

CHAPTER ONE HUNDRED EIGHTY FOUR

The use of sand in breakwater design

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Abstract

The design scheme of the Zeebrugge Outer Harbour, Belgium, consists of two breakwaters protruding into the sea as far as 1,750 m beyond the existing môle or 3,000 m out from the coastline. The west outer breakwater is 4,280 m long, the east breakwater runs 4,030 m out from the sea-wall. The east outer harbour will accommodate terminals for liquid bulk products such as LNG. The west outer harbour will provide space to install two harbour basins to suit general cargo, hazardous cargo, container and ferry traffic.

In the paper emphasis is put on the use of sand in the design of the breakwaters, firstly as part of the foundation by replacing the little resistant top layers by dumped sand, and secondly as part of the core, by dumping sand above the original sea-bottom. In as far as proposed criteria were not fulfilled, the dumped sand had to be compacted. Two methods are discussed : compaction by vibrating needles and compaction using explosives.

1. INTRODUCTION

The construction of the existing port of Zeebrugge has been a royal decision pronounced by H.M. King Leopold II in 1881. Almost a century later, in 1970, the Belgian Government decided to build a new inner harbour and finally in 1976 the decision was taken to the extension of the outer harbour. The outer harbour should provide a protection for the access channel to the new inner harbour. Moreover the outer harbour was necessary to accommodate the national LNG-terminal.

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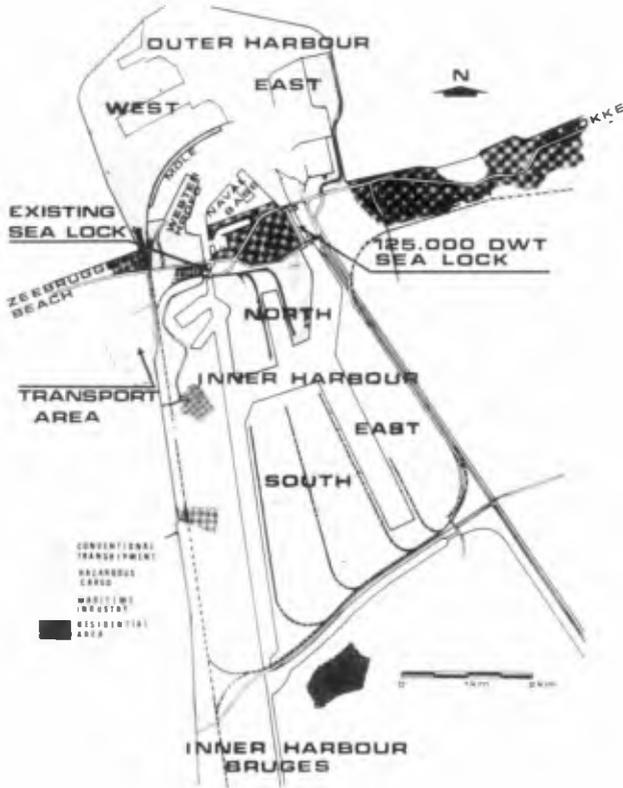


Fig. 1 : general layout of the Zeebrugge Port Extension Scheme

The masterplan finally adopted provides 3,370 ha sub-divided as follows (fig. 1) :

- outer harbour, including the existing port	1,165 ha
- inner harbour	1,705 ha
- port area of Bruges	450 ha
- transport area	50 ha.

The design scheme of the outer harbour consists mainly of two breakwaters protruding into the sea as far as 1,750 m beyond the existing môle.

The breakwaters are from the rubble-mound type. The west outer harbour breakwater is 4,280 m long. The east breakwater runs 4,030 m out from the sea-wall (fig. 2). The east outer harbour will accommodate terminals for liquid bulk products, such as LNG. A LNG-terminal is at present under construction and will be operational mid 1987. The west outer harbour will provide space to install two harbour basins to suit hazardous cargo, general cargo, container and ferry traffic.

2. DESIGN OF THE BREAKWATERS

The design of the breakwaters has been discussed in a paper on the Coastal Engineering Conference in Cape Town.

By developing the design the rubble-mound breakwater has been judged to be the best viable alternative taking into account costs, technical risks, construction problems and flexibility by changing environmental conditions.

An important economization in the design was the use of sand in the breakwater as soil replacement and as raised foundation.

Indeed, because the existing and future navigation channels to the harbour have to be deepened, some 30 million m³ of relatively coarse sea-sand was available.

The quarry stone is won at distances of 100 km to 200 km off Zeebrugge and is rather expensive. So every possible substitution of quarry stone by sea-sand was an economical operation.

Technically the design with soil replacement has been assessed by evaluating the alternative of a breakwater built directly on the existing sea-bed, where the overall stability is given by equilibrium embankments at both sides.

3. ENVIRONMENTAL DESIGN CONDITIONS

3.1. Bottom depth

The bottom depth in the alignment of the outer harbour breakwaters ranges fairly between Z - 5.00 and Z - 7.00 (chart datum Z = mean low low water spring + 0.08).

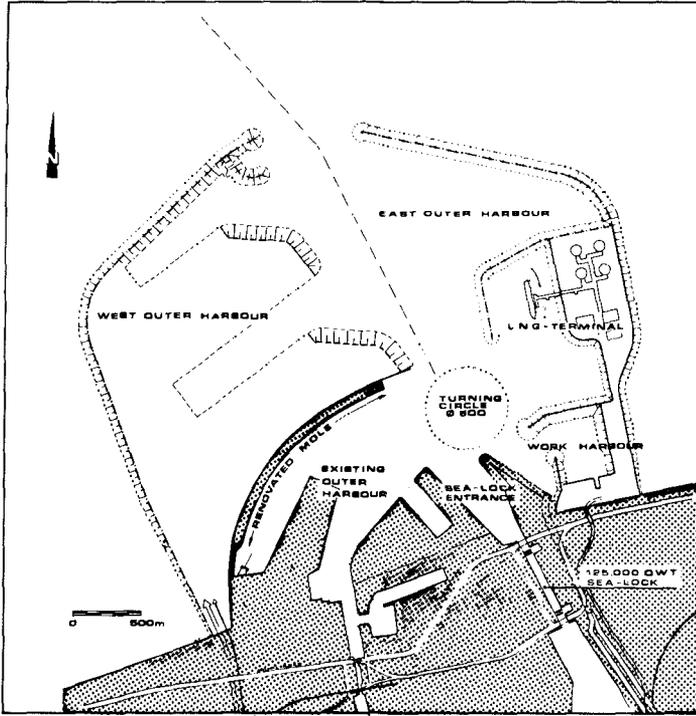


Fig. 2 : layout Masterplan Zeebrugge Outer Harbour

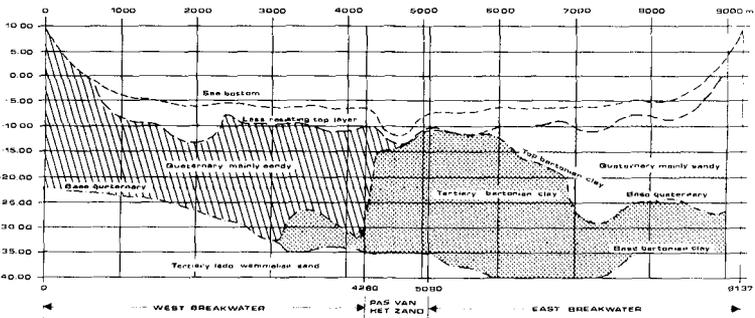


Fig. 3 : geological profile outer harbour breakwater alignment

3.2. Tidal conditions

The tide is semi-diurnal with levels ranging from amplitudes of 4.30 m at mean spring tide to 2.80 m at mean neap tide. The meteorological set-up is up to 2.45 m in the defined design period and probability of exceedance. The tidal currents at the final breakwater alignment will be (at surface) 1.2 m/s to 2.00 m/s at spring flood tide and 1.0 m/s to 1.6 m/s at spring ebb tide.

3.3. Soil conditions

The new outer harbour at Zeebrugge is located in the Scheldt Estuary which is characterized by a very heterogeneous soil composition of the top layers. In behalf of the construction of the new outer harbour a very extensive site investigation program consisting of continuous seismic profiling, CPT tests type M4 and borings with undisturbed sampling and laboratory tests is performed.

A geological profile along the axis of the breakwaters, resulting from the performed site investigation, is shown in fig. 3.

At the location of the new harbour the quaternary cover consists of loose sands and soft clays to a depth of about 4 to 6 m, underlain by a medium dense to dense sand layer. In the north eastern part of the new outer harbour the dense sand layer is not present and the top loose sands and soft clays are immediately resting on a stiff tertiary clay layer (Bartonian Clay).

The layer of soft clay and loose sand has cone resistances ranging between 0 and 1.5 MN/m².

4. SAND USED AS PART OF THE BREAKWATER

4.1. As part of the foundation : soil replacement

Over large lengths of the breakwaters the bearing capacity of the surface layers was insufficient.

Only locally it was possible to construct the breakwaters directly on the natural soil layers on condition to provide a heavy toe construction.

Based on technical and economical considerations a soil replacement was decided over about 8 km of the breakwaters. The top loose sands and soft clays are dredged by large sea-going cutter suction dredgers and hopper suction dredgers and are replaced by relatively coarse sea-sand won in the existing and future navigation channels to the harbour. The mean diameter of the dumped sand is about 170 to 300 μ m. The cross-section of the dredged trench with a width of about 100 m is shown in fig. 4.

It is generally known that by the execution of a soil replacement by dredging and filling a trench, a transitional layer occurs at the bottom of the dredged trench. Special precautions had to be taken to

minimize the extent and thickness of that layer. In addition to that phenomenon, at Zeebrugge a supplementary supply of mud in the dredged trench is caused by waves and strong tidal currents. In order to minimize the sedimentation of mud in the dredged trench, immediately after preparing a compartment with a length of about 150 to 250 m, the sand was dumped at a quick rate with hopper suction dredgers.

The sand dumping was done with advancing slope in order to push forward the eventual mud sediments in the direction of the open part of the trench, where the mud was sucked by the hopper dredger after completing its dumping activity (the so-called cleaning phase).

The theoretical bottom line of the trench could be reached very smoothly. The deviation in depth varied between 0 and 1.00 m. The trench slopes varied between 1:10 to 1:15 for hopper suction dredgers and between 1:4 to 1:6 for cutter suction dredgers. In the case of hopper suction dredgers, the transition layer consisting of silty mud at the bottom of the excavated trench was less pronounced.

In order to reduce the cost of the superstructure of the breakwaters, the sand was dumped as high as possible, on an average about 1 m above the original sea-bottom. This was only possible by using hopper suction dredgers with slidebottomdoors and split-hopper trailer dredgers. In order to limit the loss of dumped sand by the tide-run the sand was covered as soon as possible with about 1 m of unscreened sea-gravel.

The controlled dumping of the sand was executed without any mooring. High accuracy in positioning could be reached especially by using hoppers equipped with bow thrusts or propellers.

4.2. As part of the core : raised foundation

In the zone of the LNG-breakwater, where the soft and loose top soil layers were dredged for the exploitation of the existing harbour, sand was dumped immediately on the sea-bed over a height of about 5 m in order to reduce the quantity of quarry stone of the breakwater. The sand was on both sides protected with gravel banks (fig. 5).

5. SOIL IMPROVEMENT

5.1. Mechanical characteristics of the dumped sand

To control the quality of the soil replacement, CPT tests type M4 are performed at regular distances in the soil replacement areas. In fig. 6 the results of 3 CPT tests are given as an example. On each CPT diagram of fig. 6 the bottom of the dredged trench is indicated. Although precautions were taken to minimize the quantity of mud at the bottom of the trench before dumping the sand, it can be deduced from the results of the CPT tests that this could not be prevented completely. In each diagram of fig. 6, a thin layer with relatively small cone resistances can be distinguished at the bottom of the previously dredged trench, indicating the presence of a relatively thin layer of silty or clayey sand. In general however, from a depth of about 1 or

2m underneath the top of the dumped sand layer, the cone resistance in the dumped sand layer reaches values of about 6 to 10 MN/m², or even more. The gradual increase of the cone resistance in the upper 1 to 4m of the dumped sand is to be attributed to the effect of depth.

The design criteria of the quality of the dumped sand were made bearing in mind that liquefaction of the dumped sand foundation of the breakwaters was not admitted.

Based on practical experience with natural soil layers and with hydraulic sand-fills on land, and considering CPT tests type M4 with measurements of the cone resistance every 20 cm, the following criteria were put forward for the evaluation of the quality of the dumped sand :

- underneath the critical depth in the CPT test, the mean cone resistance over any meter of depth must be higher than 5 MN/m² : $q_{c,mean} > 5 \text{ MN/m}^2$;
- over any meter of depth, at most 2 values of the cone resistance may be smaller than 4 MN/m².

Answering to the criteria was checked by one CPT test M4 per area of about 4000 m².

Considering for instance the CPT test diagrams of fig. 6 :

- at the vertical of the CPT test ZW-GZ/D29 of fig 6.c the dumped sand layer has a thickness of 7.60 m. At the bottom of the backfilled trench only one relatively small value of the cone resistance is found indicating the presence of a thin clayey sand layer. Up to a depth of 2.00 m underneath the sea-bottom the cone resistance increases nearly linearly with depth and reaches values ranging between about 7 MN/m² and about 10 MN/m². Only at a depth of 5.80 m one value of the cone resistance smaller than 4 MN/m² is measured. The above-mentioned evaluation criteria are thus fulfilled over the whole depth of the dumped sand layer.
- At the vertical of the CPT test NW-GZ/D40 of fig. 6.b the dumped sand layer has a thickness of 6.80 m. At the bottom of the backfilled trench a layer with relatively small cone resistance is found over a thickness of about 0.40 m. Apart from that layer only at a depth of 3.40 m one value of the cone resistance is smaller than 4 MN/m² and the evaluation criteria can be considered as fulfilled.
- At the vertical of the CPT test ZW-GZ/D19 of fig. 6.a the dumped sand layer has a thickness of 7.60 m. At the bottom of the backfilled trench a layer with relatively small cone resistances is found over a thickness of about 0.40 m. The evaluation criteria are fulfilled up to a depth of 3.80 m. From that depth and up to the bottom of the trench the criteria are not fulfilled. In as far as the criteria were not fulfilled, the dumped sand had to be compacted.

Handling the criterion for density control, compaction was only necessary over a length of about 25 % of the breakwaters where dumped sand has been used.

5.2. Deep compaction using vibrating needles

At Zeebrugge a compaction in depth based on vertical vibration is applied using a patented vibration probe constituted of three steel plates welded together with a cross-section of a three pronged star, hence the name "starprofile". The probe is hung in an electric or hydraulic vibrator block and is lowered into the dumped sand from a jack-up platform by means of a heavy crane. The energy of the vertically vibrating probe is transferred to the soil mainly through a series of ribs working as individual pounders (fig. 7). The vibrating probe is introduced in the ground to a depth of about 2m into the natural soil layers underneath the bottom of the filled trench.

Based on experience a vibration time of 15 minutes per "compaction prick" has been used. The distance between the compaction centres on a triangular pattern was determined depending on the results of the CPT tests performed before compaction, and on the cone resistance to be obtained in the dumped sand after compaction.

For the compaction of the dumped sand foundation layer of the northern breakwater of the Workharbour, compaction grids of one prick per 4 m², one prick per 6 m² and one prick per 8 m² have been used. Fig. 8 shows the results of CPT tests performed before and after compaction, according to the different spacings of the compaction centres on a triangular pattern. The post-compaction tests were positioned in the centre of the treatment pattern remote from the compaction centres. Fig. 8.a gives the influence on the cone resistance in the dumped sand layer of one compaction prick per area of 8 m². Fig. 8.b gives the influence of one prick per area of 6 m² and fig. 8.c of one prick per area of 4 m². From fig. 8 it can be deduced that the influence of the compaction on the cone resistance in the dumped sand layer is higher the smaller the spacing of the compaction centres. It may be remarked that the transitional layer at the bottom of the previously dredged trench could not be eliminated completely by compaction, which is to be attributed to the clay content of that layer.

A diagrammatic picture of CPT tests performed before and after compaction is given in fig. 9. Before compaction, due to the influence of depth, the cone resistance in the dumped sand layer increases nearly linearly with depth from the sea-bottom up to a depth z_0 where the cone resistance reaches a value q_{c,z_0} .

After compaction the cone resistance increases more rapidly with depth from the sea-bottom up to approximately the same depth z_0 , but reaches a higher value q_{c,z_0}^n at that depth.

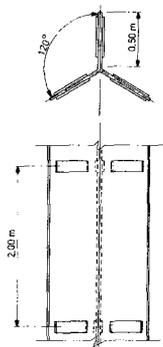


Fig. 7 : vertically vibrating probe

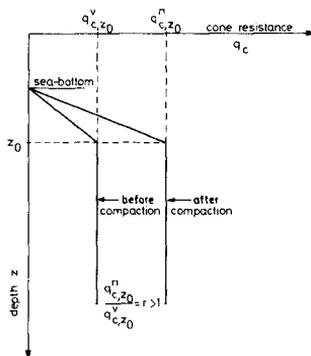


Fig. 9 : diagrammatic picture of CPT tests before and after compaction

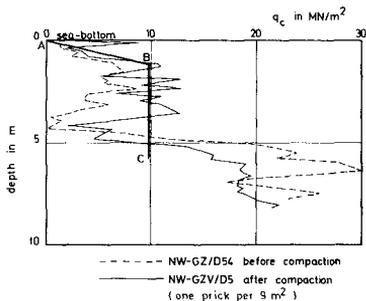


Fig. 10 : comparison of predicted and measured cone resistance profile after compaction

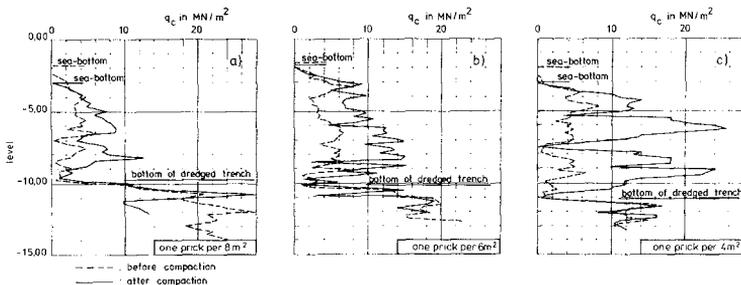


Fig. 8 : results of CPT tests before and after compaction

The ratio of the cone resistance at the depth z_0 after and before compaction is given by

$$r = \frac{q_{c,z_0}^n}{q_{c,z_0}^v} > 1$$

From a systematic analysis of 14 pairs of CPT tests performed in a zone with a compaction grid of one prick per 6 m² a ratio r could be derived, ranging between 3.3 and 1.5, with a central value $r_6 = 2.4$. From 11 pairs of CPT tests performed in a zone with a compaction grid of one prick per 9 m² a ratio r , ranging between 2.7 and 1.2, with a central value $r_9 = 1.7$ was deduced. From 16 pairs of CPT tests performed in a zone with a compaction grid of one prick per 12 m² a ratio r , ranging between 2.9 and 1.1, with a central value $r_{12} = 1.5$ was obtained. The values of r_6 , r_9 and r_{12} reflect the effectiveness of the different compaction grids. Having at disposal the results of CPT tests which doesn't fulfil the quality criteria, a rough estimation of the cone resistance profile after compaction can thus be made and thus a choice of the compaction grid.

In fig. 10 the schematic cone resistance profile ABC, predicted from the results of CPT test NW-GZ/D54 for a compaction grid of 1 prick per 9 m², is compared with the results of CPT test NW-GZ/D5, performed after compaction. The prediction can be considered as fairly good.

5.3. Deep compaction using explosives

Although the deep compaction with the vibrating probe has given satisfactory results, a program was set up to examine the feasibility of in situ densification using explosives.

5.3.1. Design of the blast program

The impact of the explosive charge causes momentary liquefaction of loose saturated sands which subsequently adopt a more dense, stable structure under the weight of the overburden and increased drainage. Literature presents empirical relationships for single concentrated charges relating size of charge, depth of charge and spacing of blast holes based on extensive field and laboratory test data. However, no clear information is presented in the literature about size and depth of two concentrated charges placed at different depths in the same blast hole.

The depth of the lower charge was selected considering that the upper layer was liquefied by the upper charge and thus could be neglected when the lower charge was detonated; in this reasoning the lower charge was detonated within one to two seconds after the detonation of the upper charge.

The sequence of blasting was planned bearing in mind that successive blasts are more effective than a single heavy blast or different small blasts at the same moment. For the time interval between successive blasts at least 4 hours was chosen as suggested in literature.

5.3.2. Set-up of charges

5.3.2.1. Explosive

A high explosive "Blastogel" with a density of 1.4 kg/dm³, primarily composed of nitroglycerine (50-60 %) and containing no ammoniumnitrate (NH₄NO₃) was used. This explosive is initially waterresistant but decomposes under water in about one month. An eventually not detonated charge couldn't present a latent danger for the achieved breakwater. Blastogel has an equivalence factor of 1.0915 with regard to TNT. Blastogel was delivered in cylindrical blocks \varnothing 85 mm of 5 kg mass.

5.3.2.2. Firing system

As the electrical firing system and the use of detonating cord were excluded for underwater blasting, a non-electrical firing system NONEL was chosen, being waterresistant and presenting sufficient strength against water currents and accidental pull.

5.3.2.3. Charges

The upper and lower charges of one vertical were prefabricated on the deck of the drilling platform. A scheme of the building up is given in fig. 11. The upper charge weights 4.5 kg, the lower charge 5.5 kg.

5.3.2.4. Round

For sake of security each charge was fitted with two detonators NONEL with suitable delay. The four NONEL tubes of the upper and lower charge of one vertical were connected together above the waterlevel and bundled with the NONEL tubes of the charges placed in the other boreholes of the same blasting series. An electric detonator was then coupled to the bundle of NONEL tubes and was detonated from a blasting initiator placed on the deck of the drilling platform.

5.3.3. Blasting program

The blasting program was carried out in the foundation layers at the extremity of the north western breakwater. The jack-up platform was equipped with three jetting rigs and the platform was moved in parallel lanes in order to minimize movements of the platform anchors.

The charge holes are jetted in parallel lines at distances of 7.5 m and the explosive charges are placed at depths of 4.5 m and 9.0 m below the sea-bottom. Blasting is executed in a triangular pattern, the charges at the corners of one triangle are detonated at least four hours after detonation of the charges of the adjacent triangle. The

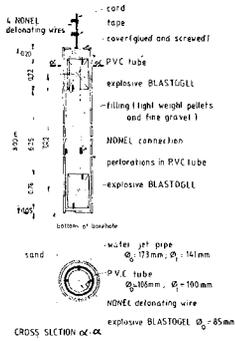


Fig. 11 : prefabricated couple of charges

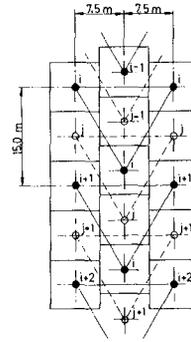


Fig. 12 : principle of arrangement

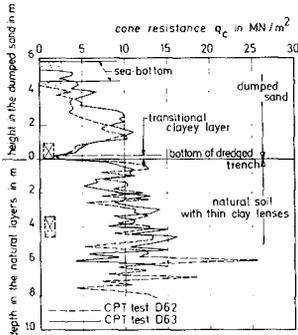


Fig. 13 : CPT tests D62 and D63

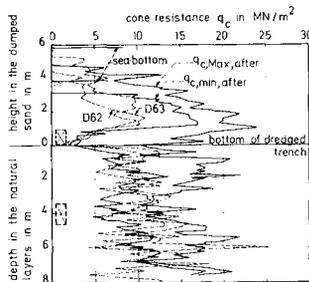


Fig. 14 : min. and max. q_c values of tests S10 to S13 after blasting with tests D62 and D63 before blasting

principle of the arrangement for the blasting is shown in fig. 12. In a first pass of the platform in a lane the charges of triangles $i-1$, i , $i+1$, ... are detonated consecutively ; the charges of triangles $j-1$, j , $j+1$, ... are detonated consecutively in a second pass of the platform in the same lane. After all charges of one lane were detonated, the platform was moved to the adjacent lane. In this way the same volume of soil was influenced by different blasts at different times.

A total surface area of about 14,000 m² was compacted over a period of fifteen days. Compaction was carried out over a thickness of layer of about 11 m using about 15.3 g of Blastogel per m³ of soil. In the compacted area 242 borings with a total length of 2,180 m were carried out in waterdepths varying between 8 and 13 m, 2,300 kg of explosives were detonated.

The increase in density of the sand after blasting was evaluated by performing a number of CPT tests.

5.3.4. Blast results

Before starting the blast program, two CPT tests numbered D62 and D63, are performed (fig. 13).

After completion of the whole blasting program over an area of 13,612 m², another 4 CPT tests numbered S10 to S13, were performed. In fig. 14 the maximum and minimum cone resistances of these tests are compared with the results of the two virgin tests D62 and D63. The results of the tests S10 to S13 confirmed the rather important increase of the cone resistances in the dumped sand layer. In the natural layer above the level of the lower charge, the minimum cone resistances in the tests S10 to S13 are of the same magnitude as the minimum cone resistances in the virgin tests D62 and D63 ; however, the maximum cone resistances after blasting are higher than in the tests before blasting. The thin clay lenses after blasting thus influence the minimum cone resistances as before blasting ; in the sand between the clay seams higher cone resistances are obtained after blasting.

6. CONCLUSIONS

- 6.1. The soil improvement technique by replacing in open sea the soft top layers by dumped sand was successfully developed.
- 6.2. Even by equal costs, the soil improvement profile still offers a more controlled construction and a foundation pad with a higher quality level. By using this profile less settlements may be expected.
- 6.3. Compaction of the dumped sand can be done as well by using a vertical vibrating probe as by using explosives.