LARGE SCALE SOIL IMPROVEMENT IN THE ZEEBRUGGE OUTER HARBOUR

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ABSTRACT

For the development of a quay wall on the south side of the Wielingen dock in the outer harbour of Zeebrugge, a huge area has been reclaimed by hydraulic filling. This work raised the levels from about –10.00 m TAW to +8.00 TAW, in an area where the tide normally ranges around about 5 meters. Water depths to -15.00 m TAW allow for the loading and unloading of paper for the paper industry. Behind the cargo handling area a large number of warehouses has been built (transit and short stay warehouses, distribution 1 and 2). Reclamation works started about January 1999 and by the spring of 2001 about 40.000 m2 of warehouses have been built. The record period of construction necessitated from the very start the availability of a fill with adequate deformation-free load bearing qualities.

The hydraulic fill was primarily subject to two types of deficiencies, being on the one hand the inclusion of soft cohesive silt and peat layers, and on the other the insufficient density of the sand layers. Several soil improvement techniques have been put at work, depending on the in situ soil profile, and depending on the time available: the two most important were dynamic compaction and accelerated consolidation under preloading.

The evaluation of the performance of the soil improvement techniques was monitored through the continuous execution of cone tests. The consolidation under the pre-load was checked by settlement and pore pressure measurements. A large number of settlement measurements were executed through PE-tubes installed under the applied fill. This allowed to judge the settlement evolution in time and adjust the numerical model to come to a more accurate prediction of the consolidation period. At its extreme, the total settlement mounted up to about 2 meters.

In the post-construction period, the settlements of the warehouses will be monitored by measurement on a grid of reference points, fixed to well-chosen structural elements.

Keywords: Dynamic compaction, Consolidation ratio, Performance monitoring

1. INTRODUCTION

The major part of the activity in the outer harbour of Zeebrugge is situated in an area that has been gained on the sea. In the late nineties the outer harbour of Zeebrugge has been selected by the leading paper manufacturer StoraEnso as their future main gateway in Western Europe for the transhipment of paper. An area of 20 ha alongside the Wielingen dock has been envisaged as the ideal location. The location of this recovered area is indicated in figure 1.

Since the spring of 2001, the StoraEnso terminal consists of:

- a complex of four industrial warehouses (one Transit warehouse, one Short Stay warehouse and two distribution warehouses called Distribution 1 and Distribution 2), for a total area of about 40.000 m2.
- ca. 150.000 m2 of paved cargo handling area (a combination of light and heavy duty pavement).

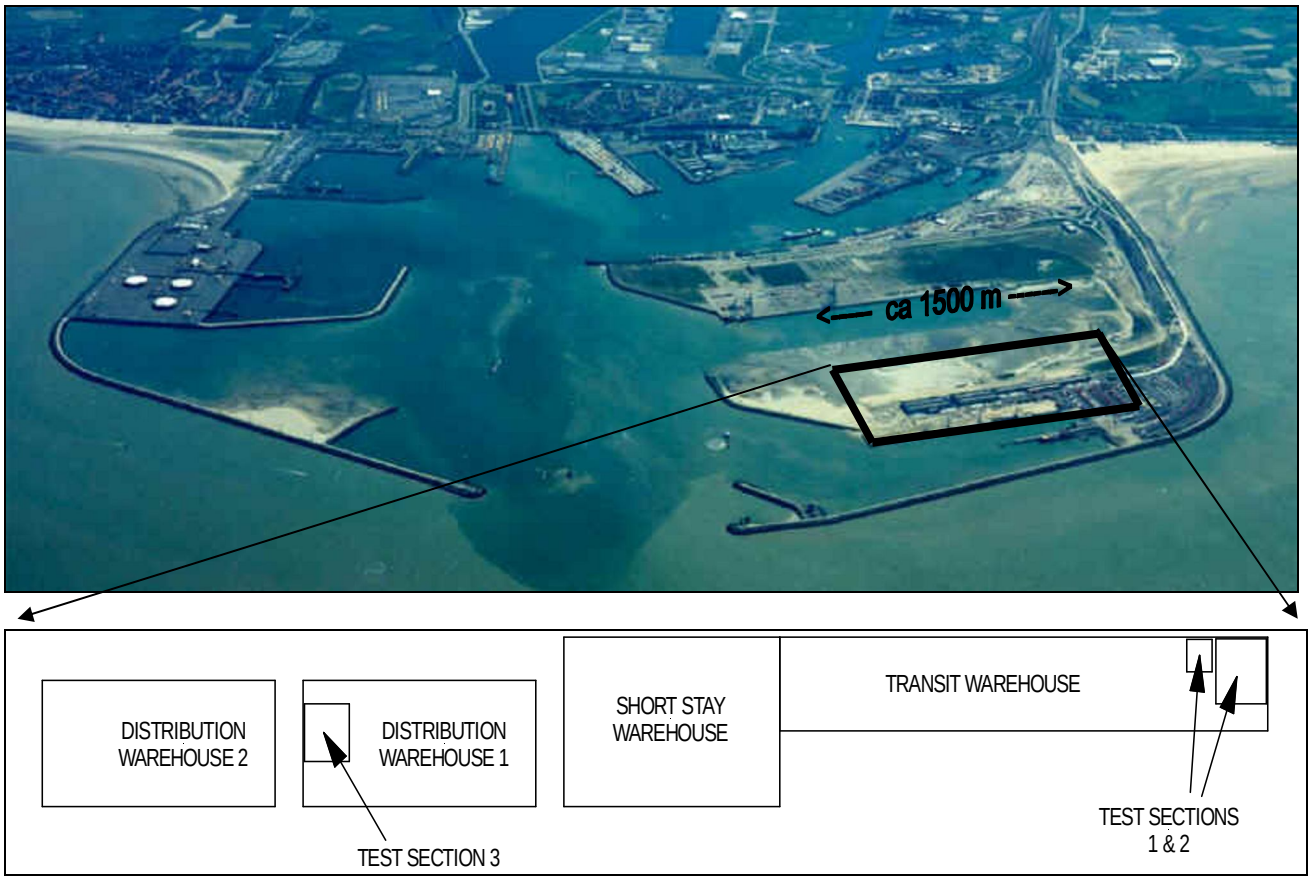


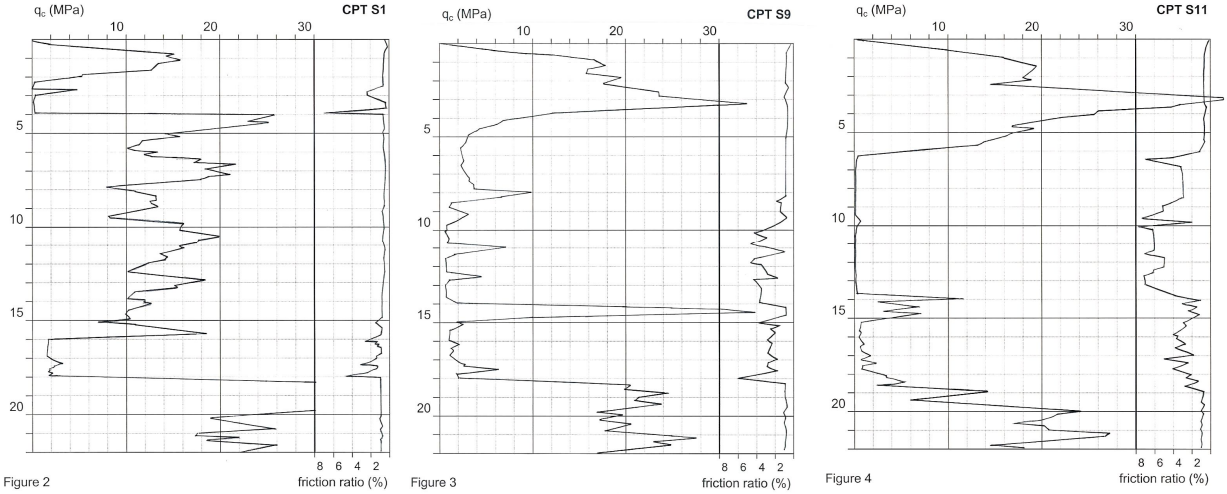
Figure 1: Aerial view on the outer harbour of Zeebrugge.
Layout of the warehouses of the StoraEnso terminal.

2. GEOLOGY

The construction of the terminal area was accomplished by hydraulically placing fill-material (sand) upon the existing seafloor. The seafloor was naturally covered with a layer of non-cohesive silt. Cost studies proved that the removal of this sludgy material, before applying the landfill, would be several orders of magnitude higher than a treatment afterwards of the underground in such a fashion that it would be suitable for the construction.
Investigation of the soil conditions showed a significant variation in the thickness and the depth of the sludge layers over the construction area. This resulted in two distinct zones where different soil improvement was necessary.

The first and largest zone comprised the Distribution1, Short stay and Transit warehouses. Here the low resistance layers (consisting of sludge) were located at depths ranging from 2 to 5 meters. The underlying layers had sufficient bearing capacity to allow for the construction and exploitation of the terminal.

The second zone below the Distribution 2 warehouse was very different from the previous one. Here, very soft layers were found between 6 and 14 meters of depth. A typical CPT diagram for each area is given in figure 2 (CPT S1, CPT S11). Below this layer the cone-penetration-resistance remained rather low. Between these areas there is a transition area. A CPT typical for this area is given in the centre of figure 2 (CPT S9).



<u>Figure 2</u>: Geological situation in Zone 2.
Three typical cone penetration test profiles.

3. DESIGN ISSUES

At feasibility stage, several soil improvement techniques have been recognized for evaluation:

- deep foundations on piles and/or gravel columns;
- dynamic compaction;
- dynamic compaction in combination with preloading.

These technical solutions have been examined carefully with respect to 3 aspects as to find the optimal one:

- the geological situation, given the zoning and subzoning as discussed in the previous chapter;
- the financial costs, including the ones related to time implications;
- the design criteria.

The floors of the warehouses had to be designed for a maximum load of 100 kN/m^2 (with a quasi-permanent part of about 60 %). Only very small differential settlements were allowed, both with respect to the floor behaviour as to the warehouses' structural behaviour. These warehouses were designed as classical reinforced concrete structures, with a very high percentage of precast elements.

It was not surprising that as a result of the variable nature of the subsoil, different treatment methods had to be implemented in each of the two zones.

<u>Zone 1: Distribution 1, Short stay and Transit Halls</u>

In view of the relatively shallow depth of the soft layer, the client (Sea-Ro Terminal nv) finally decided for a ground treatment with dynamic compaction. This method proved to be by far the most cost-effective treatment.

For the StoraEnso project, the dynamic compaction was performed with two main purposes:

- to obtain a soil replacement at the location of the impacts;
- to obtain an improvement of the soft soil layer between the impact locations.

Calculations with a finite element model have been performed in order to determine the differential settlements of the floor of the warehouse under a uniform load of 100 kN/m^2. These calculations have been performed for the axi-symmetric situation.

- The soil replacement was simulated by introducing a column with a diameter of 1,5 m and an elastic modulus of 60 MPa.
- For the soft layer different values of the elastic modulus have been introduced in order to determine which improvement had to be realised.

From the results of these finite element calculations it could be observed that the replacement soil column has a real bearing function. Within this replacement column a concentration of vertical stresses could be observed.

Zone 2: Distribution 2

As mentioned before, the soil conditions at the Distribution 2 area were significantly different with low soil bearing capacities to depths going down to 17 m depth. It was clear that the treatment implemented in zone 1 would be insufficient even as measurements in zone 1 had shown that the soil improvement was still clearly visible at 8m depth or more.

The heavy design criteria for the distribution plant (10 tons per square meter) it was necessary to consolidate all the soil up to the deep very soft layer (between 9 and 11m depth) to keep the absolute and the differential setting within the design limits (respectively 25mm and 1,5 per thousand)
This could only economically be obtained by a static preloading of the area with a weight higher than the design load. This is a very well known and reliable technique but the consolidation takes a very long time if no extra measures are taken.

Figure 3: Overview of the construction site during the soil improvement process.

Because of the time limit the client decided for a treatment method consisting of preloading and a drainage system to accelerate the consolidation.

In zone 2, the consolidation module of a finite element programme was run to predict the time necessary for consolidation under the preload fill. The actual measurements in the field served as a calibration for the calculation model and allowed to improve its accuracy.

4. PRACTICAL APPROACH

Zone1

The success of the dynamic compaction method depends on the use of the most effective technique in a given soil condition.
To determine these application parameters, 3 test sections were performed. The location of the test sections is indicated on figure 1.

The goal of the test sections was to determine:

- the number of phases that had to be applied;
- the number of impacts needed for each phase;
- the grid for each phase;
- the impact pressure that had to be used to obtain the best results (drop height en impact surface of the drop weight);
- the time needed for the soil to consolidate and to release the pore pressure induced by the dynamic compaction.

Two drop weights with different impact section but each having a weight of 15 metric tons were used for the tests. The drop height was 15 m for the deep soil treatments and 8 m for the surface treatment.
During the treatment no significant uplifting of the terrain surrounding the impact point was observed which means that the energy of the impacts was effectively compacting the soil underlying the impact point.
The test sections showed that the best results were obtained using the drop weight with the smallest section and resulted in the design of the following compaction program:

- phase 1 using a grid of 12m by 12m;
- phase 2 using the same grid as phase 1 but the compaction points were interlaced with the grid in phase 1 resulting in an overall grid of 8,5m by 8,5m;
- phase 3 re-compacts the impact points of phase 1 and 2 and some additional point to establish a treated grid of 6m by 6m;
- phase 4 reduces the grid to 4,25m by 4,25m;
- phase 5 surface treatment (one drop of the weight continuously over the complete surface).

After each phase the craters produced by the drop weight are filled up with sand.

Figure 4 shows 2 different phases of the dynamic compaction process as well as the surface treatment at the Zeebrugge site.

Figure 4: Dynamic compaction in zone 1 at work.

Zone 2

The chosen drainage system was a flat wick drain of 10cm length and 0,5cm thick. The length of the drains put in place was 15 meters, therefore covering the muddy upper soil layers till the first (thin) resistant layer. It was considered that the low strength layers beneath this depth would have limited impact on the differential setting of the soil.
The drains have been installed at the following intermediate distances:

- 1,20 m x 1,20 m underneath the warehouse;
- 1,50 m x 1,50 m in a zone with a width of 30 m around the building.

The placement of the drain was performed in the following way:

1. Creating a working platform of filtering material. This working platform would work as a surface drain to evacuate the water brought up by the drains. The hydraulically placed fill served this purpose. In a limited area obstacles were encountered at depths ranging from 6 to 12 m. To enable the placement of the drains in this area a campaign of pre-drilling with augers was performed. The drains were then placed in the pre-drilled holes.
2. Setting up the equipment for placement consisting in backhoe with a hollow driving punch wherein the drain slides.
3. Placement of the rolls of prefabricated drains on the machine.
4. Sliding the drain through the driving punch and fixing an anchor on the drain. The anchor remains in the soil holding the drain while the driving punch is removed.
5. Pushing the driving punch into the soil to the required depth.
6. Retrieving the driving punch and cutting of the drain about 15 cm above the working platform.

After the placement of the drains a 5 m layer of fill was placed on the area of Distribution 2 to simulate the load to expect from the future plant.

Figure 5: Accelerated consolidation in zone 2.

(top): working platform of filtering material
(right): placement technique of the vertical drains

5. PERFORMANCE EVALUATION – QUALITY ASSURANCE

Zone 1

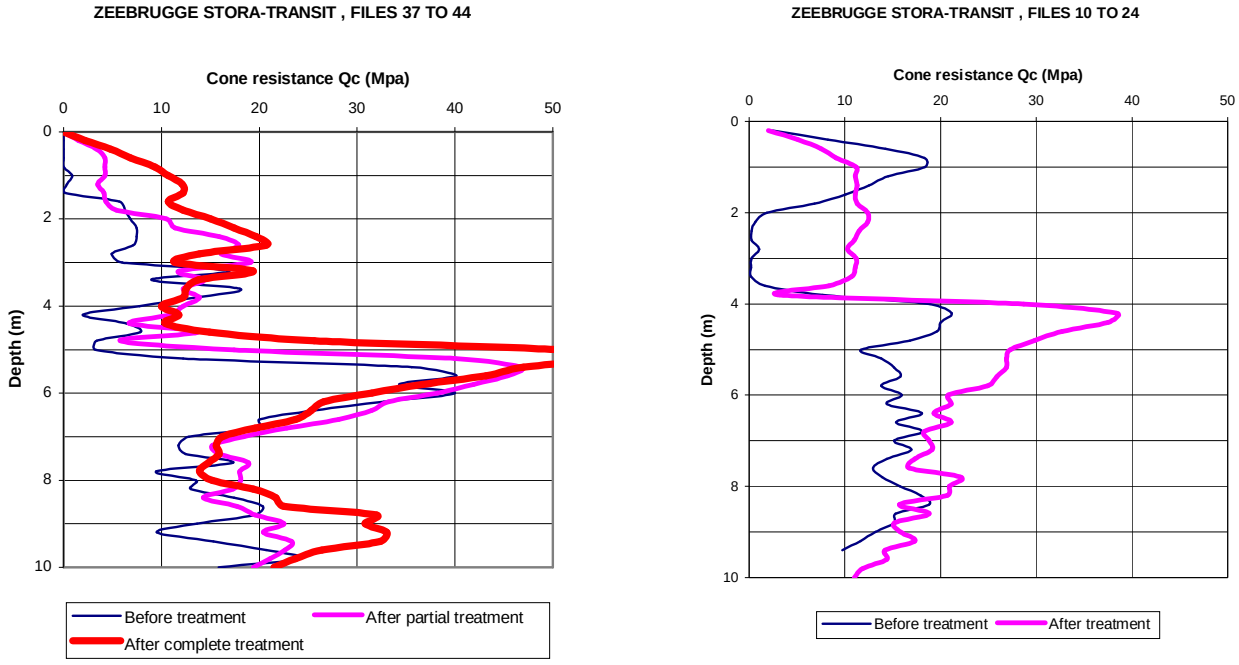


Figure 6: Comparison of average values of Cone resistances Qc before and after treatment (dynamic compaction).

After each compaction phase the soil improvement was checked with a cone penetration test. Pore water pressure sensors were used to determine when the next phase of the compaction could be started.

The results of the test sections showed that the soil conditions could be improved by the dynamic compaction that the resulting absolute settlements in working conditions of the StoraEnso terminal were smaller than 25 mm and the differential settlements less than 1,5 per thousand.

Figure 6 shows the measured improvement of the soil after implementation of the dynamic compaction. The soil improvement is best up to 5 meters depth but is observable to a depth of 10 m.

Zone 2

The behaviour of the subsoil was monitored through a combination of four types of techniques :

- cone penetration tests ;
- laboratory testing;
- direct settlement measurement via settlement tubes;
- water pressure measurements.

Comparative cone penetration testing (before and during treatment) learned that differences were only marginal. Soil improvement in zone 2 was essentially obtained through consolidation of highly saturated silt and peat layer (improvement of the compression moduli). The improvement can hardly be deducted from a classical CPT-test. In theory, comparative laboratory testing on small sized samples would be needed to find improved compression moduli. However, it was found that laboratory testing on small sized samples has a number of important disadvantages:

- are the small samples representative?
- have they been recovered undisturbed?
- laboratory testing is mostly expensive;
- direct interpretation/extrapolation to the field remains difficult, but necessary.

Evaluation of the consolidation effect by direct measurement of the settlement effect therefore seemed the most interesting way to go. To this purpose, plastic tubes have been installed along 4 profiles. Furthermore, piezometers have been installed in order to measure the variation of the pore water pressures.
Before the installation of the fill, 5 cm diameter plastic pipe were placed horizontally at 4 different locations over the complete width of the backfill area. These pipes are flexible enough to follow the movement of the ground under the weight of the backfill and would serve as settlement measuring pipes. At regular time intervals a probe is pulled through the pipe making a recording of the vertical location of the pipe.

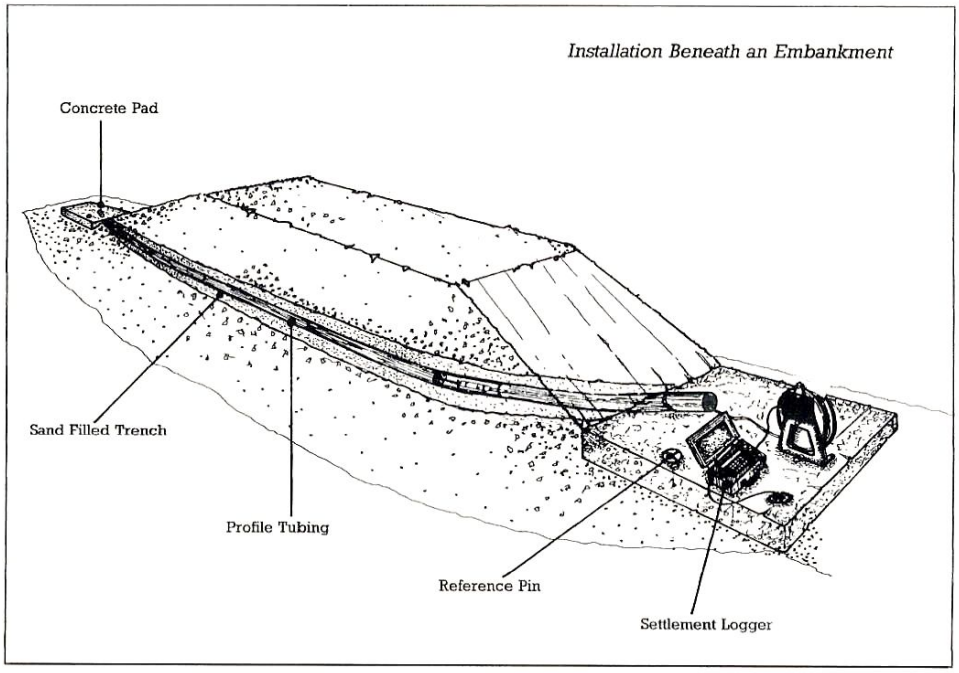


Figure 7: Principle of the settlement monitoring under an embankment (or under the preload fill as applied in zone 2) via settlement tubes

After the installation of the monitoring devices a 5 m thick sand layer was placed on the area of distribution 2 to simulate the load to be expected from the future plant. A 5 m thick sand layer was considered to be enough as the mean load of the warehouse floor was maximally 60 kN/m². So even after the expected settlement of about 1 m, the preload charge remained larger than the mean load on top of the very soft layer.

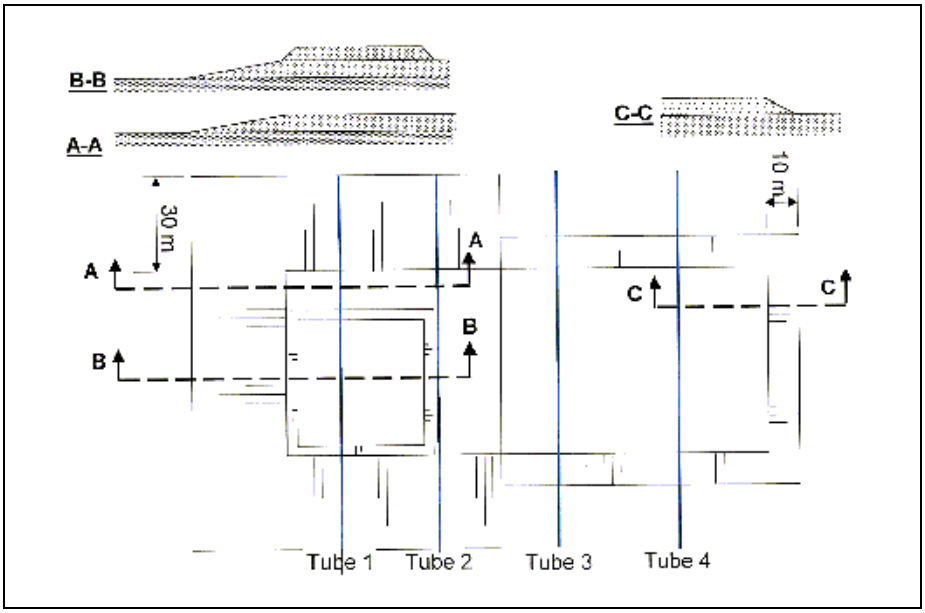


Figure 8: Contours of the preload fill and location of the settlements tubes in zone 2.

To follow up the evolution of the pore pressures during the consolidation, pore pressure meters were installed at different locations over the preloaded area. However, this technique of in situ

measurements proved to be quite vulnerable to the physical dangers of a construction site. It gave no interpretable results.
The results of the settlement measurements are given in figure 9 for profile 1.

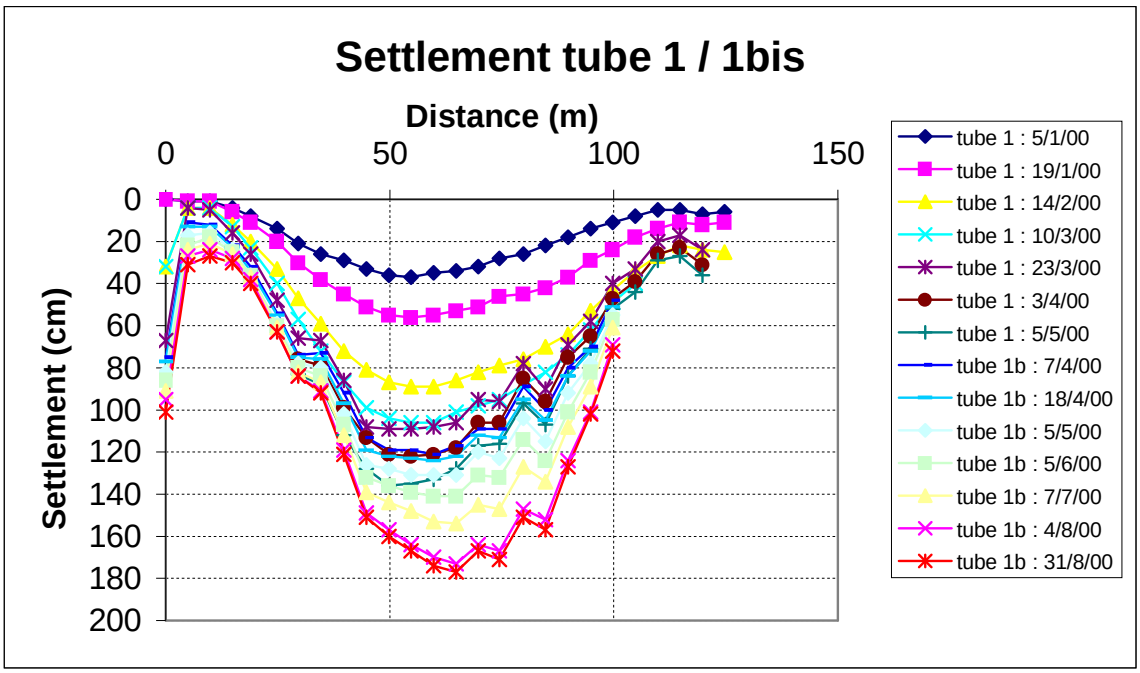


Figure 9: Evolution of the settlements along tube 1 with time.

And the variation with time of the maximum settlement is given for the tubes 1 and 2 in figure 10. From this figure one can see that after 3 months (90 days) the settlements were almost finished at the location of tube 2 but were still increasing at the location of tube 1.
Four months after the installation of the fill layer, the tube at location 1 was blocked at about 20 meters from the starting point of the measurements. So the measurements had to be performed from the other end of the tube. As there was no zero measurement of the tube from this point an interpretation of the measurements had to be done and therefore the tube has been renamed as tube 1b (or 1bis).

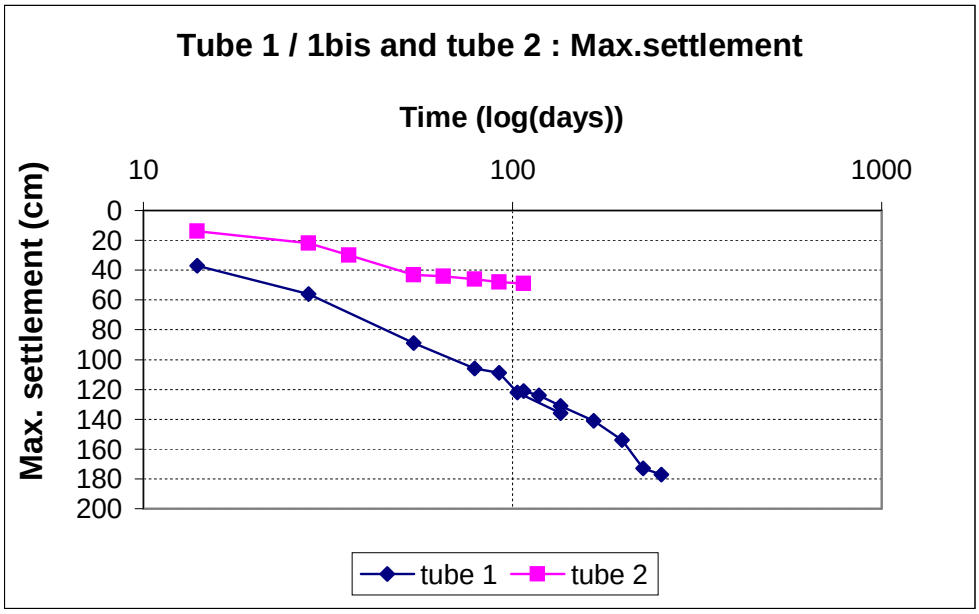


Figure 10: Evolution of the maximum settlements with time.

When after six months the settlements were still increasing, it was decided to install an additional fill layer of 3 meter over the area where settlements were still increasing (as indicated in figure 8).
This additional load was maintained for a period of three months and a maximum settlement of about 180 cm had been measured. Although at that time the full consolidation of the very soft layers was not obtained under that load, nor demonstrated under the (smaller) previous load, it was decided to remove the preload and to build the warehouse.

As an additional measure the columns of the warehouse structure situated within the area where settlement was still taking place, have been partially founded on precast concrete piles. In the post-construction period, the settlements of the warehouses will be monitored by measurement on a grid of reference points, fixed to well-chosen structural elements. If needed, this will trigger extra interventions to maintain the structural floor integrity (e.g. by injections).

5. CONCLUSIONS

A lot of experience has been gained with the evaluation of the performance of soil improvement techniques. Whereas comparative cone resistance testing proved to be an excellent instrument for evaluation of the effect of dynamic compaction in loosely packed sandy layers, better results were obtained with direct measurement of the settlement effects when judging consolidation of peaty silt layers under a preload fill.
A nearly continuous monitoring of the global behaviour of the subsoil under the preload was obtained via the technique of the settlement tubes.
In this particular project, monitoring did not prove to be just a sterile check of the quality of the execution. It proved to be capable of much more:

- calibration of (finite element) design models;
- direct impact on the execution methods;
- triggering of post-construction interventions.

It is believed, that in the given design circumstances, monitoring was a versatile tool to obtain an economically justifiable result.

6. REFERENCES

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Figure 11: Overview of the construction site (summer 2000)