of 0.1 arc seconds and 0.1 mm respectively.

The procedure for performing a survey involves selecting survey stations to maximize the number of targets that can be seen from each. The targets should be seen from at least two stations to obtain redundant observations as this considerably increases confidence in the measurements.

The locations of the survey stations are determined each time a set of measurements is made by reading to reference targets placed on buildings outside any zone of influence.

Measurements are made from each station to all visible targets. Note that when the angle between the instrument and target becomes too oblique it is often not possible to measure distance. It is essential that angle measurements are made from two stations to such targets. Angle measurements are made on both faces of the instrument to eliminate face errors and improve accuracy.

The processing and analysis of the results from such types of measurement are performed using a computer program which adjusts all observed values of horizontal and vertical angles and distances by the method of least squares. Observations are properly weighted to make the observation equations dimensionless.

The accuracy of measurements that has been achieved on buildings monitored by the LINK team is typically ± 1.5 mm. The accuracy that can be obtained is dependent on the distance between the instrument and the point being measured and their relative positions.

**Electrolevel inclinometers**

At the outset of the JLE works it was intended to monitor real-time building movements using electrolevel inclinometer systems. However, in some instances problems were encountered which made this difficult, especially where the systems were exposed to the elements. One of the causes appears to be related to temperature instability arising out of the mounting of the electrolytic inclinometers on aluminium beams.
An intense study was carried out at Imperial College in order to develop a more stable mounting system (Barakat, 1996). It was found that, by suspending the electrolevel on a constant tension wire (CTW), the influence of temperature change on the mounting system could be eliminated. Such a system is much cheaper than aluminium beams and is also much more adaptable since it can be used for various spans of much longer length if necessary. Results from the CTW system installed at the Palace of Westminster, London are given by Burland and Standing (1996).

**Crack monitoring**

Monitoring existing cracks and construction and expansion joints should not be overlooked as these features often affect the way a building deforms and its subsequent damage.

There are several proprietary crack-monitoring systems available. The LINK research team used a manually read demountable mechanical (Demec) strain gauge. This is a relatively simple and inexpensive device which is very reliable and is operator insensitive. Sets of studs, to which measurements are made, are positioned around the discontinuity in one of several configurations that can be adopted. It is possible to measure components of movement normal and parallel to the crack and also those in the intact material. Details of these stud arrangements and the measurement and analysis techniques are given by Viggiani and Standing (1997), a case study with measurements is also included in the paper.

The accuracy of measurement obtained for the LINK research was usually within ±0.005 mm.

It is worth noting that the techniques of precise levelling and geodetic surveying were used to monitor shallow surface ground movements at the two greenfield control sites. Comparing measuring techniques gave confidence in the accuracy of the readings (Nyren, 1998). Electrolevel devices were also used at these locations to measure subsurface displacements.

**Two case studies from the jubilee line extension project**

In this section two case studies from the LINK research project are given. Two forms of structure have been chosen for which different prediction methods were applied. The first is a reinforced concrete frame building the movement of which was predicted using conventional techniques with the building stiffness taken into account. The Treasury and some of the construction activities that affected it have been modelled using finite element analyses.

**Elizabeth House: Ten-storey reinforced concrete frame structure**

As part of the LINK project, class A predictions (Lambe 1973) were performed to estimate the responses to tunnelling of a number of selected buildings. The effects of building stiffness were taken into account in these predictions using the approach outlined in Section 8 of this paper. These predictions were made by Professors R.J. Mair and R.N. Taylor of the Geotechnical Consulting Group, London.

One of the case studies is presented below, the predictions for which involved four key steps:
- assessing the appropriate volume loss and trough width parameters to estimate greenfield settlements;
- applying the principle of superposition and summing settlements for twin tunnels or separate phases of excavation;
- assigning suitable relative stiffness values to the structure (using the Potts and Addenbrooke (1997) approach) in order to compute the modified settlement;
- judging how closely the structure would conform to the calculated profile.
Elizabeth House is a multi-storey reinforced concrete framed building constructed in the 1960s with two levels of basement and is founded on a monolithic 1.4-m thick reinforced concrete raft. The length of the building affected by the JLE tunnels has ten storeys above ground level and has dimensions approximately 80 m long by 18 m wide with the height of the building from formation level about 45 m.

The underside of the raft foundation beneath the lower basement is about 8 m below ground level at this location. There is generally about a 1 m thickness of gravels beneath the foundation, underlain by some 30 m of London clay. The tunnels are in London Clay and their axes are about 23 m below foundation level.

Figure 10 shows the plan of the tunnels that were constructed beneath the building. Two running tunnels of 5.6 m diameter were constructed using the sprayed concrete lining technique (NATM), except for a short initial length of the westbound tunnel where segmental linings were used.

![Figure 10: Plan of Elizabeth House with underlying JLE tunnels.](image)

The bending stiffness of the building was assessed (Mair and Taylor, 1996) to obtain estimates of the modification factors that could be applied to the deflection ratios. The modified shape of the settlement profiles were then estimated. It was concluded that in the transverse direction, i.e. over its 18 m side, the building would behave almost rigidly to tunnel construction. Conversely, in the longitudinal direction, i.e. over its 80 m length, it would be expected to behave almost perfectly flexibly. In view of these assessments the predicted greenfield settlements were modified to allow for rigid behaviour of the building in the transverse sense, while no modifications were applied in the longitudinal sense. Examples of the predicted profiles along two sections through the building for immediately after construction of the
running tunnels are shown in Figure 11. Also shown are the corresponding measured settlements. Excellent agreement is evident, particularly with regard to the influence of the building stiffness in the two directions.

Figure 11: Predicted and measured settlements at basement level of Elizabeth House.

The Treasury
Results are given from numerical analyses that were performed to model the behaviour of the Treasury building which was also influenced by the construction of two JLE tunnels and where additionally protective measures in the form of compensation grouting were implemented. Only a summarised description of the analyses and the results are included, further details are given by Standing et al. (1998).

Description of structure and construction works
The Treasury is a massive stone-clad brick-masonry structure approximately 210 m long and 100 m wide with four storeys above ground and two basement levels. The foundations consist of strips and pads connected by an unreinforced concrete slab founded in the Terrace Gravels which overlie London Clay. The top of the foundation is approximately 6 m below ground level.

As part of the JLE project, two running tunnels were excavated under one corner of the building as shown in Figure 9. These two tunnels run with the eastbound tunnel above the westbound. Beneath the corner of the Treasury the axes levels are 24 m and 34 m below ground level and approximately 10 m apart with the westbound tunnel axis about 8 m away from the corner of the building, as shown in Figure 12. The tunnels are 4.95 m in diameter and were excavated using an open-face shield tunnelling machine. Concrete segments form the linings and these were placed and expanded immediately behind the shield.
Due to the historical nature of the building complex precautionary measures were taken to prevent building damage involving the use of compensation grouting which was implemented after driving the westbound tunnel and during construction of the eastbound tunnel. The level of the tubes-a-manchette (TAMs) used for this work is roughly 16 m below ground level extending beneath the basement between the two tunnels and the foundation slab (Figure 13).

The results presented here focus on: (1) movements after construction of the westbound tunnel; (2) movements 18 weeks after the compensation grouting works and the construction of the eastbound tunnel and; (3) long-term predictions after consolidation is complete.

Adjacent to the Treasury building (less than 100 m away) in St. James’s Park, further instrumentation has been installed to measure greenfield behaviour (Standing et al. 1996). This allows comparisons to be made with the building movements, finite element analyses have also been performed for this site.
Finite element analysis
The finite element program ICFEP (Imperial College Finite Element Program) was used to carry out the analyses reported here. Plane strain conditions were assumed and the cross-section AA shown on Figures 12 and 13 was analysed. The soil stratigraphy was obtained from the site investigation performed as part of the JLE and is indicated on Figure 13. The detailed part of the finite element mesh in the vicinity of the tunnels is shown in figure 14.

Figure 14: Detailed section of finite element mesh.

Information relating to the constitutive models adopted, the boundary conditions and pore pressure distributions are given by Standing et al. (1998). It should be noted that volume loss was used to control the analyses, appropriate values were prescribed from settlement data from St. James’s Park, Standing et al. (1996).

Compensation grouting was modelled by applying a pressure within a row of interface elements which were included in the finite element mesh at the location of the TAMs (Figure 13). In the finite element analyses the construction activities and associated durations recorded during the works were followed. The analyses were then extended to investigate long-term behaviour.

Two finite element analyses were performed, one with and the other without the grouting activities modelled. Comparison of these analyses enable the effects of the grouting to be quantified.

Results from the analyses
Figure 15 shows the settlement troughs after excavation of the westbound tunnel (i.e. the first tunnel to be constructed). Field measurements at ground surface from St. James’s Park (greenfield) and at foundation level for the Treasury are presented. Comparison of the observations from these two sites indi-
cates the influence of the building. It reduces the maximum settlement but increases the width of the settlement trough. The maximum settlement beneath the Treasury is beneath its corner, slightly offset from the tunnel centre-line. Also shown on Figure 15 is the finite element prediction for the Treasury. The agreement between the prediction and the field observations is excellent.

Figure 15: Ground settlements after construction of the westbound tunnel.

Figure 16 presents settlement troughs at foundation level for the Treasury, 18 weeks after construction of the eastbound tunnel. Both observations and finite element predictions are presented. To be consistent with the field data the predictions are from the run which modelled the grouting operations both before and during construction of the second tunnel. The agreement between prediction and observation is again excellent.
Figure 16: Ground settlements 18 weeks after construction of the eastbound tunnel.

Figure 17 presents settlement troughs at foundation level in the long term after all excess pore water pressures in the soil have dissipated. As long-term conditions have not yet been achieved in the field there are no field data and only finite element predictions are given. As discussed above these analyses have reproduced the observed behaviour to date and this gives confidence in their ability to predict the future. Comparison of the result from the two runs quantifies the effects of the grouting on long-term settlements.

Figure 17: Long-term ground settlements.
The grouting operations reduce the total maximum settlement from 68 mm to 48 mm. However, the difference in these values is achieved during the short-term control of movements, the settlements associated with the dissipation of excess pore water pressures, after construction and grouting are complete, are the same for both runs (i.e. maximum settlement approximately 41 mm). Consequently grouting does not appear to reduce the long-term components of movement due to pore pressure dissipation.

**Conclusions**

This paper summarises briefly a rational and coherent approach to the assessment of risk of damage to buildings due to tunnelling and excavation. The approach is based on the integration of a number of studies relating to prediction of ground movements, categorisation of damage and the factors controlling cracking of masonry and brickwork.

A new approach to predicting potential building damage is described where the axial and bending stiffness of the building can be taken into account. This method is based on the results from an extensive series of numerical parametric studies that were carried out varying building stiffness and geometry.

The approaches described have an analytical framework and it is evident that much reliance is placed on experience and case records. There has been a conspicuous shortage of well documented case histories of measured building response to ground movements. In a concerted effort to remedy this, the opportunity offered by the construction of the Jubilee Line Extension has been used to carry out a major cooperative research programme into the behaviour of selected buildings along the route.

Some of the monitoring techniques used and developed during the LINK research project have been described. Simple, reliable and robust instruments are required to monitor ground and building response.

Results from the LINK research project have been used to illustrate the effectiveness of different prediction methods and the response of different structural forms to tunnelling-related activities. It is hoped that the results from this research programme will continue to progress our understanding of the tolerance of structures to ground movement and in the assessment of risk of damage.

One case study has been used to illustrate how a structure can modify the ground movements and how taking the building stiffness into account in a systematic manner can lead to improved predictions of movement and damage.

Another building influenced by the JLE has also been used to show how its stiffness can modify the settlement profile by comparison with nearby greenfield observations. Complex protective measures were implemented on this building. These have been modelled numerically using finite element analyses which show good agreement with the observed movements and have allowed long-term trends to be estimated.

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Abstract
This paper presents the implementation of advanced monitoring and compensation grouting techniques for critical phases of the Tren Urbano Project in Puerto Rico: Construction of a 14 m diameter cavern whose crown is located 5 m below existing structures, and construction of two parallel running tunnels below historic buildings. SOLDATA, a company specialising in the implementation of highly technical monitoring schemes on small and large projects, provided both an automatic monitoring system consisting of two computer guided theodolites, and the complete database package allowing acquisition, storage and presentation of the results. Compensation grouting methods have also been improved over the last years by SOLETANCHE BACHY. The latest developments allow prediction of the settlement at any time thanks to an iterative approach and a very close integration with the monitoring results. Specific examples show the monitoring results recorded both with and without compensation.

Rio piedras project
The Tren Urbano Project in San Juan, Puerto Rico, is a 17.2 km long new railway line, the first in this city. Most of the project is over ground, but in the Rio Piedras area, it consists of two parallel TBM driven tunnels arriving into the critical Rio Piedras Station. This latest consists of a large vault about 140 meters long, 18 meters high and 23 meters wide. Its construction is taking place completely underground because of the important business activity and the heavy traffic on the Ponce de Leon Avenue. The man made excavation method, digging 3x3 meters drifts, was chosen for this part of the works considering the small depth of the layer above head (as little as 2 meters). The 15 interlocking drifts will be backfilled with concrete to form the permanent structure of the station. After drift construction, the interior of the station will be mined. The 2 TBM driven tunnels are of 6 meters diameter and are running 10 to 15 m below ground surface. The soil in the area consists of 2 m of fill overlying 4 m of plastic clay (CL to CH). The clay is overconsolidated (OCR of 6 to 9) with secondary structure consisting of high angle fissures. The compensation had to be carried out within that thin layer. Below the clay, one can find interbedded layers of sand, silt and clayey sand (loose to medium dense). Maximum predicted settlements are in the order of 60 mm (30 mm for the running tunnels) for a 2 % predicted face loss, and up to 150 mm (56 mm) for the actual construction face loss of 5 %, with a settlement specification limit of 25 mm. Ground treatment and settlement control measures are therefore required. The Rio Piedras Contract has been awarded to the Joint Venture Kiewit Kenny Zachry as main contractor, with Soletanche Inc as subcontractor for technical support for the compensation grouting operations.
Automatic control of movements

Introduction

It is of the utmost importance when carrying out ground treatment associated with tunnelling in soft ground to measure and analyse virtually in real time the ground and structures movements. Various instruments exist nowadays on the market. But one must also be able to treat and study the huge amount of results continuously produced by these instruments.

The original monitoring requirements were limited to the manual levelling. The measurements were to be taken at best one to twice daily. The delay for getting this information linked to access difficulties to the control settlement points and problems of accuracy of the results were considered as unacceptable. It was not possible to follow the reaction of the ground and adjust the compensation grouting techniques within a satisfactory span of time.

SOLDATA proposed efficient answers both to this problem and to the requirement for a data treatment system able to cope with the specificities of monitoring results with automatic acquisition systems and measurement devices on one hand, and a monitoring database on the other hand. After a period of test where both the traditionnal and SOLDATA systems were used, the new equipment proved its superiority and further elements were installed.

CYCLOPS systems

In Puerto Rico, SOLDATA chose to install its CYCLOPS (CYClic Optical Surveyor) system: It consists of a motorised theodolite guided by a computer. It aims continuously at targets organised in pre-defined cycles, and sends results of the movements measured “in real time”. This system has been developed by, and is now used in collaboration with, the IGN, the French National Geographic Institute, for the past two years. Each target gives 3 displacements values: 2 angles and one distance, translated as X, Y, Z movements. The certified accuracy at 120 m is 1 mm or better in the 3 dimensions, in actual site conditions, which are difficult in Puerto Rico: High temperatures, storms...

Two CYCLOPS have been installed on the San Juan site. The first one monitors the station area: The aim is to monitor, in real time, 13 buildings over the 150 m long Station of Rio Piedras. A total of 50 prisms targets have been installed above the Station. The targets located on the roof of the buildings are tracked by the CYCLOPS, and their movements recorded. The prisms have been fixed on the roof for once and do not require physical connection or access.

The theodolite base station is on the roof of the tallest building, to offer good lines of sight. 9 reference targets have been installed in adjacent areas, outside the settlement trough area, to allow CYCLOPS to correct and compensate in real time for the movements of its own building. In about 10 minutes, all the targets are remotely checked by the CYCLOPS.

The second CYCLOPS is set-up in a similar manner, to monitor the most sensitive buildings located above the TBM drives. It is located on the roof of an aisle of the University. It continuously sights targets located both on the roof of the University (a classified historical building) and on other roofs and at street level along the path of the TBM drives, in the area to be treated with compensation grouting.

Monitoring database: GEOSCOPE.

From then, the GEOSCOPE system takes on, to allow rapid and efficient use of the data: Results are transferred to the office through a numerical link, and are presented on screen through clear and interactive visualisation programs in real time, with automatic management of alarms. All the results from the CYCLOPS are then stored in a specially designed monitoring database, alongside the other monitoring data: surveyors, inclinometers...

The interest of this system is the following:

- Virtual Real time monitoring: 1 cycle in about 10 minutes.
- Continuous monitoring (night, week-ends, holidays), with possible connection with alarms, pagers, fax,... in case of unforeseen movements above set limits.
- Precision: 1 mm or better.
Monitoring of the back of the buildings, for the detection of differential movements even though these were not accessible for conventional survey. For compensation grouting purpose, specifically during excavation of the drifts, when concurrent grouting is required, continuous real time monitoring is of great help to fine tune the volume required as soon as decompression of the ground is detected, as explained below.

**Improvements to the compensation grouting methods**

**Compensation grouting**

Compensation grouting is a reasonably new technology the use of which seems to be widespread. It consists in injecting small and well defined quantities of grout in precisely selected locations, in order to replace the volume loss caused by underground excavation: During underground excavation works, surface settlement are caused by the ground deformation towards the unsupported or loosely supported excavation. If the volume lost can be replaced by grout before the deformation reaches the surface, all disorders are avoided.

**Improvements implemented on the Tren Urbano Project.**

**Introduction**

The theory behind the compensation grouting process is simple, but the details of its implementation are complicated: In order to replace the volume loss before it reaches the surface, one must be able to predict this volume, its exact location and the timing of its appearance in function of the tunnel works below. The ideal is to compensate the soil movements as close as possible from their starting position, to input the smallest possible corrections, and finally, the phenomenon being dynamic, to anticipate slightly.

The prediction of the volume of ground loss for a given project needs to take into consideration such elements as nature of ground, size and depth of the excavation, composition of the ground cover etc... and is therefore very difficult to estimate accurately from theory.

In effect, most “compensation grouting” sites to date have had no other choice but to estimate roughly the amount of grout necessary, and to review their program following surface movements (heave if excessive grouting was carried out, settlement in the other case). The effect is to obtain intermediate surface movements of importance which are very damaging to the structures, even if the final total movements can be adjusted to low values.

**Figure 1.** Surface movements during compensation grouting operations: « What should be done » (dotted line) versus « what is usually done » (continuous line).
In London’s Jubilee Line Contract 101 in 95-97, SOLETANCHE BACHY implemented a new technique particularly well suited to slow manual excavation and hard London clay (the “pressure method”), which does not require exact preliminary knowledge of the volumes to be grouted. However, for soft soil and for a fast moving TBM, the grouting volumes must be accurately known and adjusted in real time if one wants to follow exactly the compensation grouting theory. This is why SOLETANCHE-BACHY designed improvements to the system, first in Madrid (Madrid Metro Linea 1) and finally in Puerto Rico.

Active movement control

We have seen that settlement mechanisms are dynamic, complex, and difficult to estimate. Therefore the chore of the new method consists in organising an iterative field regulation cycle to:

- forecast and estimate settlement. Methods like finite elements settlement calculations are time consuming and incompatible with site progress, and a simple settlement model must be used.
- estimate grouting quantities and best point of injection as to better mitigate settlements.
- observe ground and building movements as they occur and adjust grouting program in real time.
- compare daily the settlement predictions to actual field measurements and fine tune the predictive model.

A special software has been written to perform all these calculations and actions: COGNAC. It defines the program of grouting depending on the program of excavation and on past grouting and monitoring data. COGNAC calculates settlements using a simple method based on the use of only two parameters: the face loss coefficient and the shape coefficient of the Gaussian curve modelling the settlements. These 2 coefficients are estimated in a first phase for example on the finite elements calculations of the Engineer, and they are later adjusted regularly depending on the monitoring and grouting results. This is the key point of the system: A numerical model, as sophisticated as it can be, will never be able to reproduce perfectly the reality of site conditions, and will not be able to adjust to unforeseen or unpredictable events like the crossing of an unknown old river bed or other.

COGNAC then calculates the total volume to be grouted and its precise repartition, sleeve by sleeve, amongst the chosen boreholes. The two parameters are adjusted daily in function of the previous monitoring and grouting results.

From this point, SOLETANCHE BACHY can make full use of its other innovations in the domain of grouting: The complete chain of commands is fully automatised, the grouting instructions are used by the grouting control program to regulate the grout pumps in volume, pressure and flow rate, the position of the packer in front of each selected sleeve is measured using bar code on the grout line, the detailed grouting results are stored for future analyses, and the software are designed to allow cross analysis of grouting and monitoring results.

Example of monitoring and grouting results

In March 99, the first of the 2 TBM went through the compensation grouting area of the University of Puerto Rico. Ahead of the treated area, the measured surface settlements reached values of 50 mm. The TBM went through the area very rapidly: 15 m per day. The compensation grouting therefore had to be extremely accurate and rapid. The treatment was a complete success in the University area: Movements were continuously measured by the CYCLOPS, giving vital real time information to the grouting engineers. Small adjustments were made on COGNAC’s parameters up to three times daily on the first days, allowing a very accurate compensation to be carried out. The area settled by less that 1 mm, with maximum intermediate movements of -2 and +1 mm. In the next group of buildings, a problem with the grout production stopped the injection for 5 hours one night. The settlements increased rapidly to 5 mm, and were stopped at that value when the grouting resumed. It would have been possible to lift the buildings back to their position, but this would have been contrary to the compensation grouting “philosophy”. The level was kept stable for the rest of the TBM drive.
The Graph on figure 2 shows one of the possibilities from GEOSCOPE: the volume grouted in the various phases (top graph) is plotted on the same time scale as the results from CYCLOPS targets (bottom graph). The serie with triangles shows the theoretical settlements if no compensation grouting was carried out.

![Grouting phases](image)

![Surface movements](image)

Figure 2. Grouting phases versus surface movements.

**Conclusion**

Although what is called compensation grouting has become more and more common over the last few years, it is extremely difficult to stick to the actual philosophy of the method and it is rarely done perfectly, especially for a fast moving excavation and for difficult soft soils. However, thanks to new developpements of its grouting tools and methods, and by making full use of the modern methods of real time monitoring implemented by SOLDATA, SOLETANCHE BACHY have shown than compensation grouting can be carried out « to the letter », even in difficult soil. And the CYCLOPS and GEOSCOPE systems have proved their usefulness and the benefits they can bring when knowledge of the soils and structures movements in real time is vital to a project.
GROUTING TO INCREASE THE BEARING CAPACITY OF PILE FOUNDATIONS

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Abstract
This paper outlines how grouting can be used to increase the bearing capacity of pile foundations. Several injection techniques, like permeation grouting, jet grouting and compensation grouting (both fracturing as well as compaction grouting) are used in “Full Scale Injection Test” in Amsterdam. This test is part of the North/South metroline. Objective of this test is to determine the (positive) influence of injecting / grouting near to (Amsterdam) pile foundations.
Outlined are the principle of grouting to increase the bearing capacity of the piles, the test set-up including specific site conditions, characterisation of the pile foundations, the monitoring program and the actual test program. The test-results will hopefully be presented at the oral presentation during the congress.

Introduction
Scope of research
The research program is focussed on two main issues:
- compensation of displacements or loss of bearing capacity of pile foundations;
- increasing the bearing capacity of pile foundations.

The PhD research conducted at Delft University of Technology focuses on the latter issue, the North/Southline research program focuses on the first issue.
The use of grouting to increase the bearing capacity of foundations is not a new technique, nor is it new to compensate settlements caused by tunneling. However, using grouting combined with pile foundations is not so common. Both jet grouting as well as compensation grouting have not been used on (typical Dutch) pile foundations.

General Information North/South metroline
The North/South metroline Amsterdam will use the shield tunnelling method for construction of two (6.5 m metro tunnels. Therefore, due to settlements, complications with some historical buildings founded on pile foundations may occur. To prevent these complications, mitigating measures by means of stabilising the soil using injection-techniques, are planned at these locations. The lack of experience with injection in the vicinity of pile foundations, in combination with the fact that the non homogeneous, soft, stratified soil in Amsterdam has been injected only limited, has led to the development of the “Full Scale Injection Test”.

Scope of the paper
The grouting methods used on the Full Scale Injection Test pile foundations are:
* permeation grouting;
* jet grouting;
* compensation grouting (compaction grouting and fracturing).
The Full Scale Injection Test was completed in October 1999 and the first results will be available during the Congress.
Organisation
The PhD study is conducted at Delft University of Technology at the Foundation Technology sections of the Faculty of Civil Engineering & Geosciences and at the Faculty of Architecture. The Full Scale Injection Test is, because of national interest, a co-operation of:
- Design Office North/South metroline (75%);
- Centre for Underground Construction Studies (in Dutch: COB *, 20%);
- Delft University of Technology (TU Delft, 5%).

*The COB is a co-operation of Dutch contractors, engineering consultants, specialist institutions and others involved in underground construction.

Figure 1. Test Site: Overview.

Principle of increasing bearing capacity of pile foundations
Permeation grouting
By injecting a silicagel under and / or near the pile toe using permeation grouting, the effective pile toe surface is increased. This way the bearing capacity of pile foundations will increase. The grouted soil near the pile toe has a higher stiffness and a better (apparent) cohesion compared to the virgin soil. During the installation of the piles temporary relaxation of the soil adjoining the borehole will occur. The nearer the borehole is to the pile, the higher the settlements due to installation will be.
FEM analysis has been conducted, which can only give an indication of the increase in bearing capacity, because the driving of a pile is nearly impossible to simulate in FEM calculations. Both this FEM analysis as well as analytical predictions show that the bearing capacity of the pile (or its stiffness) can rise up to 50%.

Jet grouting
By jet grouting a small column (1 meter height) next to and under and above the pile toe, the effective pile toe surface is increased. A pile with an enlarged base is created in situ. This way the bearing capacity of pile foundations will increase. The grouted soil near the pile toe has a higher stiffness and a better (apparent) cohesion compared to the virgin soil. During the installation of the piles the soil adjoining the pile will be totally liquefied by the erosion of the jet grouting process. The nearer the column is to the pile, the higher the settlements during the grouting will be.
FEM analysis has been conducted, which can only give an indication of the increase in bearing capacity, because the driving of a pile is nearly impossible to simulate in FEM calculations. Both this FEM analysis as well as analytical predictions show that the bearing capacity of the pile (or its stiffness) can rise over 100%. Settlements due to installation will occur, however they are relatively small and can probably be accepted.

**Compensation grouting (fracturing)**
Fracturing is not being used to increase the bearing capacity of pile foundations but to compensate for settlements. Because of the complexity of fracturing combined with the modeling of pile foundations, no FEM analysis has been conducted so far. Fracturing has up to now been an injection process in which experience plays a key role. Research that is being done at Cambridge University can hopefully be combined with the test results of Amsterdam to be able to make predictions in the near future.

**Compensation grouting (compaction grouting)**
The main principle of the compaction grouting is to increase the (horizontal) effective stresses in the soil. By injecting a very stiff grout in the sand layer in which the pile toes are placed, the bearing capacity of the wooden pile foundations should be increased considerably. In figure 2 the principle is illustrated. Figure 2a shows part of the plan and a cross section of a typical Amsterdam building. The masonry walls are supported by a wooden pile foundation which toes are placed in the first sand layer. Figure 2b illustrates the accessibility of the structure by injection tubes. It shows that when the tubes are installed from the surface problems will occur when piles that are located in the center of the structure have to be reached. This problem can be avoided by working from horizontal shafts. However, these shafts are considerably more expensive.

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**Figure 2a. Principle of compaction grouting**

**Figure 2b. Foundation Accessibility (plan).**

(upper part = plan, lower part = cross section).
Aim of the full scale injection test
The aim of the Full Scale Injection Test is to gain (additional) knowledge concerning the various above mentioned injection techniques, with regards to:
* the influence of (the installation of) the injection body with regards to pile bearing capacity;
* displacement and deformation of the foundation piles;
* soil deformations;
* changes in (effective) soil stresses and water pressure;
* mastering and controlling the injection process;
* the feasibility of some design variants.

Roughly, a distinction can be made between the (possible) negative influences and the (possible) positive influences. During installation of a injection body, settlements and a reduction of bearing capacity may occur, whilst when the injection body has hardened the pile may have both a higher bearing capacity as well as more rigid settlement behavior.

Specific conditions
Normative conditions
To simulate the designed mitigating measures for the North/South metroline, in the design of the Full Scale Injection Test ideally three conditions had to be satisfied. Firstly, the soil conditions of the test site had to be similar to those at the building locations. Secondly the foundation piles should behave correspondingly to those of typical Amsterdam houses. And finally, a TBM should pass the test location to determine the effectiveness of the injection body's. Since none of these three conditions could be satisfied simultaneously, it was chosen to satisfy only the first two, because the aim of the test could be achieved best this way. In addition, the last condition doesn't apply when increasing the bearing capacity of pile foundations is concerned.

Soil conditions
A detailed description of the soil conditions is given in previous papers (ref). Only a brief summary is given here (with NAP = Amsterdam Ordnance Datum, the Dutch reference level for vertical measurements):
* street level at NAP + 2 m; groundwater level at NAP -0.4 m;
* NAP + 2 m to NAP -13 m: Holocene package (respectively sand, rubble, peat, clay, silty/sandy layers, compressed peat);
* NAP -13 m to NAP -16 m: 1st sand layer; cone resistance (CPT) 6 - 30 MN/m²;
* NAP -16 m to NAP -19 m; In-between layer; cone resistance 2 -12 MN/m²;
* NAP -19 m to NAP -28 m; 2nd sand layer; dense to very dense; cone resistance 15 - 45 MN/m²;
* NAP -28 m to NAP -45 m; Eem Clay layer.

The piezometric surface of the 1st and 2nd sand layer is NAP -2 m.

Pile foundations
In general, buildings constructed before 1945 have a wooden pile foundation, placed on the first sand layer. Most (mayor) buildings constructed afterwards have concrete piles, placed on the second sand layer. More detailed information concerning the buildings and damage criteria are given by Netzel et al (1999). Table 1 shows the characteristics of the Amsterdam wooden and concrete foundation piles.
<table>
<thead>
<tr>
<th>Feature</th>
<th>Wooden pile</th>
<th>Concrete pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter tip (mm)</td>
<td>ø 230</td>
<td>Ø 350</td>
</tr>
<tr>
<td>Diameter toe (mm)</td>
<td>ø 130</td>
<td>Ø 350</td>
</tr>
<tr>
<td>Pile toe lever (m NAP)</td>
<td>-13,5</td>
<td>-20,0</td>
</tr>
<tr>
<td>Length (m)</td>
<td>15</td>
<td>22</td>
</tr>
<tr>
<td>Material</td>
<td>Soft Wood</td>
<td>Concrete B35</td>
</tr>
<tr>
<td>Pile toe resistance (%)</td>
<td>75-85</td>
<td>65-70</td>
</tr>
<tr>
<td>Pile shaft resistance (%)</td>
<td>15-25</td>
<td>30-35</td>
</tr>
<tr>
<td>Service bearing capacity (SLS)</td>
<td>135 kN</td>
<td>1000 kN</td>
</tr>
<tr>
<td>Ultimate bearing capacity (ULS)</td>
<td>205 kN</td>
<td>3410 kN</td>
</tr>
</tbody>
</table>

Table 1. Pile Characteristics.

**Construction**

Based upon the above-mentioned desired conditions, a test site was chosen in the northern part of Amsterdam. Here a construction as illustrated in Figure 1 and Figure 3 was built (note the similarity with the Test Pile Project (TPP; Teunissen, 1998).

The construction consists of:
* 9 wooden piles used for permeation grouting (and later on for fracturing);
* 3 wooden piles and 3 concrete piles exclusively used for the jet grouting;
* 6 wooden piles used for compensation grouting;
* hydraulic jacks to adjust the load on the piles;
* various steel profiles to redistribute the load;
* ballast (concrete piles and big bags filled with sand).

During the tests, the jacks were used to maintain the SLS loads on the piles (no redistribution of loads). This way the displacement as a function of time (injection process) at constant load rate were obtained.
Figure 3. Cross Section of the Ballast Frame.

**Monitoring program**

**Introduction**

Because of the large quantities of data that were to occur from the test, very strict regulations regarding these data were given. Some important aspects are mentioned here:

* the distinction that is being made between continuous measurements and measurements with regular intervals; where continuous is described as the lowest possible interval with the prescribed monitoring equipment.
* time synchronizing of the monitoring equipment (every hour);
* required format of the data (time, temperature & reference location registration; units; accuracy);
* presentation (daily digitally (CD-RW) & hardcopy, graphs, concept & final report).

In the next paragraphs, a survey of the measurements / monitoring equipment is given. Generally a distinction is made between the several different stages. This because although some of the monitoring methods are the same for all the stages, the application can differ. Essential for the design of the test is that injection takes place at different distances from the pile (toe).

The pile load test to determine the (ultimate) bearing capacity of the pile and the used monitoring equipment are the same for all stages and are therefore discussed independently.
Pile load tests
The pile load test consisted of applying increments of static load to a test pile and measuring the deflection of the pile. The load was jacked onto the pile by using the dead weight of the ballast frame.
All the piles used in this test were loaded at least 3 times. The first two load tests were conducted before injecting the soil, the third afterwards. Results of the TPP show that the first and second load test can differ considerably; the aim of the first test is therefore to be able to compare representative tests before (2nd) and after (3rd) injection to determine the difference in bearing capacity.

Because the pile behavior is not known exactly before the load tests, the first test was used to determine the appropriate load increments and ultimate bearing capacity $F_{u,l}$. The ultimate bearing capacity is defined as the load at which the vertical pile displacement reaches 10% of the pile toe diameter. The load scheme is shown in table 2. When in the first load test the pile didn’t reach $F_{u,l}$ by step 9, an increment of $+10$ kN / $+200$ kN was used until $F_{u,l}$ was reached.

<table>
<thead>
<tr>
<th>Step</th>
<th>Wooden piles</th>
<th>Concrete piles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test 1</td>
<td>Test 2 &amp; 3</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
<td>0,50 * $F_{u,l}$</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>0,70 * $F_{u,l}$</td>
</tr>
<tr>
<td>3</td>
<td>120</td>
<td>0,80 * $F_{u,l}$</td>
</tr>
<tr>
<td>4</td>
<td>140</td>
<td>0,85 * $F_{u,l}$</td>
</tr>
<tr>
<td>5</td>
<td>160</td>
<td>0,90 * $F_{u,l}$</td>
</tr>
<tr>
<td>6</td>
<td>180</td>
<td>0,95 * $F_{u,l}$</td>
</tr>
<tr>
<td>7</td>
<td>190</td>
<td>1,00 * $F_{u,l}$</td>
</tr>
<tr>
<td>9</td>
<td>200</td>
<td>1,05 * $F_{u,l}$</td>
</tr>
<tr>
<td>9</td>
<td>210</td>
<td>1,10 * $F_{u,l}$</td>
</tr>
<tr>
<td>&gt;9</td>
<td>dF=+10</td>
<td>dF=+0,05 * $F_{u,l}$</td>
</tr>
</tbody>
</table>

# Approximate calculated ULS (ref. table 1)

Table 2. Load increments for pile load test.
Special monitoring equipment

Among the monitoring equipment used are water pressure meters, soil stress meters, load cells, extensometers (figure 8), inclinometers, CPT's and horizontal and vertical levelling equipment (figure 7). Most of the equipment is used in a regular way, but an exception is made for the piezometers and the soil stress meters. Results of the TPP showed that it was possible to integrate them in the pile, as shown in figure 5 and figure 6, with satisfactory results. A big advantage is that when this equipment is installed this way, the soil isn’t disturbed like when installing them in a borehole. The piezometers could be installed by pushing them into the soil; therefore boreholes are not used.
Monitoring frequency
The used frequency differs throughout the test period. During installation of the TAM’s and during injection, most of the monitoring equipment is read out every minute. When no injection takes place the frequency is reduced at least 5 times. After injection the frequency is reduced each week. Monitoring stops four weeks after injection (unless the results show further monitoring is necessary).

The full scale injection test
Permeation grouting near wooden piles
In figure 9 the plan of the permeation grouting injections is shown. Figure 10 shows some characteristic cross-sections.
Tube à manchettes (TAM’s) were used to create a more or less spherical permeation grouting body either just beside / under the pile toe or surrounding the pile toe. Only wooden piles (1st sand layer) were used. During the process of installing the TAM’s and during the injection process vertical displacements of the piles were intensively monitored. Also monitored were the horizontal stresses and the water pressure, the first only in the piles, the latter both in the piles as well as in the soil. The piezometers in the soil were placed at increasing distance from the injection points, to monitor the influence of the injection pressures. Note that during the installation of the TAM’s, the placing accuracy is of the utmost importance.
Figure 10. Permeation grouting near pile toes.

**Jet grouting near wooden & concrete piles**

Before jet grouting four CPT’s (S1 - S4) were taken at varying distance from the intended edge of jet grouting body A. After jet grouting another four CPT’s (S5 - S8) are taken to determine the differences. For the creation of the jet grouting bodies, a one and two-phase jet grouting system (double jet) were used. Injection bodies A through D reach from NAP -10 m to NAP -35 m. The deeper part (from NAP -28 m) is used to test the groutability of the Eemclay layer, a/o. because in the station design struts are integrated there (De Wit, 1999).

Two injection bodies with a diameter of 1 meter (A & B) and two with a 2-meter diameter (C & D) are created respectively. During the grouting process both horizontal as well as vertical displacements of the piles are intensively monitored. Also monitored are the displacements at depth (extensometers and inclinometers), horizontal stresses and the water pressure.

Two injection bodies (E&F) with a height of 7 meter were created near the toes of the concrete piles. Three injection bodies (G, H, I) with a height of 1,0 meter were created near the toes of the wooden piles.
Compensation grouting near wooden piles
This stage consists of three different injections. The first is a supplement of the permeation grouting and consists additional fracturing, the second is fracturing on 3 piles and the third compaction grouting on 3 piles.

Permeation grouting and fracturing (supplement of the permeation grouting)
Some additional permeation grouting bodies were created under the toes of the piles of the permeation grouting stage. After one day (the silicate has hardened) fracturing was used to crack the soil under the permeation grouting bodies (and the piles) and lift them.
The permeation grouting bodies were created to spread the pressures that are induced by the fracturing. This way it was prevented that the soil is pressed between the piles.

Fracturing
Figure 11 (left) shows a characteristic cross-section for the fracturing (compensation grouting). For the fracturing a plastic grout was used to crack the soil horizontally. The upper and lower manchette were used for preconditioning, the middle was used for the actual lifting of the soil. Lifting took place in steps of 2-3 mm / injection until a total uplift of approximately 25 mm was reached.
In this stage, no permeation grouting was used. This way the spreading effect of the permeation grouting bodies can be evaluated.

Compaction grouting
In figure 11 (right) characteristic cross-section for the compaction grouting is shown. For the compaction grouting a very stiff grout was used to tighten the soil. The horizontal effective stresses were increased this way. The injection tube is used one time only for grouting. Because of the high stiffness of the grout, the tube is pulled while grouting with high pressure.
Predictions

Introduction
Because of the complex nature of injecting in soft soil near pile foundations, predictions have been made with increasing convergence.

First an extensive desk study has been made. This was followed by analytical, empirical and 2D and 3D FEM analysis.

Desk Study
The desk study consisted of a literature study and interviews with contractors and engineering agencies. The result of the study showed that there was very little experience with any of the injection techniques applied near to deep pile foundations. Most of the experience lies with the contractors, and as a consequence (to protect their investments in technical developments) little has been published. Also, only very limited numerical analysis concerning soil stresses or deformations has taken place.

Analytical Predictions
Determining of the dimensions of the permeation grouting have been made using Raffie & Greenwood (1961). For the jet grouting, Kauschinger (1992) will be applied (in a somewhat altered form). For the compaction grouting, Vesic is used.

No relations were found for the influence of any of the injection bodies on pile foundations.

FEM Analysis
Both 2D (PLAXIS) FEM analysis as well as 3D (ABAQUS, DIANA) is being conducted now. Problems that are being encountered especially relate to the modelling of the (driven) pile.

Most likely, the FEM analysis is ideal for interpreting the test results.

References
Raffle, J.F., D.A. Greenwood, 1961. The relation between the reological characteristics of grouts and their capacity to permeate soil, Proceedings of the 5th International Conference on Soil Mechanics, Paris
MONITORING WITH GROUND PENETRATING RADAR

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Abstract
An overview of the background, operation and diagnostic possibilities of the ground penetrating radar is given. First the basic physical principles behind the “seeing” with this instrument are discussed, followed by a technical description of how the measured electromagnetic signal is translated into an image of the shallow subsurface of the earth. These theoretical considerations lead to a strategy for how a ground-penetrating survey should be carried out in practice. The meaning of the image in terms of contrast of dielectric constants is discussed and the relations of these contrasts to the actual physical contrasts: the water layer in the earth, rot in a wooden pole, water appearance in concrete. The paper is concluded with some practical case studies.

Introduction
Looking into the deep earth, up to 4 km, with the aim of detecting potential oil and gas reservoirs is the occupation of the applied geophysicist. By making an explosion he sends sound waves into the earth and records with small microphones, the so-called geophones, when and how strongly the reflected sound wave returns to the surface of the earth. This measuring technique is known as the seismic method. From the outcome of these measurements, the geophysicist is able to determine the structure of the earth’s subsurface. The physical principle of the seismic method is simple: the sound wave that arrives later comes from deeper regions in the subsurface and the amplitude strength of the wave is an indication of the contrast, hence the nature of the geological layers. The results are presented in a seismogram; a space-time map of the reflected sound waves which looks like a cross-section of the earth with a resolution in the order of 10m. Then the geophysicist, together with the geologist, indicates where to drill for oil or gas. The expensive drilling operation is no longer an educated guess, but follows from a sound technical measurement.

So much for our interest in the deeper layers. The a non-destructive tool is also needed to monitor the constitution of the shallow subsurface, down to 20m, for example to perform an environmental investigation or to establish the soil-mechanical conditions. In this case, we also could use sound waves, but they have a restriction in resolution. In the last twenty years ground-penetrating radar or georadar has become popular for shallow-subsurface monitoring. The georadar employs electromagnetic waves to delineate the subsurface. The waves in this case are different, electromagnetic waves instead of sound waves, but the physics behind the measurement is the same: reflected waves that originate from the deeper parts arrive later. For a good historic account of georadar measurements, I refer to the pioneering work of Annan and Davies (1976). A good survey of the existing literature can be found in Grandjean and Gourry (1996). In the context of geotechnical applications, I refer to the excellent work of Saarenketo et al. (1992). In the next section, I illustrate the principle of the georadar with the well-known common radar that is used to detect targets at a distance.

Object detection with electromagnetic waves
Radar is an acronym for RAdio Detection And Ranging. In World War II, this apparatus was extensively used to detect the whereabouts of the enemy. The working of radar is best illustrated by the schematics of figure 1.
The source antenna launches an electromagnetic wave in the direction of the object, whereupon the receiver antenna registers the arrival time of the reflected wave. In most radar applications, source and receiver coincide. The distance from the radar to the object then simply follows from half the arrival time of the reflected wave multiplied by the velocity of the electromagnetic wave in the surrounding medium, in this case the air. However, in the case of the georadar, matters are much more complicated, as is schematically depicted in figure 2.

We are situated with our source and receiver at the surface of the earth and our object is buried in unknown earth layers. In this situation how are we able to detect the location in depth and the form of our object? We could start in the following way. Let us assume we know the layers in the earth and the propagation velocity of the electromagnetic waves in these layers. Then we can compute the path of the travelling wave and thus the location of our object, but how do we know where the layers are and how do we determine the corresponding velocities? Well, we can also 'see' the layers in our radar measurements.
because the contrast in propagation is the reason why a wave is also reflected back to the surface from the interface between the layers and will arrive sooner than the reflected wave from the object. The amplitude strength of this wave is directly related to the velocity contrast. How convenient, but how difficult to solve. We are confronted with a problem with a hierarchy of nested complexities: to determine the object, which is visible as a velocity contrast we have to know the velocities. We have the same problem with the seismic method and it is known as the velocity paradox. To complicate the matter further, we have assumed that the waves follow simple straight lines, like light rays. If this were the case we could direct the transmitting wave and receive the reflected wave in a preferred direction and, in doing so, be able to scan the object and hence determine its form. In practice, however, the source is emitting waves in all directions according to a certain radiation pattern and in the same way, the receiver collects the reflected waves from all directions, again according to a certain pattern. To be able to scan the object, to simulate the transmitting and receiving beam, we have to combine the emitted waves from several sources and the reflected waves from several receivers. Operating in this way and giving different weights to the transmitting array and the receiving array, we can probe the object. We must realise that such an approach requires many sources and receivers or we have to repeat the experiment many times for different locations. Indeed this strategy is followed with the seismic method. However for investigations of the shallow earth in general from an economic point of view this is not feasible. Enough questions, let us see in the next section how we can arrange the georadar measurement to deal with the problems.

The georadar theory
The theoretical framework behind the propagating of waves is furnished by system of coupled partial differential equations. To simplify the discussion I employ a simplified generic notation. In general, the equations that govern the wave propagation are given by

\[ D_1 \partial_x F_1 + M_1 \partial_t F_2 = S_1, \]
\[ D_2 \partial_x F_2 + M_2 \partial_t F_1 = S_2. \]

In Eqs. (1) and (2) the quantities \( F_1 \) and \( F_2 \) represent the propagating wave fields. For example in seismics they represent the pressure and the particle displacement, in electromagnetic wave propagation they stand for the magnetic and electric wave fields, respectively and the system of equations is known as the equations of Maxwell. The operator \( \partial_x \) means partial differentiation with respect to space and \( \partial_t \) signifies partial differentiation with respect to time. The operators \( D_1 \) and \( D_2 \) are tensor operators and typical for the problem at hand. For the electromagnetic case, they are both rotation tensors. The operators \( M_1 \) and \( M_2 \) are medium operators and, in general, also tensors. They constitute the medium in which the wave fields are propagating. In fact, they are the targets of our investigations. There would be no wave field, if there were no sources. Their action is accounted for on the right-hand sides of Eqs. (1) and (2) by the quantities \( S_1 \) and \( S_2 \). The structure of the wave equations is clear. A change in space expressed by the spatial differentiation leads to change in time expressed by the temporal differentiation. In this way we see how wave propagation, as a phenomenon that changes in space in time, is organised in the wave equations.

In our geophysical measurement we consider the action of one type of source say \( S_1 \). Further, we assume that \( S_1 \) is a point source given by
\[ S_1 = s_1 \delta (x - x_s), \]  

where \( \delta \) denotes the Dirac delta function operative at the source position \( x_s \). Due to shift invariance in time we consider the wave equations in the frequency domain with the benefit that \( \partial_t \) is replaced by \( j\omega \) (\( j = \sqrt{-1} \)). Then Eqs. (1) and (2) are rewritten as

\[
D_1 \partial_x F_1 + j\omega M_1 F_2 = s_1 \delta (x - x_s),
\]

\[
D_2 \partial_x F_2 + j\omega M_2 F_1 = 0,
\]

where \( F_1 = F_1 (x \mid x_s, \omega) \) and \( F_2 = F_2 (x \mid x_s, \omega) \), indicating that the wave fields at the position \( x \) origin from a source at \( x_s \). The aim of the geophysical measurement is to determine the medium parameters from the knowledge of the wave fields. Let us assume that we know \( M_2 \) and that the target of our investigation is \( M_1 \). Next we define the incident wave field \( F_{1,\text{inc}} \) and \( F_{2,\text{inc}} \) that propagates in a known background medium, the host medium, with the same source as in the actual case. The corresponding wave equations are

\[
D_1 \partial_x F_{1,\text{inc}} + j\omega M_1^B F_{2,\text{inc}} = s_1 \delta (x - x_s),
\]

\[
D_2 \partial_x F_{2,\text{inc}} + j\omega M_2 F_{1,\text{inc}} = 0.
\]

The difference between the actual and the incident wave field is denoted as the scattered wave field \( F_{1,\text{set}} \) and \( F_{2,\text{set}} \) follows from

\[
D_1 \partial_x F_{1,\text{set}} + j\omega M_1^B F_{2,\text{set}} = j\omega (M_1^B - M_1) F_2
\]

\[
D_2 \partial_x F_{2,\text{set}} + j\omega M_2 F_{1,\text{set}} = 0.
\]

Observe that the actual wave field has been decomposed into an incident and a scattered wave field with the provision that \( F_1 = F_{1,\text{inc}} + F_{1,\text{set}} \) and \( F_2 = F_{2,\text{inc}} + F_{2,\text{set}} \). The scattered wave field is not a physical wave field, but a computational construct with a source that finds its origin in the contrast between the actual and background medium (cf Eq. (8)). With this decomposition we are able to formulate our geophysical strategy in operational terms. Using the global form of the reciprocity theorem (Fokkema and van den Berg, 1993) the scattered wave field can be expressed in terms of the contrast as follows

\[
F_{2,\text{set}} (x_r \mid x_s, \omega) = \int G_2 (x_r \mid x_s, \omega) j\omega (M_1^B - M_1) F_2 (x \mid x_s, \omega) \, dv,
\]

in which \( G_2 \) is known as the Green's function of the background medium and is defined as \( G_2 = s_1 F_{2,\text{inc}} \).

The geophysical problem is now formulated as follows: we measure \( F_{2,\text{set}} \) at the receiver position \( x_r \) due to the action of the source at \( x_s \), we know the Green's function of the background medium, are we able to determine the medium contrast \( (M_1^B - M_1) \)? That is still problematic since we do not know the actual wave field \( F_2 \) in the contrast domain. Actually, this
The determination problem is non-linear and again we are confronted with the velocity problem discussed in the previous section. Usually the problem is linearised by assuming that the actual wave field in the medium is the same as the incident wave field, hence

\[ F_2 \approx s_1 G_2. \]  

This is the so-called first Born approximation. With this approximation Eq. (10) is written as

\[ F_2^{\text{scat}} (x_r \mid x_s, \omega) = \int G_2 (x_r \mid x_s, \omega) j \omega (M_1^B - M_1) G_2 (x \mid x_s, \omega) s_1 \, dV. \]  

In the case of the georadar \( F_2^{\text{scat}} \) represents the electric wave field and is measured as a voltage with the receiving antenna, which is usually an electric dipole. The quantity \( s_1 \) represents the spectrum of the source antenna and is associated with the enforcing current of the antenna system. The medium contrast of interest is the dielectric or permittivity contrast

\[ (M_1^B - M_1) = (\varepsilon_r^B - \varepsilon_r) \varepsilon_0, \]

where \( \varepsilon_0 \) is the permittivity of air and \( \varepsilon_r \) is the dimensionless relative permittivity. There is no contrast in the electric permeability and for the whole medium \( \mu_0 \) of air is assumed. The propagation velocity in air \( c_0 = 3 \times 10^8 \text{ m/s} \) or related to nanoseconds \( c_0 = 0.3 \text{ m/ns} \). The propagation velocity in the other media follows from \( c = 0.3/\sqrt{\varepsilon_r} \text{ m/ns} \). From the relative permittivities presented in Table 1, some insight is obtained regarding the contrast we may encounter in practice. A water layer will be clearly visible in most geological environments. For media without losses the permittivity is real, however in real life this is in general not the case, then \( \varepsilon_r \) is complex. The imaginary part is related to the losses and expressed as the conductivity \( \sigma \). The conductivities for the different materials are also presented in Table 1. This constant is the reason that the propagating wave field is attenuated and hence is the limiting factor in the penetration depth of the georadar signal.

### Table 1. Electrical constants for some common materials measured at 100 mHz.

<table>
<thead>
<tr>
<th>Material</th>
<th>Relative permittivity ( \varepsilon_r )</th>
<th>Conductivity ( \sigma ) (mS/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Distilled water</td>
<td>80</td>
<td>0.01</td>
</tr>
<tr>
<td>Fresh water / Sea water</td>
<td>80/80</td>
<td>0.5 / 4000</td>
</tr>
<tr>
<td>Dry sand / Saturated Sand</td>
<td>3-5 / 20-30</td>
<td>0.01 / 0.1-1.0</td>
</tr>
<tr>
<td>Dry asphalt / Wet asphalt</td>
<td>2-4 / 6-12</td>
<td>1-10 / 10-100</td>
</tr>
<tr>
<td>Dry concrete / Wet concrete</td>
<td>4-10 /10-20</td>
<td>1-10 / 10-100</td>
</tr>
<tr>
<td>Dry clay / Saturated clay</td>
<td>2-6 / 15-40</td>
<td>1-100 / 100-1000</td>
</tr>
</tbody>
</table>

In Eq. (12) the last Green's function on the right-hand side represents the wave propagation in the background medium from the source antenna to the object, while the first Green's function covers the propagation from the object to the receiving antenna. The process of imaging the shallow subsurface by using the measured georadar response entails integral operators that operate on the left-hand side of Eq. (12), annihilate both Green's functions and, in doing so, map the contrast and its form to the right depth location in the medium. We observe that this operation would require many sources and receivers as...
was indicated in the previous section. In the next section the use of georadar measurement in practice will be discussed.

**Georadar data acquisition in practice**

In figure 3 a schematic view of the so-called common-midpoint configuration (CMP) is depicted. This approach is used to obtain information on the propagation velocities in the different geological layers (Van der Kruk et al., 1999). The offset of both the source and receiver is increased away from a fixed midpoint. At all distances between source and receiver with a fixed midpoint, most of the energy that is reflected by not too steeply dipping interfaces originates from the same subsurface position below that midpoint. Different events can be distinguished in the received signal as is schematically shown in Figure 4.

In figure 5 an example of a CMP measurement with a 900MHz PulseEKKO 1000 radar system from Sensors & Software is presented. The radargram (compare seismogram) shows the two-way travel time as a function of offset. Note that the velocity in the ground just under the surface changes when the offset is 2.5m, which is observed from the change in the slope of the ground wave. The numbered events can be compared with the corresponding numbers in the schematic pictures of figure 4.

A complete coverage of the subsurface would entail a repeated CMP measurement for every midpoint of
the surface. This would then result in the multiple-source multiple-receiver acquisition approach mentioned before. In practice, this is not feasible. The practical field surveys are conducted in the common-offset mode. In figure 6 a schematic view of this acquisition strategy is presented. This strategy is used to detect objects or to investigate lateral changes in the subsurface. A fixed distance separates the source and receiver antennas. Subsequently the antennas are moved for the next measurement. This is repeated along the survey line. The method is fast and therefore cost-effective. In figure 7, the results of a pipe detection survey in Delft are shown. For this survey we employed a 200 MHz system. The subsurface at this position is due to the presence of clay not ideal for a georadar measurement. On the left side of figure 7 the common-offset results are presented with the maximum position at 41m, on the right-hand...
side the common-midpoint result at 41m is shown. We clearly recognise the contributions from the air and ground to the common offset. The steeply dipping slopes of the hyperbolas indicate the presence of buried pipes. The appearance of the object as reflection hyperbolas in the radargram are best explained with the schematic pictures of figure 8.

Assuming that there are no velocity changes between the surrounding material, the host, and the object, and assuming that the object is small compared to the wavelength used, the reflection pattern is a perfect hyperbola. In figure 8 (a) the actual location of the object with respect to the source and the receiver

Figure 8. The shortest travel path from source and receiver to and from the object (a) and the corresponding schematic space-time diagram (b).
antenna is shown, together with the shortest travel paths of the electromagnetic signals. The consequences for the radargram are depicted schematically in figure 8 (b). The arrival times for the reflection events are positioned below the mid-point position between source and receiver and this causes the hyperbolic appearance in the radargram.

In a low-loss environment, the sand dunes along the North Sea coast of the Netherlands, measurements have been carried out with three different centre frequencies: 110, 225 and 450 MHz for the same location and are presented in figure 9. The level of detail clearly increases with increasing frequency.

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Figure 9. Comparison of common offset measurements with a georadar system on the same location for different center frequencies.
```

The depth of penetration of the 450 MHz is less than that of the 225 MHz antenna. The event at about 50 ns, visible for all three frequencies is identified as the ground-water level. An excellent overview of typical radar-stratigraphic features that are characteristic for the environments of the Netherlands can be found in Van Overmeeren (1996). At a low frequency, a low resolution and only horizontal reflection events are visible. Eastward dipping layers can be recognised at the high frequency of 450 MHz. Furthermore, we observe that in the 450MHz radargram smaller objects, shown as localised disturbances, become visible. The ultimate goal would be to combine these different radar views of the same subsurface to obtain an improved image of the subsurface at this position. The data cannot simply be added. A

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Figure 10. Rot detection in wooden pole with a 1200MHz antenna.
Courtesy Sensor & Software
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much more sophisticated approach is required and this is still one of the topics of our ongoing research. In figure 10 a georadar survey is shown of the mapping of the internal structure of a wooden pole. Variations in water content most often indicate that rot or insect infestation is present.

Figure 11 shows the georadar results of a survey with a 450 MHz system. The data were collected on a major road in Kuala Lumpur. The contractor needed to know where he could safely dig without hitting a utility while installing fibre optic communication cables. The different cables and pipes are dominantly present through their characteristic hyperbolas.

In figure 12 results are presented of a 1200 MHz georadar system that was used to detect a void around a sewer. The wet areas are clearly visible, as is the regular pattern of the steel enforcement.

Conclusions

![Utility mapping with a 450MHz antenna](image)

*Figure 11. Utility mapping with a 450MHz antenna. Courtesy Sensor & Software*

The georadar is an important tool in geotechnical investigations, but likewise it also important to know how the object of investigation appears in the radargram.

Acknowledgement

The research reported in this paper is being carried out in the framework of an STW research grant (DMB.3649), which support is gratefully acknowledged. The author is appreciative of the co-operation with Peter Annan of Sensors & Software and the help from Jan van der Kruk and Evert Slob. Finally, the co-operation with TNO-NITG is acknowledged.
Figure 10. Sewer mapping with a 1200MHz antenna. Courtesy Sensor & Software

References

EXPERIMENT WITH FOAM CONCRETE

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Introduction
This presentation focuses on an experiment with an alternative method for foundation rehabilitation conducted in the Zuideramstel quarter in Amsterdam. Part of the buildings in this quarter come under the “20-40 belt” of Amsterdam, an area with many foundations of wooden piles which were not designed to withstand the impact of negative shaft friction.

The key objective of the experiment was to reduce the settling speeds which resulted from a lack of bearing capacity of the pile foundation – in turn caused by negative shaft friction. This reduction was achieved by replacing the fill material by a lighter substance, in this case foam concrete. To label the operation “foundation rehabilitation” is therefore incorrect, as the foundation itself was not corrected in any way.

Reason for the experiment
Around the turn of the century, the Municipality of Amsterdam planned to meet the ongoing housing need by constructing houses in the surrounding polders; this plan was to be realised from around 1920 onward (that is why it was called the “20-40 belt”). The first decades of the century were spent in a preparatory frenzy of activities, such as raising the polder levels about 4 to 5 metres by sand fill. The subsidence resulting from this fill is still in process as secular settling in the subsoil (see figure 1: typical cone penetration test in the Zuideramstel quarter).

The often block-shaped four-storey buildings were founded on a framing of concrete beams resting on wooden piles, often capped by concrete sleeves, in the upper layer of sand (some 12.5 m below sea level) (see for details of the foundation figure 2 on the next page).

At the time, the phenomenon of “negative shaft friction” seems to have gone unrecognised, or perhaps its consequences were grossly underestimated. In any case, the designed pile foundations were not capable, with any margin of safety, of tolerating the forces.
affecting them. The buildings therefore started to subside. The speed with which this happened was largest immediately after the construction, then slowed down with time. The absolute subsidence of the buildings varies from 100 to 500 mm. Such subsidence will not be disastrous as long as the settling depths of the buildings show no mutual variations; little damage will then be incurred, and safety remains guaranteed. The damage is done when adjoining buildings subside at different paces, which will often be the case; this may well threaten the desired safety margins. Even now, the unequal subsidence resulting from negative shaft friction continues to damage the buildings. This may eventually lead to demolition.

Around 1993, a working group revived a concept which dated back to 1984, and brought it to the attention of the municipal politicians. The idea was that negative shaft friction to the pile foundations could be decreased by the partial replacement of the sand fill by foam concrete. This renewed attention led to the implementation of an experiment with six buildings in the Uiterwaardenstraat in Amsterdam, supervised by the Housing Department of Amsterdam. The sand fill beneath these 6 houses was replaced in 1995; as it was followed up by monitoring of the settling process over a period of years, it was not until 1998 that the experiment and its results were reported on.

**Objective of the experiment**

The experiment was aimed at the investigation of the extent to which the settling speed of buildings – the foundations of which were affected by negative shaft friction – could be decreased by the (partial) replacement of sandfill by a lighter material, which in this case consisted of foam concrete. The sandfill could not be removed entirely, since it also provided horizontal supporting pressure for the pile foundations and acted as a precipitation drain, amongst other things. A pre-condition for the experiment was that the effect of the decreased soil load on the settling speed be studied in some detail. This pre-condition also played a role in the selection of the method of execution, as we will see on the next page.
Implementation of the experiment

The diagram in figure 3 (site plan of the houses) indicates a levelling of the fronts, showing the relative settling differences of the buildings. Prior to the test, the subsidence and also the settling speed at the location of the tested buildings evidently was more pronounced than in the rest of the housing block. The (partial) replacement of the sand fill was aimed at levelling out these differences in settling speeds.

Figure 4 shows a cross-section of a main wall, showing its state before and after the experiment. Some 1.5 m sand was removed and replaced by a layer of foam concrete some 1.75 m thick (relieving the soil load by about 13.2 kN/m²). A 1.3 m thick layer with a weight by volume of 5 kN/m³ was incorporated in this layer. The general rule saying "The lighter, the cheaper" worked in our favour, since a second rule, saying "The lighter the material, the less of it is required to achieve the same relief of soil load aimed at", also applied here.

Furthermore, the structure of foam concrete is such as to absorb very little water (some 5%). Its low weight by volume makes it float on the water; when the concrete is applied below the current groundwater table, the upward water pressure can therefore also be deployed. We deliberately chose to disregard the upward force of the groundwater for the purpose of this test, because we wanted to be able to measure the effect of the relief of the soil load on the settling speed in greater detail, without interference from the effect of the groundwater pressure. That is why the foam concrete was kept apart from the construction (see insulation materials near the pile and the foundation beam), and a layer of heavy foam concrete (20 kN/m³) was applied to prevent buoyancy (equilibrium fill). It should be emphasised that, with a normal works, the choice should be made in favour of using the upward pressure of the groundwater and in favour of connecting the concrete fill to the foundation itself. This will result in a thinner layer, requiring only a single, lighter, kind of foam concrete. The only reason why the experimental method differs from the common one, is to gain the most unbiased monitoring results possible. It should furthermore be noted that foam concrete is not only available in low weights by volume, but also has the following beneficial characteristics:

*Foundations*
The material is cohesive and thus capable of absorbing forces. This is important with a view to the exertion of horizontal supporting pressure both during and after the execution of the works.

The material absorbs water only to a limited extent and can therefore also improve the relative humidity in the crawl space.

The material represents an acceptable environmental load from an ecological point of view.

Figure 5 indicates a cross-section of the front of the building containing the realised replacement. Steel sheet piling some 4.0 m long was driven at the front to prevent the street surface from subsiding, and to reduce the influx of groundwater when the groundwater table would be lowered (by pumping). The sheet piling had originally been planned closer to the front, but the presence of cables and pipelines as well as bay windows made it necessary to keep to a minimum distance of 2.5 m. This long distance to the house fronts required a rather expensive strutting. To retrieve these costs to some extent, the contractor decided to use permanent formwork.

Figure 6 shows a cross-section of the back front of the building. A (permanent) wooden sheet pile wall of some 3.0 m was sunk there for the same reasons as the one to the front.

**Measuring results**

Figure 7 shows a plan of the foundation of the test objects, including a cross-section and a longitudinal section, the latter indicating that the transitions to the untreated adjoining buildings were fitted with slopes. For the monitoring programme, three measuring bolts were placed in every main wall of the tested buildings, one in the front wall, one in the back front wall and one inside the house. They were measured prior to, during and after the execution, thus monitoring the settling process during the works from start to finish. The graph in figure 7 shows the bolt measurements per front wall.

Figure 8 shows the measuring results for a single bolt, no. 7, indicating an aspect which, to some degree, was common to all measuring bolts: the spectacular phenomenon of the buildings' rising, between start and finish of the excavation works, to the extent of 5 to 7 mm in about 7 weeks' time! This observation
alone proved that the replacement was successful. After the foam concrete had been inserted, the subsid­ence of the houses resumed, although at a slower speed once the pumping was stopped. Lowering the groundwater table increased the negative shaft friction, which in turn speeded up the settling process. Allowing the groundwater table to rise decreased the negative shaft friction and reduced the settling speed. (The lowering of the groundwater table had a lagging impact on the subsidence behaviour of the adjoining houses: the faster settling speeds prompted by the lowering kept that pace for some time before returning to their former level).

A closer look at the rising as indicated in Figure 7 reveals that the rising was most pronounced in the middle, diminishing closer to the adjoining sites which remained untreated. The fact that the effects of the excavation gradually decreased nearer to the adjoining, untreated buildings was considered an additional benefit of this method. Such transitions tend to be more abrupt with traditional methods of foundation rehabilitation (where additional piles are inserted). The measuring bolts 10 and 19 offer a view of the changes in settling speed as they occurred before and after the replacement. These two measuring bolts had been installed in the houses, and were being monitored, long before the project was initiated. Measuring bolt 19 was positioned in the transition between treated and untreated sites, whereas no. 10 was in the centre of the treated site.

Figure 9 shows the changes in the settling process as measured via measuring bolt 19. Two regression lines were drawn to estimate the change in settling speed, for which purpose it was assumed that the subsidence behaviour would occur in a mostly linear way, as it is a derivative from the ground level subsidence. As the latter is entirely caused by secular subsidence in the subsoil, it may be assumed that this behaviour can be estimated reasonably accurately by way of a straight line. The settling course as measured via measuring bolt 19 showed very little difference between the settling speeds prior to and after the test. This was expected, however, since this particular bolt was positioned in the transition to the adjoining buildings (see figure 7).
testing site. With this bolt, too, two regression lines were calculated and incorporated in a graph (see figure 10). These lines show that the settling speed was decreased from about 2.2 mm/year to about 1.3 mm/year, which is a considerable improvement. Environmental and Geo-technical Research Consultants Amsterdam (OMEGAM), a company which participated in the project, analysed all measuring bolt results, and concluded that the decrease in settling speeds averaged some 0.8 mm/year. OMEGAM expected on geo-technical grounds that the settling speeds would be reduced further over time.

Figure 11 shows the link between the achieved soil load relief and the reduction in settling speeds which can be achieved (source: OMEGAM). This connection has to be regarded with some reservation, as it is really only valid in the circumstances prevailing at the test site, and needs further confirmation from experiences elsewhere. This graph should therefore be used as a guideline only.

**Advantages and disadvantages of the method**

**Advantages**
- Recalculations by the contractor showed that a “common” operation is about 50% cheaper than a traditional rehabilitation of a foundation (18,000 versus 36,000 Euro per house). “Common” is used here in the sense of leaving out the insulation materials, which were meant to keep the foam concrete separate from the building itself, and an equilibrium fill (application of heavy foam concrete), as well as replacing the steel sheet piling by a (permanent) wooden sheet piling to the front. Costs may also be reduced, wherever possible, by omitting the use of sheet piling altogether.
- It will generally not be necessary to evacuate the inhabitants. If the buildings remain in use, some noise nuisance is to be expected, in consequence of the work carried out beneath the ground floor. This should be a point of attention during the execution of the works.
Transitions from treated to untreated buildings are smooth; the method is therefore quite suitable to treating only sections of blocks of buildings. The more abrupt transitions in traditional rehabilitation of foundations carry a greater risk of additional damage to adjoining buildings. See Figure 7.

The humidity in the crawl space is improved by ensuring that the foam concrete be installed at a minimum of 50 mm above the highest known groundwater table.

Disadvantages:

- The settling process is slowed down, not stopped. That is why, for monuments which have to be preserved for posterity at all costs, a more preferable method is the full rehabilitation of the foundation to bring the subsidence to a complete standstill. (In Amsterdam, this means driving piles down to the second sand layer, which has a larger bearing capacity). In such cases, one has the obligation to provide the adjoining buildings with such a foundation, too; otherwise the remedy may be worse than the disease.

- The conditions of the demolition and excavation works in the crawl space are rather degrading; the demolishing of surface concrete which is still intact, in particular, is a dangerous job requiring professional supervision. On the other hand, excavation can make it very easy to remedy certain shortcomings in the crawl space, such as replacing leaking (or lead) water pipes, sewage pipes, decayed beams/floor parts, which can now be performed in an upright instead of a prone position.

- The number of buildings which can be dealt with simultaneously has been limited to about six, for reasons of construction. A rehabilitation project which involves more than six buildings will require a phased approach. During the preparatory stage of our project, detailed calculations were made with regard to the stability of the piles during and in the final stages of the works. The handling of more buildings at the same time might cause problems of stability in the foundation.

- A large-scale block-by-block application of foam concrete beneath buildings might have an adverse effect on the draining function of the sand fill with regard to precipitation.

Conclusions

The objective of the pilot project as formulated beforehand was to demonstrate the theoretically presumed effect of the method on the reduction of the settling speed in the field. On the basis of the conducted measurements it may be concluded that this objective has been achieved. It may also be concluded that this innovative — and cost-effective — method of correction is a welcome supplement to the corrective methods already available. The method can be deployed exclusively with buildings of which the foundation has no other defects, apart from a lack of supportive capacity resulting from negative shaft friction. It is eminently suitable to levelling of the mutual differences in settling speed between buildings which are part of a larger whole, such as a block of houses. Practical experience will have to be gained to strike the right balance between the volume of sand fill to be excavated and the desired reduction in settling speed.

It should furthermore be noted that the costs of the method may be reduced further, for example by letting the thickness of the excavated layer taper off from the front to the back. The impact of negative shaft friction is in principle always larger at the front than at the back, due to the continuous raising of the street level to the desired height, which in time also contributes to the thickness of the sand fill in front.
Following the experiment, the foundations of the adjoining buildings (Uiterwaardenstraat 5 through 19) were also corrected by partially replacing the sand fill by foam concrete. These houses were basically very similar to the ones involved in the pilot project, but it is interesting to assess the performance in a true project based on the outcome of the experiment. No measuring results are currently available, however, to observe the impact on the settling speed.

This project consisted of 8 buildings directly adjoining the 6 buildings from the experiment. The activities were conducted in two stages of 4 houses each. Stability calculations during the implementation stage of the project had indicated that a single-stage operation for 8 houses would not be feasible. Contrary to the experiment, the layer of foam concrete here was not kept separate from the construction and could be used to mobilise the upward pressure of the groundwater. The groundwater pressure will introduce moments which cause tensile stress and compressive stress in the foam concrete. With a view to this tensile stress, as well as shrinkage stress, the foam concrete should be reinforced. Possibly occurring shrinkage cracks would be expected to originate beneath the main walls, where the cross-section was weakened by some 15% due to the presence of concrete pile sleeves. It was decided to have the layer of foam concrete rest on two points, like a beam, and to let the shrinkage cracks originate from these two supporting points. Aragrid, which is a reinforcement made from aramid fibre, was used to reinforce the upper side, in 65 mm strips applied every 150 mm crs (centre to centre) across the top, with a 50 mm covering (see figure 12).

For practical purposes, the concrete layer was applied with a thickness of some 1.60 m. The contractor wanted to be able to drive a small excavator underneath the foundation beams, which therefore had to clear a space underneath of at least 1.50 m. The top side of the layer of foam concrete was determined by the current groundwater table.

The realised relief of the soil load, including the upward pressure, thus amounted to some 23 kN/m². As the graph in Figure 11 demonstrates, this brought the achievable reduction in settling speed to 1.5 mm/year, far more than the reduction observed with the test buildings. The transition to the testing site was considered gradual enough not to expect problems in this field. It was further decided to excavate slopes at the front and back, instead of using sheet piling. These buildings were also equipped with measuring bolts to monitor the progress of various aspects, but so far no results have become available.
Institutions involved in this project were:

*Housing Department of Amsterdam*
Client / Project supervision / Consultation / Report writing

Zuideramstel quarter
Contacts inhabitants and owners

*OMEGAM (Environmental and Geo-technical Research Consultants Amsterdam)*
Scientific and practical foundation of geo-technical aspects

*Fugro De Waal Engineering Consultants*
Precision levelling, front levelling, water-level measures and soundings

*Selie Contractors*
Execution of the foundation rehabilitation works

*Housing/Energy Consultancy (Adviesbureau Woon/Energie)*
Ecological comparison of replacement materials

*Foundation for the 'Steering Committee for Experiments in Public Housing'*
Financial contribution
INTERACTION BETWEEN EXISTING AND NEWLY ADDED PILES

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Introduction

This paper deals with the possibilities of reinforcing existing foundations. Following the introduction which aims to outline the extent of the foundation problems in The Netherlands, an examination is made of the causes of poorly functioning foundations, followed by the possibilities for repairing foundations, and the costs involved.

Random sampling of the quality of foundations (Tol, 1994) showed that the quality of piled foundations constructed in The Netherlands since the end of the last century until the present day has markedly improved. Figure 1 shows development of the percentage of poor wooden piled foundations built between 1900 and 1940 in Amsterdam and Rotterdam. Converted into numbers, this means that approximately 10,000 wooden piled foundations of poor quality are in place or have been in place in both cities (some of these foundations have already been demolished). Repair of these piled foundations would require an investment of approximately Dfl. 500 to 700 million. The costs of repairing piled foundations amounts to some Dfl. 50,000 to 70,000 per premises, generally consisting of a two or three story building. The direct value of these premises is frequently too small for such an investment to be justified. For this reason, studies are being undertaken into less expensive methods for repairing existing foundations.

The quality of post-war piled foundations is much better. This is due to the improved quality of soil investigations, (the development of soil penetration tests), the introduction of concrete piles, and greater understanding of the behaviour of piled foundations, such as negative shaft friction and calculation rules which were subsequently developed. The number of damage cases resulting from inadequate piled foundations since 1965 was used to deduce a failure risk of approximately 1:3000 (Tol, 1994).

Figure 1: Percentage of poor wooden pile foundations, as a function of the year of construction
The usual objective of an investigation into the quality of existing foundations is to determine a maintenance period, or to evaluate the need for foundation repairs. When carrying out such an investigation, it is useful to divide the type of building into the following categories:

- pre-war housing (where investigation usually take place within the framework of an urban renewal programme);
- post-war building;
- special buildings (historical monuments etc.).

The outcome of a foundations investigation is extremely important in the case of the first category. If repairs to the foundations appear to be necessary, the high costs involved often means that a decision is taken to demolish the building. The cost of foundation repairs is very high compared with the direct value of pre-war housing if it is collectively owned by, for example, a housing association. The book value is then frequently small. Local foundation reinforcement only appears to be economically feasible if it results in the preservation of a large amount of housing.

The above conclusion regarding the potential for foundation repairs is not true of for the other two categories. In the case of pre-war buildings and special buildings such as monuments, the repair of foundations can often be justified in economic terms. The situation is also different for privately-owned pre-war houses, as the direct value is then much higher.

When determining the quality of the foundations, it is also important to consider whether any changes in circumstances must be taken into account in the future, for example load increments, lower groundwater tables etc.

The following section examines the reasons behind the deterioration of foundations.

**Causes of foundation deterioration**

**Shallow foundations**

Inadequate performance by shallow foundations is mostly due to continuous, uneven subsidence. This continuous subsidence is nearly always caused by compression of cohesive layers due to loading of the foundations, or fill that was placed during or after construction preparation activities. Sand layers, even if loosely packed, lead to one-off subsidence. Exceeding the ultimate load-bearing capacity is rarely the cause of poor foundations in the case of shallow footings.

Figure 2, showing subsidence of several houses in one blocks built on shallow foundations in Vreewijk, a neighborhood Rotterdam, indicates that continuous subsidence does not always have to result in poor foundations. These buildings were constructed during and soon after the Second World War, when a scarcity of materials led to the selection of shallow foundations. Piled foundations were only used at locations where there had previously been polder ditches. In spite of the enormous amount of subsidence, the majority of these shallow foundations have proved satisfactory until well into the 1970s. Absolute subsidence was a more acute problem in this particular instance because the margin between the bottom of the crawl spaces and the groundwater level became too small, leading to regular flooding of the crawl spaces.
Wooden piled foundations

As mentioned in the introduction, poor piled foundations are nearly only found in buildings constructed before 1940 where wooden piled foundations were used. There are two reasons for the deterioration of wooden piled foundations:

- decay of the foundations’ wood;
- overloading of the piles, mostly caused by negative shaft friction, sometimes combined with a high pile load or a low bearing capacity.

The pile itself settles in the latter case, while in the first case, the upper building subsides with respect to the pile. Figure 3 shows the construction of piled foundations found in Amsterdam and Rotterdam. Wooden foundations above the groundwater table generally result in more serious consequences for Amsterdam piled foundations than for those in Rotterdam. There are two reasons for this. Firstly, the higher position of the horizontally-positioned wood means that it will be damaged first. Secondly, certain capillary actions which arise because of the direction of the fibers in the wood means that the pile wood remains wet. For an Amsterdam foundation, damage to the cap (or cross) beam can mean that the foundation wall is eventually adjacent to the pile. This leads to one (or even both) of the piles being pushed alongside the foundation wall. This subjects the adjacent pile trestle to overloading and may lead to collapse as well. The consequences can therefore be serious. In the case of the Rotterdam foundation, decay of the horizontally-positioned wood will initially result in some compression of the wood. Although this results in subsidence, it is limited to a few centimeters. Wooden piled foundations with pile trestles are nevertheless also used in Rotterdam.
Overloading of wooden piles particularly occurs in areas where there is a thick parcel of soft layers. Figure 4 gives an example of the forces working along a wooden pile in West Holland. It appears that the load on the pile together with the negative shaft friction approximates the positive skin friction in the sand and the bearing capacity of the pile tip, but is even greater in many cases. This does not necessarily lead to pile failure because the consequent subsidence of the pile converts part of the negative shaft friction into positive shaft friction, as the pile settles more than the surrounding soil. Shaft friction is nonetheless a frictional force and the direction is therefore dependent on the relative movement. The friction in the soft layers will then redistribute itself in such a way as to achieve equilibrium, so that the downwards-directed forces are identical to those in an upward direction. The point where the pile settles at the same rate as the surrounding soil, the so-called neutral point, can be found by observing the equilibrium of forces along the pile. In figure 4b, this point lies 11m - ground surface for example. Assuming that the ground surface settles at a rate of 15mm/year due to fill that was brought up before the construction of the houses and lowering of the groundwater table, then figure 4b shows the possible distribution of the rate of subsidence at depth. It can be assumed that the subsidence rate remains at zero at the upper surface of the deep sand layer. The relation between subsidence and depth is sometimes assumed to be linear as well. The rate of the subsidence at depth can be used to deduce the rate at which the piles settles. At the level of the neutral point, in this case 11m - ground surface, the surrounding soil settles at a rate of 3mm/year. The pile therefore settles at the same rate. Such values are realistic, and have been confirmed in both Amsterdam and Rotterdam by long term levelling. The above observations are also true in terms of subsidence rather than rate of subsidence.
The progression of subsoil subsidence will be dominated by the secular effect, namely that subsidence will progress with the logarithm of time, especially if construction took place a long time ago. If fill is regularly brought up on the ground surface, for example to compensate for subsidence, subsidence will be somewhat faster. A conservative assumption is that subsidence will proceed linear with time after a preliminary period, as in the case of subsidence of the Vreewijk shallow foundations (figure 2).

**Repair of piled foundations**

The method for repairing piled foundations is determined by the cause of the subsidence.

**Decay of wooden foundations**

The required measure involves lowering the wooden piled foundations, by cutting away the damaged pile head until the healthy wood is reached and is sufficiently beneath the groundwater level. A durable pile head is then fitted, usually made of concrete. Two different methods are used:

1. a method where the piles remain loaded throughout
2. the piles are reloaded using a flat injection jack once the piles are shortened and a concrete piece has been positioned.

Method 1 is illustrated in figure 5, and method 2 is shown in figure 6. Both methods require consider-
able craftsmanship and skill.
The costs of such measures are determined by the following factors:
- the excavation: Determining factors are whether excavation can take place from outside by machine,
or must be carried out by hand from the interior outwards in a restricted space, the depth of the
excavation, etc.
- groundwater level: Work must always take place underneath the groundwater table and a drainage
system is necessary.
- quality of the masonry: Particularly where the brickwork is poor, numerous mostly unforeseen repairs
must be made to the masonry.

![Figure 5: Method where the piles remain loaded](image)
![Figure 6: The piles are reloaded using a flat injection jack](image)

It is also extremely important that these types of activity are carried out by a specialised contractor. The
costs of lowering the foundations using these methods varies between Dfl.1,500 and Dfl. 2,500 per pile,
depending on the factors given above.

**Insufficient load-bearing capacity**
The following options are available to repair piled foundations with insufficient load-bearing capacity:
- replacement of the entire piled foundation;
- installation of additional piles;
- increasing the load-bearing capacity of the existing piles;
- reducing the load;
- reducing the negative shaft friction.

**Installing additional piles**

It is relatively easy to replace the entire foundations. The new foundations can then be designed in accordance with standards for new buildings. Such a solution, however, is extremely expensive. In addition, adjacent buildings have often been constructed on similar foundations so that new differential settlement may arise. To overcome this problem and at the same time work in an economically justifiable way, partial load-bearing capacity is added. Additional piles are installed in the building section where there is the greatest subsidence. It is of course important that no fixed points are created to prevent new differential subsidence and fracturing occurring. The approximate rate of subsidence of old wooden piled foundations is a few millimetres a year. Fixed points can be avoided by:
- opting for low safety regarding the load-bearing capacity of the additional piles;
- selecting a so-called elastic pile, which undergoes considerable deformation when subjected to loads.

![Figure 7: Driven steel tubular piles sections, used in restricted work space](image)

Figure 7: Driven steel tubular piles sections, used in restricted work space

![Figure 8 (r): Injected steel tubular pile, used in restricted work space](image)

Figure 8 (r): Injected steel tubular pile, used in restricted work space

In the first case, a concrete-filled driven tubular pile is generally used measuring more than some 200mm in diameter (see figure 7). Deformation of elastic piles is mainly due to a thick-walled steel tube measuring up to 100 mm in diameter, with a grouted section in the load-bearing sand layer (see fig 8). In the second case, the non-rigidity of the pile means that it has to be prestressed. The deformation characteristic of both pile types and of wooden piles is shown in fig 9. In both cases, if the non-strengthened section of a building subsides and the additional piles are over
loaded, then the piles will also settle. The structure required to transfer the loads from the superstructure to the new piles should always be designed in accordance with normal safety, or preferably be over-designed. The two types of piles described can be made in sections for a limited working height. A large number of other pile types also exist which are suitable for use inside buildings: an overview is given in SBR(1985).

In situations where the surroundings are subsiding, the major problem of designing the additional load-bearing capacity necessary is far more difficult than when designing foundations for new buildings. If there is a clear difference in load or negative shaft friction, this can sometimes be used to calculate the extra load-bearing capacity required. For example, if a one-sided embankment has been constructed alongside a building and the building has therefore subsided at an angle. The subsidence difference is rarely simple enough to be converted into an obvious loading difference.

Figure 9. Deformation characteristics of different pile types

The additional load-bearing capacity required can be calculated using the subsidence difference which has occurred. Figure 10 gives an example of such a calculation. The left-hand side of this figure shows the subsoil subsidence as a percentage of the ground surface subsidence. The distribution of negative shaft friction along the pile is shown on the right-hand side. In the example given (dotted lines), the wall subsidence is 50% of the ground surface subsidence. The adjacent walls subside by 20%, which is to say that the neutral point of the piles of the rapidly settling wall must be brought from NAP-7.2m to NAP-10.8m of the walls in the surroundings. By moving these points to the right-hand part of the figure, it can be seen that 29kN shaft friction per pile occurs across the zone from 7.6m to 10.8m. This shaft friction has a positive effect in the case of the rapidly subsiding pile, and a negative one for the slowly subsiding pile. This means that two times 29 kN load-bearing capacity must be added per pile. Multiplying by the number of piles gives the load-bearing capacity to be added per wall. This is a lower value of the bearing capacity to be added. In general the settling parts of a building will have transferred load to the parts founded on a stiffer foundation. A somewhat larger load-bearing capacity than that calculated will therefore be selected in practice. Low safety regarding the ultimate load-bearing capacity ensures that the strengthened wall does not act as a fixed point.
Figure 10: The calculation of the load-bearing capacity to be added

Figure 10 also shows an example of a pile whose subsidence is some 23% of the ground surface subsidence and surroundings which subside by approximately 13%: load-bearing capacity 2*22.5 kN per pile (solid line) to be added.

In Rotterdam this method to calculate the additional bearing capacity has been applied in a small number of projects. It appeared that the capacity of the additional piles was indeed a minimum. In one case the behaviour was satisfactory and in one case the settlements of the reinforced parts were still somewhat larger than surrounding points.

Cost indication

The cost of reinforcing foundations by the installation of additional piles depends on:
- sequential approach for a single house or load-bearing wall;
- combined or not combined with renovation;
- pile length.

If reinforcement work is combined with renovation activities, no additional construction costs are incurred for making the building accessible to equipment. Recalculations for several projects carried out together with renovation activities resulted in the following indication:

Recalculation of a number of projects in Rotterdam (combined with renovation)
- Dfl. 40,000 - 45,000 per load-bearing wall (sequential approach)
- Dfl. 75,000 - 87,500 per premises
- Dfl. 50,000 - 55,000 per premises (sequential approach)
Increasing the load-bearing capacity of existing piles
This can be achieved by pushing existing piles deeper into the subsoil or grouting around the pile tips.

To push existing piles deeper, the exact length of the piles must be known and the depth to which these must be pushed. It appears that piles removed from pre-war buildings are sometimes broken or are of varying lengths. This approach can be used if the piles are nearly all the same length. The second method of grouting around the tips of wooden piles has not yet been applied in housing construction projects and is an item of research on Delft University of Technology (Stoel, 1998).

Reducing negative shaft friction
This can be achieved by removing an amount of relatively heavy soil next to or underneath the premises, and replacing it with lighter materials such as argex grains or foam concrete. A project has recently been carried out in Amsterdam involving the removal of approximately 2m of soil which was replaced by foam concrete.

Shallow foundations
As has already been discussed, the cause of poorly-functioning shallow foundations is usually differential settlements which result from the compression of soft cohesive layers. Reinforcement of the foundations should aim to slow down this subsidence. This is only possible by reducing the load, which is often impossible, or by dispersing the load over a much larger surface area. This reduces the soil stresses and the settlements.

A good example of this is the foundation reinforcement project carried out in 1967 at York Minster Cathedral, England. York Minster stands on a shallow foundation, with compressible clay, silt and fine sand layers up to approximately 50m beneath the foundations. A high tower standing where the central nave crosses the transverse nave had subsided more than the rest of the cathedral because of the high load to which it had been subjected. This led to substantial damage. The tower was founded on several individual bases. The foundations were reinforced as follows (see figure 11):

- soil was excavated from alongside the individual bases, until the lower surface of the foundations was reached;
- huge concrete slabs where then placed on the unloaded clay;
- flat jacks were next positioned on these slabs;
- a concrete supporting structure was placed on top;
- this was pre-stressed on the existing foundations using tension rods;
- using the flat jacks, a substantial part of the existing load could then be transferred to the new foundations.
Figure 11 Reinforcing the foundations of the York Minster

The latter step is essential, of course, to prevent considerable subsidence of the existing foundations before the new foundations are loaded. The jack-screws were inflated several times to compensate for compression of the previously unloaded clay.

This procedure was successfully applied, substantially reducing the subsoil stress and equalizing the subsidence differences.

**Conclusion**

This paper has shown that there are two reasons for poorly-functioning pile foundations in The Netherlands: decay of the wooden piles and overloading of the piles because of a high negative skin friction. In the first case a traditional repair method has successfully been carried on many occasions. In the case of overloaded piles a traditional reinforcement with new piles with a normal safety factor which take over the total load of the building can result in new differential settlements in the future. Only part of the load has to be carried by additional piles in such cases. A method to calculate the additional bearing capacity is given. The method is not yet applied on large scale but show satisfactory behaviour in some cases.

**Literature**

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THE LEANING TOWER OF PISA

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Introduction
The aim of this paper is to present the current condition of the leaning Tower of Pisa, updated till this very moment.
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 stabilization 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.

Fig. 1 - Piazza dei Miracoli - Air view

Historical background
The Monuments of Piazza dei Miracoli, see figure 1, including the Tower, are: the Cathedral, the Baptistry 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 figure 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 stair case 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 figure 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 (figure 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.

\[ V = 142 \text{ MN}, \quad M = 327 \text{ MNm}, \quad e = 2.3 \text{ m} \quad (*) \]

\( (*) \) Situation in year 1990

Fig. 2 - Leaning Tower of Pisa - Cross-Section

Fig. 3 - Construction History
It was during the second construction phase that the curvature in the axis of the Tower began to appear, see figure 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.

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 plumb 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 figure 5. The aim of the catino was to uncover the bases 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 figure 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.
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).

Based on all the above mentioned geotechnical investigations, it is possible to determine the following soil profile (according to the designations of main Horizons adopted in MPW), see also figure 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 $A_0$: 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 A₂ becomes increasingly sandy. Overall, a comparison of the cone resistance (qc) yielded by CPTU's reveals that resistance of Horizon A is markedly lower at South when compared to the North side, see Figure 8a. The CPTU’s also showed that the qc profiles on the East side yielded an average lower cone resistance than on the West side, see Figure 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 figure 9. In addition to the profile of shear wave velocity Vs, the figure shows the trend of qc vs. depth which confirms what emerges from figures 8a and 8b.

Fig. 7 - Subsoil Conditions

Fig. 8a - Cone Resistance in horizon A, North-South Cross-section.
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 Resistance in horizon A, West-East Cross-section.](image)

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 C₁; 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 C₂; from elev. -65.0 to -75.0; greenish clayey and silty sand.

* Layer C₃; from elev. -75.0 down to the maximum explored depth, grey changing to a shade of green in lower sand.

Figure 10 is representative of the groundwater conditions. Three different piezometric levels exist in the Horizon A layer B₁ 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 B₂ 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:
y = bulk density; \hspace{1cm} G_s = \text{specific gravity}; \hspace{1cm} W_n = \text{natural water content};

LL = \text{Liquid Limit}; \hspace{1cm} \text{PI} = \text{Plasticity Index}.

### Table 1 - Grading of main soil layers

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

### Table 2 - Index Properties of main soil layers.

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

Based on the information concerning the piezometric levels and with reference to the values of \( y \) determined in laboratory, the variation of the effective overburden stress (\( \sigma' \)) with depth shown in figure 7 has been established. The value of \( \sigma' \) in combination with preconsolidation pressure \( \sigma'_p \), 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, 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

\[
I_v = \frac{e - e_{100}^*}{e_{100}^* - e_{1000}^*} = \frac{e - e_{100}^*}{C_c^*}
\]

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, figure 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 $o^\prime$ vs. void index ($I_v$), the latter defined [Burland (1990)] as follows:

\[ e = \text{current void ratio of tested specimen} \]
\[ e^\prime_{100} = \text{void ratio at } o^\prime = 100 \text{ kPa determined reconstituted specimen starting from } LL < W_n < 1.5 LL \]
\[ e^\prime_{1000} = \text{as above but referring to } o^\prime = 1000 \text{ kPa} \]
\[ C^\prime_c = \text{compression index of reconstituted clay.} \]

Table 3 - Compressibility indexes from oedometer tests.

<table>
<thead>
<tr>
<th>Horizon</th>
<th>Layer</th>
<th>$C_{c1}$</th>
<th>$C_{c2}$</th>
<th>$C_{c1}/C_{c2}$</th>
<th>$C_s$</th>
<th>OCR range</th>
<th>$C_{ae}/C_{c1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>A_3</td>
<td>0.243</td>
<td>0.243</td>
<td>1</td>
<td>0.023</td>
<td>2.4 (\pm) 4.1</td>
<td>0.011</td>
</tr>
<tr>
<td></td>
<td>B_1</td>
<td>0.909</td>
<td>0.640</td>
<td>1.42</td>
<td>0.072</td>
<td>1.3 (\pm) 2.0</td>
<td>0.035</td>
</tr>
<tr>
<td>B</td>
<td>B_2</td>
<td>0.266</td>
<td>0.266</td>
<td>1</td>
<td>0.030</td>
<td>2</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td>B_4</td>
<td>0.280</td>
<td>0.280</td>
<td>1</td>
<td>0.057</td>
<td>1.3</td>
<td>0.023</td>
</tr>
</tbody>
</table>

$C_{c1} = \text{Primary compression index immediately beyond } \sigma^\prime_p$

$C_{c2} = \text{Primary compression index at } \sigma^\prime_v \gg \sigma^\prime_p$

$C_s = \text{Swelling index}$

$C_{ae} = \text{Secondary compression index immediately beyond } \sigma^\prime_p$

---

**Fig. 11 - Compression curves of upper Pisa clay in term of Void Index.**
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 $o'v > o'p$ are significantly steeper than SCI and ICL, and only at $o'v$, one order of magnitude higher than $o'p$ 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 A3, B2 and B4 may be found in the work by Lancellotta et al. (1994). The results collected by these authors led to the following values of $C' / C''$ ratio for the tested clays: 1.4 for B1, 1.0 for B4, B2 and A3. Table 3 summarizes the characteristics of the different clay layers tested.

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

<table>
<thead>
<tr>
<th>Horizon</th>
<th>Layer</th>
<th>$\phi'$ ('')</th>
<th>$C'$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>A3</td>
<td>31</td>
<td>0 to 20</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>33</td>
<td>0</td>
</tr>
<tr>
<td>B</td>
<td>B1</td>
<td>22</td>
<td>6 to 20</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>28</td>
<td>12 to 30</td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td>34</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>B4</td>
<td>27</td>
<td>0 to 5</td>
</tr>
</tbody>
</table>

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

### Table 5 - Normalized undrained shear strength of upper Pisa clay

<table>
<thead>
<tr>
<th>$\sigma'_u / \sigma'_v$</th>
<th>$0.23 (OCR)^{0.84}$</th>
<th>$0.29 (OCR)^{0.84}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TEST</td>
<td>DSS - CK₀U</td>
<td>TX - CK₀U</td>
</tr>
</tbody>
</table>

OCR = overconsolidation ratio
DSS = direct simple shear
TX = triaxial test
CK₀U = consolidated in K₀ - condition undrained

The representative drained peak shear strength characteristics $o'$ 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 $N_{SP}$.

The angle of friction at critical state has been determined only for clay of layer B1, 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 Ko-consolidated undrained triaxial compression tests (TX-CK₀U) and Ko consolidated undrained direct simple shear tests (DSS-CK₀U). 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.

The initial soil stiffness $G_o$, at a strain less than the linear threshold strain, has been inferred from Vs 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 the results of in situ and laboratory tests, in terms of $G_o$ are reported in figure 12. Additional information concerning these tests may be found in the work by Jamiolkowski et al. (1994).
Seismic cone
Resonant column
Bender elements

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

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.

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 Figure 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 (figure 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 (figure 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.

An overall picture of the Tower tilt on the North-South plane since 1911 is shown in figure 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 figure 15, show a slow but steady increase in the rate of tilt 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 through 109, see figure 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 Figure 16, located externally on the Tower plinth.
* Twenty-four bench marks, 1 through 24, see figure 16, used to monitor the movements of Piazza dei Miracoli by means of precision levelling.
* Deep datum point, DD1 in figure 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 figure 17, are located on the ground floor in the entrance to the Tower. The inclinometers and the automatic hydraulic levellometers, shown in the same figure, allow for the continuous measurement of change in monument tilt over a short term.

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, figure 18 shows the reciprocal relationships between the inclination of the monuments and its overhanging as well as that between the plinth tilt and the relative settlement of its South edge.
Inclination and overhanging in May 1993 before application of counterweight:

\[ \theta = 5^\circ 33' 36'' \]
\[ h = 4.47 \text{ m} \]

\[ \theta = \text{inclusion} \]
\[ \delta = \text{relative settlement of South edge with respect to North edge; } \]
\[ \delta (\theta = 1'') \approx 0.095 \text{ mm} \]

\[ h = \text{overhanging referred to 7th "cornice"; } \]
\[ h (\alpha = 1'') \approx 0.22 \text{ mm} \]

\[ \theta = \alpha + 11'25'' \]

N.B. \[ 1^\circ = 60' = 3600''; \quad \theta_{1998} = 19971'' \]

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 (figure 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 figure 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, Figure 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 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 x 10^{-2} and 3 x 10^{-3} respectively”.

**Structural features**

As shown in figure 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 figure 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 : \( N = 142 \text{ MN} \); average foundation pressure: \( q = 497 \text{ kPa} \);
* total height : \( H = 58.36 \text{ m} \); height above G.L.: \( =55 \text{ m} \);
* distance from the centre of gravity to the foundation plane \( h_g = 22.6 \text{ m} \);
* annular foundation, inner diameter; \( D_i = 4.5 \text{ m} \), outer diameter \( D_o = 19.6 \text{ m} \);
* area of the annular foundation: \( A = 285 \text{ m}^2 \);
* present inclination : \( \alpha = 5^\circ 28' 09'' \);
* present eccentricity of \( N \); \( e = 2.3 \text{ m} \).

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 figure 21.
Table 6 - Mechanical properties of Pisa Tower Masonry

<table>
<thead>
<tr>
<th></th>
<th>$\sigma_c$ (MPa)</th>
<th>$\sigma_t$ (MPa)</th>
<th>$E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Giuliano</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Marble Facing</td>
<td>110 - 190</td>
<td>4 - 8</td>
<td>70.000 - 90.000</td>
</tr>
<tr>
<td>Infill Masonry</td>
<td>4 - 8</td>
<td>0.3 - 1.3</td>
<td>5.000 - 7.500</td>
</tr>
</tbody>
</table>

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

$\sigma_c$ = Compression strength
$\sigma_t$ = 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 evidenced in figures 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.
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**

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, 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 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 at 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 consists of 18 lightly post-tensioned tendons located in the places shown in figure 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 placing 6 MN of lead ingots on the North edge of the plinth as shown in figure 24. The lead ingots have been placed gradually (figure 25) on the prestressed concrete ring shown in figure 24 generating a stabilizing moment of 45
MNm. The counterweight placed in the period between May 1993 and January 1994, see figure 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 figure 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.

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

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

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 a 0.7 to 0.8 m 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 figure 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 [Figure 30] of the lead ingots on the North edge of the plinth.

Fig. 29 - South section of Catino - Actual configuration

Ever since, the Tower has been motionless as far as its inclination is concerned, see Figure 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 underexcavation intervention described in the following.

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

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 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 Figure 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 evidenced 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 (~10%) inclination 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. 31 - Tilt of Pisa Tower since May of 1995

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

Fig. 33 - Underexcavation field trial
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 figure 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 Figure 33. The under excavation was performed extracting gradually the soil from Horizon A by means of a procedure, shown in figure 34, which made it possible to reduce the inclination of the trial plinth by almost 1000" of arc, as documented in figure 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 Figure 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 figure 36. Based on the response of the monument, referring to the scheme shown in figure 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 figure 37.

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

Fig. 37b - Cable Stay Structure - Plan
"Stop press" addendum - At the end of 1998

In February 1999, the preliminary underexcavation intervention under the Tower was started. The intervention consists in a very gradual controlled ground extraction from 12 holes shown in figure 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 figure 38.

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 m³ of the ground has been extracted. Since the start of the intervention, the Tower has responded positively rotating towards the North. With the instantaneous center 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 completion 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 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.

Fig. 38 - Preliminary underexcavation scheme
"Stop press" Addendum - at September 30, 1999

The preliminary underexcavation under the Tower was started on February 8th, 1999. The intervention consisted in a very gradual extraction of 7 m³ 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 rotating Northward at a reduced rate. On September 2nd, 1999 it was achieved a reduction of the inclinometer of 132 arc-seconds.
- During this period of time the Southern edge of the plinth rises by 1.5 mm while the Northern edge settles at 11 mm. The observed behaviour corresponds to two favorable phenomena:
  * instantaneous point of rotation located within the plinth;
  * small reduction of contact stress under the Southern edge of the foundation.

The overall reduction of the Tower inclination obtained as a result of the lead weight application (50') and following the completion of the preliminary underexcavation is evidenced in figure 42.

At present the Committee for the Safeguard of the Leaning Tower of Pisa is ready to undertake the final underexcavation aimed at reducing the Tower inclination by 1500 to 1800 arc-seconds.

This intervention will be carried out using 41 extraction holes penetrating 2.5 up to 3.5 m under the plinth. The achievement of the above reduction of inclination 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 39: Hole for soil extraction
Figure 42: Tower inclination in North-South plane (Girometti-Bonecchi inclinometer)

### 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 Geotechnica

### References


Instrumentation in Geotechnical Engineering, Hong Kong, Institution of Engineers.


Work started on the building of the Amsterdam concert Hall in 1880 by driving over 2000 wooden piles of 13 m on which foundation-beams out of wood and brickwork were made. During construction the design was changed. This was due to the latest German developments, requiring double walls instead of single, so that the space in between could be used for ventilation and heating. Therefore the pile load, originally designed at approximately 100 kN, increased even to 170 kN for some piles, causing settlements and unacceptable cracks in the brickwork. Settlements of up to 18 cm were measured.

In 1985/1986 around 400 vibration-free Tubex-displacement-piles were installed under the Concert Hall, with a shaft of Ø355 mm, a pile shoe of Ø560 mm and a pile length of approximately 18 m. After these Tubex piles had been installed they took over the complete weight of the whole building, thus eliminating further settlements and damage. 95% of all the construction activities took place while the concerts went on. The chosen solution also made it possible to create a new basement under the Concert Hall as an extension for dressing-rooms etc.

Introduction

The construction of the Amsterdam Concert Hall or “Concertgebouw” as it is called in Dutch, was started in 1880 by driving more than 2000 wooden piles to a depth of 12-13 metres. On top of these piles a wooden framework was constructed on which brick walls were to be built. However during construction, the board of directors of the Concert Hall became very impressed by a new technique, which had been developed in Germany. The technique consists of the application of a double brick wall with a space of about 30 cm in between; which can be used for heating and ventilation.

This system also had to be used for the Concert Hall in Amsterdam. Design and construction were changed to cover this new system, but without any changes to the foundation already constructed.
Therefore the pile loads on the wooden piles increased from about 100 kN (original design) to 170 kN (new design with double brick walls). Around 1982, cracks in the brick work and progressive settlements up to 18 cm!! made a complete renovation of the existing foundation inevitable to save the Concert Hall from total loss.

At first the settlements were believed to be caused by settlements of the overloaded piles, but profound investigation in 1983 made it clear that about 80% of the settlements were caused by compression of the wooden framework on top of the piles carrying the brick walls.

Settlements of the wooden piles, even though overloaded, would not have led to severe damage.

The renovation as it was started in 1984, consists of four aspects:
1. Renovation of the foundation.
2. Creating more room for musicians audience and personnel.
3. Renovation of the building itself.
4. New technical equipment.

This contribution to the Conference will deal mainly with the first aspect. The renovation of the foundation.

**Solution**

The contract of the renovation for which 4 Dutch contractors were invited to bid, was awarded to Strukton. Strukton with Fundex Piling developed a system that enables them to install a new foundation without having to close down the Concert Hall during construction.

With about 600 musical performances per year, the board of the Concert Hall concluded that this was the best proposal they could get and accepted the bid. For the installation of the piles, a special rig was constructed to work in very restricted space. The height of the rig is less than 2 m and it is only 1,50 wide. The machine is electrically powered, to avoid unacceptable noise nuisance and exhaust-gasses. The rig is used to drill so-called Tubex piles, a vibration free installed soil-displacement pile.

**The tubex-pile**

The Tubex pile consists of a steel tube, in this case 0355,6 x 8 mm, which is welded to a conical pile shoe 0560 mm. Depending on the limited weight available,
a tube-section of 2-4 meters long is clamped in the drill-table of the rig and drilled into the ground vibration free, together with the pile shoe. After the first section, another pile section is taken and welded on top of it, and again the pile is drilled in another two or three meters. This procedure is repeated until the required depth has been reached or sufficient resistance to penetration is encountered. Finally the pile is filled with concrete and a full or top reinforcement. In order to displace the soil and to overcome friction on the tube during drilling, considerable power is required for the drill-table. The electric power-pack provides the drill-table of this rig with a capacity of 400 kNm torque and a pull-down force of approximately 100 kN (80% of the weight of the rig) on the axis of the drill-casing. By using a gauge installed in the hydraulic circuit changes in the ground resistance can be monitored by the pressure readings. There is a fairly accurate correlation between the pressure shown on the gauge and the ground resistance, which has been measured before, by means of SPT, DPT (Dynamic Penetration Test), CPT (Dutch Cone Penetration Test) or pressiometer tests. These pressure readings are used as a yardstick for the installation of piles at larger distances from the nearest CPT (or other type of soil investigation). Discontinuities in the bearing stratum can be traced on every pile location and appropriate steps can be taken, such as, using longer or shorter piles if the bearing stratum is found at various depths. Occasionally, if deviating pressure readings are encountered, additional soil investigation may be necessary. These pressure readings provide a good check in reaching the various pre-determined levels and increase the liability of the system, and therefore of the structure for which the foundation is constructed. Because of the radial displacement of soil during driving, the soil is being compacted, which increases the effective stress and shear strength and gives a strong bond between the ground and pile, both affecting the pile-bearing capacity positively.
Design
For the design of the new foundation CPTs were carried out to investigate the soil. The SPT's show typical Dutch soil conditions:

- 0 - 12m highly compressible clay and peat layers
- 12- 14m medium dense sand (foundation level of wooden piles)
- 14- 15m clay
- 15- 17.5m medium dense sand
deeper than 17.5m : dense sand layers

The Tubex-piles, shaft Ø6355, 6 mm and pile-shoe Ø560 mm, had to be installed to a depth of 18m to carry pile-loads from 500 - 800 KN

The safety factor to prevent failure for this project was to be $n = 3$. Some piles even had to be installed to a depth of 22 m since the bearing stratum was found at various depth.

Since the piles had to be installed very close to the existing wooden piles, there could be some risk of additional settlements during the installation of these Tubex piles. A simple test has been done to prove that the soil-displacing Tubex pile would not involve such a risk. Therefore a wooden pile was driven in on a test site and loaded to 85 kN by two reaction piles.

At a distance of about 85 cm, a Tubex -piles was drilled in to a depth deeper than the pile tip level of the wooden pile similar to what was to be done under the Concert Hall. The results of this test were convincing and also during the actual pile installation for the Concert Hall, no additional settlements were measured.

Execution
At first a sheet-wall was placed around the Concert Hall which allowed lowering of the ground water table without any risk of damage to adjacent buildings. Then an excavation was made around and under the Concert Hall to a level just above the existing wooden framework on top of the wooden piles. Now piling could be started, at first outside the building, later in and under the Concert Hall. Piles were installed in two rows along each wall. Steel beams were placed on top of the piles. Between the two rows of piles, holes were drilled in the existing brickwork in between. Steel beams were placed through these holes in the brickwork and placed across and on top of the beams over the rows of piles. By jacking up these crossbeams the Tubex piles will take the entire weight of the Concert Hall thus eliminating further settlements and damage.

Taking over the weight of the Concert Hall, to the new foundation was the most critical and essential but also the most spectacular stage in the renovation of the foundation. After the installation of Tubex piles and steel framework it would be easiest just to remove the old
foundations and the structure would have to find its way to the new foundation. This method of course would lead to unacceptable deformation (i.e. more cracks in the brickwork). Therefore the entire Concert Hall had to be jacked up to the new foundation. But, if this had been done in one stage, additional deformation could have been expected, unless the whole structure was jacked up simultaneously. This, of course, is very difficult if not to say impossible. Besides the elastic behaviour of the wooden piles (100-170 kN mainly friction) is quite different from the steel Tubex piles (500-850 kN end bearing). By taking over the entire weight in 3 steps, each time only small (acceptable) differences in deformation may occur without leading to more cracks or other damage. Each step would involve 30-35\% of the load. The time between each step of loading would be 2 weeks. Once this important stage is finished, the ground-water-table is lowered another two meters and further excavation is done to the same depth, making it possible to create a basement under the Concert Hall which is under construction right now. When this basement is ready, the ground water table will be brought back to its natural level and the sheet-wall around the Concert Hall can be extracted again. Some of the Tubex piles now may have to take tensile loads up to 300 kN due to uplift of the basement under the groundwater table. The total renovation is expected to be ready about one year from now. And so it will become possible for future generations to enjoy music in the Amsterdam “Concertgebouw”.
EXTENDING OF SHALLOW FOUNDATIONS BY JET-GROUTING

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Introduction

In “pr EN 12716 Execution of special geotechnical works jet-grouting” the following definition is given for jet-grouting:

“The jet-grouting process consists of the disaggregation of the soil or weak rock and its mixing with and partial replacement by a cementing agent; the disaggregation is achieved by means of a high-energy jet of a fluid which can be the cementing agent itself.”

The aim of jet-grouting is to inject solidified elements known as ‘soilcrete’ into the soil. In most cases, cylindrical column elements are produced but it is also possible to create planar panel elements or even half cylindrical column elements. The elements are most often interconnected to provide underpinning to structures, excavation support, groundwater control or in-situ stabilisation for civil engineering and environmental applications.

Jet-grouting is normally performed as follows (figure 1):
- A rod system is lowered into the soil by means of the direct flushing method. In most cases water is used as the flushing medium.
- At a predetermined depth, grout is pumped through horizontal nozzles at a high pressure. In this way a jet with a high velocity is obtained. This energy causes the erosion of the ground and the placement and mixing of grout in the soil.
- The rotating drilling rods are then lifted up to a constant height (for instance 4 cm) while grout is pumped continuously. During this grouting operation there is a continuous flow of spoil returning to the surface (surplus of grout and finest elements of the in-situ soil).

Figure 1

It should be noted that for the jet-grouting technique, the high pressure of the grout is only necessary to obtain a high-energy jet that makes it possible to disaggregate the soil up to a certain distance from the drilling rods.
The following methods can be used to execute jet-grouting (figure 2):
- The mono-jet or single fluid system. A jet of cement grout is used to disaggregate and cement of the soil.
- The bi-jet or double fluid (air) system. An air jet shroud is used around the grout jet to increase the efficiency of the grout jet, in this way elements with a larger diameter can be realized.
- The double fluid (water) system. The disaggregation of the soil is achieved by a high-energy water jet and the cementing is simultaneously obtained by a separate grout jet.
- The tri-jet a triple fluid system. The disaggregation of the soil is achieved by a high-energy water jet assisted by an air shroud and its cementing is simultaneously obtained by a separate grout jet.

Figure 2

Extending of shallow foundations
Jet-grouting is a very appropriate technique for extending shallow foundations, for the following reasons:
- it allows the creation of rather large elements underneath existing foundations
- a direct load transfer takes place from the existing foundation to the jet-grout element.
- the jet-grout elements can be installed without a considerable change in the stress situation in the existing foundation and in the soil underneath the existing foundation.

When the jet-grouting technique is used, rather large elements (diameter 0.50 à 2.0 m) can be realized underneath existing foundations. Therefore, only relatively small holes with a diameter of 150 to 200 mm have to be made through the existing foundation. The core drilling technique is normally used to make these holes.

As the new grout columns always have a much larger diameter than the hole through the existing foundation, a direct load transfer from the existing foundation to the grout element can take place (Fig. 3a). This is an important advantage over the micropile technique, where the load transfer from the existing foundation to the micropile has to take place by friction between the foundation material and the micropile (Fig. 3b). Therefore, the load that can be transferred to the micropile depends largely of the quality of the foundation material. For foundations of weathered masonry, it is sometimes very difficult to determine the load that can be safely transferred to the micropile.

In normal conditions, the stress situation in the foundation and the soil underneath the existing foundation is not significantly changed by the installation of the jet-grout elements. Since in the first stage only a rather small hole is drilled, the risk of decompression of the surrounding soil is very limited. During the disaggregation and cementing phase the hole that has been created is always filled with a water-cement-soil mixture. Thus the potential flow of water from the hole to the surrounding soil has a stabilizing effect and the possible decompression of the surrounding soil remains limited. However, it must be remarked that the stabilizing effect of the fluid in the hole that has been created is greater when the mono-jet technique is used than when the bi-jet or tri-jet technique is used.

In some case precautions have to be taken during the installation of the jet-grout elements:
- When the jet-grout elements have to be installed through unstable soils (fills, very loose sands) grout has to be used as flushing fluid for the drilling of the preliminary holes.
- Underneath very weathered foundations, the diameter of the grout column has to be limited in order to avoid local instabilities within the existing foundation.
- When the safety factor of the bearing capacity is smaller than the factor normally applied, it is also necessary to limit the diameter of the grout elements. In this way, excessive settlement can be avoided during the jet-grouting operation.
Design of jet-grouting works

When designing the jet-grouting elements that have to be installed underneath shallow foundations it is necessary to:
- determine the dimensions and characteristics of the jet-grouting elements to be installed
- choose the system to be used.

Based on this data, the jet-grouting parameters have to be set by the jet-grouting specialist. Before any jet-grouting work is started, it is always necessary to determine the dimensions of the existing foundations.

a) Diameter of the elements:
The diameter of the columns to be installed is determined by:
- the grout pressure
- the grout flow rate
- the lifting speed
- the erosion characteristics of the soil

and for the bi-jet and tri-jet methods also by:
- the air pressure and air flow rate
- the water pressure and water flow rate.

Table 1, taken from pr EN12716, gives the ranges of the jet grouting parameters usually adopted for the different systems.

The disaggregation effect, and in this way also the diameter of the jet-grouting element, is obtained by the high velocity of the jet, which is mainly dependent on the pressure of the fluid used for the disaggregation. The pressure in the fluid used for disaggregation is maximal at the nozzle exit and decreases with increasing distance from the nozzle.

Figure 4 gives the variation of the pressure in the fluid for different conditions:
- monojet with smaller energy (small nozzle diameter)
- monojet with higher energy (high nozzle diameter)
- bi-jet or tri-jet.

This figure is based on the results of measurements performed at the Catholic University of Leuven. The pressure that is necessary for disaggregation of the soil depends on the soil type. Different types of soil exhibit different erosion characteristics (figure 5).

Based on available experience, the diameter that can be obtained with the different systems and in different soil types is given in Figure 6. The lower bond values are obtained with a grout pressure of about 35 MPa and a grout flow rate of about 150 l/min. The upper bond values are obtained with a grout pressure of 45 MPa and a grout flow rate of about 300 l/min.

Actually there are no consistent relations available between the diameters of the jet-grouting element, the execution parameters and the soil type. One has to place trust in the available specialist experience.
to assess which system, method and procedure will furnish soilcrete that achieves the goals of the project. When no comparable previous experience is available, representative preliminary in-situ trials have to be carried out and the columns that have been realized must be excavated in order to check the geometry and the strength characteristics.

b) Strength of the soilcrete:
The strength of the jet-grouted materials depends on:
- the jet-grouting system selected
- the jet-grouting parameters employed
- the soil type and its heterogeneity.

When the mono-jet system is applied, the following compressive strength values are currently obtained:
- in clay 4 N/mm$^2$
- in loam and silt 6 N/mm$^2$
- in sand 8 to 15 N/mm$^2$. In very clean sands compressive strengths of 25 to 30 N/mm$^2$ can be obtained.

When the bi-jet method is applied, only half of the given values are currently obtained.

The strength of the jet-grouting material is considerably influenced by the quantity of grout added. Depending on the soil type, the minimum value of the grout quantity to be added varies between 30% and 70% of the volume of the jet-
grouting elements to be realized. If necessary, the strength of grouting elements can be increased by increasing the quantity of added grout. When performing jet-grouting in clays, pre-cutting is often performed. Pre-cutting consists of a preliminary disaggregation phase with a jet of water. In this case, complete substitution of the soil is the objective and a grout volume of at least 100% of the volume of the jet-grouting element has to be introduced into the soil.

c) System to be used:
When jet-grouting is executed underneath the existing foundation, preference must always be given to the mono-jet method. The bi-jet and tri-jet systems can be used only when there is a very limited risk that damage may occur due to a local settlement or upheaval of the soil underneath the foundation.

d) Dimensions of the existing foundations:
For the underpinning of existing foundations, it is very important to have precise data about the characteristics of the foundation materials. When dealing with foundations of old monuments or historical buildings there may be some particular problems.

In most European cities, the historical buildings are located along rivers and in places where sandy soils with a rather high water level are encountered. Therefore, the excavations had to be stopped at a limited depth underneath the groundwater level because the soil was loosened due to the upward gradient of the groundwater flow. To improve the bearing capacity of the loosened soil often a layer of stone cobbles or shale was installed on the bottom of the excavated trench. In other cases, very short wooden piles had been installed through the loosened soil layer (figure 6).

Nowadays in such cases when inspection pits are dug in order to determine the dimensions of the existing foundations, it is usually impossible to reach the lowest foundation level. This is because there is a lot of water coming through the transition zone consisting of stone cobbles, shales or the loosened soil between the short wooden piles (see Fig 7). Inspection of the lowest foundation level and of the transition zone is then only possible after the groundwater level has been lowered over a rather large area. Such lowering of the groundwater level is usually beyond the scope of a normal foundation inspection and is therefore omitted.

Jet-grouting cannot be performed underneath foundations with such a transition zone because the spoil disappears through the transition zone and no returning of spoil can be obtained at the surface. Therefore, in most cases a preliminary treatment of the transition zone is necessary before the jet-grouting can be started. This often leads to considerable additional costs. In such cases due information on the dimensions of the foundation can be obtained by performing destructive borings through the foundation (see figure 8).

When the drilling parameters are recorded during the execution of this destructive borings, valuable information can also be obtained on the characteristics of the foundation materials.

For mediaeval buildings, it is very important to have precise information on the characteristics of the foundation materials. Indeed, in many cases the walls of such buildings consist of two separate masonry plates and between them a fill with very variable characteristics. Before the execution of jet-grouting can be started, one has to know if such a system will also be encountered in the foundations.

Problems that may occur during jet-grouting
During the execution of jet-grouting works the following problems may occur:
- Lift up of the foundation due to a blockage of the return-spoil
- Settlement of the foundation due to a collapse of the hole that is created.
- Divergence from the specified dimensions.

**a) Spoil returning:**
During the execution of any jet-grouting work a continuous flow of return spoil to the surface has to take place. When the spoil returning of is interrupted, an overpressure arises in the already completed grout column. This leads to a decrease of the column diameter, as can be seen in Figure 4. In some case hydrofracturing may take place, giving rise to lifting of the surrounding soil and the underpinned foundation. In order to avoid this it is really necessary to assure a continuous flow of return spoil to the surface during jetting.

Blockage of the annulus around the drilling roads can be avoided by:
- increasing the diameter of the hole through the existing foundation
- using the mono-jet system.

When the bi-jet or tri-jet systems are used the flow of the return spoil is always more irregular and it is also more difficult to detect when an interruption of spoil returning takes place
- Performing a pre-cutting in very cohesive soils.

**b) Stability of the created hole:**
When jet-grouting is performed in unstable soils such as fill layers and very loose sands, due attention has to be given to the stability of the hole created during jetting. In these types of soil a collapse may occur due to the lack of cohesion of the surrounding soils. In such cases, special measures have to be taken such as:
- decreasing the diameter of the columns, for instance, by installing more columns of 0.50 m diameter instead of less columns with a diameter of 1.0 m.
- using only the mono-jet system.
- When the mono-jet system is used a more continuous flow of return-spoil takes place and an almost hydrostatic pressure is maintained within the grout of the already completed part of the column. With the bi-jet or tri-jet system, a more irregular flow of return-spoil takes place and sometimes underpressures may occur within the completed part of the column.

c) Dimensions:
When a jet-grouted diaphragm has to be realized according to the primary and secondary sequence method, due attention must be given to the diameter of the primary columns. When a primary column with a diameter that is too large has been injected, problems may occur when the secondary column is installed. The excess diameter of the primary column may give problems during the drilling of the preliminary hole and deviations from the vertical or the prescribed axis may occur. When the preliminary hole of the secondary element is drilled through the excess diameter of the primary element, the fabrication of a grout column is impossible over the height where the preliminary hole is situated in the primary column (Fig. 9). In this way an untreated soil area remains and gives rise to a discontinuity in the diaphragm wall. This problem is very important when a watertight wall has to be made.

Case histories
Deepening of existing shallow foundations may be executed in order to:
- increase the bearing capacity of an existing foundation, for instance when owing to the renovation or extension of a building, larger loads have to be transferred to the soil
- deepen the level of the existing foundation when excavations have to be performed along the existing foundation
- limit the settlements of an existing foundation.

a. Concentra - Hasselt
At Hasselt, the foundation of an existing hall had to be strengthened to accommodate an extension. However, as the newspaper ‘Het belang van Limburg’ was printed every day in this hall, a strengthening method that could be executed without interruption of the activities in the existing hall had to be chosen. So jet grout piles were installed underneath the existing foundation according to the arrangement shown in Figure 10.

b. Grand Bazar - Antwerpen
In Antwerp the level of the existing shallow foundations, carrying loads of more than 5000 kN, had to be lowered for the installation of a department store in the basement level. The following method was used for the installation of a new foundation at a lower level, (Figure 11):
- in the first stage a corset of reinforced concrete was installed around the existing columns (a) and holes of 200 mm diameter were drilled through the existing foundation (b)
- through these holes, jet-grout columns were installed to a depth of about 7 metres underneath the foundation level (c) and 140 mm diameter steel tubes were inserted in the jet-grout columns (d)
- A steel structure was set up between the steel tubes and the corset around the column, including four flat jacks (e)
- After hardening of the jet-grout columns, the flat jacks were pumped up in order to transfer the loads from the column to the steel tubes. During this operation the column was lifted up for a few millimetres
- In the next stage, the existing foundation was demolished and a new foundation resting on the jet-grout columns was installed (f). Finally the column was extended and the steel structure and corset were removed (g).

c. ‘Huis van Brecht’ - Breda
For the construction of a new basement level, the existing foundations were deepened according to the arrangement given in figure 12.

d. Archive building - Antwerpen
In Antwerpen, a more than 7 m deep excavation has been dug around a historical archive building of 6 m by 10 m and with a height of 12 m.
In a first stage, vertical and near-vertical grout columns were installed underneath the existing foundations. Afterwards the excavation were performed in three stages and three rows of near-horizontal nails were installed (figure 13).

During the execution of this job, levelling of the building was performed regularly. The most important movements occurred during the installation of the first row of near-horizontal nails, when an uplift of 4 to 6 mm was measured. After the completion of the excavation an uplift of 2 mm remained.

e. Cathedral of Sint-Salvator at Brugge
During the restoration of the tower of the Cathedral of Sint-Salvator, it appeared that the foundation level of the tower was situated just above a rather compressible loam layer. In order to avoid any further settlement of the tower during its restoration, it was decided to extend the foundation to the sand layer underneath the loam layer. Therefore beams of reinforced concrete were installed inside and outside the tower and linked to each other through the existing foundations. Afterwards, vertical jet-grout columns were installed underneath these beams on both sides of the existing foundations. Preference was given to the underpinning method, as it was very difficult to obtain reliable information about the existing foundations and the characteristics of its materials.
Grand Bazar – Antwerpen

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Introduction

Hydraulic fracturing, generally called claquage, is a frequent phenomenon that occurs during the performance of permeation grouting. It may take place either from the beginning of the grouting operation, if the grout or the grouting technique is suitable for the permeation or at the end when all accessible voids in the ground have been filled up.

Most of the time it is considered that the claquages have a detrimental effect (undesired grout losses and ground movements). It is common practice to record the soil movements when performing permeation grouting in order to identify undesired soil movements at an early stage and to avoid damage.

With experience, the idea of using the uplifting effect of the claquages correct building settlements emerged. However, in contrast to normal permeation grouts, it is necessary to implement grouts with reduced mobility in order to limit and to control the penetration distance of the grout. This led to the development of stiffer and thicker grouts. The compensation grouting technique, developed during the last decade to compensate the settlements resulting from the tunnelling operations, makes extensive use of these grouts.

When using mortars with very low fluidity and plasticity, it is possible to place large quantities of grout without fracturing the soil but rather by compacting it. This is the basic idea of the compaction grouting technique which was started almost 50 years ago in the USA and which nowadays finds broad application in Europe.

SOLETANCHE BACHY played a major role in the development of both the compaction and the compensation grouting techniques in Europe. The aim of this paper is to give a brief glimpse of the operating principles for these techniques and their fields of application.

Compaction Grouting

Mechanism

Compaction grouting is a ground improvement technique in which a highly frictional mortar is forced into the soil under high pressure. The mortar can neither impregnate nor fracture the soil; it can only push back the material surrounding the injection point. A volume of mortar grows more or less regularly and compresses the surrounding soil as injection progresses.

Figure 1 (l): Process of soil compaction by solid grout injection.

Figure 2 (r): Grouting Material-Grain Size
A complex system of radial and tangential forces develops in the soil around the injected mass (figure 1). A plastic zone near the mortar mass, with important shear and compressive stresses, followed by a pseudo elastic zone with important shear forces. Shear deformations and, to a lesser extent, the compression stresses may result in a significant increase in the relative density of the soil. This causes the surrounding soil to be compacted. Furthermore, the network of grout columns contributes a reinforcement effect, thanks to the mechanical characteristics of the column material itself, which may range between a high friction material and a solid mortar, depending on its composition.

**The grouting material**

The grout to be used should neither impregnate nor fracture the soil. It must therefore be very rigid and at the same time very stable, since the bleeding-out of water would prematurely stop the progress of the mortar. Experience shows that the grain size distribution of the material must be selected in the range shown in figure 2. The main component is a sandy material, whose grain size distribution must be extended towards the fines by addition of silts, fillers, puzzolans etc. The binding material, which contributes to possible mechanical resistance after setting, also contributes to the increase of the fines content.

The amount of water is one of the critical points in the composition. Water is necessary to make the mortar pumpable. The minimum water content \( w_m \) varies widely from one grout to another, depending on the graduation, quantity and nature of fines, admixtures, temperature, and time elapsed since mixing. The higher the water content, the easier the grouting will be, but at the same time the bigger the risk of soil fracturing. A water content \( w_c \) above which, in given grouting and soil conditions, fracturing occurs can be determined. The interval \([w_m, w_c]\) for suitable grout is very narrow. Figure 3 shows qualitative variation as a function of the grouting power (grouting power = grouting pressure * pumping rate). Both values can be empirically determined on site by means of tests.

![Figure 3: Zone of grout suitability (for a given grout and soil)](image)

Mortar characteristics are generally checked on site with the ABRAMS’ cone slump test (figure 4). A maximum value of 3 to 5 cm is commonly considered satisfactory. However, this test does not give a complete idea of the mortar characteristics, in particular those of the grout filtration under pressure and the evolution of the shear parameters. SOLETANCHE BACHY, together with the Technical University of Darmstadt, developed a special testing device for this (figure 5).

**Operating Principle**

The operation starts with the excavation of the grout hole. Usually, a rotary drilling machine is used. Drilling is performed down to the desired bottom level of the hole and then the grouting operation is carried out continuously, stagewise from the bottom up, with the drill rod placed at the top of the stage (Fig. 6). The stages are generally 0.5 to 2.0 m high. In some particular cases, it may be necessary to perform the grouting operations from the top down but this is more time consuming.
Most of the time grouting is carried out until a refusal pressure is reached. This pressure can be as high as 6 to 8 MPa at the outlet of the pump. The criterion of limited quantities, which is usual in conventional grouting, may also be applied to the primary grout holes when a more even distribution of the grout is requested. In any case, grouting must be stopped as soon as uplift is detected, unless this effect is sought (see next section: compensation grouting).

Each grout hole is considered as a separate elementary treatment. The response of the soil is directly related to its state of compaction and therefore to the volume to be added (figure 7).

This is why it is particularly interesting to record the drilling data (feed rate, pressure on the tool, vibrations reflected back) and the grouting data (flow, pressure, and volume) for each hole. A permanent survey of the surface uplift is necessary. It must be completed by surveying with deep uplift detectors and inclinometers when underground constructions exist at levels close to the treatment zone.

Foundations
Applications

Compaction grouting is a versatile technique and the field of application is therefore very broad, for example it includes settlement reduction of shallow foundations, liquefaction risk reduction in seismic areas, improvement of the slope stability of embankments.

The treatment performed for the Atlantic Docks of the Port of Dunkirk may be considered as a typical example to highlight this versatility.

The West Port Docks were dug in sand, but in spite of the gentle incline of the retaining banks of the port (1/10), several breaches of the embankment occurred during dredging (figure 8). The origin of these breaches was the presence of a 1 to 4m thick layer of silt and loose sand of very poor characteristics (qc of 0.4 to 1.0 MPa), at a depth of 20 to 23 m below a very compact sand (qc of 10 to 40 MPa).

It was decided to improve the characteristics of the loose layer by compaction grouting.

The treatment was implemented in steps, by making primary, secondary and tertiary holes. The corresponding treatment area was respectively 50, 25 and 12.5 m² per borehole. Since the weak layer was encountered at varying depths and was of varying thickness, the drilling was performed by using a parameter-recording device. This allows very precise determination of the position of the soft layer in each borehole and the performance of a tailor made treatment.

Thanks to the very homogeneous characteristics of the layer, it was possible to implement an entire injection treatment by using predetermined quantities: an incorporated mortar volume of 4% in the silt and 8% in the loose sand.

As usual for soil improvement techniques, the improvement brought by the treatment is checked by carrying out in situ tests before and after the treatment, preferably by pressure meter tests or static penetration tests. In this case static penetrometer tests were performed.

The results were as follows:

Table 1: Comparison of CPT Tests before and after the treatment

On average, the shearing parameters of the soil were multiplied by 2. The average strength increase in the soil mass will even be more, since the characteristics of the mortar inclusions themselves can be taken into account.

Compensation grouting

Together with the remarkable improvement in tunnelling techniques during the last decade, the number of underground excavation works has considerably increased but whatever the precautions taken, when constructing a tunnel, some decompression of the soil is unavoidable. This decompression is caused by the excavation itself, by the overdig (free space between the excavated soil and the steel ring of the
TBM), by the reduced diameter of the linings at the tail exit and, finally, by the stress redistribution around the linings (figure 10). The settlements at surface resulting from decompression, and especially the differential settlements, can cause severe damages to the constructions on the surface. Various preventive techniques are available to protect the structures against damage. These include structural strengthening, underpinning, various in-tunnel measures, but increasingly compensation grouting appears to be the most flexible option to provide convenient solutions particularly in congested urban areas. Two main grouting techniques have been developed for the compensation of ground loss between structural foundations and the advancing tunnel, namely compaction grouting and compensation grouting. The aim of both techniques is to limit the movements of a structure to acceptable figures by introducing grout into the soil between the structure and the tunnel.

Compaction grouting has been discussed in the previous section. It is mostly implemented after the settlement has occurred, i.e. as a curative measure. The injection points are then often located directly beneath the superficial foundations (pad or strip footings). However, compensation grouting has a more preventive feature. The aim is to compensate the settlements progressively as soon as they appear.

**Operation principle**
A successful compensation grouting project requires the input of several advanced technologies, such

<table>
<thead>
<tr>
<th>Results Dutch Cone Qc (MPa)</th>
<th>Before Treatment</th>
<th>After Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>0.4</td>
<td>1.4</td>
</tr>
<tr>
<td>Maximum</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td>Average</td>
<td>0.7</td>
<td>1.5</td>
</tr>
</tbody>
</table>

![Figure 10: Settlement Evolution along a TBM alignment](image-url)
as:
* a correct tool to predict the soil movements and the corresponding differential and absolute movements of the construction foundations, together with a detailed structural assessment to determine allowable settlement figures
* advanced instrumentation technology for the real time recording of the soil and the structures to be protected
* a grouting technology able to produce an up-heave à la carte at different points in the soil and the foundations
* a computer system able to process all the collected data in real time.

The grouting process is based on the Tube à Manchettes (TAM) grouting technique, which was developed by SOLETANCHE BACHY some decades ago but which nowadays is an international standard technique for soft soil permeation grouting, although the French wording Tube à Manchette, which means Sleeved Tube, has been retained. Before the passing of the tunnelling machine, grout holes are drilled between the future tunnel and the structure to be protected. These grout holes are equipped with metallic or PVC tubes, about 2" in diameter and with grout openings every 33 or 50 cm. The grout openings are covered with rubber sleeves, which act as non-return valves and avoid the return of the grout once the grouting step completed. Thanks to these non-return valves, it is possible to re-grout the different grout holes as many times as required.

The density of the boreholes may vary in a wide range, typically between 1m*1m to 4m*4m. Main parameters governing this choice apart from the soil parameters, are the distance between the tunnel and the foundation to be protected, the importance of the expected settlements, the specifications regarding the allowable settlements and in particular the differential settlements.

After installation of the TAM, the first grouting step is performed to pre-stress the soil and to obtain a first up-heave effect so that an immediate reaction will be obtained when performing the next grouting steps. Accurate data collection on the movements during this phase will allow definition of the up-heave reaction of the soil or the foundations and in particular the efficiency coefficient, which is the ratio between the up-heave volume and the grouted volume. This ratio, which depends on many parameters but in particular the soil and the grout, ranges most of the time between 10 and 50%. It is an important key to the further treatment steps.

The second grouting step will be performed when the tunnel drift passes the area to be protected. This is the main grouting phase, because more than 50% of the total settlements caused by the tunnelling operation occur at this moment and at a high rate. Before the start of this grouting step a detailed anticipated grouting scheme is set up, based on the expected settlements on one hand and the efficiency coefficient of the grouting points on the other. During the crossing of the tunnel drift and the simultaneous grouting, real time monitoring of the movements is performed, and recorded values are compared with the expected values. Corrective actions are undertaken as soon as the recorded values fall beyond pre-determined limits.

A third grouting step may be required after the passing of the tunnel to correct the movements beyond the tail. These movements may have a significant amplitude especially when the tunnelling is performed in cohesive soils, but they are much slower than the movements during the previous grouting step. This third step can therefore often be performed as a homeopathic treatment.

THE GROUTING MATERIAL
The choice of the correct grouting material associated with a correct grouting procedure is of utmost
importance. The main target is to reach an optimum efficiency coefficient (i.e. ratio [uplift volume / grouted volume]) with a limited spread in order to be able to fine-tune the à la carte treatment highlighted above.

In practice, the material will have intermediate features between the high friction mortar used in the compaction grouting technique and the fluid bentonite-cement grout classically used in permeation grouting. The high friction and low plasticity mortar cannot be used here, firstly because this type of mortar is not compatible with the TAM grouting technique and would cause a premature plugging, and secondly because of the low efficiency coefficient. On the other hand, the spread of a fluid bentonite-cement grout is more or less uncontrollable and usually results in a soil fracture with a residual thickness of less than 1 or 2 mm. The intermediate material consists of stable bentonite-cement grouts but with additives to adapt the rheological parameters of the mixture before setting: viscosity, rigidity, and setting time.

RECENT IN-HOUSE DEVELOPMENTS
Facing the new requirements of compensation grouting sites, SOLETANCHE-BACHY developed a new set of specific tools, called ContAcTS (for Control of Active Tunnel Settlements):
* prediction of the movements, with permanent adaptation of grouting phases to the advance of the tunnel and the observed response of the ground,
* accurate and real-time monitoring of structural movements, with a comprehensive system of alarms,
* daily update of the model from site results,
* and an efficient connection between these two by networking computers on site.

Applications
Jubilee Line Extension
The first industrial application of the compensation-grouting concept was conducted at the construction site of the Jubilee Line Extension Project in London in 1995, where compensation grouting was successfully used to create large excavations underneath very delicate structures.

The Jubilee Line Extension, running beneath some of the oldest, historically most important and prestigious areas in London, was subject to the most stringent damage limitation tolerances ever before imposed on a tunnelling project in the city. Protection methods at Southwark station, Jubilee Line Contract 103, were specifically designed to limit the settlement of the Victorian Railway viaduct, a brick railway viaduct with arches more than 100 years old (figure 12).

Directly beneath this Railway Viaduct are two 7m-id platform station and a central 9.2 m-id concourse tunnel, with several interconnecting passages and a permanent ventilation and escape shaft. The tunnels are at 15 m below ground level, with a minimum of 5 metre clay cover. Predicted settlements were 140 mm, whereas the specifications for the Railway Viaduct stipulated an absolute settlement less than 25 mm, a differential settlement less than 1/1000 and angular distortions less than 1/500. The considerable rail traffic could not be interrupted. The consolidation and compensa-

Figure 11: ContAcTS: Control of Active Tunnel Settlements
tion grouting, performed by SOLETANCHE-BACHY was programmed into the sequencing of the sections of the new line.

Prior to the tunnelling operations, 2" metallic TAM were installed in boreholes drilled either sub-vertically from ground level below the arches of the viaduct (Fig. 14 - Southwark Station) or sub-horizontally from shafts, 4 to 20 meter deep. More than 4000 boreholes were drilled with a total length of 55,000 m.; the grid of the boreholes ranged from 1m*1.5m in the area of Southwark Station to 3m*3m in the area of Red Cross Way. For the automatic monitoring of the levelling points more than 1200 sensors were installed, producing about 30,000 daily level data recordings which had to be analysed and processed. To meet the requirements of this major project it was necessary to install a special survey system, called Géoscope, This system has now been in use there for several years.

The real-time display is easily understandable, making the survey system extremely convenient to use. The computer stations scattered around the job sites are connected by telephone line to the central supervisory PC computer. If an alarm is initiated, the persons in charge can check the movements on their screens, directly or from remote outstations. Using a database raised no difficulties for the team of ten site technicians with no special training in computer skills.

The Compensation Grouting

Four grouting steps were performed for the compensation grouting at Southwark Station.

1. A preliminary grouting phase in order to create a roof within the terrace gravels. The aim of this was to protect the bridge foundation against the risk of vertical fracturing during the ulterior grouting steps or against a concentrated uplift; this protection was deemed necessary in this case because of the relatively small clay of the 9.6 meter diameter tunnel; this preliminary grouting phase
was performed with Silacsol, an ultra-fine cement grout able to penetrate fine sands and to provide a strong bond between the grains.

2. The first phase grouting, performed before the passing of the tunnel excavation machinery; the grouting was done in the London Clay just below the Terrace Gravels, using a classical B/C grout. This grouting phase allowed pre-stressing of the soil and determination of the efficiency coefficient, which in this case equalled about 30%.

3. The second phase grouting was performed at the same time as the tunnel excavation in the space delimited by the two previous grouting steps. This second grouting phase had to be performed in a very soft way in order not to endanger the stability of the excavation face. Figure 15 gives an idea of the complexity of the daily grouting compensation scheme in this particular case, where full compensation of the expected settlements was required. For a 14m$^3$ daily excavation volume (corresponding to the enlarging of a 4.2 meter diameter pilot tunnel to a 9.6 meter diameter tunnel), it was necessary to grout 15 sleeves shared out over 15 grout holes and with a grouting volume of only 40 litres per sleeve (or 600 litres in total).

4. The third phase grouting to compensate the settlements occurring after the tunnelling and to come to a zero remaining settlement.

Figure 16 shows the settlements observed on pillar n° 95, which is located above the 9.6 meter diameter concourse tunnel, the two 7.4 meter diameter station tunnels and the three inclined tunnels of 6 meter diameter for the mechanical stairs. Several tunnel drifts were performed simultaneously, which makes it difficult to identify the different excavation phases. Over a period of one year, the pillar experienced maximum amplitude of the movement of 10 mm, a maximum settlement of 5 mm., where the theoretical models anticipated a settlement of 140 mm.

Puerto Rico Subway Construction

In order to reduce chronic traffic jams created by the rapid growth of the population of the city (1.2 million people, to increase by 20% in 2010), the government of Puerto Rico decided to construct a subway line. This was to be subsidised by the US Federal Government. The underground central part of this line runs under the University of Puerto Rico, and the preservation of historical buildings located in this area is a central issue for the local authorities.

The Joint Venture consisting of KIEWIT, KENNY and ZACHRY, won the bid for two stations (Rio Piedras and Universidad), 2 tunnels of 6 meters in diameter, to be excavated over 1 kilometre by earth
Figure 15: Example of Daily Compensation Grouting Scheme

Figure 16: Comparison between real movement and anticipated movement without compensation grouting

pressure balanced shield machine, and 4 tunnels of 6 meters diameter by NATM. In addition to this, 2 shafts are to be built using the pile and lagging support method on either side of the Rio Piedras and Universidad stations.

The Rio Piedras station is a large underground vault about 140 meters long, 18 meters high and 23 metres wide, located under an important business area and the heavy traffic on the Ponce de Leon avenue. The manmade excavation method, digging 3x3 metre drifts, was chosen for this part of the works considering the small depth of the overburden: only 2 meters.

One of the priorities of the Tren Urbano site is to minimise construction-related nuisances. The steps in constructing the Rio Piedras station are:

* Digging the grouting drift between the North access shaft and the South access shaft.
* Drilling the compensation grouting holes from this grouting drift.
* Digging 15 other drifts, underneath the grouting drift, to obtain a vault design.
* Excavation of the inside of the vault.

Compensation grouting is also used at several locations along the tunnel route between stations Rio Piedras and Universidad. Works performed so far on this site confirm the efficiency of the method, as proven by the following figures obtained on a typical section of the running tunnel (University Building):

* The estimated settlement was around 25 mm from a Finite Element analysis
* The settlement observed in a nearby park area, without compensation works, was 56 mm
* The settlement observed in University Buildings with compensation works, in a first phase were limited to between +2 and -2 mm; and after a short length of tunnel and a few cycles of feedback they were limited to nearly

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Figure 17: Typical layout of compensation grouting at station Rio Piedras, in Puerto Rico.

Figure 18: CYCLOPS total station operating on site to monitor structural movements.