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Compaction by blasting in offshore harbour construction

Compactage par explosif dans la construction d'un port en mer

R. CARPENTIER, Dr. ir., Chief Engineer, Belgian Geotechnical Institute, Zwijnaarde, Belgium
P. DE WOLF, ir., Senior Engineer, Ministry of Public Works, Coastal Service, Ostend, Belgium
L. VAN DAMME, ir., Senior Engineer, Ministry of Public Works, Coastal Service, Ostend, Belgium
J. DE ROUCK, ir., Chief Engineer, Harbour and Egg Consultants, Haecon N.V., Ghent, Belgium
A. BERNARD, ir., Project Engineer, T.V. Zeebouw-Zeezand, Knokke-Heist, Belgium

SYNOPSIS The paper gives a description and the results of in situ densification tests by blasting in the sand foundation layers of the breakwaters of the new outer harbour under construction at Zeebrugge on the Belgian coast. Use was made of buried charges lowered into the sea-bottom from a small drilling platform.

INTRODUCTION

Since 1976 a new outer harbour is under construction at Zeebrugge on the Belgian coast (Fig.1). The harbour is protected against the rather rough sea conditions of the North Sea by breakwaters of the rubble mound type constructed on a sand foundation. Over large lengths of the breakwaters the bearing capacity of the upper in situ soil layers was insufficient. These layers are dredged and replaced by relatively coarse dumped sea sand. The quality of the dumped sand foundation was evaluated by means of CPT tests performed from small jack-up platforms. In as far as the used evaluation criteria, based on the results of the CPT tests, were not fulfilled, deep compaction of the dumped sand layer was carried out by lowering vibrating probes from a jack-up platform into the ground (De Wolf et al., 1983).

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

Densification of loose cohesionless deposits by blasting for foundations of dams and other structures was described by Lyman (1942), Kummeneje and Eide (1961), Hall (1962), Wild and Haslam (1962), Prugh (1963), et al.. However the most detailed treatment of the subject was provided by Ivanov (1967, translated to English in 1972) who described extensive Russian experience with surface, deep, and underwater densification on numerous projects. Despite the economic attractiveness of in situ densification using explosives (Mitchell, 1970) very little data on the subject have been published in the technical literature in recent years (Klohn et al., 1981 ; Pilot et al., 1981).

Most of the applications of in situ densification using explosives are situated on land. Only a few publications deal with underwater densification (Ivanov, 1967 ; Dembicki et al., 1980) using charges loaded in water and exploded above the surface of the soil.

In the application of the method of in situ densification by blasting at Zeebrugge, use was made of buried charges lowered into the sea-bottom from a small drilling platform.

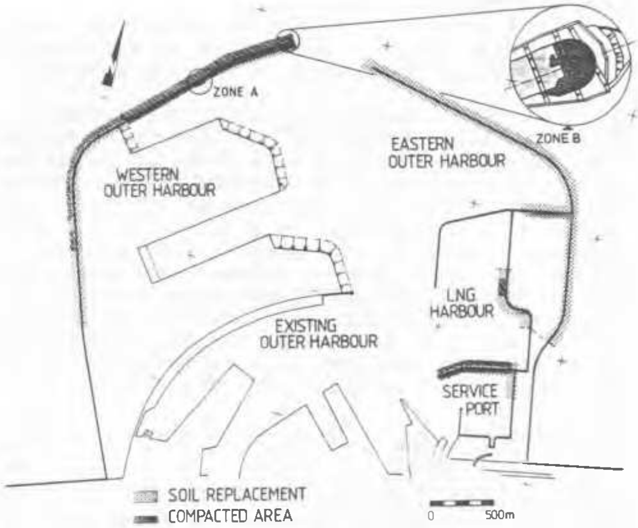

Fig.1 - Lay-out of the new outer harbour at Zeebrugge.

ENVIRONMENTAL CONDITIONS

The blasting program is carried out in the two zones A and B (Fig.1) of the north western breakwater, some 3 to 3.5 km outside the seawall. In zone A the sand foundation layer was dumped up to the level Z - 6.0 and covered with gravel up to the level Z - 5.0. In zone B the sand was dumped up to the level Z - 9.0 and covered with gravel up to the level Z - 8.0.

In the harbour area MLWS is situated at the level Z + 0.32, and MHWS at the level Z + 4.62, thus presenting waterdepths of about 5.5 m to about 12.5 m over the test areas.

At the time of the tests the significant wave height amounted to about 2 m, and the mean water current at mean tide was 1.34 m/s increasing up to 1.6 m/s at high tide in the area of the north western breakwater.

Due to the nature of the sea-bottom the water is charged with fine material and the visibility in the water is nil.

SOIL CONDITIONS

After dredging the top loose sands and soft clays, a 3.5 m (zone B) to 6 m (zone A) thick sand layer was realized by dumping sea sand in the dredged trench. The dumped sand layer was protected against erosion with a 1 m thick dumped gravel layer.

A typical grain size distribution curve of the dumped sand in the test areas is given in Fig.2.

Although special precautions were taken during dredging and dumping, a more or less clayey transitional layer could not completely be avoided at the bottom of the dredged trench (see for instance Figs. 4 and 11).

The underlying natural soil in zone A (Fig.4) was a rather dense sand with a less resistant layer with varying thickness up to about 2 to 3 m at a depth of about 11 m underneath the sea-bottom. In zone B (Fig.11) the underlying natural soil was a medium dense to dense sand with different less resistant inclusions at varying depths.

At the time of the design, shell borings had shown the sandy nature of the underlying natural soil layers. However, a more detailed investigation with continuous undisturbed sampling after blasting revealed the existence of very thin clay lenses with thicknesses of a few mm in the less resistant layer in zone A and in the natural ground in zone B, and which were not distinguished in the earlier performed shell borings. In Fig.2 also some grain size distribution curves of the layers of sand with thin clay lenses are given, showing the more or less influence of the clay lenses on the grain size distribution.

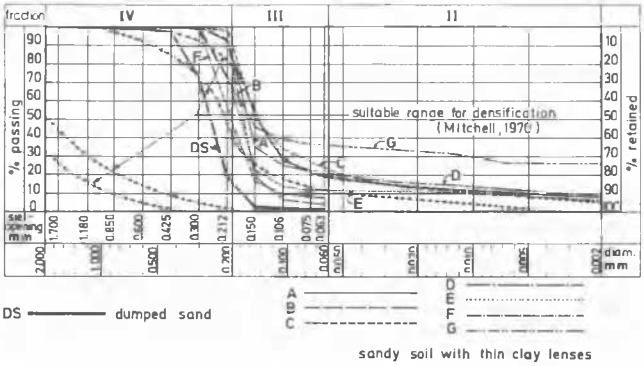

Fig.2 - Grain size distribution curves of the dumped sand in the test areas and of the sand layers with thin clay lenses.

In Fig.2 also the range of soil grain sizes suitable for densification by vibroflotation and according to Mitchell (1970) also by blasting, is represented. From Fig.2 it follows that the grain size distributions of the natural layers containing thin clay lenses are very near to the lower boundary of suitability for densification by blasting. However, the grain size distribution of the dumped sand is completely situated in the suitable range.

DESIGN OF THE BLASTING PROGRAM

A program was set up to examine the feasibility of densification by blasting of the dumped sand layer and part of the underlying natural soil in the given offshore conditions. The total thickness of layer taken into consideration amounted to about 14 m in zone A and about 11 m in zone B.

As suggested by Ivanov (1980) for considerable thickness of layer, the technique of layer by layer structure destruction was chosen, considering separately the dumped sand layer and part of the underlying natural soil layer.

Ivanov (1967 ; 1980) 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 empirical relationships of Ivanov were used as guidelines in preparing the blasting program and resulted in selecting the basic parameters for the upper charges in the blast holes : size of charge, depth of charge and grid spacing.

For the lower charge in a blast hole, about the same size as for the upper charge was chosen. 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 large one or several small ones detonated simultaneously (Hall, 1962 ; Prugh, 1963 ; Mitchell, 1970). For the time interval between successive blasts at least 4 hours was chosen as suggested by several authors (Hall, 1962 ; de Groot and Bakker, 1971).

The blasting program was carried out in two stages. Stage 1 was a test stage and consisted of a grid of 25 blast holes over an area approximately 1540 meters square to confirm : the size and depth of the charges, the grid spacing, the amount of densification, the feasibility of the method in the given offshore conditions.

The Stage 1 test blasting program confirmed that the choice of the parameters was suitable for soil and environmental conditions at this site and consequently the same rules were used for the Stage 2 blasting program which was the production stage.

SETUP OF CHARGES

Explosive

A high explosive "Blastogel" with a density of 1.4 kg/dm^3, primarily composed of nitroglycerine (50-60 %) and containing no ammonium nitrate was used. This explosive has initially a good water resistance but deteriorates under water in about one month. Blastogel has an equivalence factor of 1.0915 with regard to TNT. Blastogel was delivered in cylindrical blocks Ø 85 mm of 5 kg mass.

Firing system

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

Charges

The upper and lower charges of one vertical were prefabricated on the deck of the drilling platform. A scheme of the prefabricated charges is given in Fig.3.

Circuit

For sake of security each charge was fitted with two detonators NONEL with suitable delay. The four NONEL tubes of the upper and lower charges of one blast hole were connected together above the waterlevel with the NONEL tubes of the charges placed in 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.

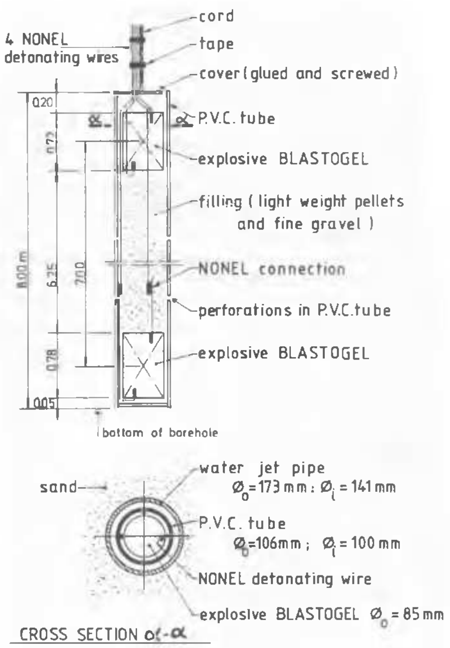

Fig.3 - Prefabricated couple of charges in zone A.

STAGE 1 TEST BLASTING PROGRAM

The test blasting program was carried out directly in the foundation layers of the north western breakwater (zone A in Fig.1).

The total test area of 81 x 38 m^2 was divided into two squares of 40.5 x 38 m^2, the first one serving as a trial area for a first choice of the blasting parameters and to obtain practical experience with the execution procedure in the given offshore circumstances, the second one serving for the eventual adaptation of the blasting parameters.

The subsoil conditions before blasting were obtained from CPT tests. Fig.4 shows a typical subsoil profile at the Stage 1 test site.

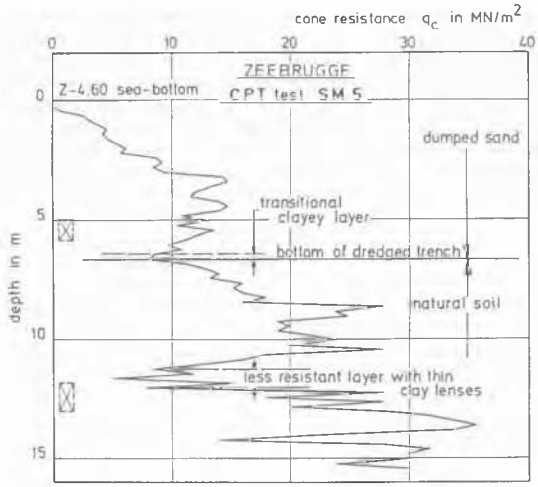

Fig.4 - Typical CPT test in zone A before blasting.

The lay-out for the Stage 1 blasting test program is shown in Fig.5. It contains 25 blast holes in a square grid of 7.5 m side, divided in 4 successive blast series as indicated in Fig.5. It was planned to make and charge the holes of each series after blasting the foregoing series.

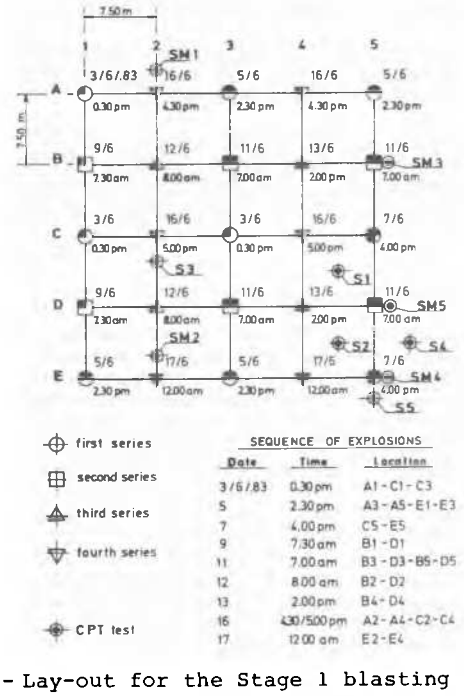

Fig.5 - Lay-out for the Stage 1 blasting test program.

The explosive charges were placed at depths of 5 m and 12 m below the sea-bottom in 173 mm diameter holes.

In blast series 1 and 2 an upper charge of 5.5 kg of Blastogel and a lower charge of 6.0 kg of Blastogel were used. In series 3 and 4 both upper and lower charges were increased with 0.5 kg of Blastogel to take into account an increase in density caused by the former blast series.

In the Stage 1 blasting program two jetting rigs were used on the drilling platform for making the charge holes. The water jet pipe served also as a casing and held the hole open while the charge was placed. Each charge, prepared on the deck of the drilling platform, was lowered into the casing by its own weight down to the bottom of the jetted hole. The casing was then withdrawn and the hole was not backfilled above the upper charge.

Blast series 1 consisted of 9 blast holes to be jointly detonated. However, already after execution of four holes it revealed that it was impossible to keep the installed NONEL tubes intact due to the strong changing currents and the movements of the drilling platform over the test site. It therefore was decided to make and blast the holes in groups of maximum four holes of the same series. The final sequence of blasting is indicated in Fig.5.

In the Stage 1 test the total soil volume of 40.5 x 38 x 14 = 21.546 m^3 was improved by detonating 297.5 kg of Blastogel, or 13.8 g of Blastogel per m^3 of soil.

An attempt was made to measure the settlements of the sea-bottom after each blast series. However this had to be disregarded due to several practical reasons (lack of visibility in the water, rough sea conditions, out of plumb of the settlement stakes by the blasting or by pushes of the spuds of the drilling platform).

No pore pressure readings were made. Observations of small geysers of water and gas could only be made at the water surface.

As an indirect method of assessing the soil improvement after blasting a number of CPT tests were performed to estimate the increase in density of the sand after each blast series.

STAGE 1 BLASTING RESULTS

Before starting the blasting program a series of 5 CPT tests (prefix SM) was performed. At different intermediate stages a total of 4 CPT tests was performed. A few days after the last detonations a series of 5 CPT tests (prefix S) was performed. The locations of all CPT tests SM and S in the test area are given in Fig.5.

The virgin CPT tests SM showed comparable results although with some local differences from one vertical to another. For comparison purposes the boundaries of the minimum and maximum cone resistances measured in the five tests SM are represented in Fig.6 with the bottom of the dredged trench as a reference line.

The CPT tests S1 to S5 are performed from the fifth day after the last blasting. Taking into account the location of the tests (Fig.5), the results are compared in Figs. 7 and 8. In Fig.7 the maximum and minimum cone resistances of the four tests S1, S2, S4 and S5 are compared with the results of the nearby virgin tests SM 4 and SM 5. In Fig.8 the maximum and minimum cone resistances of all tests S1 to S5 are compared

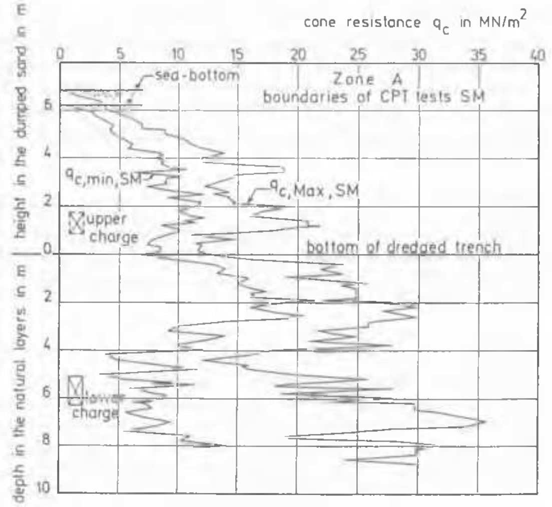

Fig.6 - Boundaries of minimum and maximum cone resistances measured in 5 CPT tests in zone A before blasting.

with the minimum and maximum cone resistances of all virgin tests SM 1 to SM 5.

From Fig.8 it can be stated that in the dumped sand layer the minimum cone resistances after blasting are situated between the minimum and maximum cone resistances of the virgin tests, while the maximum cone resistances after blasting are higher than the maximum cone resistances of the virgin tests. From Fig.7 it can be stated that in the dumped sand layer the minimum cone resistances after blasting are situated in the same range as the cone resistances of the virgin tests SM 4 and SM 5, but that the maximum cone resistances after blasting are much higher.

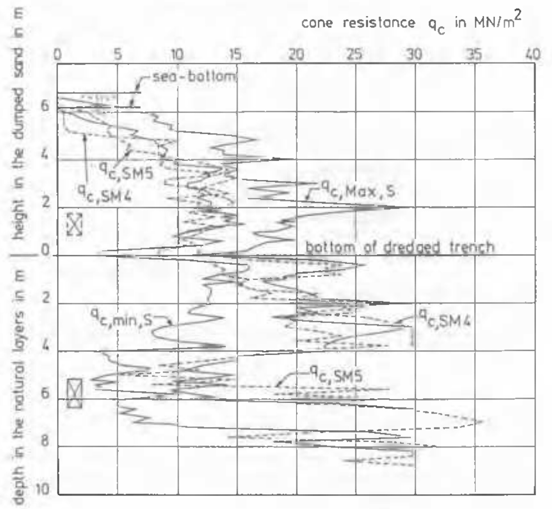

Fig.7 - Comparison between minimum and maximum cone resistances of CPT tests S1, S2, S4 and S5 after blasting, and CPT tests SM4 and SM5 before blasting.

From Fig.7 it can be stated that for the natural soil layers the maximum cone resistances after blasting are in the same range as the maximum cone resistances of the virgin tests. The minimum cone resistances after blasting however, are

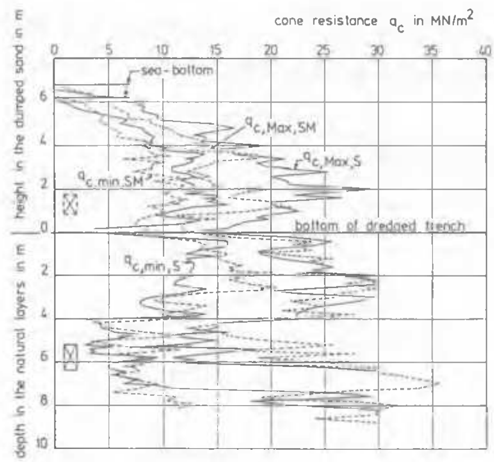

Fig.8 - Comparison between min. and max. cone resistances of tests S1 to S5 after blasting, and min. and max. cone resistances of tests SM1 to SM5 before blasting.

much lower. It must however be remarked that test SM4 reached only a few metres into the natural soil layers. From Fig.8 it can be concluded that the minimum cone resistances after blasting are only localy lower than the minimum cone resistances in the virgin tests which can be due to local differences from one vertical to another.

The following conclusions were drawn from the results of the CPT tests after blasting

- Although already rather high cone resistances were measured in the dumped sand layer before blasting, still higher cone resistances were obtained by blasting ;
- The results of an intermediate CPT test performed in the immediate vicinity of a blast hole one hour after detonation confirmed that a liquefied zone was created around the blast hole ;
- Intermediate CPT tests indicated that even within a period of at least 18 hours after detonation the influence of the blasting was still noticeable in the natural soil layer with thin clay lenses. Control tests thus have to be performed after sufficient time after blasting.
- The results of CPT tests S1 to S5 performed from the fifth day after blasting do not indicate a marked influence on the cone resistances in the natural soil layers. Even the minimum cone resistances remained unchanged which is to be attributed to the influence of the thin clay lenses ;
- In general a more homogeneous cone resistance diagram than in the virgin tests was obtained after blasting (see for instance the results of tests S2 in Fig.9 and SM5 in Fig.4).

STAGE 2 BLASTING PROGRAM

The stage 2 blasting program was carried out in the foundation layers at the extremity of the north western breakwater (zone B in Fig.1) having in view a high production rate of in situ densification by blasting. The jack-up platform was equiped with three jetting rigs and the platform was moved in parallel lanes in order to minimize handling 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-

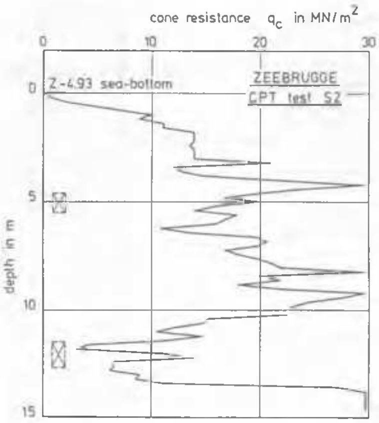

Fig.9 - Results of CPT test S2 in zone A after blasting.

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 principle of the arrangement for Stage 2 blasting is shown in Fig.10. In a first pass of the platform in one 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. In the first pass an upper charge of 4.0 kg Blastogel and a lower charge of 5.0 kg of Blastogel were used. In the second pass both upper and lower charges were increased with 0.5 kg of Blastogel. 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.

By working continuous shifts, twenty fours a day and seven days a week a total surface area of about 14 000 m^2 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 m^3 of soil. In the compacted area 242 borings with a total length of 2180 m were carried out in water depths varying between about 8.5 m and 12.5 m. 2300 kg of explosives were detonated.

No pore pressure readings were made and no settlements were measured.

The increase in density of the sand after blasting was evaluated by performing a number of CPT tests from the jack-up platform.

STAGE 2 BLASTING RESULTS

Before starting the blast program, two CPT tests D62 and D63, shown in Fig.11, are performed.

After completion of the whole blasting program over an area of 13.612 m^2, 4 CPT tests numbered S10 to S13 are performed. In Fig.12 the boundaries of the maximum and minimum cone resistances of these tests are compared with the results of the two virgin tests D62 and D63. The tests after blasting indicate a rather high increase of the cone resistances in the dumped sand layer. In the underlying natural layer also the maximum measured values are somewhat higher after blasting.

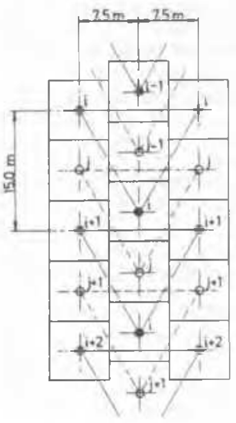

Fig.10 - Principle of arrangement for Stage 2 blasting.

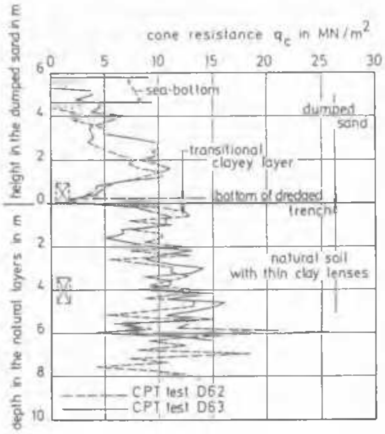

Fig.11 - CPT tests D62 and D63 in zone B before blasting.

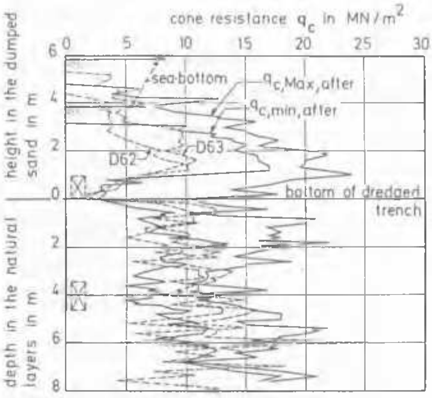

Fig.12 - Min. and max. q_c values of tests S10 to S13 after blasting with tests D62 and D63 before blasting.

However, above the level of the lower charge, the minimum cone resistances in the tests S10 to S13 after blasting are of the same magnitude as the minimum cone resistances in the virgin tests D62 and D63. As the receding peaks in the CPT diagrams in the natural soil layer are caused by thin clay lenses, it can be concluded that the existence of thin clay lenses has a great influence on the final result of an in situ densification by blasting.

CONCLUSIONS

The in situ densification of rather loose, saturated, granular soils by the use of explosives has provided an effective and economical means of improving the properties of the deposit in the given offshore conditions.

The presence of thin clay lenses has an important influence on the efficiency of the blasting technique.

Although data are available (Ivanov 1967 ; 1980) to guide the engineer in planning an in situ densification program by blasting, a prediction of the in situ behaviour of loose granular deposits when treated by blasting is difficult. A full scale field test is necessary to determine the design parameters : area of influence, degree of densification, rate of pore pressure dissipation, size of charge and minimum blast hole spacing required.

For the moment insufficient data are available for the technique of layer by layer structure destruction and further experience in this field has to be gathered.

Densification by blasting has given satisfactory results in the dumped sand layer at Zeebrugge although a mean cone resistance of 15 MN/m^2 was demanded by the Owner.

The blasting technique has been economically competitive with the compaction technique by vibrating probes in the given offshore conditions at Zeebrugge.

REFERENCES

de Groot, W., and Bakker, J.G. (1971). An investigation for compaction with explosions (in Dutch). LGM-mededelingen (14), 3, 65-89.

Dembicki, E., Kisielowa, N., Nowakowski, H., and Osiecimski, R. (1980). Compactage des fonds marins sableux à l'explosif. Proc. Int. Conf. on Compaction, 1, 301-305, Paris.

De Wolf, P., Carpentier, R., Allaert, J., and De Rouck, J., (1983). Ground improvement for the construction of the new outer harbour at Zeebrugge, Belgium, Proc. 8th ECSMFE, 2, 827-832, Helsinki.

Hall, C.E. (1962). Compacting a dam foundation by blasting. ASCE J. Soil Mech. and Found. Div. (88), SM3, 33-51.

Ivanov, P.L. (1967). Compaction of noncohesive soils by explosions. Translated from Russian in 1972. U.S. Dept. of Commerce TT70-57221.

Ivanov, P.L. (1980). Consolidation of saturated soils by explosions. Proc. Int. Conf. on Compaction, 1, 331-337, Paris.

Klohn, E.J., Garga, V.K., and Shukin, W. (1981). Densification of sand tailings by blasting. Proc. 10th ICSMFE, 3, 725-730, Stockholm.

Kummeneje, O., and Eide, O. (1961). Investigation of loose sand deposits by blasting. Proc. 5th ICSMFE, 1, 491-497, Paris.

Lyman, A.K.B. (1942). Compaction of cohesionless foundation soils by explosives. ASCE Transactions (107), 1330-1348.

Mitchell, J.K. (1970). In-place treatment of foundation soils. ASCE J. Soil Mech. and Found. Div. (96), SM1, 73-110.

Pilot, G., Colas des Francs, E., Puntous, R.,and Queyroi, D. (1981). Compactage par explosif d'un remblai hydraulique. Proc. 10th ICSMFE, 3, 757-760, Stockholm.

Prugh, B.J. (1963). Densification of soils by explosive vibrations. ASCE J. Constr. Div. (89), CO1, 79-100.

Wild, P.A., and Haslam, E.F. (1962). Towers and foundations for project EHV. ASCE J. Power Div. (88), PO2, 69-111.