# 9. A new approach to the execution and control of dynamic compaction

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SYNOPSIS. Two case histories of soil improvement by dynamic compaction are presented. It is demonstrated that by careful field monitoring, it is possible to develop the optimal compaction procedure on site. Furthermore, a novel electronic compaction control procedure is described, using dynamic measurements during the impact of the pounder, to determine the increase of soil stiffness. In this way it is possible to minimize the required compaction energy, and to document that the required soil densification has been achieved.

## INTRODUCTION

1. The concept of soil compaction using a falling mass, often refered to as dynamic compaction (DC), is already known since 1936 (ref. 1). However, only since 1969 this soil improvement technique has become generally accepted especially as a result of the pioneering work by L. Ménard. DC is mainly used to improve cohesionless, granular soils and fills. However, some projects reported in the literature describe also improvement of cohesive soils. The objectives of DC treatment can be

(a) to increase the shear strength of the soil
(b) to decrease the soil compressibility
(c) to increase the soil homogenity in depth as well as in plane
(d) to decrease the liquefaction potential of a loose sand deposit.

2. Fig. 1 gives an overview of a typical dynamic compaction site. The efficiency of soil improvement by DC is affected by several factors which should be carefully considered before the start of the project : soil type and soil properties, initial stress conditions, location of ground water level and capillarity effects, soil stratification/ layering, dynamic response of subsoil, required

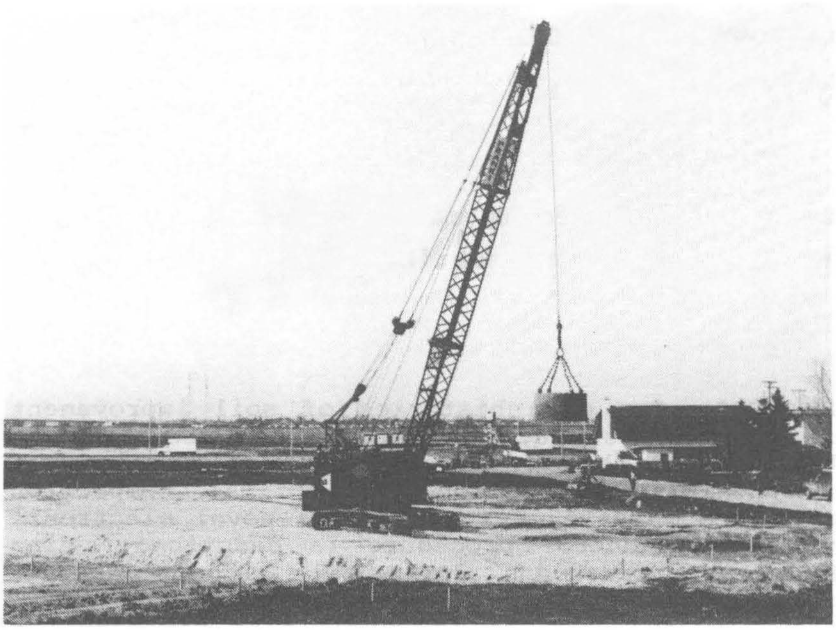

Fig. 1. View of a dynamic compaction site

depth and degree of compaction, and vibration effects on adjacent structures.
The method of execution must be chosen depending on the prevailing conditions. In order to achieve efficient soil compaction, it is important to avoid

(a) the paving effect : a very dense layer is formed at the soil surface. This layer absorbs a large amount of the compaction energy and prevents densification of deeper layers.
(b) the mustache effect : the soil in the immediate vicinity of the impact point is heaved, i.e. it is displaced but not compacted.
(c) the lateral remoulding effect : during the progressive deepening of the crater, the soil is laterally displaced along shear failure surfaces but not compacted.

Generally the following compaction procedure is recommended : first and only if necessary, the site is covered with a layer of granular soil in order to provide a working platform and to reduce the effect of lateral remoulding. Then the first compaction pass is executed : the pounder is dropped a predetermined number of times in a

normally square grid. The craters formed are then refilled. This first pass is followed by one or several more passes during which the pounder is dropped at points on a grid inbetween the previous compaction points. These passes are intended to achieve most of the soil compaction ("high energy pass"). Thereafter the "ironing pass" is executed. The aim of this last pass is to level the surface which is achieved by using reduced drop heights on a narrow grid. If necessary, the surface can be compacted afterwards by means of a rolling vibrator.
In order to verify the design criteria, it is recommended that a compaction test is carried out. This will assure that an economical and technically satifactory procedure is used.
The results of the compaction work should be checked both during and after execution. Both projects discussed below will illustrate the importance of this aspect.

SITE AT CHARLEMAGNE

3. On this project DC was specified in order to enable the use of a shallow foundation, i.e. spread footings (width of 0.6 m) for a two-storey residential building at Charlemagne, 15 km east of Montreal. The spread footings had to satisfy the following requirements

(a) allowable bearing capacity of 0.1 MPa with a factor of safety of 3 with regard to failure
(b) a maximum total settlement of 25 mm and a maximum differential settlement of 15 mm at a pressure of 0.1 MPa.

Furthermore, it was required that the execution of DC might not generate vibrations harmful to the neighbouring buildings.

4. Laboratory and in-situ tests indicated the presence of a fine, uniform sand layer below the 0.15 m thick organic surface layer. This sand layer was very loose below 1.5 m depth and contained a silty sand layer at around 3 m. Stiff very sensitive silty clay was encountered below 4.5 m on top of till, beginning at 15 m depth. The phreatic surface was located at a depth of 1.5 m.

5. The drop mass, its geometry and the drop height were determined in advance using the method proposed by M. Wallays (ref. 2), which makes it possible to calculate the induced vertical dynamic stress increase and the pounder penetration per blow as a function of depth (Fig. 2). The

settlement of the spread footing was calculated assuming a preconsolidation pressure equal to the calculated vertical dynamic stress. The number of blows required to obtain the desired densification was deduced from the calculated pounder settlements. The 3 compaction parameters were changed until a combination was found which satisfied the above specified requirements, taking into account the replacement of the surface layer by a 0.5 m thick sand layer. Furthermore, for this specific project the drop mass was chosen such as to limit the dynamic stresses in the clay in order to prevent remoulding of this layer.

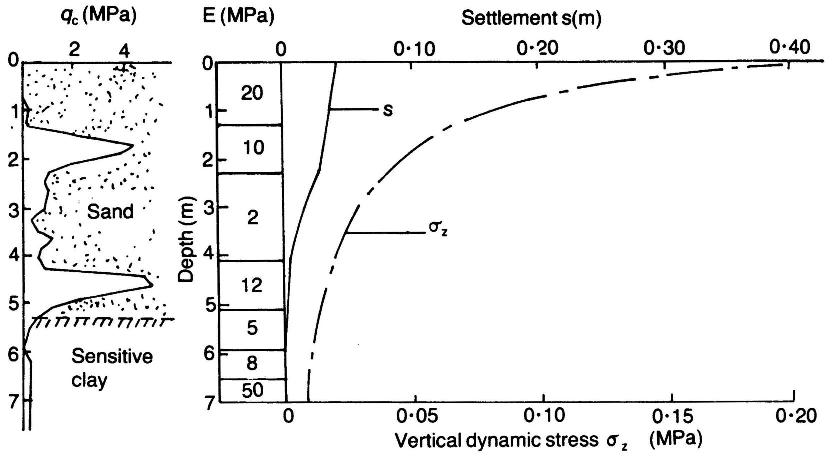


Fig. 2. Calculation of pounder settlement and stress change according to ref. 2.

The following compaction procedure was finally adopted

- pounder mass of 7 tons with a diameter of 1.8 m
- effective drop height of 4.2 m, corresponding to an actual drop height of 6 m
- 2 passes using a grid with a spacing of 5 m
- 8 to 18 drops per impact point with an average of 13
- specific energy of around 45 tm/m$^2$ of treated surface.

Vibration monitoring

6. Vibrations due to different drop heights were measured adjacent to the nearest building,

using three-directional geophones. It was observed that in general, the vertical vibration amplitudes were most pronounced and that the vibration frequency was around 6 to 7 Hz. Furthermore the measurements showed that it was safe to use a drop height of 6 m, resulting in a maximum measured particle velocity of 6 mm/s (Fig. 3).

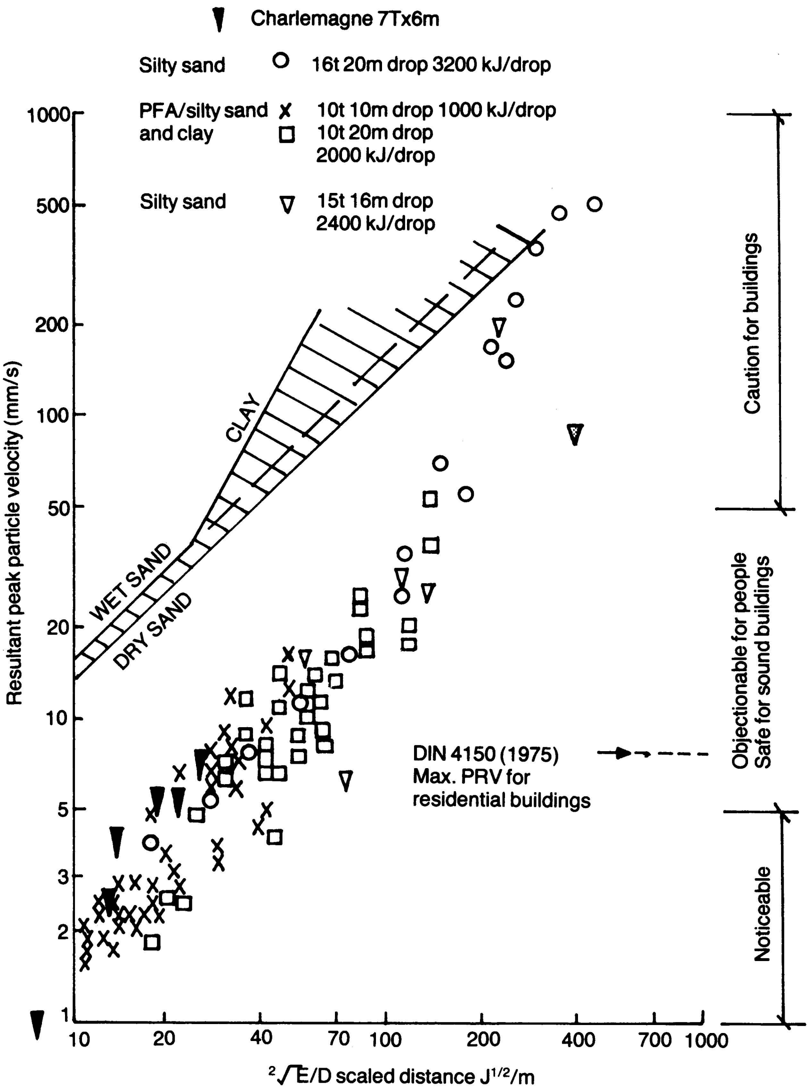


Fig. 3. Results of the vibration measurements from dynamic compaction, cf. ref. 3.

Monitoring of soil displacement

7. At certain impact points, the pounder penetration and the crater form were measured as a function of the number of blows, as shown in Fig. 4 and 5. Fig. 4 illustrates that pounder penetration decreases with increasing number of blows, which means that the soil is gradually densified. Fig. 5 allows to determine on an empirical basis a curve relating the volume of the crater to the pounder penetration.

8. During execution of the DC project, pounder penetration was measured at all impact points. These measurements allowed to increase the specific energy at those points where the pounder penetration at the previous pass exceeded the average penetration value. Furthermore, it was possible to deduce the total displaced soil volume from the measured pounder penetrations, using the experimental curve mentioned above. This total displaced soil volume can be related to an average brute soil compression value which however has to be corrected for the heaved soil around the impact points, to obtain the average net soil compression.

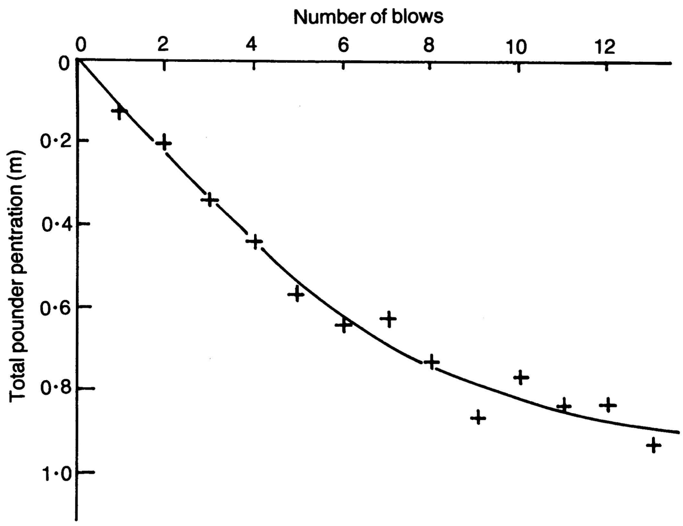


Fig. 4. Total pounder penetration as a function of number of blows

At an individual impact point, it was found that the net soil compression corresponded to 40 % of the soil displacement. This value decreased to 33 % for the total grid of impact points.

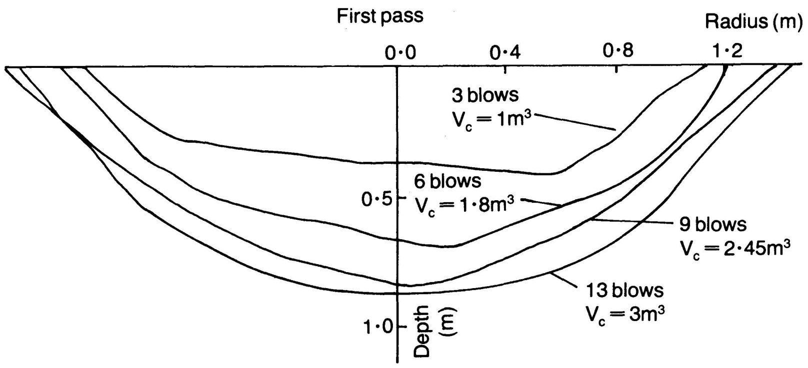


Fig. 5. Crater-form as a function of number of blows

Measurements of pounder penetration during the 4 passes (Fig. 6) suggests that

(a) the pounder penetration and the average brute settlement decrease with the number of passes
(b) the difference between the maximum and the minimum pounder penetration decreases with the number of passes, which means that the soil deposit becomes more homogeneous
(c) the average net settlement does not vary significantly and amounts to 0.2 m for all 4 passes together.

Geotechnical monitoring

9. The pore water pressure was monitored using open tube piezometers. The phreatic level was temporarely raised by 2 m. Moreover, CPT tests carried out 6 hours after completion of DC showed no significant soil improvement and also suggested the presence of excess pore water pressure. It was then decided to use 4 passes instead of 2, which increased the applied specific energy to 90 $tm/m^2$.

10. In order to evaluate the compaction results, CPT, SPT and PMT tests were executed 5 to 7 days after completion of the fourth pass. The measured values before and after DC at one specific point are shown in Fig. 7. Significant improvement of soil conditions can be observed at depths between 1 and 4 m, except for a thin zone at about 3 m, which contained silty layers. The average improvement values for depths between 0.7 and 4.3 m equaled to

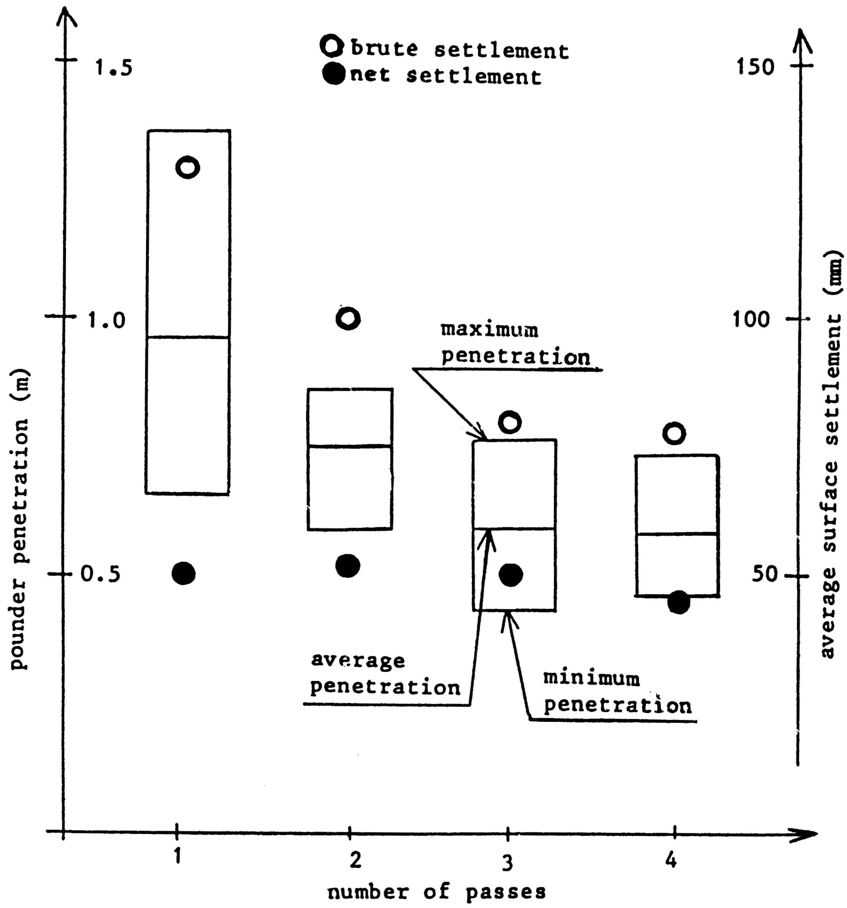


Fig. 6. Pounder penetration and surface settlements as a function of compaction passes

1.8, 2.1, 2.5 and 2.3 for the cone resistance, the SPT blowcount, the pressiometer modulus (E) and the limit pressure ($p_\ell$), respectively. It should be noted that also the ratio $E/p_\ell$ has increased, indicating that the soil has become overconsolidated as a result of DC. The settlements calculated on the basis of the PMT results were reduced by a factor of 4.

SITE AT GROS CACOUNA

11. A predominantly granular fill at the right side of the Sain-Laurent river, situated 250 km downstream of Quebec had to be densified. This

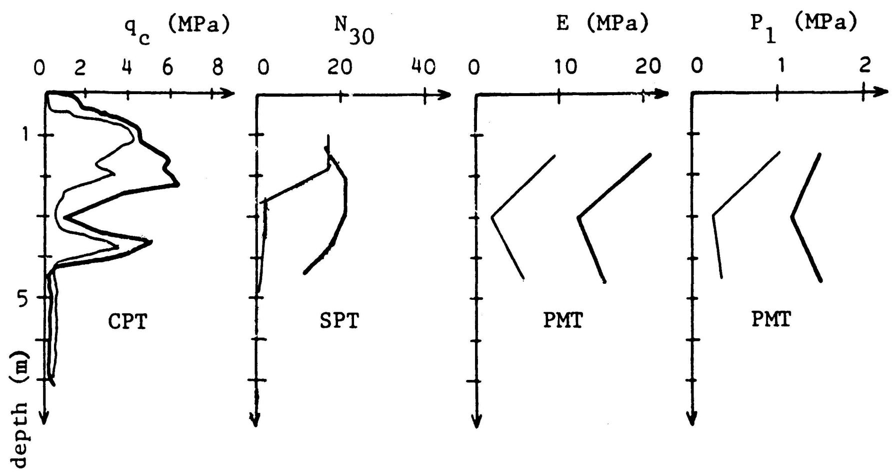


Fig. 7. Comparison between geotechnical tests before (-) and after (-) dynamic compaction

fill contained cobbles and boulders and had a variable thickness.

It was anticipated to found the structure on spread footings (2 m by 2 m) at a depth of 0.5 m. The subsoil had to be improved to sustain an admissable pressure of at least 300 kPa, with a factor of safety of 3 and a maximum angular distortion of 1/500. The first criterion corresponded to a limit pressure of about 0.6 MPa, the second criterion to an absolute settlement of 25 mm using the charts of Bjerrum (ref. 4) which required a pressiometer modulus of 3.5 MPa.

The soil densification was obtained by DC using a 16 ton mass and a drop height of 20 m. The total applied energy varied between 284 and 437 tm/m$^2$ as a function of the fill depth, at a grid spacing between 5 and 10 m. The compaction energy was achieved by 2 to 3 high energy passes and one ironing pass.

12. It was initially suggested to check the results of DC by 30 pressuremeter tests. However, the presence of cobbles and boulders caused problems and the reliability of the test results was considered doubtful, because of the heterogenity of the fill. Therefore, it was proposed to use the impact of the pounder as a dynamic load test.

By measuring the retardation of the large (5 m$^2$) mass upon impact it was possible to back-calculate the dynamic response of the subsoil. In this way, one could estimate the settlement characteristics (modulus) of the soil.

Preliminary calculations

13. Initially, a parametric study was carried out in order to determine the optimal drop height of the pounder. A computer program was developed which analyzes the dynamic interaction between the pounder and the subsoil (ref. 6). This study was performed using the following soil properties

(a) specific soil weight : $\rho$ = 2000 kg/m$^3$
(b) hyperbolic stress-strain relationship at small deformations : G = 6.75 MPa, $\nu$ = 1/3
(c) at failure : $q_r$ = 0.9 MPa.

The required drop height had to generate a dynamic pressure corresponding to the admissable pressure of 0.3 MPa. Table 1 shows the main results of this parametric study and it was decided to adapt a drop height of 0.5 m, which would generate a dynamic pressure of 0.5 MPa.

Table 1. Main results of the parametric study

| Drop height H (m) | 0.25 | 0.50 | 1.00 |
|---|---|---|---|
| Impact velocity $v_i$ (m/s) | 2.21 | 3.13 | 4.43 |
| Maximum dynamic force $F_M$(MN) | 2.0 | 2.5 | 3.1 |
| Maximum dynamic settlement $s_M$ (mm) | 30 | 46 | 70 |
| Load duration T (ms) | 58 | 59 | 62 |

Test execution and interpretation

14. For each drop it was necessary to measure the drop height (H), the permanent pounder penetration ($s_R$) and the acceleration (a). The acceleration signal was obtained using a Brüel and Kjaer accelerometer installed close to the gravitational center of the pounder. This analogue signal was recorded and stored permanently by a field data acquisition system. In order to obtain meaningfull test results the pounder should have a perfect contact with the soil. Therefore it was necessary for the control tests to let the pounder fall from a reduced height (comparable to the above determined value) before realistic measurements could be made.

15. The test results are essentially based on the measured acceleration time records. The results are given by 3 graphs (Fig. 8), showing the following :

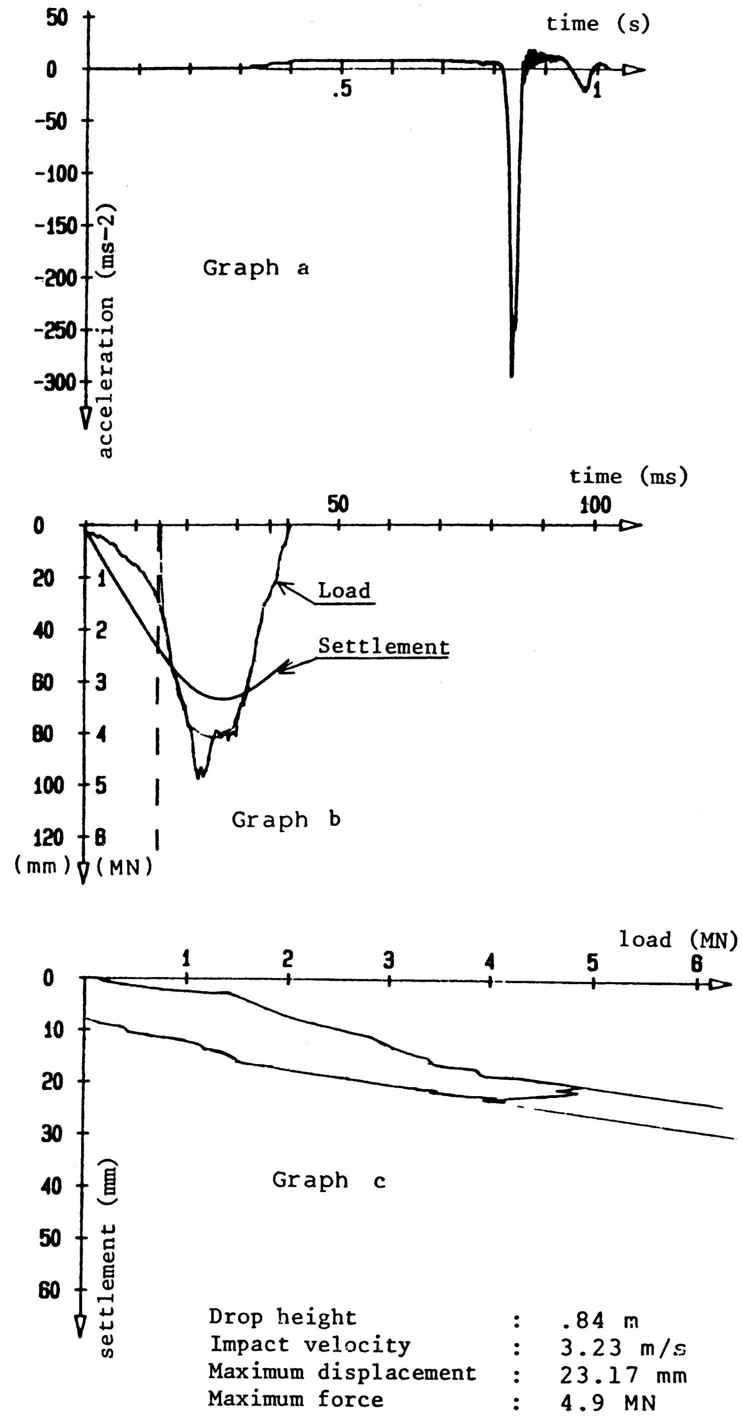


Fig. 8. Results of a dynamic control test

(a) the acceleration signal (a) as a function of time (Graph a). This is the raw signal measured on site and can be separated into 4 parts : before fall (a=o), during the fall (a ≈ 2/3 g), during the first impact (a < o, deceleration) and the rebound.

(b) the settlement s and the force F (Graph b). The settlement s is obtained by integrating the acceleration twice with time, and the force F by application of Newton's law.
Only the impact part of the acceleration signal is utilized to calculate the settlement and the force. The force-time diagramme indicates two stages corresponding to imperfect and perfect pounder-soil contact during the impact. Both phases are separated on Graph b by a vertical line.

(c) the force-displacement diagramme (Graph c) obtained from a combination of the settlement and the load diagramme of Graph b.
The displacement value used is the pounder displacement after perfect contact is obtained.

The soil stiffness (coefficient k) can then be calculated and is used to determine the static settlement $s_E$ of the square foundation (2 by 2 m) subjected to a load of 300 kPa. This soil stiffnes coefficient k is the average of 2 values : $k_R$ and $k_T$. The value $k_R$ equals to the slope of the unloading part of the force-displacement diagramme the value $k_T$ is calculated from a dynamic response analysis of a single degree of freedom system, assuming the period equal to the measured load duration T.

Results

16. Twenty-four dynamic control tests were executed. The results are shown in Table 2.

17. Table 2 indicates that all tests satisfied the above specified acceptance criteria. Moreover, the test data also show that the site has become more homogeneous as a result of the DC treatment.

18. Two PMT tests (Fig. 9) were performed in order to calibrate the dynamic control tests. However, the measured deformations were so small that the limit pressure could not be determined. A lower boundary value of the limit pressure was estimated to be about 2.6 MPa, which indicates an admissable pressure of at last 3 times the required one. Comparison of the settlements based on DC control tests and PMT tests indicates that the values of the dynamic control tests are on the safe side (by a factor of about 2).

Table 2. Results of the dynamic control tests

| Test number | H (m) | $s_R$ (mm) | $v_i$ (m/s) | $s_M$ (mm) | $F_M$ (MN) | T (ms) | $k_T$ (MN/mm) | $k_R$ (MN/mm) | $s_E$ (mm) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.95 | 12 | 3.29 | 25.2 | 3.22 | 25 | .253 | .179 | 6.2 |
| 2 | 0.77 | 8 | 3.29 | 20.7 | 2.72 | 35 | .129 | .170 | 9.0 |
| 3 | 0.75 | 10 | 3.14 | 23.6 | 4.96 | 16 | .617 | .370 | 2.8 |
| 4 | 0.71 | 13 | 3.29 | 24.6 | 4.70 | 23 | .299 | .179 | 5.7 |
| 5 | 0.76 | 12 | 3.25 | 29.8 | 3.30 | 30 | .175 | .155 | 8.1 |
| 6 | 0.76 | 13 | 3.19 | 25.7 | 2.76 | 26 | .234 | .188 | 6.3 |
| 7 | 0.77 | 10 | 3.20 | 28.8 | 2.80 | 32 | .154 | .255 | 6.7 |
| 8 | 0.82 | 8 | 3.27 | 19.8 | 3.36 | 21 | .358 | .358 | 3.7 |
| 9 | 0.75 | 12 | 3.06 | 17.9 | 3.48 | 15 | .702 | .500 | 2.2 |
| 10 | 0.78 | 10 | 3.19 | 34.9 | 3.63 | 28 | .201 | .155 | 7.5 |
| 11 | 0.75 | 8 | 3.17 | 37.3 | 2.84 | 31 | .164 | .130 | 9.1 |
| 12 | 0.90 | 8 | 3.26 | 27.0 | 4.07 | 16 | .617 | .376 | 2.8 |
| 13 | 0.84 | 9 | 3.23 | 23.2 | 4.90 | 22 | .326 | .275 | 4.4 |
| 14 | 0.74 | 7 | 2.71 | 10.5 | 3.68 | 17 | .546 | .385 | 2.9 |
| 15 | 0.78 | 10 | 3.10 | 34.0 | 4.96 | 30 | .175 | .132 | 8.7 |
| 16 | 0.80 | 11 | 3.04 | 34.3 | 3.68 | 24 | .274 | .154 | 6.5 |
| 17 | 0.69 | 6 | 3.11 | 20.3 | 3.51 | 22 | .326 | .328 | 4.1 |
| 18 | 0.80 | 11 | 3.14 | 18.3 | 4.31 | 20 | .395 | .500 | 3.0 |
| 19 | 0.77 | 9 | 3.10 | 17.4 | 5.04 | 18 | .487 | .477 | 2.8 |
| 20 | 0.80 | 18 | 3.49 | 15.6 | 4.05 | 18 | .487 | .238 | 3.9 |
| 21 | 0.88 | 7 | 3.51 | 40.2 | 3.73 | 27 | .217 | .275 | 5.4 |
| 22 | 0.80 | 7 | 3.11 | 32.0 | 4.03 | 20 | .395 | .214 | 4.6 |
| 23 | 0.80 | 11 | 3.49 | 27.5 | 4.90 | 24 | .274 | .274 | 4.8 |
| 24 | 0.75 | 6 | 2.94 | 17.2 | 3.24 | 22 | .326 | .210 | 5.1 |

## SUMMARY AND CONCLUSIONS

19. One objective of the paper was to demonstrate that by careful monitoring of the dynamic compaction procedure, it is possible to minimize the required compaction energy. This has several advantages such as reduced dynamic forces and thus lower vibration levels, light-weight equipment and significantly lower costs.
Another important aspect of careful site monitoring is the possibility to optimize the compaction procedure. Measurements of soil displacements as a result of pounder impact provide valuable information on the efficiency of soil compaction.

A novel compaction control procedure was described as well as its successful application on a project. By measuring the acceleration during the pounder impact, it is possible to monitor the compaction process. From the acceleration time history a dynamic soil modulus can be calculated which is

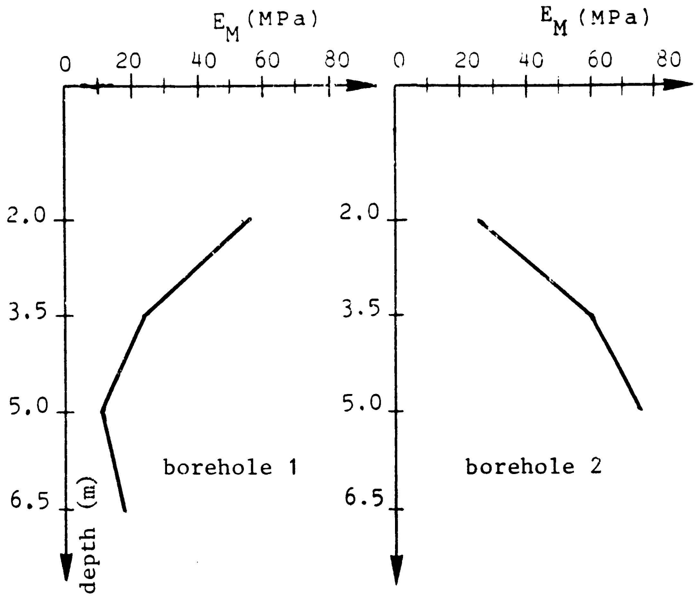


Fig. 9. Measured pressiometer profiles, after dynamic compaction treatment

representative of a relatively large soil mass. Field tests have given good agreement between the dynamic soil modulus from dynamic compaction and other geotechnical tests.

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