It is a quite challenging objective to aim at covering in one paper the most recent evolution in piling and deep foundations technologies. This report has to be understood within the limits of the session 2b. It will therefore concentrate on the technological evolution of deep foundation techniques, addressing design issues only when directly related to execution aspects. For the same reason, it will not address the related codes, as a separate session of this conference is specifically dedicated to this topic. Monitoring and deep excavations being covered in other sessions, they will only be addressed when clearly relevant. We will try to show, from a practical viewpoint what are the main changes and the most exciting or critical issues over the last years, as they appear from the topics addressed in the literature or in the papers published in this session of the conference. As the paper mostly intends to explain trends and critical topics, it cannot be exhaustive, neither can it give the full description of the case studies. The author invites the reader to read the full papers, for a more complete description.

We have divided our report in three main topics: Piling, soil improvement and treatment and retaining walls. Anchors and tie-backs will not be treated, neither tunneling.

INTRODUCTION

Before addressing the technical issues, it is worthy to look at the current trends in today’s Deep Foundations market. We have been questioning hundreds of practitioners about their interpretation of their respective foundations market. The results of our inquiry will be the red line throughout our Report.

Pile foundations remain by far the most used deep foundations system worldwide (~50 %). Ground improvement technique keep progressing, though differently in the different continents (Fig.1)

The interaction between technology and design plays a determinant role in the evolution of our profession. The introduction of hydraulics and the exponential increase in available power on one hand, the miniaturisation and the introduction of electronic components on the other hand exerted a major influence on new foundation systems. The recent technological evolution has sometimes been so quick that our understanding of the phenomena hasn’t been following at the same tempo.

PILING

Very few general reports or papers on piling intend to present a classification of the different pile systems. Referring to the recent advances made in the Belgian NAD working group, we propose the following classification, mainly based on the stress state resulting from the pile installation procedure.
**CATEGORY I: HIGH SOIL DISPLACEMENT**

**DRIVEN PILES**
- precast concrete without enlarged base
- cast in situ, without enlarged bottom plate
  - with lost driving tube
  - shaft in plastic concrete
- cast in situ, with enlarged bottom plate ($D_b > 1.1D_s$)
  - with lost driving tube
  - shaft in plastic concrete
- cast in situ, with shaft in dry concrete, in situ formed enlarged base
  - steel pile
  - close ended
  - open ended with plugging

**SCREW PILES**
- without lost driving tube
- with lost driving tube

**CATEGORY II: LOW SOIL DISPLACEMENT OR LOW SOIL RELAXATION**

**DRIVEN PILES**
- open end steel pipe without plug
- H-profiles and sheet pile wall elements

**CFA piles with special provisions to limit soil relaxation**
- with overpressure
- with casing
- with large diameter of the hollow stam and small flanges

**CATEGORY III: SOIL EXCAVATION**

**CFA PILES without special provisions**

**BORED PILES**
- executed with temporary casing or under thixotropic liquid

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### Table 1. Pile classification

The piling market worldwide, as it appears out of our survey is shown on fig.2. The use of one or another piling technique is related to several factors:
- Geological issues and soil conditions
- Local historical factors
- Local construction habitudes

Looking closer to the figures, we can come to a few conclusions:

1. Driven piles are still extensively used, even considering the stricter regulations with respect to noise and vibrations in still more urban areas. The technological developments of economical alternative vibration-free systems (displacement auger piles, screw piles or jacked piles) offer a competitive answer without succeeding to put the driven piles out of the market. This is particularly the case in the “soft soils” regions, like the delta areas: Netherlands, Sweden, North America (partially). We will report on recent development and research and on a few papers presented in this session.

2. Alternatively, vibratory driven piles are non-equally used, though an increasing interest is expressed by the research community. Although no papers have been presented on this topic, we will analyse the recent advances in the understanding of these systems.

3. Displacement auger piles (ADCIP or screw piles) continue their development, but their application still varies geographically. This can partially be explained by soil conditions, but not completely. How can we explain, for example, that displacement screw piles represent 60% of the small diameter piling market in Belgium, but are still marginally used in the soft alluvial areas in Italy, or in the London clay. We will come back on this topic.
4. Bored piles are still widely used, especially where more difficult or heterogeneous soil conditions are encountered: France, Germany, North-America. A still larger part of this group is constituted by CFA (ACIP) piles, and cased augercast piles. We will try to see how the reliability of these systems has been evolving.

We will begin this report by quickly stressing which progresses have been made in the understanding of the global behaviour of piles and the influence this had on the evolution of the piling market. We will then concentrate on the technological evolution in piling techniques (new processes, machines...) together with specific lessons learned by the practice for each of these techniques. Specific case histories or particular references will be given when appropriate.

UNDERSTANDING OF THE PILE BEHAVIOUR AND RECENT EVOLUTION IN DESIGN

The authors believe the one of the most critical evolution in the understanding of the bearing behaviour of piles is due to the analysis of instrumented full-scale load tests with a particular emphasis on the works by W. Van Impe and M. Bustamante. Instrumented load tests have enabled to assess separately the end bearing and the shaft bearing capacity of piles.
The understanding that many piles bear the most of their loads by friction, certainly in the range of settlements, which are usually authorized, exerted a capital influence on the evolution of codes and design methods, which no longer put the accent on base resistance only. This evolution was particularly beneficial to piles installed by other means than driving and contributed indirectly to the increase of use of piles installed by lateral displacement of the surrounding soil.

The second largest evolution, partially linked to the first one, is the better understanding of the deformational aspect of pile design (Van Impe, 199?, Poulos, 2003, Mandolini, 2005). The fact that piles cannot be considered as no-settlement elements and hence the increase in use of piles as settlement reducers instead of rigid supports for concentrated loads. The developments of combined foundations, piled rafts and the works by Poulos, Mandolini or Katzenbach have provided for a new era of unusual pile foundation concept.

Last but not least the introduction of the Eurocode 7 and the partial safety factors approach, coupled to the better use of statistic approach to soil investigation enables for a more accurate design when properly applied. In this frame, new technological developments came to the market, trying to fully benefit of the evolution in design trends.

The evolution in piling techniques was oriented by the lessons learned out of the research as explained here above and made possible by the tremendous evolution of the technology over the last decades.

**Driven piles**

**Key issues**
- Environmental aspects (noise, vibrations)
- Monitoring (PDA, ...)
- Capacity prediction, set-up

For the first half of the 20th century, most piles were driven piles and their control was mostly based on the measurement of the set. Driven piles are still widely used worldwide (nearly 50 % worldwide), in no urban centres. Looking back to the answers to our questionnaire, we deduce that the market of driven piles is divided between cast-in-place (10%), precast (21%) and H or pipe piles (10%). The choice between these different systems again varies in function of soil, applied loads, or local factors.

Diesel hammers have been the reference for many years, but recent concern for noise impact and recent technological improvements have lead to the larger development and use of hydraulic hammers and more recently of vibratory driving. Hydraulic hammers have gained more and more popularity over the last years. Their Energy Transfer Ratio is higher than the one of diesel hammers (see table 2), but the advantages of the diesel hammers:
- relatively light weight,
- sturdy construction,
- no external power supply needed,
- lower cost
often lets the balance weight in the favour of diesel, at least for on-shore driving. In order to answer the environmental concerns, noise-reducing systems have been developed for application of diesel hammers in noise sensitive environment.

![Fig. 4 Driving silencer: special sound enclosure](image)

**Technical details of impact hammers**

Table 2 gives the technical details of most hammers in use (after Rausche, 2000).
Table 2. Hammer types and characteristics

<table>
<thead>
<tr>
<th>Type of hammer</th>
<th>Ram weight</th>
<th>Rated energy</th>
<th>Efficiency transfer Ratio (ETR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diesel</td>
<td>&lt;200</td>
<td>0.4-600</td>
<td>31 % (steel) 25% (concrete)</td>
</tr>
<tr>
<td>Hydraulic drop hammers</td>
<td>&lt;150</td>
<td>&lt;200</td>
<td>55-85 %</td>
</tr>
<tr>
<td>Self-monitored hydraulic</td>
<td>&lt;1500</td>
<td>&lt;200</td>
<td>55-85 %</td>
</tr>
</tbody>
</table>

The selection of the proper equipment is critical. A few brief rules should be respected:
- For friction piles, blow counts should be less than 80 blows per 25 m.
- For end bearing piles (hard driving of relatively short duration), one could extend this to 200 blows per 25 m.
- Compressive concrete stresses should be less than 85 % of the concrete strength.
- Tensile concrete stresses should be less than 70% of the equivalent yield strength for regular reinforcement or prestress plus 50 % of concrete tensile strength.
- For steel piles, stresses should be less than 90% for yield.

For specific jobs, the wave equation will be the most reliable tool to predict hammer and driving system characteristics.

Recent developments

The clean diesel-hammer: Berminghammer has recently introduced the “green” diesel hammer, without smoke emission. This diesel impact hammer is reported to ensure super clean combustion and unsurpassed fuel economy (90% less fuel), together with matching the energy transfer of the best hydraulic hammers.

The “hollow-hammer”: this recently developed hydraulic hammer enables a free-fall hammer to pass through the cylinder of the hammer, for the realization of an enlarged base with dry concrete (Franki type). The well-known Franki piles are still widely used worldwide because of their reliability. The enlarged base in dry concrete enables to limit and control the length of the piles and to reach high bearing capacities even in weak soils. The conventional Franki piles, despite their high geotechnical standards, are slow in execution and relatively expensive. The idea of using a hydraulic “hollow-hammer” gave a new impulse for a quicker execution of the Franki pile in Belgium and the surrounding countries.

Monitoring of driving operations: driving formulae

As previously mentioned, the main construction control applied for driven piles over many years has been the measured set for each piles. Driving formulae have been developed, from the very simplistic Dutch or Engineering News formulae to more sophisticated Gates or other proprietary formulae like the Danish or the Delmag formula.

Based on energy considerations, those formulae strive to provide a direct relationship between the measured set and the ultimate bearing capacity. As stated by Holeyman, (1997) the large factors of safety typically applied to derive the allowable value of bearing capacity underline the low reliability of these formulae. Major issues are (Rausche, 2000):
- The static resistance at the end-of-driving (EOD) is often lower than the total resistance (static and dynamic) at the EOD.
- The long-term capacity after a set-up period is higher than the static resistance at the EOD.

There has been a recent increasing interest for the change of capacity with time: the long-term ultimate pile capacity may range between 50 % and 1000 % of the EOD capacity. Sometimes relaxation occurs, but usually the capacity will increase with the time. Table 9-19 of the FHWA Manual on the Design and Construction of Driven Pile Foundations.
gives set-up factors in function of soil type. Recommended set-up factor varies from 2.0 for clay to 1.0 for sand and gravels, though observed values may reach 5.5 for clay and 2.0 for sand. Although it appears that set-up will usually be beneficial, the large scatter observed makes the non-calibrated use of a single value either conservative or dangerous. EOD formulae should therefore be used with much caution, as they can lead to misinterpretation. The only objective approach is the calibration of the driving curve with respect to the previous soil investigation and calculation of bearing capacity. Blow on Restrike (which is the testing of the pile after a certain waiting time) is now recommended, but it puts scheduling and time burden on the construction process. Rausche (2000) suggests that, for an economical and reliable capacity test, a 24 h restrike seems to be better than any other EOD based method or formula. We would like to add, unless a good correlation can be found with a sufficient and reliable soil investigation. In this context, automatic monitoring and dynamic analysis have been introduced (e.g. Rausche et al, 2004). Practitioners from North America even identify the generalised use of Pile Driving Analysis as one of the major advances in the field of deep foundations. Noteworthy, systems like PDA are much more widely used in the US than in Europe. Another session being dedicated to monitoring, we will not address this issue more in detail.

Vibratory driving

Key issues
- Vibro-driveability
- Bearing capacity

The use of vibrators in the field of deep foundations is known for several years now and has been widely applied, and reported by many authors. Viking (2002) reports that the first production units were built in Russia during and after the Second World War. In favourable soil conditions, the vibratory driving of H-profiles or tubes offers an economical alternative to impact driving. The technique uses a harmonic, vertical excitation force generated by a vibrator fixed on the top of the element to be driven. The recent developments of high frequency vibrators (up to 100 Hz) and variable frequency have helped vibratory driving to gain in popularity. Our survey identifies a little 10% of the market. The uncertainties of the method and the lack of reliability probably play a role in the limited propagation of these systems.

Holeyman (2002) provides a lot of useful information for a better understanding of the soil under vibratory loading. He summarizes the key engineering issues related to vibratory driving (Fig. 6)

![Fig. 6 Major issues in vibratory driving](image)

Table 3a and 3b out of Holeyman (2002) provide a good summary of vibrator’s data and recommendations for different types of soil and piles. Vibrator choice, however, remains mostly based on experience and field observations.

As stated by Rausche (2002), the method is still fraught with uncertainties. Among these, we can list:

Vibro-driveability
The vibratory driving of piles is based on the reduction of the static resistance of the soil due to the degradation of soil strength and the build-up of excess pore pressures (Holeyman, 2002). Although sophisticated models were proposed by many authors, the reliability of vibratory pile driving remains elusive. The models usually should allow predicting the required penetration time and enable the choice of the adequate hammer. Raussche (2002), though, recognises that the question of which hammer could drive the pile to the required depth often cannot be answered with sufficient certainty. Premature refusal can often be encountered.

In an interesting report from a research project at the BBRI, Huybrechts et al(2002) propose a simplified model for
prediction of driveability. The authors also provide a list of different reasons for premature refusal attributed to the soil, the vibratory hammer or the piles:

**Attributable to the vibratory hammer**
- Insufficient weight of the non vibrating part of the hammer.
- Insufficient eccentric moment of the vibratory hammer leading to a deficient vibration amplitude

**Attributable to the pile**
- Although the weight of the pile plays a favourable role, a too heavy pile reduces the vibration amplitude.

<table>
<thead>
<tr>
<th>Type</th>
<th>Frequency range (rpm)</th>
<th>Eccentric moment (kg.m)</th>
<th>Maximum centrifugal force (kN)</th>
<th>Free hanging double amplitude (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;Standard frequency&quot;</td>
<td>1300-1800</td>
<td>up to 230</td>
<td>up to 4,600</td>
<td>up to 30</td>
</tr>
<tr>
<td>High frequency</td>
<td>2000-2500</td>
<td>6 to 45</td>
<td>400 to 2,700</td>
<td>13 to 22</td>
</tr>
<tr>
<td>Variable eccentricity</td>
<td>1800-2300</td>
<td>10 to 54</td>
<td>600 to 3300</td>
<td>14 to 17</td>
</tr>
<tr>
<td>Excavator mounted</td>
<td>1800 to 3000</td>
<td>1-13</td>
<td>70 to 500</td>
<td>6 to 20</td>
</tr>
<tr>
<td>Resonant driver</td>
<td>6000</td>
<td>50</td>
<td>20,000 (in theory)</td>
<td>Self destructing</td>
</tr>
</tbody>
</table>

**Table 3a. Vibrator types**

<table>
<thead>
<tr>
<th>Cohesive soils</th>
<th>Dense cohesionless soils</th>
<th>Loose cohesionless soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>All cases</td>
<td>Low point resistance</td>
<td>High point resistance</td>
</tr>
<tr>
<td></td>
<td>Heavy piles</td>
<td>Light piles</td>
</tr>
<tr>
<td>High acceleration</td>
<td>Predominant shaft resistance</td>
<td>Predominant end resistance</td>
</tr>
<tr>
<td>Low displacement amplitude</td>
<td>Requires high acceleration for fluidization</td>
<td>Requires high displacement amplitude and low frequency for maximum impact to permit elastoplastic penetration</td>
</tr>
<tr>
<td>Predominant shaft resistance</td>
<td>Requires high acceleration for fluidization</td>
<td></td>
</tr>
</tbody>
</table>

**Table 3b. Vibrators classifications (Holeyman, 2002 – after Rodger and Littlejohn)**

<table>
<thead>
<tr>
<th>Recommended parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>v &gt; 40 Hz v : 10-40 Hz</td>
</tr>
<tr>
<td>a: 6-20 g a: 5-15 g</td>
</tr>
<tr>
<td>s : 1-10 mm s : 1-10 mm</td>
</tr>
</tbody>
</table>

**Attributable to the soil**
- When the pile is too elastic (e.g. too small pile), the vibration amplitude can be insufficient at the pile base, too much transverse vibration can occur preventing the pile from penetrating.
- Clutch friction of poor quality sheet piles

As can be seen out of the last section, the driveability in cohesive soils pose more questions than in cohesionless soils; interestingly, it has been less studied. A recent case study (Tavernaro, 2006)
reports the vibro-driving of large caissons, 3 m in diameter, at a depth of 45.7 m depth in 30 m alluvium and fine sands and clays, and finally into weathered rock. The vibatory-driving proved to be challenging, according to the authors themselves, and the skin friction could in some cases not be overcome by the largest vibratory hammers available today. However, and mainly due to economic reasons, systems based on vibratory driving have been gaining in popularity like the VCC system (Vibrated Concrete Columns - Keller), or similar systems by Liebherr or Bauer.

Large scale applications have recently been reported for the vibratory driving of large concrete tubes diameter 12 m to a depth of 25 m for the foundations of a bridge on the Yang-Tse river in China. The driving operations employed four APE 4B hammers with 683 kgm eccentric moment and 750 kW each with frequencies between 19.4 and 20.8 Hz. Most of the penetration occurred prior to driving due to the static weight of the hammers and transfer beams. One of the issues here was caisson thickness: the caisson was diaphragming to an oval shape and then back to a round shape at a rate consistent to the speed of the vibratory hammer.

Bearing capacity

There are relatively few publications providing guidelines for calculating the bearing capacity of vibrated piles. Raussche (2002) explains that many attempts have been made to deduce the pile capacity on the basis of the driving resistance but that, to date, it is usually still required to re-drive the pile with an impact hammer for acceptance. Viking (2002) correctly states that too few case studies provide enough detailed information to increase the knowledge in this field. Borel & Guillaumé (2002) showed that, for permanent structures, no example could be found of accepted vibratory driven piles without field load tests.

Some recent analysis and comparative tests in Europe and in the US (e.g. Borel, S. et al, 2002) have shown that the bearing capacity obtained by vibrated (sheet) piles can be significantly lower than the one obtained by driven piles in the same soil conditions. Further analysis is needed, but the extrapolation of installation factors corresponding to impact-driven piles should be used with caution.
Jacked piles

Key issues
- Equipment capacities
- Relaxation

Jacked piles are becoming a more popular displacement pile option for urban development (Lehane, 2005). For many years, jacking systems have been presented as alternative to driving or vibratory driving of sheet piles or tubes in urban areas. The limited penetration capabilities of the systems and the relative high cost of the installation remained a limitation for a larger use. Both the need of vibration-free systems and the increase in equipment capabilities have been enhancing the possibilities offered by this technology. This new equipment, however, as stated by Poulos (2003) can be affected by significant side effects due to its weight. This aspect should be analysed by the promoters of these systems in future publications. Two papers presented at this conference deal with new developments of jacking techniques. The first one (Filip, 2006) is dealing with pile elements; the second one (Goghe et al, 2006) covers more the aspects of retaining walls and will be treated later in this report. Both authors stress the “sustainable” character of the systems.

Filip describes the design and construction of high capacity steel elements installed with the use of purpose built quiet and vibration-less equipment. The steel bearing piles were prolonged out of the ground and used as bridge columns and abutments, and then cast directly into the soffit of the bridge deck. The piles were installed into a glacial boulder clay, stiff becoming very stiff, and containing sandy lenses. The piles were installed at a depth of 12 m for the abutments and 16 m for the piers. They were formed out of 4 Hoesch Larssen 43 sheet piles, clutched together to form a 750 mm square box. The piles were considered to plug during installation, but it is not clear if this was observed during execution. Installation occurred by a four-cylinder “push-pull” system, assembled and mounted on a Liebherr LRB255 leader rig, with a 30 m high fixed lead. (fig. 11).

Once the pre-assembled box piles lifted up into location, each of the sheet piles are pushed down in sequence, relying on the skin friction generated by the other three sheets and self-weight of the equipment. Two maintained load tests were carried out to 1,5 times the safe working load (1440 kN), two weeks after driving and prior to partial filling with concrete. A lateral load test on a pile-abutment was also carried out.
One of the often cited advantages of jacked piles is that the installation load is measured and can be considered in sand to be equivalent to (or less than) the pile’s capacity. However, compiling the data from different authors, Lehane (2005) discusses the possible “relaxation” affecting jacked piles in sands. Ratios of installation loads to static capacities are plotted on fig 12 against the so-called equalisation period. These data appear to suggest that the static ultimate base capacity will be less than that mobilised during installation.

2. Bored piles

Cased bored piles (drilled shafts)

Key issues
- Curing of borehole
- Post-grouting
- Drilling fluid: polymers

Conventional bored piles or drilled shafts remain popular (>25 % worldwide). The main reason for this is their ability of carrying out high concentrated loads even in difficult ground conditions. There exists a large variation of cranes, rigs and attachments, … to install large diameter bored piles in the most variable site conditions. In the same time, there exist a lot of variations of execution, with respect with the temporary support of the excavation: casing, bentonite, and more recently polymers. Typical case studies illustrating this variety are submitted by a few authors. Davie et al (2006) give a case history on drilled piers of diam. 1.93 m installed in Tenesse, USA for the foundations of a massive gantry crane. The piles, installed under partial temporary casing were installed with a truck-mounted rig. Farouz and Failmezger(2006) explain how the design of drilled shafts installed in a cobble formation, were optimized on the basis of pressuremeter tests.

In Europe, the most recent developments are directly linked with the increase in machine power and capacity and the improvement of the excavation tools, which enable the easier installation of bored piles in very resistant rock. The last generation rigs are characterized by high installed powers resulting in high available torques and crowd.

When addressing the issue of construction related concerns, bored piles certainly concentrate most of the questions. A number of key factors influence the axial capacity of piles (O’Neill (2001), Poulos (2003), Maertens (2003))

- Borehole roughness in soft rocks
- Presence (or absence) of highly degraded smeared rock at the interface
- Effects of seams or discontinuities in the rock.
- Cleaning of the pile basis
• Influence of the drilling fluid on the bearing capacity.

This type of questions has led to the increased control of the bored piles execution.

**Execution control and monitoring**

As far as the quality control of the piles is concerned, recent experiences have been leading to the generalized sonic coring of the piles together with bottom core drilling in order to check the integrity and the soil-concrete contact at the bottom of the piles and the integrity of the shaft (e.g. Maertens et al-2003).

Sonic coring (Cross-hole sonic logging) is a low strain integrity testing method commonly used in Europe and the US to evaluate the condition of concrete within cast-in-place piles and more specifically bored piles. The integrity of the concrete can be evaluated by determining the propagation time and relative energy of an ultrasonic pulse as it travels between two ultrasonic probes or transducers. Fig. 13 shows typical results of a sonic coring test performed on bored piles diameter 800 mm installed down to rock at a depth of 19.00 m. The panel 2-3 shows a slight irregularity at the bottom of the pile, enlarged on the detail shown on fig.26 c

![Fig. 13 a and b: Results of Sonic coring in bored piles – two panels of measures.](image)

![Fig. 13 c Enlargement of slight default zone on panel 2-3](image)

This testing method is usually completed by a core drilling of the base of the pile through a reservation tube placed on the reinforcement before concreting (fig. 14)

![Fig. 14: Core drilling through base of bored piles](image)

These controls have been questioning the correct curing of the pile base in many cases.

**Post-grouting**

Alternatively, post-grouting of both the shaft and the base of the piles has been developed in order to minimize the settlements and to increase the reliability of the piles. Both Dapp et al and Klosinsky and Szymankiewicz present a paper dealing with base grouted bored piles, respectively in Poland and in the US.

The construction of normal bored piles is subject to some physical and construction limitations, as repealed by Dapp et al (2006):
- Strain incompatibility between side shear development (required movement 0.5 to 1% of pile diameter) and base resistance mobilisation (10 to 15% of pile diameter).
- Soil stress relaxation due to the process, especially in cohesionless soils. Correct cleaning and curing of the pile base as explained here above.

The post-grouted bored pile is normally constructed, but a grout delivery system is foreseen which enables the injection of grout under high pressure once the concrete has gained sufficient strength (Bruce, 1986). The post-grouting can concern both the shaft and the base of the pile. Base grouting can be realised by two main mechanisms (Dapp et al (2006)):
- the flat-jack: grout delivery tube to a steel plate with a rubber membrane wrapped underneath
- the tube-a-manchette (sleeve-port): U-tubes covered by a tight fitting rubber sleeve and arranged in various configurations at the bottom of the pile.

Dapp et al report on the results of post-grouted piles on 10 projects since 2003. These projects include 17 full-scale load tests and nearly 600 bored piles (drilled shafts). The authors explain the normal quality assurance program practiced in the Southeastern US:
- pilot grouting
- load testing program on the initial piles. The main purpose of this program is to: 1) corroborate the design; 2) evaluate the construction process; 3) establish grouting criteria. These criteria consist of three main components:
- grout pressure
- upward displacement
- minimum grout volume

Evaluation of the skin friction (taking into account the direction of the loading) will be necessary in order to determine the target maximum upward displacement criterion. This value, according to Dapp et al, is typically around 6 mm, but higher values (up to 15 mm) can be allowed in very clean sands. The authors review various case histories, showing how the measured upward displacement can be linked with some forms of construction difficulties, waiting times, caving of the shaft, concrete overrun, etc.. They introduce the Verified Load Ratio (VLR):

\[
VLR = \frac{\text{Applied Grout Force}}{\text{Design Load}}
\]

The Applied Grout Force is the total of the upward and downward grouting force, and thus equal to two times the grout pressure times the nominal shaft diameter. The applied grout force can be considered as a verification of load capacity, given that downward directed side shear is at least as great as the mobilized side shear from the upward portion of the grout force. The upward displacement and VLR for various bored piles is shown on fig. 15.
Klosinski and Szymankiewicz report on the Polish experience of post-grouted bored piles, mostly 1.0 m to 1.5 m in diameter, founded in granular and cohesive material. The soil investigation results are not enough detailed to get the full benefit of the load tests results. Depending on the soil conditions and the pile construction, the authors report a reduction of settlement of 30 to 50 % under working load (see for example fig.16)

Both Dapp et al and Klosinsky emphasize the enhancement of the reliability and the possibility to mitigate construction problems, as bottom hole cleanout.

Polymers

Another aspect of the execution of bored piles, which regularly comes back in recent papers, is the interaction between the drilling fluid and the bearing capacity, and the increase of interest for polymer-based slurries. Theoretical and laboratory testing as well as case studies have recently been reported by several authors (Majano and O'Neill (1993); O'Neill and Reese (1999), Thasnanipan (2003); Bustamante (2005)).

Bentonite and polymer slurry do react differently when in contact with soil:
- Bentonite maintains the soil particles in suspension and stabilises the borehole wall by its density and the formation of cake.
- Polymers do not suspend sand. Sand particles will therefore sediment at the base of the borehole. This sedimentation is stable with the time due to progressive agglomeration of the slurry and migration into what O'Neill and Reese (1999) call "oatmeal" material. Sometimes, as reported by Thasnanipan (2003), a small percentage of bentonite is added to the polymer slurry, in order to reduce the high filtration in the sand layers.

Fig. 16 Full-scale load test on post-grouted and no post-grouted piles – Klosinski (2006)

Fig. 17 Generalised behaviour of slurries in borehole (a) bentonite slurry (b) polymer-bases slurry – Thanasnanipan (2003)

Although the use of polymer-based slurry is getting more accepted, information on the desirable properties remains limited. Table 4, out of Thasnanipan (2003) shows the range of the properties specified by the ADSC (1999), O'Neill and Reese, and those commonly applied in Bangkok soils.
Table 4. Properties of polymer slurries specified by ADSC and polymer-based slurry used in Bangkok - Thanasanipan (2003)

<table>
<thead>
<tr>
<th>Property</th>
<th>ADSC (at 20 °C)</th>
<th>Majano et al., 1994</th>
<th>Common Practice in Bangkok (PBS***)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cc)</td>
<td>1.02</td>
<td>0.995 to 1.01</td>
<td>Maximum</td>
</tr>
<tr>
<td>Viscosity (cP)</td>
<td>40 - 90*</td>
<td>33 - 45</td>
<td>Maximum</td>
</tr>
<tr>
<td>pH</td>
<td>7 - 12</td>
<td>8 - 11.7</td>
<td>Maximum</td>
</tr>
<tr>
<td>API Sand Content</td>
<td>1.0</td>
<td>0 to 1</td>
<td>Less than 1%</td>
</tr>
</tbody>
</table>

*or as recommended by manufacturer and approved by geotechnical engineer; ** Polymer-based slurry

Most of the studies we refer to are based on the use of partially hydrolysed polyacrylamide (PHPA) polymers. In a paper reported at this conference, Lennon et al (2006) report the use of a new polymeric technology specifically designed for use in piles and diaphragm walls. This new polymer system comprises several different polymers that work together to help provide the required criteria. They mostly combine the use of CDP and MPA base products. The authors report the successful use of this vinyl polymer support fluid on several jobsites in Glasgow, Scotland. The case studies include limited geotechnical information, and no detailed technical information (density, viscosity, pH,..) about the polymer used. The results of full-scale load tests show limited settlements at working load. It is interesting to note that Bustamante (2005) reports on three case studies using polymer slurries for the execution of very long piles of large diameter, which the only successful one is reported to be the one using CDP-based polymer slurry.

**Hybrid systems and new applications**

Considering the extremely strict requirements in terms of settlement for the foundations of a new viaduct for the high-speed railway train in Battice (Belgium), a new hybrid bored pile system was used, in order to overcome some of the risks connected with conventional bored piles (too large displacements). The structure is conceived in inclined supports as shown on fig 18. Each pier was founded on 30 piles, both vertical and inclined.

![Fig. 18 Typical cross-section – Battice viaduct - 2003](image)

The piles diameter 620 mm were drilled under casing like conventional bored piles. Once the required depth reached (top of the rock: varying between 8.00 and 32.00 m), a steel H-profile was inserted. This profile was provided with a steel plate underneath and reinforcement rings were welded on the profile (see fig.19). The profile was then top-driven for a few blows, ensuring the plate with the knife to be anchored within the weathered rock layer. The pile was then concreted using high slump concrete.

![Fig. 19 Steel bottom plate and knife – Post driven bored piles – Battice 2003](image)

The piles were instrumented and load-tested by the LCPC and showed very good results, combining a very good shaft friction with an enhanced end-bearing capacity.
Continuous Flight Auger piles (Augercast piles)

Key issues
- Loosening of the surrounding soil
- Influence of the equipment
- Proprietary systems

The most spectacular penetration of the piling market worldwide over the three last decades is certainly this one of CFA piles (22 %, worldwide according to our survey, reach 50 % in some areas). Despite its apparent simplicity, or even because of it, the system has been the topic for large controversial debates among the piling specialists. The main risk is associated with the sensitivity of the system to the rig operator and to the capacity of the equipment itself, which, in case of insufficient installed power and hence of insufficient penetration rate can lead to “over-excavation” of the soil. Van Weele, very early, described the effect of a badly installed CFA pile on the surrounding soil. A lot of recent papers have been producing data on this topic. The most recent information can be found in Hannink & Van Tol (2005), Evers et al (2003). Most of these case studies, however, report too few information about the equipment used, which is of outstanding importance on the final result.

A lot of authors have covered the systems specificities as well as the mechanics of soil decompression in connection with an insufficient penetration rate (Bottiau & Massarsch, 1988; Viggiani, 1988) and the importance of the concreting phase (Van Impe, 1997).
Based on these considerations, a second generation of CFA piles has been growing through the development of proprietary systems as the Starsol pile (Soletanche) or the PCS pile (Socofonda). The main advantages of these more recent systems, are the following:

- High capacity Rotary drive units combining maximum torque > 20 Tm with rotational speeds > 8 to 10 rpm.
- Crowd (pull-down) forces exceeding 10 T.
- High capacity concrete pumps (50 to 70 m$^3$/h).
- Auxiliary retractable concreting tube
- Full monitoring of the installation measuring in real time.
  1. The vertical thrust $N_d$ on the auger while drilling in (kN);
  2. The vertical penetration speed during drilling $v_i$ (m/s)
  3. The torque $M_i$ while drilling in (kNm).
  4. The rotational speed $n_i$ during drilling in (r/min).
And during concreting:
  5. The total volume of concrete (or grout) injected and the over-consumption;
  6. Ideally, the detailed and continuously monitored volume of concrete injected
  7. The vertical upward speed while drilling out.

The monitoring system allows the operator to follow and react in real time; it allows also for an adequate documentation of the pile installation by providing out-prints of the recorded data (see fig 39). The correct use of this abundant information is questionable. Bustamante (2003) published an interesting paper with respect to the potential limitations of monitoring if not properly used. In particular, he showed that the integration of the recorded parameters in the model of Viggiani, could lead to false conclusion. The piles installed in the sandy soils of Le Havre, didn’t satisfy the theoretical equation of Viggiani: $V > np$

$V$ = rate of penetration of the auger (m/min)
$n$ = revolution rate of the auger (r/m)
$p$ = pitch of the auger (m/r)

Both the pile load tests and the CPT tests performed after the pile installation, showed however that no soil loosening did occur.

Bustamante finally states that engineering judgement should never be forgotten when analysing the lot of information coming from monitoring systems. We moreover fully support his statement that monitoring remains a relatively effective and reliable tool provided the presented records

- Come above all from rough data which were not subjected to prior corrections or smoothing
- The sensor selected to measure each respective parameter has the capability to do it
- Is examined in its entirety by taking into account the real soil conditions, the rig characteristics, but also the many possible incidents, which are so common on site.

Finally, considering the importance of the equipment parameters and the installation factors, the final monitoring using control soil investigation (e.g. CPT executed after pile installation) or the certification of proprietary systems remain the best way to evaluate the real installation influence.

![Fig 23 Typical out-print of the monitoring system for PCS piles](image-url)
Auger piles with larger stems have also been developed, also called partial displacement like the PCS-lambda pile. This pile was used extensively for many foundations for the construction of bridges or piled embankments for the Highspeed Railway line in Belgium. The main advantages are the following:

- Applicable in nearly all soil conditions, even in dense sands and gravels, where displacement piles will be difficult to install.
- Rigid auger allowing for tight verticality tolerances.
- Large stem enabling the placement of a reinforcement cage before concreting.
- Partial displacement provided the installed power of the piling equipment is correctly dimensioned with respect to the soil conditions.

Fig. 24 shows the first application in Belgium of PCS-lambda piles diameter 60 cm at a site of Brussels-South, presenting a resistant sand layer with variable resistances and thickness. The piles had therefore to be installed to level +3.00.

![Fig. 24 PCS lambda pile at Brussels South – F. Theys - 2003](image)

Full-scale load tests were performed on several piles showing a very good behaviour. They can be consulted in F. Theys, 2003.

**Cased CFA piles or front-of-the-wall systems**

**Key issues**

- Soil loosening
- Pile walls

Between the conventional bored piles and the auger-cast piles, new systems have been introduced called Twin Rotary Drive Drilling System or Front-of-the-wall system. The principle is based on the simultaneous drilling of casing and auger by two rotary-drives rotating in opposite directions. The soil, which is loosened at the auger tip is transported to the surface (gate at the top of the tube underneath the rotary drive) on the auger flights.

![Fig. 25 Cased CFA pile](image)

This system has been specially developed for the execution of secant pile walls just in front of existing buildings for small to medium excavation pits in urban centres as we will explain in our section on retaining walls. The system also evolved to the drilling of larger diameters, for bearing piles. The speed of execution and the compactness of the equipment offer an economical alternative to more conventional systems. Here again, the technological evolution was more rapid than its analysis. Few correctly documented load tests have been published on this topic (e.g. Theys, 2003). Bustamante (2001) identified a series of advantages with regard to CFA piles:

- Less disturbance (loosening) of the surrounding soil, especially in sandy soils
- Limitation of the over-volume of concrete.

We believe that this point-of-view needs to be nuanced. Particularly, the influence of the equipment should not be underestimated. It should be observed that most twin rotary drive units are characterised by low installation torques. Within these limits, the beneficial influence of high capacity CFA piling rigs as well as the injection of the concrete under pressure will eventually result in higher bearing capacities. The (limited) available load tests show that the system clearly
should be considered as a bored pile with a reduced base bearing capacity. We give the example of a control CPT after the execution of cased CFA on a jobsite for the High-speed Railway Line (Loenhout-Belgium, F. Theys, 2003). Fig.26 clearly shows the reduction in cone resistance at the level of the pile base. The shaft friction, in the same time, will not benefit of the stress improvement linked to the concreting phase of a CFA pile.

Conceptually classified in the category of “auger” piles, the displacement auger piles are characterized by some major differences. Starting from the beginning, it is desirable to (re)define:
1. auger piles allowing soil de-compaction;
2. auger piles with lateral soil displacement.

Fig. 26 Reduction of the cone resistance for cased CFA pile – Loenhout - Belgium

Full-scale load tests showed that installation factors were clearly those of bored piles.

**Displacement auger piles (Screw piles)**

**Key issues**
- Equipment and systems specifics
- Dependence on soil variations
- Installation control

Displacement auger piles know an important development worldwide (currently 6% of the total piling market, the screw piles are differently represented in the different regions: up to 30% in Belgium, 20% in Japan). In his general report at the Conference of Osaka, Lehane (2005) stated that it is only a matter of time before they will dominate the market of medium scale bored piles.

Without intending to classify the systems, it is interesting to show which are the main differences in term of shape, and execution process.

Many authors have been publishing about this topic. One can refer in particular to Van Impe (1988, 1989, 1994), Bustamante (1988, 1993), Huybrechts (2001).

Most of the first generation of displacement auger piles are characterized by a short auger head intending to displace the soil very rapidly. The Atlas auger head for example (fig.28 Atlas auger head) results in a very stiff displacement and in a helical shaped pile.

This pile has been, in its category, the most extensively used and studied over the last three decades. Its particular
shape, realising a perfect pile-soil contact and hence a very high shaft friction and its capability to achieve very high bearing capacities even in mediocre soil conditions helped to make it one of the best-seller of its category. It is installed by means of a purpose built machine, with a base rotary drive.

![Atlas auger head](image1)

**Fig. 28** Atlas auger head.

![Atlas pile](image2)

**Fig. 29** Atlas pile.

Recent purpose-built Atlas rigs (BT 60) offer an installed power of more than 40 T.m and enable to install up to 180 m of daily pile production even in difficult soil conditions.

Other more recent systems allow for some soil excavation at the base, transporting the soil from the tip, and displacing it upwards. This of course is favourable in terms of penetration into dense soil layers, but the influence on the achieved bearing capacity should not been underestimated. Fig. 30 shows the displacement process of more recent systems.

![Omega auger head and Berkel auger head](image3)

**Fig. 30** Second generation displacement screw piles

Finally, the concrete casting procedure exerts a major influence, which is not always considered with the importance it should request. Some systems just cast the concrete under gravity, others inject under overpressure as CFA piles. The influence of the concreting phase is important and has been described among others by Van Impe (1988).

The execution process as described above obviously influences the final result. Main systems used in Belgium were part of two comprehensive load-test campaigns— one can in this respect refer to the proceedings of the two Symposia on Screw piles, Brussels, (A. Holeyman, 2001) and J.Maertens & N.Huybrechts(2003)). These campaigns result in global conclusions and installation coefficients, but some particularities associated with each system were also noticed. We want to present here some general points of attention, which are valid for all displacement auger systems.

In a paper presented to this conference, Gwizdala et al describe recent experience in Poland with the Atlas pile and shows two case studies with well-described soil investigation including CPT tests and Static Load Tests in mostly clayey and sandy soils. He clearly introduces the
particularity of the screw pile system and its dependence towards local soil resistance variation. We try to explain hereunder why screw piles should be considered as a completely specific piling system.

The combination, which governs the successful installation of displacement auger piles is:

- Intensive and reliable soil investigation.
- Specific design method.
- Adequate and reliable control of the pile asset during execution.

Displacement auger piles are more sensitive than other pile systems to the interplay between soil type and equipment capacity. Furthermore, the final bearing capacity highly depends on the precautions taken during the execution. This pile type is geotechnically speaking at the confluent between bored piles and (driven) displacement piles so that it should not be treated by one or another design approach, but well specifically.

Displacement auger piles, if treated by analogy with bored or CFA piles, will be requested to penetrate into dense layers much more then needed for achieving a comparable bearing capacity as well as impossible to achieve technologically. On the other hand, if treated by analogy with driven piles, because of the soil compaction, the problem of the correct choice and reliable control of their asset is of outstanding importance.

For this reason, this category of piles has to be specifically approached. Although a more reliable form of control becomes available (energy measurements, see below), the importance of an extensive soil investigation is more crucial than with other systems.

As displacement auger piles, by their conception, “feel” quicker then other similar systems any change in soil condition, their depth (or asset) will vary more according to the surrounding soil conditions. The essence of system is to benefit as much as possible of locally better resistances, but of course this should be related to previous soil investigation. Insufficient available soil information can have more dramatic importance with this pile type then others. Especially in changing soil conditions, the density of CPT, boreholes or SPT data should be increased. This is common practice in Belgium, but less evident in other countries. Though, Petersen et al (2002) have presented an interesting paper in this respect showing the importance of an adequate “guidance” of displacement auger piles by additional CPT execution on a project in Florida. Fig. 31 shows, as a matter of example, a longitudinal soil profile with typical depths of displacement auger piles, which clearly indicates a large variation following the soil resistance.

Fig. 31 Typical variable soil profile and asset of displacement auger piles (Petersen, 2002)
**Control of the execution**

As previously explained, the choice of an adequate asset level should be directly related to the in-situ soil investigation. The larger variation of depths should be accepted as an intrinsic component of the system. It should not be tried to achieve impossible anchorages into resistant layers but instead to benefit of any increase of the soil resistance.

The logical question that arises is: how can we control the correct execution and the respect of the asset. Van Impe (Ghent, 1988) or more recently Bottiau et al (Stresa, 1989; Vienna, 1999) have been emphasizing the importance of the monitoring of a number of parameters during the pile performance. The way these parameters are controlled can vary between the careful annotation by the foreman to the most sophisticated monitoring systems as explained before.

These parameters can be used for comparison between the available soil information and the calibration of the execution. The common use should be the following. Based on the preliminary soil investigation campaign, the site is divided in different areas of (slightly) different asset levels.

The first installation of the piles should be “calibrated”, in order to characterize the encountered layers of soil and to determine the adequate torque and/or penetration speed to be used as a reference. The first piles should be installed in the close vicinity of reliable soil data. The layers of soil are then defined and the installation parameters are measured.

For each pile installed far apart from soil information and, it is then possible to compare the installation parameters and to subsequently proceed to the control of its bearing capacity on site.

Fig. 32 shows the installation torque in parallel with the soil profile (CPT) for an Atlas pile.

The method can be refined by the calculation of the installation energy. If all the above-mentioned measurements are available, a specific installation energy $E_s$ can be derived,

$$E_s = N_d v_i + n_i M_i / \Omega . v_i \text{ in kN/m}^2.$$

with

- $N_d = \text{Thrust force (pull-down) in kN}$
- $v_i = \text{downward speed in m/sec}$
- $n_i = \text{rotational speed in m/sec}$
- $M_i = \text{Torque in kN.m}$
- $\Omega = \text{pile section in m}^2$

This installation energy can be related to the soil investigation. The author has previously explained (Vienna, 1998) how the installation energy, correctly calibrated, can be used to control on site the bearing capacity of installed piles.

Fig. 33 shows the measured bearing capacity derived from the energy formulae vs this one calculated on the basis of CPT tests.
**Micropiles and small diameter grouted piles**

To fulfil the picture of recent pile developments, this category would need another paper. Many recent technological developments have been made, especially with the construction of equipment combining higher capacities on compacter rigs. The introduction of electronic recording of the drilling parameters and injection volumes have provided for a better installation monitoring.

We will not cover this topic in detail.

**SOIL IMPROVEMENT**

Soil improvement techniques are gaining in popularity and understanding, especially for large infrastructure projects or logistical platforms. Here again, the technological developments have driven new practical applications.

Terashi and Juran (2000) have provided an excellent, though limited, State-of-the-Art report in Melbourne and we will use some of their tables to introduce the classification needed between all the existing Ground Improvement methods. They classify the wide variety of methods in the following way:

- **Replacement**: the simple replacement of soft inappropriate soils by a good quality foreign material.
- **Densification**: the densification of loose granular soils, waste or liquefiable soils by one of the following methods:
  - Vibro-compaction (vibro-flotation, vibro-rod)
  - Heavy tamping (Dynamic Compaction, dynamic consolidation)
  - Sand Compaction Pile
- **Consolidation/Dewatering**: preloading associated with vertical drainage, vacuum consolidation, are typical applications of consolidation
- **Grouting**: placement of a pumpable material which will subsequently set or gel in pre-existing natural or artificial openings (permeation grouting) or openings created by the grouting process (displacement or replacement grouting), (ASCE, 1995)
- **Admixture stabilization**: mixing chemical additives with soil to improve the consistency, strength, deformation characteristics, and permeability of the soil. The most frequently used additives in the current applications are lime and cement. These are the well-known Deep-Mixing methods.
  - Thermal stabilization: heating and freezing.
  - Reinforcement: creating a composite reinforced system by inserting inclusions in predetermined directions to improve the shear strength characteristics and bearing capacity of the soil. This includes several techniques as: deep-mixing elements, fiber reinforcement and geo-synthetics, ground anchors and soil nailing, lime-cement columns, micropiles, vibro concrete columns, stone columns...
  - Miscellaneous

It is obvious that we will not aim at discussing the whole range of these techniques in detail. A few recent research works or new developments seem to emerge, which we would like to discuss, assisted by the papers presented at this session.

**Deep Mixing Methods**

**Key issues**

- Reproductability
- Strength scatter

The Deep mixing methods originate in the 70’s, together in Japan and in the Nordic countries, first using lime as the binder. The techniques evolved in Wet Mixing using a cement slurry in Japan, whereas a dry lime/cement mixture was used in the Nordic countries. According to Massarsch (2005), deep mixing can today be classified with regard to the method of mixing (wet/dry, rotary/jet-based, auger-based or blade based) or the type of binder used. Bruce and Bruce (2004) provided a classification of 24 types of Deep-Mixing systems clearly showing the explosion of the systems.

Dry deep mixing is mostly used in the Nordic countries (3 to 4 millions linear meters yearly), with the following typical figures (Massarsch, 2005):
- **Mixture**: 50%lime/50%cement
- **Targeted strength**: 100 kPa to 200 kPa
- **Amount of binder**: 80 to 200 kg/m3 of stabilised soil
Wet Deep Mixing uses a cement slurry, mixed in the soil by means of several different tools (see table xx), in order to create a relatively rigid column or wall element (Massarsch, 2005).

**Mixture cement**

- **Targeted strength**: > 1 MPa
- **Amount of binder**: 80 to 450 kg/m³ of stabilised soil

More details of the compared Nordic and Japanese methods are given in table 6, from Massarsch, 2005.

### Comparison of European and Japanese dry mixing techniques

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Details</th>
<th>Nordic technique</th>
<th>Japanese technique</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixing machine</td>
<td>Number of shafts</td>
<td>1-3</td>
<td>1-4</td>
</tr>
<tr>
<td></td>
<td>Diameter of mixing tool</td>
<td>0,4 m to 1,0 m</td>
<td>0,8 m to 1,3 m</td>
</tr>
<tr>
<td></td>
<td>Maximum depth of treatment</td>
<td>25 m</td>
<td>33 m</td>
</tr>
<tr>
<td></td>
<td>Position of binder outlet</td>
<td>the upper pair of mixing blades</td>
<td>Bottom of shaft and/or mixing blades (single or multiple)</td>
</tr>
<tr>
<td>Injection</td>
<td>Variable</td>
<td>Maximum 300</td>
<td></td>
</tr>
<tr>
<td>Pressure</td>
<td>400 kPa to 800 kPa</td>
<td>Kpa</td>
<td></td>
</tr>
<tr>
<td>Batching plant</td>
<td>Supplying capacity</td>
<td>50 kg/min to 300 kg/m³</td>
<td>50 kg/min to 200 kg/min.</td>
</tr>
</tbody>
</table>

### Comparison of European and Japanese wet mixing techniques

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Details</th>
<th>On land, Europe</th>
<th>On land, Japan</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixing machine</td>
<td>Number of mixing rods</td>
<td>1-3</td>
<td>1-4</td>
</tr>
<tr>
<td></td>
<td>Diameter of mixing tool</td>
<td>0,4 m to 0,9 m</td>
<td>1,0 m to 1,6 m</td>
</tr>
<tr>
<td></td>
<td>Maximum depth of treatment</td>
<td>25 m</td>
<td>48 m</td>
</tr>
<tr>
<td></td>
<td>Position of binder outlet</td>
<td>Rod</td>
<td>Rod and blade</td>
</tr>
<tr>
<td>Injection</td>
<td>500 kPa to 1000 kPa</td>
<td>300 kPa to 600 kPa</td>
<td></td>
</tr>
<tr>
<td>Batching plant</td>
<td>Amount of slurry storage</td>
<td>3 m³ to 6 m³</td>
<td>3 m³</td>
</tr>
<tr>
<td></td>
<td>Supplying capacity</td>
<td>0,08 m³/min to 0,25 m³/min.</td>
<td>0,25 m³/min to 1 m³/min.</td>
</tr>
<tr>
<td>Binder storage tank</td>
<td>Maximum capacity</td>
<td>30 t</td>
<td></td>
</tr>
</tbody>
</table>

Table 6 Comparison between Nordic and Japanese mixing techniques – Massarsch 2005

Without intending to explain all different systems, we think we can currently identify three main systems:

1. Systems using multiple (usually three) mixing tools (augers, mixing rods), as shown on fig. 34 & 35. These systems will be mainly used to form columns walls for retaining function.

![Fig. 34 Triple rod mixing system (Bauer)](image)
Fig. 35 Triple rod mixing system (Bauer)

2. Nordic mixing system, used to form lime-cement columns and mainly used for soil reinforcement purposes.

Fig. 37 Swedish Limix system

3. Cased systems where the mixing tool acts under protection of an outer tube, again mainly used for forming retaining walls. Primary soilmix piles can be used in conjunction with secondary concrete piles.

Terashi, 1997, 2000 summarizes the potential factors affecting the strength increase (table 7).

<table>
<thead>
<tr>
<th>I</th>
<th>Characteristics of hardening agent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Type of hardening agent</td>
</tr>
<tr>
<td>2</td>
<td>Quality</td>
</tr>
<tr>
<td>3</td>
<td>Mixing water and additives</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>II</th>
<th>Characteristics and conditions of soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Physical, chemical and mineralogical properties of soil</td>
</tr>
<tr>
<td>2</td>
<td>Organic content,</td>
</tr>
<tr>
<td>3</td>
<td>pH of pore water</td>
</tr>
<tr>
<td>4</td>
<td>Water content</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>III</th>
<th>Mixing conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Degree of mixing</td>
</tr>
<tr>
<td>2</td>
<td>Timing of mixing/re-mixing</td>
</tr>
<tr>
<td>3</td>
<td>Quantity of hardening agent</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>IV</th>
<th>Curing conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Temperature</td>
</tr>
<tr>
<td>2</td>
<td>Curing time</td>
</tr>
<tr>
<td>3</td>
<td>Humidity</td>
</tr>
<tr>
<td>4</td>
<td>Wetting and drying/freezing and thawing, ..</td>
</tr>
</tbody>
</table>


He also identified, in 2000, tasks for the coming decade, which we believe are not all solved yet. We list hereunder those, which seem critical in the daily applications:

- Precision of the strength prediction, taking into account the soil variability
- Clearly understand the failure mode, especially for bending (as the use of deep mixing for retaining walls is increasing)
- Identify, group and understand the influence of the different manufacturing processes.

Other issues, directly influenced by the execution process are regularly discussed:

- Homogeneity of the columns, especially in stiffer cohesive soils
- Reproductability of the mixing process
- Execution tolerances, especially for columns walls
- Overlapping of columns in columns walls
One of the critical issues here is the one of quality and reliability of the treated soil. Quality control is therefore essential. As stated by Kitazume, 2005, the quality control and quality assurance is required, before, during and after construction. For this reason, quality control of Deep Mixing should consist of:

- Laboratory mixing tests
- Quality control during construction
- Post construction quality verification through check boring and field investigation

Here again, our profession would benefit of well-documented full-scale tests, enabling to validate both the functional and the process design.

Three papers are presented in this session of the conference, that deal with Deep Mixing:

The first one, by Breitsprecher and Stihl, show comparative tests between dry deep mixing columns and concrete columns used as soil reinforcement. We will discuss the result of this paper in the section on soil reinforcement.

Both Stötzer et al (2006) and Mathieu et al (2006) introduce a recent developed Deep Mixing Cutter (CSM). The CSM System differs essentially from traditional techniques, by using a mixing tool that rotates about a horizontal axis. This new technique is derived from the cutter (Hydrofraise) technology. The tool is mounted on a kelly-bar, which allows a good accuracy of position and verticality of the soil-mixed panels (Fig. 38). The soil is mixed with self-hardening slurry, which is simultaneously introduced into the soil mass, to produce a Deep Mixing Wall.

Stötzer et al report a number of jobs with various applications: retaining walls, or foundation elements. In particular, they give an overview of a test site in xxx, where values of strength between 6 MPa and 10 MPa are reported. Unfortunately, very little information concerning the untreated soil is provided, so that it is difficult to correlate the obtained results with other similar experiences. Nevertheless, the system seems promising particularly in terms of homogeneity of the treated soil and respect of execution tolerances.

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Stötzer et al report a number of jobs with various applications: retaining walls, or foundation elements. In particular, they give an overview of a test site in xxx, where values of strength between 6 MPa and 10 MPa are reported. Unfortunately, very little information concerning the untreated soil is provided, so that it is difficult to correlate the obtained results with other similar experiences. Nevertheless, the system seems promising particularly in terms of homogeneity of the treated soil and respect of execution tolerances.
Fig. 39 UCS control tests in CSM panels at Le Havre – Mathieu et al (2006)

It can be observed that there is a relatively big scatter in the values of UCS obtained: 2 to 10 MPa, differing from the data provided by Stötzer et al. This scatter remains one of the main problems of everybody wants to routinely use Deep Mixing, as explained here above.

Jet Grouting

Key issues

- Reliability of the achieved diameter
- Strength scatter

According to our survey, jet-grouting represents nearly 10% of the ground improvement technologies. Jet grouting is a ground improvement process whereby the ground is eroded using a high velocity fluid jet (eventually air-shrouded) and grout is simultaneously injected to mix or replace the soil. The obtained result depends on:

- The process used (simple, double or triple jet)
- The process parameters (speed of rotation, speed of uplift, flow pressure, grout,...)
- The type of soil.

Morey (1992) reported by Schlosser (1997) or more recently Bustamante (2001) show the relationship between the diameter of the formed column and the type of soil, and the process used (Fig.40).

![Fig. 40 Diameter of Jet-grouting columns vs Soil type – Schlosser(1997) after Morey (1992)](image)

A few papers are presented with regard to Jet-grouting projects at this conference, in the session dedicated to North-South line. They illustrate interesting case studies, and commonly addressed concerns, as also stated by Chu (2006).

- The achievable and achieved column diameter for a given set of operational parameters
- The strength of the grout column.

One usually specifies for the latter the cement content or the water-cement ratio. Trial laboratory tests are needed to confirm the design value. Table 8 gives the indicative values proposed by Bustamante (2001).

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Average Strength UCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>3 MPa</td>
</tr>
<tr>
<td>Clayey silt</td>
<td>5 MPa</td>
</tr>
<tr>
<td>Silty sand</td>
<td>8 MPa</td>
</tr>
<tr>
<td>Sand</td>
<td>15 MPa</td>
</tr>
<tr>
<td>Gravel</td>
<td>25 MPa</td>
</tr>
<tr>
<td>Chalk</td>
<td>12 MPa</td>
</tr>
</tbody>
</table>

Table 8. Average UCS for Jet-Grouting (after Bustamante, 2001)

The question of the control of the diameter in the field remains one of the big issue. Either trial columns, or electrical methods or mechanical devices are used, no one definitively answering the question. In-situ control tests or coring are also performed. In the papers presented in the Session on North-South line, a new device is
proposed for the in-situ measurement of the column diameter.

Ho (2006) proposes a new method, based on a theoretical model describing the jet excavation mechanism. He starts from the theoretical model proposed by Ho (2005), based on both the hydrodynamic characteristics of the jet and the soil strength. The model predicts that the limit of jet penetration is reached when the average dynamic pressure at the jet tip becomes equal to the ultimate soil bearing resistance. He finally obtains:

\[
\frac{l_j}{d_n} = 6.25 \sqrt{\frac{(P_i - P_s)}{q_{bu}}}
\]

where

- \(l_j\) = ultimate jet penetration distance
- \(d_n\) = nozzle diameter
- \(P_i - P_s\) = nozzle pressure difference
- \(q_{bu}\) = soil bearing resistance (\(N_c x S_u\))

Fig 41 Soil bearing resistance at tip of jet

Ho has identified four case histories, which contain sufficient data in order to allow a reasonably accurate comparison of the predicted diameter (or jet penetration) and the measured one. Fig. 42 plots the obtained comparison.

The reliability of the model, evaluated using results from the four case histories mentioned here above, is within ±15 %.

Fig. 42 Comparison of field data with the computed values using the method of Ho – Ho (2006)

**Soil Reinforcement – Rigid inclusions**

**Key issues**
- Controlled settlements
- Transfer layer

Soil reinforcement by means of stone columns has become a largely used foundation method even for common foundation projects, especially (but unfortunately not only) for storage halls or logistical platforms where distributed loads play a predominant role. In the answers to our questionnaire, they represent more than 15 % of the ground improvement techniques. Many papers have been published on theoretical considerations on stone columns, including the use of more or less sophisticated methods in order to estimate the improvement ratio or the settlement reduction.

Case studies have also been reported, but very few give a real monitoring of the realised improvement, comparing measured settlements without treatment and with treatment. At the recent ASEP-GI conference in Paris (2004), an interesting comparison event was organised showing the comparison between predicted settlement reduction ratio according to different methods, but also with the measured ones, both for the treated and untreated embankment.

Though the settlement reduction estimated theoretically look interesting, one can deplore the large scatter of the predictions following different methods and even more
question the real improvement obtained by the stone columns with respect to the untreated embankment, in terms of reduction of the settlements. Alternatively and increasingly, soil reinforcement using rigid inclusions has been applied. The principle of the solution is illustrated in fig. 43.

Fig 43 Rigid inclusions

The main advantages of the method are the following:

- Applicability in all soil conditions (even in very soft soils, where stone columns proved to be inefficient).
- Settlement reduction accurate and efficient, provided that a appropriate transfer layer is realised.
- Some system enable installing inclusions without vibrations.

Several systems can be used to install the so-called rigid inclusions (Vibrated concrete columns, Compaction grouting Omega, Controlled Modulus Columns, Deep Mixing, etc…). Theoretical and practical studies have been proposed by several authors (Combarieu, 1988).

The mostly used computation methods are the ones of Combarieu, Plaxis, or the German recommendations Ebgeo. Fig. 44 gives the theoretical comparison between the different approaches (Combarieu, Plaxis, Ebgeo), for one research example:

<table>
<thead>
<tr>
<th>Friction angle</th>
<th>30°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied load</td>
<td>54,25 kN/m²</td>
</tr>
<tr>
<td>Transfer layer</td>
<td>no cohesion, no geogrid, thickness Hr</td>
</tr>
<tr>
<td>Inclusions</td>
<td>diameter 44 cm</td>
</tr>
<tr>
<td>Compressible layer</td>
<td>silt (c = 2kN/m²)</td>
</tr>
</tbody>
</table>

Fig.44 Rigid inclusions: load reduction factor in function of Hr/sm - Leclercq (2005)
Documented and instrumented case studies are needed in order to correlate the theoretical results.

Three papers proposed at this session deal with rigid inclusions. The first one is the previous mentioned paper by Breitsprecher et al (2006) on the use of so-called wet stabilizing columns (actually concrete columns) compared with dry lime/cement columns. The 5600 columns, or rigid inclusions, were designed for a service load of 80 kN. The columns had a diameter of 121 mm and a varying length between 2.00 (!) and 7.00 m, aiming at a smooth transition between treated and untreated soil. The project included a load test program on both dry deep mixing columns and concrete columns, the result of which are presented.

Two other papers in this section describe a new and recently patented system of unreinforced piles used as settlement reducer named PCC pile (Liu et al (2006) and Zithao et al (2006)). The PCC pile is a cast-in-place concrete thin-wall pipe pile, installed by vibrating a double casing into the ground and pouring concrete in the space between the two casings, and finally extracting the casings. The final result is an un-reinforced concrete pipe pile.

The authors report on tests performed on the PCC pile for the improvement of embankment on soft ground for the Yan-Tong Expressway. Both vertical load tests (Liu et al(2006)) as horizontal load tests (Zithao et al (2006)), have been performed.

The characteristics of the piles are the following:
Diameters: 1000 mm or 1240 mm
Thickness: 100 mm or 120 mm
Spacing: 2.80 m ; 3.00 m ; 3.30 m
Length: 16.00 m or 18.00 m

A cap cushion is placed to ensure the pile and the surrounding soil to share the bearing of the applied loads. Following figures give the principle of the application.

Fig.45 PCC pile (a) steel pipe casing, (b) movable tapered pile shoe

Fig.46 The construction process of a PCC pile
As part of the testing program, a series of 14 piles were excavated and visually inspected. An important aspect is the relative fragility of the piles to the movement of the equipment. Load-settlement curves are shown. A maximal bearing capacity of 1650 kN is obtained, which is 9% higher, according to the authors, than the design values. Unfortunately, no design calculation is provided, but we can deduce that the contribution of the inner soil to the bearing capacity is minor.

An interesting, and not so commonly available information is the analysis of the load mechanism in the composite body: soil + inclusions. A load test of the composite foundation is provided. Fig. 48 shows the stress in the surrounding soil at various distances from the piles and above the piles, for different loading steps.

Zhitao et al. (2006) analysed this problem and provide load-deflection curves showing that a limited lateral load can be born by the PCC piles (around 120 kN), but that strengthening of the upper part of the piles is preferable as well as surface soil improvement.

All these case studies show that soil reinforcement with rigid inclusions of different kinds is increasing. Well-instrumented and documented case studies are needed in order to better understand the load transfer mechanism and to improve the design.

**RETAINING WALLS**

In the area of retaining walls, the technical solutions again vary quite a lot from country to country, and according to the type of problem. We can distinguish:
- Soldier piles and lagging usually used for small to medium excavations in no water environment.
- Sheet piles or combined walls (H-piles and sheet piles) mainly used for quay walls
- Slurry walls, cut-off walls, used for large excavations
- Bored Pile walls, used in difficult soil conditions or for smaller excavations with difficult contours.
- Deep mixing walls become more popular
- Nailing remains a case to case approach

Fig. 49 gives the repartition between the different techniques worldwide.
As another session of this conference is dedicated to deep excavations, we will limit the extent of this section to technological developments, in connection with papers submitted to the conference.

**Sheet piles**

**Key issues**
- Jacking
- Sealing

Sheet piles are traditionally installed by driving or vibratory driving. Our previous comments on the technological evolution in this field remain valid, and particularly the revival of the press-in technologies, certainly in Japan.

*Dubbeling et al (2006)* explain the recent development in press-in (jacking) technological equipment enabling to install sheet piles and H-profiles in more difficult conditions. They introduce new construction concepts using more compact and lightweight “press-in” equipment also called Silent Piler. (Fig. 50)

![Fig. 50 Press-in system – Dubbeling (2006)](image)

Combining innovations in material, auxiliary equipment like water jetting or augering, the new generation of Silent piler is reported to be able to penetrate into dense and hard soil layers.

**Water-jetting:**
High pressure water jetting reduces the pressure bulb formed during pressing-in by loosening granular soils and softening cohesive soils. Simultaneously the returning water lubricates the pile surface and reduces the tendency of the pile to plug (fig 51 and 52).

![Fig. 51 Water jetting for press-in system – Goh (2006)](image)

**Crushing system (augering)**
The Silent Piler can also be equipped with an integral Pile Auger to enable the Press-in Method to be used in hard ground where water jetting would not be effective. The authors report that hard ground layers with an UCS up to 200 MPa can be penetrated.
The compactness of the equipment enables to install piles in tight working spaces, or limited headroom. Even inclined piles or sheet piles can be installed. Systemized equipments have been developed to bring the piles on a so-called “pile runner” on the top of the just installed pile wall, to the clamp crane. This system allows piling over water or sloped embankments.

Sealing of sheet piles

Lehtonen and Laaksonen (2006) introduce a new innovative system of watertight sealing using continuous cement grouting during driving. (fig 54)

The method was tested on a field test site where the main installation parameters and permeability figures have been observed and analyzed. After two weeks of monitoring, no leak could be observed and the authors report the obtained permeability of the system to be less than $5 \times 10^{-9}$.

Slurry walls and cut-off walls

It is surprising how similar techniques can take such different directions in different countries. Slurry walls don’t escape this reality. Slurry walls are common practice in some countries, and nearly completely ignored in others.

In some countries, most excavations are executed with cut-off walls, realized with self-hardening bentonite-cement slurry, with structural elements (H-profiles, sheet-piles,...) installed into the fresh slurry.

In France, Belgium, and more recently the Netherlands, the techniques of slurry walls in concrete with watertight joints is common practice. A full section will be dedicated to the project North-South line, including the slurry walls for the Metro Stations at Vijzelgracht and Rokin, and another to deep excavations, we will therefore not cover this topic in detail. We just want to summarise the most recent technological evolution or trends that we have been noticing in this field:

- The use of hydraulic grabs
- The use of hydromills or cutters for excavating in difficult soil conditions, including limited headroom, or in tight working spaces.
- The monitoring of the execution, were key parameters are registered, as depth, inclination, deviation of the grab in the two directions.
- The use of improved systems for watertight joints

These technological improvements enable to guarantee the respect of tighter tolerances in the execution of the walls, and hence a higher quality of the finished product.

Raffaela and Crippa (2006) report a case history where two contaminated sites in a petrochemical plant, were encapsulated using a self-hardening slurry with a HDP membrane. Few detailed soil data are provided. Due to the necessary continuity of the membrane, a rather complicated execution process was chosen combining...
a pre-excavation with a hydromill or pre-boring with bored piles equipment in hard soil layers. The pre-excavation was filled by plastic concrete, which was re-excavated with a clamshell working under self-hardening slurry. The slurry was tested in function of the polluted environment.

**Secant piling**

*Key issues*
- Cased augercast
- Tolerances and quality control

Pile walls represent nearly 50% of the retaining wall market. A lot of different techniques co-exist in this segment:
- Tangent pile walls with CFA piles
- Secant pile walls using CFA piles
- Secant pile walls using conventional bored piles
- Secant pile walls using cased CFA piles.

As already mentioned, the main development in this field is the cased CFA (also called Front-of-Wall), which has literally exploded in some markets, and is reported by many practitioners as a major development in this field. In Belgium, not less than 14 rigs work with this system, typically used for one or two levels underground in urban areas. The usual diameters range from 42 to 62 cm, and the length typically reaches 16.00 m.

Main advantages of the system:
- Tight tolerances on both position (<5 cm) and verticality (<1%), due to the rigidity of the combined tube/auger system,
- Execution possible even in man made obstructions, like old foundations or brick walls.
- Possibility of execution close (<20 cm) to an existing wall or façade

Main disadvantages of the system:
- Installation of the reinforcement after pile concreting, which in some soil conditions can prove to be difficult.
- Limited length and diameter (with current equipment)

**Soil Mix Systems**

Many Deep Mixing applications offer a good alternative for the execution of retaining walls. In this case, the mixed soil is used to transfer the ground pressures
by arching to structural elements placed in
the fresh “slurry” (fig. 55)

Fig 55 Arching mechanism for soil-mix
walls.

In this case, overlapping columns are used
in order to create interlocking walls (Fig.
56). Simple or double H or U profiles are
inserted in the fresh slurry to form the
structural stability of the retaining wall, the
soil mix transmitting the ground and/or
water pressures via arching.

Fig 56 Overlapping soil-mix columns for
retaining walls.

The critical issues are then:
- As in all applications of deep mixing:
  homogeneity of the treated soil. In the
case of retaining walls under water
table, this issue can be a very
important one.
- Tolerances of execution. As the wall
  is formed by overlapping columns of
soilmix, tight tolerances are needed in
order to guarantee that the columns
are effectively overlapping. In the
case of the use of the three augers or
rods systems, the rigidity of the rods is
quite low and can subsequently lead
to problems in this respect. No real
guide beam can be constructed, as for
secant pile walls.

CONCLUSIONS

We have been trying to give a
comprehensive overview of the most
recent evolution in the deep foundation
technologies over the last years, what is
obviously an impossible task. We retain:
1. The better understanding of energy
transfer and set-up phenomenon for
driven piles.
2. The use of vibratory driving and jacked
piles in replacement of driven piles.
3. The increasing part of displacement
auger piles enabling to fully benefit of
the soil capacity.
4. The development of post-grouting in
order to increase the range of
application of bored piles.
5. The better understanding of the
negative effect of soil loosening
related to CFA pile installation and
hence the development of proprietary
systems or more recently of the cased
CFA piles.
6. The use of rigid inclusions or piles used as settlement reducers for soil reinforcement
7. The increased use of Deep Mixing tools for various deep foundation problems.

Although the technological advances have been enormous, in terms of systems or installed power, the back analysis of many case studies for deducing installation effects is fraught by the lack of well documented cases with correctly described soil investigations and above all, equipment related data.

As already pointed out by O’Neill and Finno (2001), there is a need to improve the understanding of the installation effects of deep foundations techniques on the surrounding soil and hence, the level of the information contained in our papers to provide exploitable data.

PAPERS PRESENTED in THIS SESSION


BZÓWKA Joanna, PIECZYRAK Jacek. Application of CFA piles for foundation of a shopping centre on long-standing dumping ground.

DAPP Steven D., MUCHARD Mike, BROWN Dan A. Experiences with base grouted drilled shafts in the Southeastern United States.

DAVIE John R., CLEMENTE Jose L.M., ANDERSON Myron R., HEADLAND Louise C. Drilled piers support massive gantry crane.

DUBBELING Philip, VRIEND Ad, NOZAKI Tsnunenobu. Press-in piling technology for sustainable construction.

FILIP Ray K. Recent advances in quiet & vibration-less steel pile installation and extraction.

GRANATA Rafaella, CRIPPA Carlo.. Confinement of two contaminated sites in a petrochemical plant; case history.

GWIZDAŁA Kazimierz, KRAŚNIŃSKI Adam, BRZOZOWSKI Tadeusz,. Experience gained at the application of Atlas piles in Poland.

Ho Chu E., Prediction of jet grout column diameter in cohesive soil.

KHARE Makarand G., GANDHI Shailesh R., Performance of bituminous coats in reducing negative skin friction.


LEHTONEN Jouko L., LAAKSONEN Jari P. Continuous grouting in embedding of sheet piles.

LENNON Derek J., RITCHIE David, PARRY Geraint O., SUCKLING Tony P. Piling Projects constructed with vinyl polymer support fluid in Glasgow, Scotland.

LIU Hanlong , GAO Yufeng, ZHANG Ting. Application of PCC piles on soft ground improvement of expressways.

Ma Zhitao, LIU Hanlong. Behaviour of PCC pile under lateral load.

MATHIEU Fabrice, BOREL Serge, LEFEBVRE Laurent. CSM : An innovative solution for mixed-in-situ retaining walls, cut-off walls and soil improvement.

SNELL T.J., SINGLETON M. Bearing capacity and slope instability solved by novel ground engineering solutions.

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trends in Deep Foundations, Geotechnical Special Publication n°125, Di Maggio and Hussein Eds, ASCE, Reston, VA, pp 222-238


