# VIBRATORY COMPACTION METHODS

# COMPACTAGE DES SOLS PAR VIBRATION

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SYNOPSIS : Loose granular deposits can be most suitably densified, up to great depths, by vibratory compaction techniques. Among those methods, vibroflotation and casing driving with soil replacement, and certainly vibratory probes (vibro-wings, star shaped probes) at constant or varying frequencies (resonant vibratory compaction) and applied shear strains larger than about 0,1 % up to 10 %, are the most wellknown procedures. In this paper aspecial emphasize will be put on the resonant compaction technique, being the latest development in vibrocompaction of granular material.

## INTRODUCTION

Loose granular deposits can be most suitably densified, up to great depths, by vibratory compaction techniques. Among those methods, vibroflotation and casing driving with soil replacement, and certainly vibratory probes (vibro-wings, star shaped probes) at constant or varying frequencies (resonant vibratory compaction) and applied shear strains larger thanabout 0.1 % up to 10 %, are the most wellknown procedures. Water jetting is normally used to facilitate the insertion of the compaction probe. When the maximum depth has been reached, water jetting is often reduced while the probe is slowly withdrawn. Compaction occurs as a result of lateral and torsional vibration while the probe is extracted. The soil is compacted mainly in a zone adjacent to the vibrating probe. Vibratory probes on the other hand use heavy vibrations clamped to the upper end of long steel probes, which can be either suspended from a crane or guided by a mast. The probe is excited in the vertical direction and the vibration energy is transmitted to the surrounding soil along the whole length of the probe. The soil is compacted mainly as a result of vertically polarized waves. Water jetting is normally not required, which makes the method simple to execute. Different types of compaction probes were developed in Japan, North America and Europe, Massarsch (1991). The geometric shapes of simple probes such as steel tubes or H-beams is not very efficient for soil compaction. Therefore, special probe shapes were developed for soil compaction.
In loose to medium dense saturated sands the strong ground vibrations result in a sudden increase of pore water pressure in a soil column surrounding the vibrating probe which can be considered leading to a state of cyclic mobility of the soil mass. Whenever the sand in its original density was loosely enough packed, so real liquefaction can even occur.

## VIBROCOMPACTION METHODS

In contrast to the vibroflotation - vibroreplacement technique, the vibro-compaction methods go out from the vibrator on top of the probe, delivering only vertical vibrations. Material can also be added, from the natural ground level on however, with the exception of vibrated or driven casings by means of which vibro- or dynamically compacted stone columns can be achieved.
Commonly used vibrocompaction systems are the Swedisch Vibrowing (Massarsch 1982), the Franki driven casing and the Franki Tristar (fig.1)

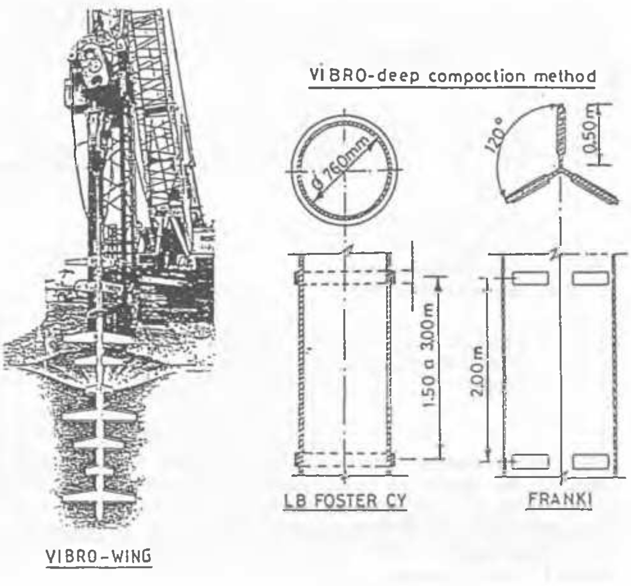
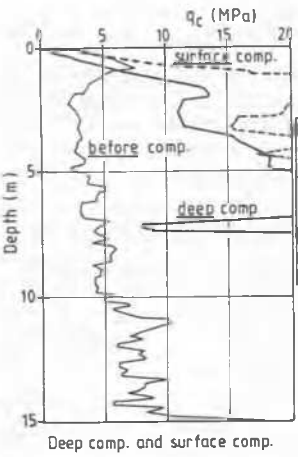



Fig. 1

The VIBROWING consist of a long (about 15 m) steel rod to which are attached 0.8 m long wings at 0.5 m spacing. A heavy vibrator (about 7 t in weight) fixed on the top of the needle, vibrates typically at a frequency of about 20 Hz (Massarsch, 1982). On the other hand, the Franki TRISTAR probe has three long steel plates 500 mm wide and 20 mm thick which are attached to a long steel rod 15 m - 20 m long, at 120° to each other. Additional steel ribs 300 mm x 50 mm x 10 mm are welded on to the two sides of each plate at 2 m intervals in order to improve further the efficiency of the probe. A motor driven vibrator mounted on top of the probe delivers vertical vibrations with frequency in the range 5 to 20 Hz.

At a site, the degree of improvement that can be achieved for a given soil, depends mainly on the duration of vibrations, the frequency, and the rate of withdrawal of the probe, the spacing between the points of insertion, and the fines content (permeability) of the original deposit. The efficacy of treatment is monitored by CPT carried out before and after densification, measurement of porewater pressures set up, the settlements, and the overall ground subsidence, etc.

Both vibro-compaction and vibroreplacement improve the subsurface granular soils through densification (Fig. 2). Vibro-replacement, though, results in a composite foundation system with stiffer elements that concentrate loads and provide drainage. This redundancy in dealing with the seismic problem gives vibro-replacement distinct advantages over other methods that only provide drainage or densification as means for mitigating liquefaction.




| Depth (m) | Shear Wave Velocity $C_s$ (m/s) before | Shear Wave Velocity $C_s$ (m/s) after | Max. Shear Modulus $G_{max}$ (MPa) before | Max. Shear Modulus $G_{max}$ (MPa) after |
|---|---|---|---|---|
| 1 | 80 | 125 | 13 | 32 |
| 2 | 90 | 128 | 16 | 33 |
| 3 | 94 | 149 | 18 | 44 |
| 4 | 110 | 165 | 24 | 54 |
| 5 | 111 | 166 | 25 | 55 |
| 6 | 133 | 171 | 35 | 59 |

Max. shear modulus $G_{max}$ determined from shear wave velocities before and after compaction

**Fig. 2** Static Cone Tests before and after VIBROWING Compaction (after Massarsch, 1986)

The improvement that results from vibro-replacement so can be analyzed, according to R.A. López and F. Hayden - 1992 in three ways : (1) improvement in surrounding material properties, (2) drainage, and (3) reinforcement of the overall soil mass.

Response of loose, granular soils to the vibrating probe results in increased relative density, shear strength, and stiffness, and reduced compressibility ( Fig. 2). In terms of soil parameters, this is generally reflected as increases in the angle of internal friction, $\varnothing$ ; shear modulus, G ; and elastic modulus, E ; and decreases in compression index, $C_c$. Furthermore soils improved by vibro-compaction and vibro-replacement have shown increased resistance to liquefaction.

The tendency of a soil to generate excess pore pressure during undrained loading is correspondent to the volume changes that occur during drained loading. Loose soils tend to contract upon shearing ; and, if loading is too quick for drainage to occur, generate excess pore pressures. For soils that derive all strength from confinement, this generation of excess pore pressures can lead to a condition of zero effective stress (when $\sigma = u + \delta u$), resulting in loss of strength and fluid-like behavior with only residual resistance to deformation.

## RESONANT COMPACTION METHOD

A promising adaption of the vibro-compaction technique is the resonant compaction system, (Massarsch 1991). As specially designed compaction probe, achieves an efficient transfer of vibration energy from the vibrator to the surrounding ground by a compaction probe with low dynamic stiffness (impedance). A heavy vibrator (centrifugal force of up to 4000 kN) with variable operating frequency, is attached to the top end of the probe. The probe is vibrated in the vertical direction only. After probe insertion, the frequency of the vibrator is adjusted to the resonance frequency of the soil layer, thereby amplifying the ground response. An important advantage of resonance compaction, compared to other vibratory methods, is that the whole soil layer oscillates simultaneously during compaction. Because of the special design of the probe and the possibility to adjust the compaction frequency, an optimal transfer of vibration energy to the surrounding soil can be achieved, resulting in a more efficient compaction process.

The capacity of the vibrator must be chosen with respect to the specific project requirements, such as soil type, initial soil density, required degree of compaction and penetration depth. The vibration amplitude required to compact the soil can be determined from a semi-empirical relationship between initial cone penetration resistance, vertical ground acceleration and soil layer depth, Fig. 3.

The resonance frequency of a soil layer can be difficult to predict theoretically but is relatively simple to measure directly on site by seismic measurements. The ground response during the switching-on of the vibrator is measured by velocity transducers at a distance form the compaction probe. The equivalent frequency spectrum (Fig. 4) indicates that resonance of the soil layer occured for example at 10,2 Hz. It can be deducted that at resonance, the vibration amplitude is strongly amplified and approximately 75 % higher than at the highest operating frequency. Frequency analyses of

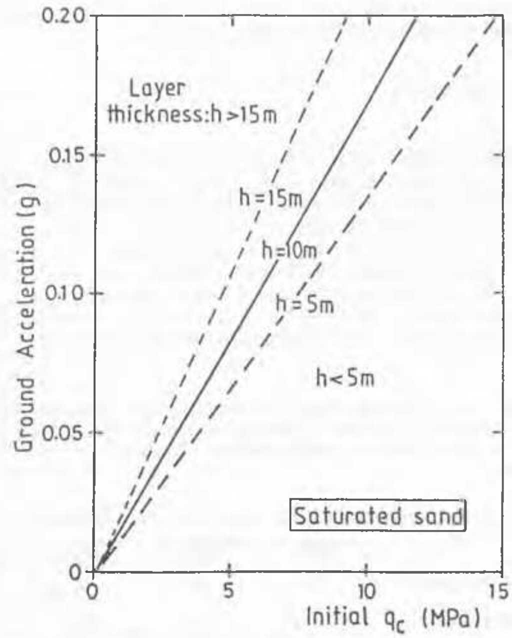



**Fig. 3** Required ground acceleration for vibratory densification of saturated sand as a function of initial soil density (CPT) and soil layer depth (Massarsch, 1991)

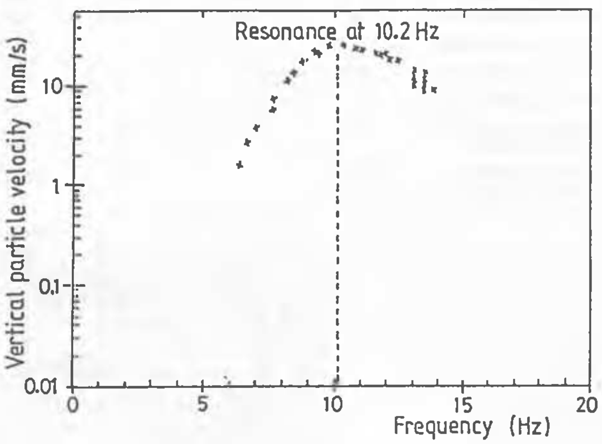



**Fig. 4** Example of the determination of resonance frequency from spectral analysis (Massarsch 1991)

ground response at the beginning of a project often show several higher vibration modes, suggesting that soil layers of varying stiffness exist. With progressing compaction, the resonance frequency increases and higher vibration modes tend to disappear, indicating more homogeneous soil conditions. The resonance frequency can be readily determined at any stage of soil compaction, and makes it possible to adapt the vibrator frequency to the optimal operating conditions.

As in the case of heavy tamping and blasting of cohesionless material, the final higher degree of relative density guarantees a more dilative deformation behavior, which implies a much higher resistance to liquefaction, since one mostly has to deal in such dense cohesionless soils with the phenomenon of cyclic mobility (Castro 1976, Van Impe 1982. As reported by R.A. López et al - 1992, Mitchell et al, 1976 studied the effects of pre-straining and soil fabric on liquefaction potential. His research determined that soils prepared to the same relative density do not necessarily exhibit similar undrained cyclic behavior. For example, soils densified by vibratory procedures produced no preferred axis of particle orientation, and exhibited better static and cyclic performance than soils prepared to the same density by either pluviation or tamping. In a similar investigation, Tokimatsu et al, 1986, found that sand that had been previously strained exhibited higher resistance to liquefaction. Yoshimi and Tokimatsu, 1991, refer to this resistance to liquefaction as ductility.
Under present practice, the liquefaction potential of mechanically improved sites is typically evaluated through the use of in situ tests, particulary the Standard Penetration Test, SPT, and Cone Penetration Test, CPT. When CPT is used, the data is either converted to equivalent SPT values, or used directly to assess liquefaction using CPT-based evaluation methods (eg, Robertson and Campanella, 1985). One should remember, though, that these correlations were developed from natural sites where no ground improvement had been performed. When soils are subjected to mechanical modification, variables associated with pre-straining, such as horizontal stresses and time effects (aging), may not be adequately represented by SPT and CPT results.
Measurements of pore pressure ratios with a piezo-cone with the porous element behind the tip show loose untreated soils generating high excess pore pressures during driving. Well treated soils, on the other hand, exhibit pressures below hydrostatic and even negative, indicating a tendency for the soil to dilate during shear. It illustrates the great importance of using the CPTU evaluation tests instead of CPT in such soil improvement analysis.

The applicability of vibratory compaction methods can be related to the soil type by means of the granulometric curve (Fig. 5a) (Mitchell

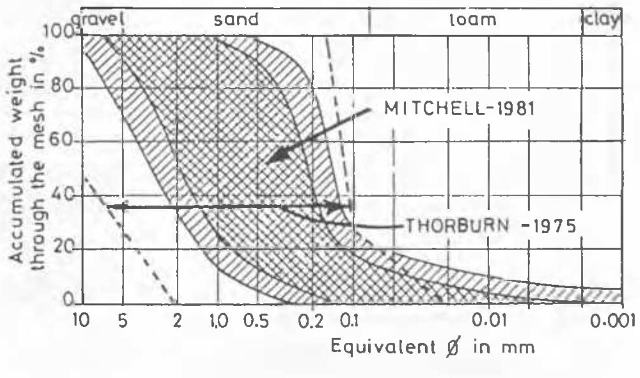



**Fig. 5a**

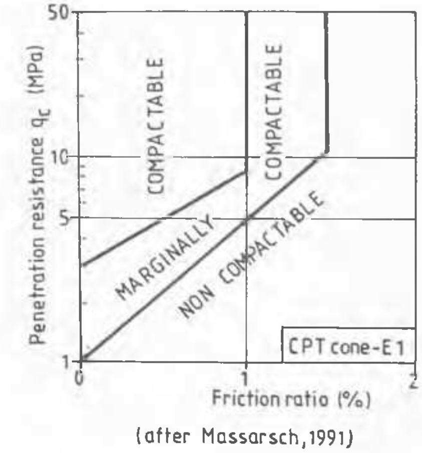



**Fig. 5b** Compactability of soils for vibratory compaction, based on electric CPT with friction sleeve measurements (Massarsch 1991)

- 1981, Thorburn 1975), to the cone resistance and corresponding friction ratio of the soil (Massarsch 1991) (Fig. 5b), and even to the dilatometerresults. Material index $I_D$ - values out of DMT varying between 1.5 and 4 should be required with respect to the vibrocompaction applicability.
Pore pressure measurements performed in connection with cone penetration tests - CPTU - can provide additional information concerning soil stratification and the existence of even thin, fine-grained layers. Soil with excess pore water pressures higher than about 10 % are often not suitable for vibratory compaction.
It is also important to establish the level and variation of the ground water in connection with a soil compaction project. Usually, dry soils or soil layers with negative pore water pressure are more difficult to densify than saturated soils and need to be identified carefully. The effect of thin impermeable seams in a soil deposit can be evaluated by measuring the permeability in situ. Soils suitable for vibratory compaction should have a permeability higher than approximately $10^{-6}$m/s.

## CASE HISTORIES

Several case histories are available. In the tabel I a and b and fig. 6 ; 7 and 8, only some of those are gathered. The interesting case of the soil improvement at Antwerp - Belgium will be in much more detail described lateron. In this case indeed, a total volume of about 100,000 $m^3$ of silty sand was improved by various methods (cfr table I a and b) controlled by CPT, CPTU and DMT before and afterwards.

Table I a : Comparison of degree of improvement by vibratory probes, casing driving with replacement and gravel replacement compaction techniques

| | Vibrowing | | Tristar | |
|---|---|---|---|---|
| Site | Rostoch (a) Germany | Zeebrugge (b) Belgium | Hamburg (c) Germany | Dordrecht Netherlands |
| - Utilized Frequencies (Hz) | 20 | 20 | 20 | 20 |
| - Depth of Treat.(m) | 10-15 | 0-7 | 11-13 | 14-19 |
| - $d_{50}$ (mm) | 0.2 | 0.3-0.35 | 0.2 | 0.3-0.65 |
| - friction ratio $R_f = f_s/q_c$ % | - | - | - | 1 |
| - % fines (60 µm) | 0 | 0 | 0 | 2-4 |
| - Grading $d_{60}/d_{10}$ | 2.5 | 1.7 | - | 1.6-3.5 |
| - Introduced mean gravel volume during replacement (t/m) | 0 | 0 | 0 | 0 |
| - Spacing of probe (m) | 2.5 | 3.0-3.7 | 2.75 | 2.2 |
| - Av . $q_c$ before (MPa) | 5 | 5.35 | 10.25 | 10.5 |
| - Av . $E_{DMT}$ before (bar) | - | - | - | - |
| - Av . $q_c$ after (MPa) | 20 | 16-21 | 24 | 22 |
| - Av . $E_{DMT}$ after (bar) | - | - | - | - |
| - Degree of improvement $q_c$ after/$q_c$ before | ± 4 | 3-3.9 | 2.3 | 2.1 |
| - Ground subsidence around the probe (m) | 0.55 | - | 0.42 | 0.6-0.8 |

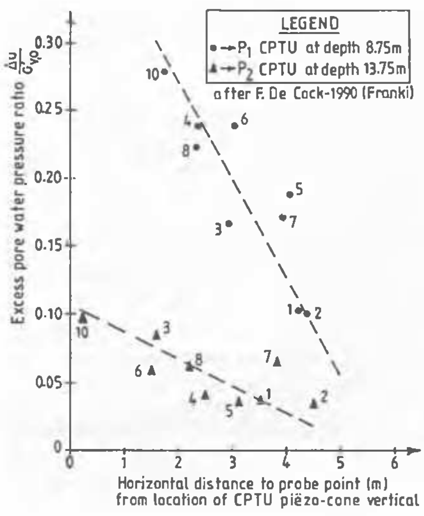



Fig. 6a

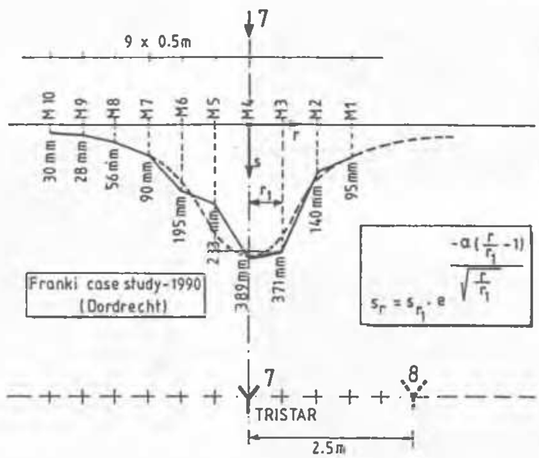



Fig. 6b

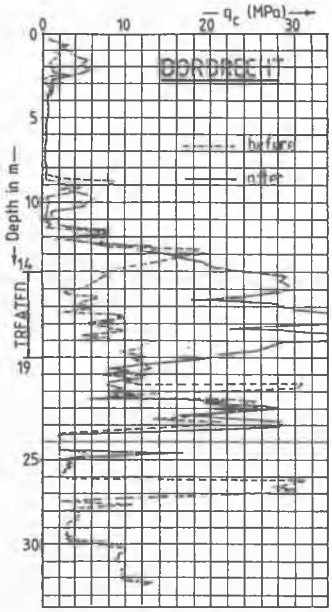



Fig. 6c

Table 1 B

| Site | Tristar | Vibroflot+ Replacement | Casing Driving + stone columns | Resonant Compaction |
|---|---|---|---|---|
| | location C30 to C38 | | | C11 - C13 |
| | Antwerp - Belgium (d) | | | |
| - Utilized Frequencies (Hz) | 20 | | | 15-24 |
| - Depth of Treat.(m) | 0-8 | 0-16 | 0-23 | 0-9 |
| - $d_{50}$ (mm) | 0.2 | 0.2 | 0.2 | 0.2 |
| - Friction ratio $w_f = f_s/q_c$ % | 0.3-0.9 | 0.6-1.0 | 0.7-1.0 | 1.1 |
| - % fines (60 μm) | 0 | 0 | 0 | 0 |
| - Grading $d_{60}/d_{10}$ | 1.6 | 1.6 | 1.6 | 1.6 |
| - Introduced mean gravel volume during replacement (t/m) | 0 | 0.8-1.25 | 0.6-0.8 | 0 |
| - Spacing of probe (m) | 2.5 | 2.5 | 2.5 | 2.5 |
| - Av . $q_c$ before (MPa) | 5.4 | 6.1 | 5.1 | 3.0 |
| - Av . $E_{DMT}$ before (bar) | - | - | - | 100 |
| - Av . $q_c$ after (MPa) | 9.4 | 16.2 | 14.0 | 13.5 |
| - Av . $E_{DMT}$ after (bar) | - | - | - | 800 |
| - Degree of improvement $q_c$ after/$q_c$ before | 1.75 | 2.7 | 2.7 | 4.5 |
| - Ground subsidence around the probe (m) | - | - | - | - |

(a) Massarsch and Broms (1983)
(b) Massarsch (1985)
(c) Franki (1986)
(d) Van Impe (1982)

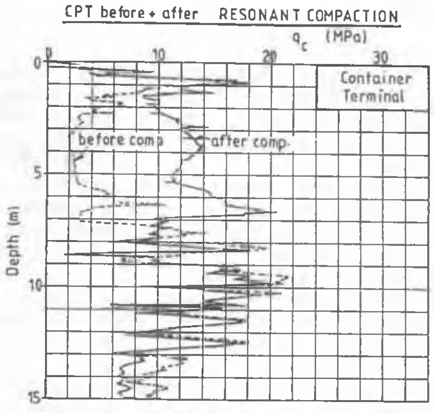



Fig. 8a

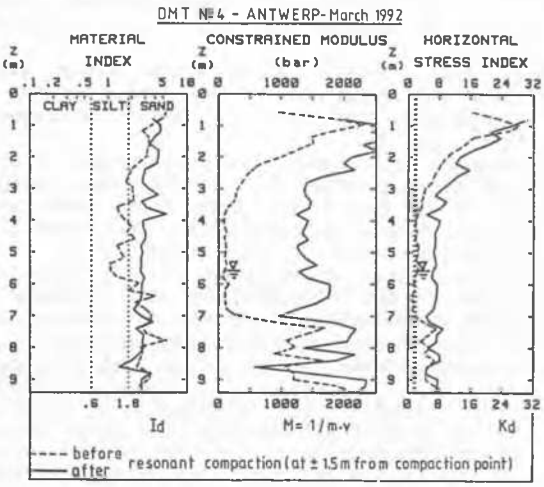



Fig. 8b

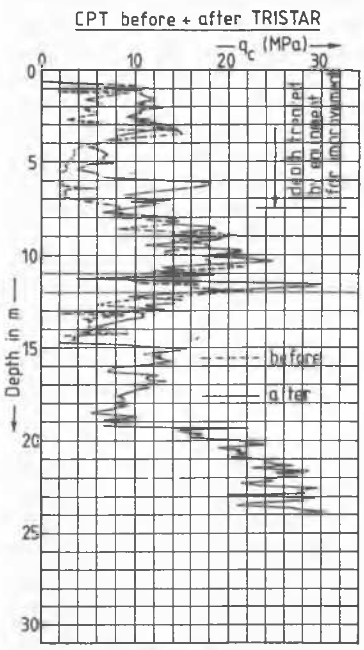



Fig. 7a

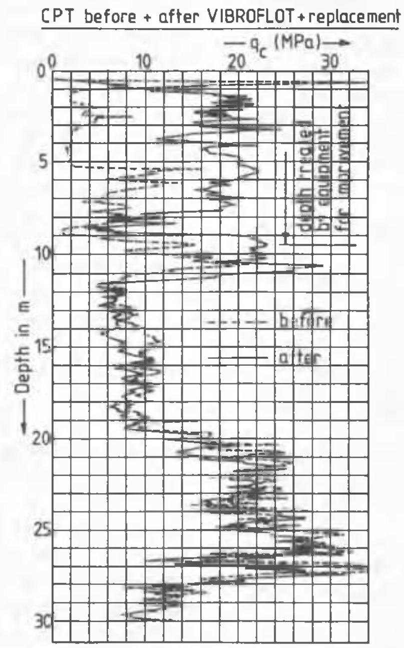



Fig. 7b

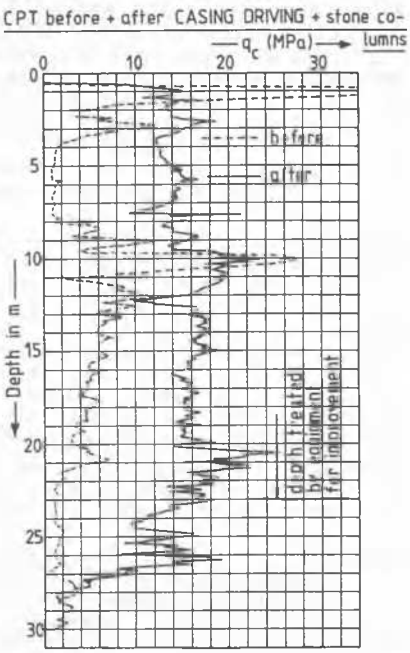



Fig. 7c

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