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SYNOPSIS Dynamic compaction resulted to be the most effective method to compact a very loose hydraulic silty sand fill, with a fine content from 25 to 90 per cent. Two levels of compaction energy were tested. The soil settlements enforced by these energies were verified with reference to the concept of saturation energy and by means of a simplified analysis of the physical model.

## INTRODUCTION

The construction of the container terminal of the port of Voltri, about 20 kilometers West of Genoa, Italy (Figure 1), is under way on a reclamation area that will have an extension of about 600 acres at the end of works.

This paper is related to the first portion of the area, with an extension of approximately 300 acres, completed in 1990. This portion was obtained partially with dumped rubble and mainly by means of hydraulic fills, using soils dredged from adjacent areas of the port of Genoa. The volume of the fills placed before 1990 was of approximately 9 millions cubic meters, out of a total of more than 20 millions cubic meters.

The construction of the terminal is developed by the Consorzio Autonomo del Porto (CAP) of Genoa. The terminal will be operated by a group of companies (VTE) including CAP, Fiat Group and others. The General Contractor in charge of Construction is Impresa Pizzarotti & C., of Parma (Italy). The site investigations were developed by Sorige (Parma) and the civil works design of the terminal, including geotechnical design, was provided by D'Appolonia S.p.A..

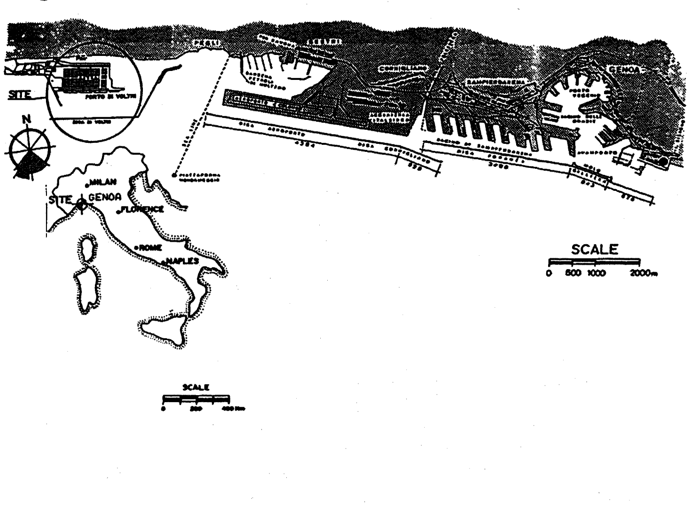


Fig. 1. Site

## PLANT LAYOUT AND FEATURES

The main issue of the foundation design of the subject plant, beside the design of the breakwater, was the design of the soil improvement measures required for the pavements of the containers storage areas. The containers will be handled in these areas by a number of gantry bridge cranes (Rubber Wheels Transtainers). These cranes have a 35 tons capacity and a total weight of 180 tons. A partial planimetric view of the terminal is presented in Figure 2. The main plant equipment is shown in the section of Figure 3. The terminal presently under construction will have the capability of handling 400,000 teus (standard container size) per year, out of a final total of 800,000 teus per year.

The incoming containers will be unloaded at the south wharf (Figure 2) by means of four 60 tons cranes (Portainers). The containers will be delivered via railway by means of two transtainer gantry bridge cranes on rails.

The runway of both portainers and transtainers on rails are founded on drilled piers, respectively of 1200 and 800 millimeters of diameter.

The yard gantry cranes on rubber wheels will be operated directly on the pavements between the storage areas. The maximum load for the transtainer wheels is 35 tons, to be transmitted to the foundation soils through a multilayer pavement structure, with bituminous and rollcrete layers. The rubber wheel cranes can be moved to parallel aisles by means of transversal aisles and of a 90 degrees rotation of the rubber wheels. During transfer operation the maximum design load per wheel is 20 tons.

## FILL PLACEMENT

The first fills were placed from 1985 to 1986 by means of bottom dumping of dredged soils from ships, for a total volume of 1.5 millions of cubic meters. The initial sea-bottom was at depths ranging from 5 to 15 meters. Two lateral dikes at the North and at the South (Figure 2) were formed by dumping soils from trucks from 1987 to 1988. Between the two dikes the fill was extended by dumping from trucks, at the east side, and from ships, at the west side, with a volume of dredged soils of 1.2 millions of cubic meters. The ship operations were extended through 1990, up to a depth of five meters below

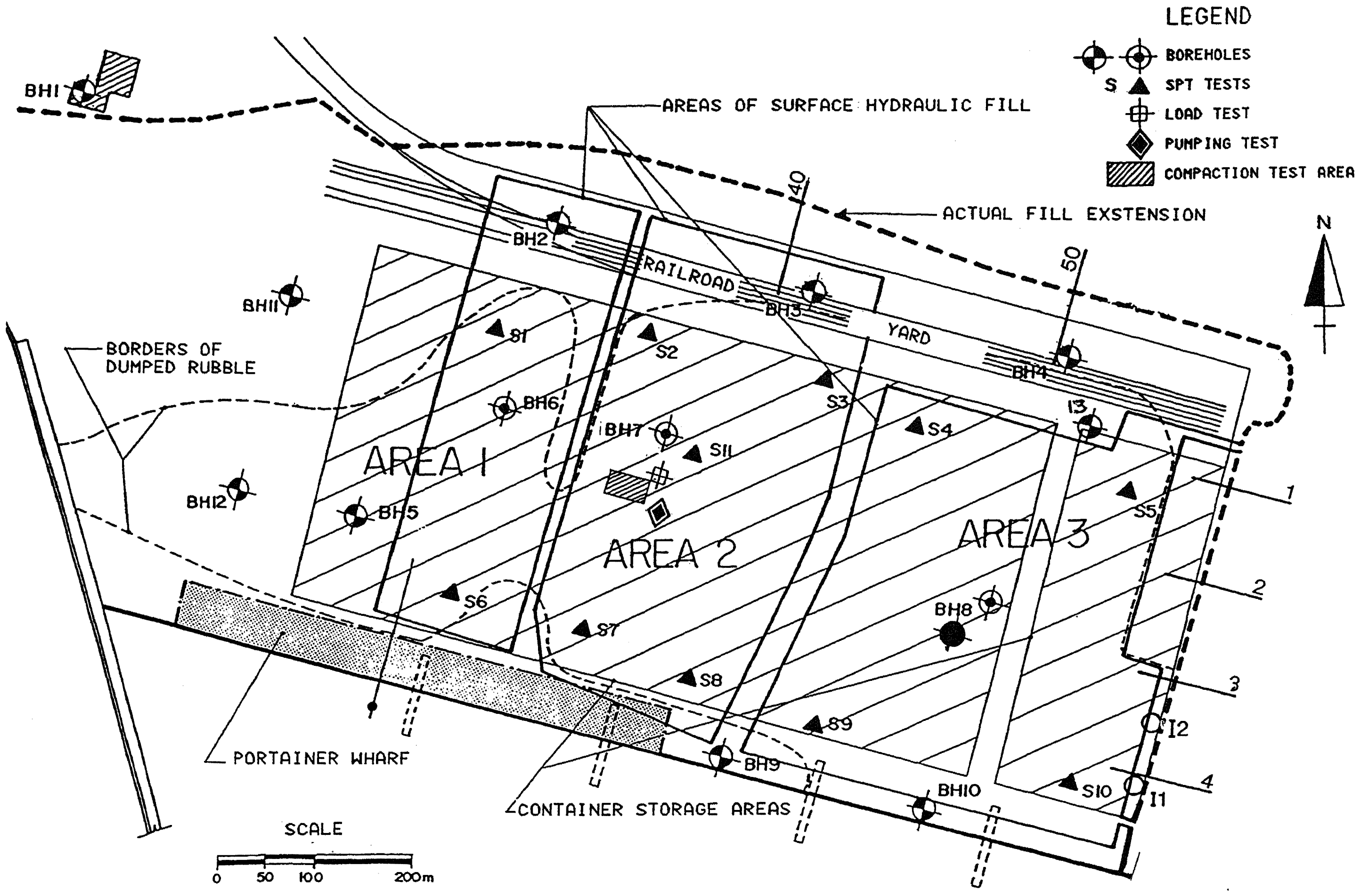


Fig. 2. Fill and soil investigation layout

the sea level. From this depth to a minimum depth of two meters the fill of dredged soils was placed by a floating pipeline, at a rate of 2.0 to 2.5 cubic meters per second.

Finally, from EL. -2.0 to about EL. +2.0 the hydraulic fills were placed by means of pipelines operated at various surface locations. Before these activities, the previous fill surface was divided in smaller areas by means of rubble dikes dumped from trucks.

The soil investigation described in the following chapter was carried out on the fill completed in August 1990.

## SITE CONDITIONS AND FILL PROPERTIES

### Soil Investigation

The properties of the fill and of the soils underneath were investigated by means of continuous sampling and SPT boreholes, surface density measures, pressuremeter tests (Menard), a full scale load test (Figure 4) and permeability tests. The laboratory testing has included the evaluation of minimum and maximum densities and of the modified Proctor (AASHTO) density for the hydraulic fill materials.

The sea soils of the area, on which the fill is founded, are medium dense to dense silty sands, based on a bedrock located at depths ranging from less than ten meters, close to the North-West corner, to more than 30 meters, close to the South-East corner. The soil profile of the fill area includes four main formations:

- Formation A: hydraulic fills, with dumped soils ($A_1$) and soils placed by pipelines ($A_2$);
- Formation B: rubble, dumped by land operated trucks;
- Formation C: sea-bottom silty sands, and
- Formation D: bedrock, formed mainly by shales, sandstone and conglomerates.

The loosest of these formations was the pipeline placed portion of hydraulic fills. The relative densities and AASHTO densities measured at the surface (Table 1) are as low as 7 per cent and 62 per cent, respectively. Good correlations between the two density values were possible, although the relative density concept should not be extended to high fine content sands. Similar densities were measured from relatively undisturbed borehole samples.

CPT tests have also indicated very loose fill conditions up to a depth of 10 meters from the ground surface, with typical values less than 1.0 MPa. The tip resistances of the ship dumped fills are higher, in the range of two to four MPa. The site silty sands below have tip resistances in the range of four to six MPa.

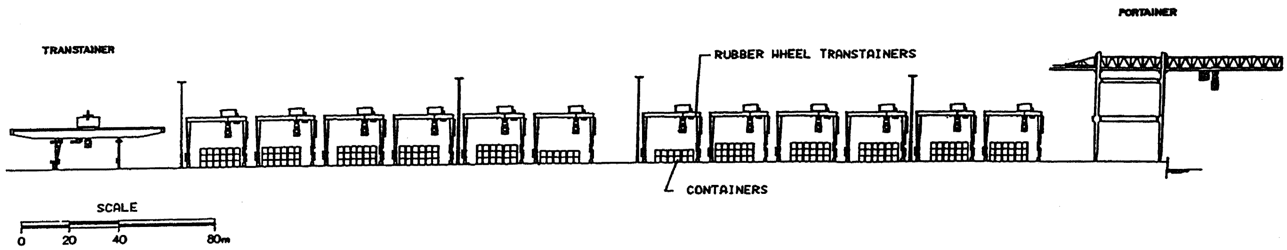


Fig. 3. Terminal section. E-W view

The fine content (passing the 200 ASTM sieve) of the granular formations is ranging from 25 to 90 percent (Formation A) and from 25 to 55 percent (Formation C). The sandy fraction is mostly fine. Formation B (Rubble) has a more distributed grain size, with cobbles and gravel and a fine content ranging from 10 to 35 percent.

Load Test

A real scale load test was developed by means of a soil fill of six meters of height as indicated in Figure 4, to evaluate the as-built soil deformation moduli of the hydraulic fill. The foundation soil settlements were measured by means of steel plates placed at the base of the fill. The maximum measured settlement was more than 50 centimeters. The test has indicated a modulus of 0.8 MPa for the Formation $A_1$.

Soil Permeability

The permeability of hydraulic fills was evaluated by means of a number of in-hole tests, and of a pumping test.

The in-hole permeability tests have generally indicated higher values than the mass permeability shown by the pumping tests. The in-hole tests have indicated values locally higher than $10^{-3}$ cm/s. The pumping test values are close to $10^{-4}$ cm/s. In consideration of the fine content (about 50 per cent) of the hydraulic fills in the pumping test area, the permeability values disclosed by the pumping test are more representative of the hydraulic fill conditions.

AS-BUILT GEOTECHNICAL EVALUATION

The generally high content of fines of the soils used for the hydraulic fills, is responsible of the loose to very loose as-built conditions, particularly for the fills placed by pipelines.

With reference to the low moduli values indicated by testing, large settlements were predicted for the pavements of both the storage yard and the runway of the rubber wheels transtainers. Bearing capacity problems were also anticipated for these last foundations, with reference to a pavement thickness of about 70 centimeters, including a mechanically stabilized base of 35 centimeters and a rollcrete layer of 20 centimeters of thickness.

Furthermore, due to the generally low

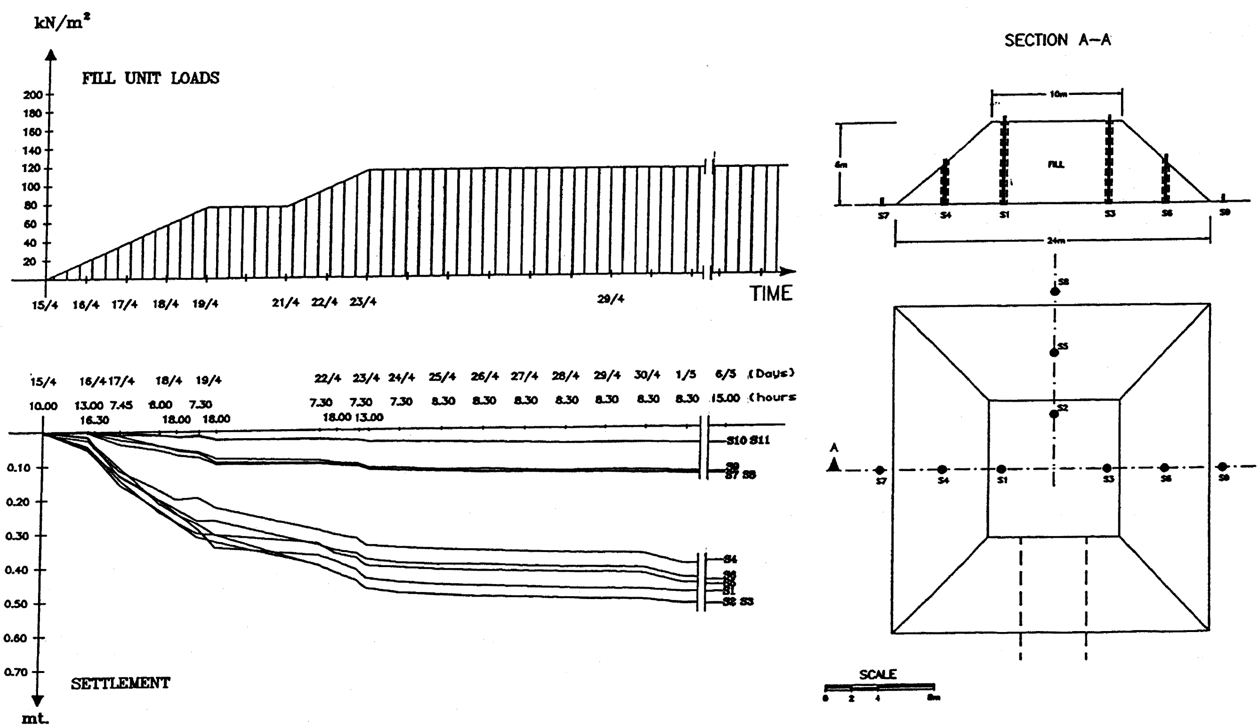


Fig. 4. Load test

permeability of the fills and to the poor drainage conditions of the site silty sands, the fills were expected to settle under their own weight for times exceeding the construction schedule.

The need of improvement measures was indicated therefore, and evaluated with reference to the summary of the as-built soil properties presented in Table I.

**TABLE I. As-built soil properties**

| Formation | $\gamma$ ($kN/m^3$) | $\varphi'$ | $S_u$ (1) | E (MPa) |
|---|---|---|---|---|
| A1 - A2 | 17 | 22 - 30 | 0.3 $\sigma'_v$ | 1 - 2 |
| B | 18 | 25 - 35 | -- | 2 - 5 |
| C | 19 | 28 - 35 | 0.5 $\sigma'_v$ | 5 - 15 |

Note:

(1) In case of undrained behavior

IMPROVEMENT MEASURES FEASIBILITY

A number of alternative improvement measures were evaluated, such as preloading, vibroreplacement, vibrowings, jetgrouting and dynamic compaction.

Preloading was obviously feasible, but involved large amount of soils. The required material was not available.

Vibroreplacement by means of vibroflotation also resulted to be feasible. This approach however would have been exceedingly costly, if extended to the full terminal area. Its consideration was only possible for the pavement foundations along the rubber wheel transtainer paths.

The use of vibrowings, developed and experienced in the last decade specifically for the improvement of hydraulic fills (Massarsh et al., 1984), was not suitable in this case, due to the soil high fine content, as pointed out also by the promoter of the method after a review of the local soil grain sizes.

Jetgrouting resulted to be feasible, particularly for the rubble fill areas. Its use was suitable for building and structural foundations.

The extensive use of this method in the container storage area resulted to be however more expensive than other methods.

The use of dynamic compaction was carefully evaluated with reference to the fine content of hydraulic fills. Previous experiences were developed by D'Appolonia on soils with a fine content up to 40 per cent (Varosio et al., 1983). Similar soil conditions were also reported in the specialized literature as suitable to the method (Mayne el al., 1983).

It was therefore concluded to proceed with a tamping test. A numerical model of the foundation pavements was used to define the minimum soil properties to be obtained by dynamic compaction.

TRANSTAINER FOUNDATION ANALYSIS

The numerical model was developed to evaluate the required foundation soil properties and pavement thickness at the base of the rubber wheels transtainer cranes in the container storage area. The analysis was initially performed with an uniform pavement thickness by means of Mindlin's solution for a point load.

The stiffer elements produced by the soil treatment (Stone Columns, Tamping Prints) were then modeled by means of the ANSYS finite element code.

Finally the elastic beam approach was used to verify the thicker pavement zones for the foundations of the transtainer wheels (Figure 5).

The final output of these analyses provided the required values for the soil moduli, as an input for the dynamic compaction test: eight MPa at the base of the Transtainer wheels, and five MPa for the pavement foundations in the container areas.

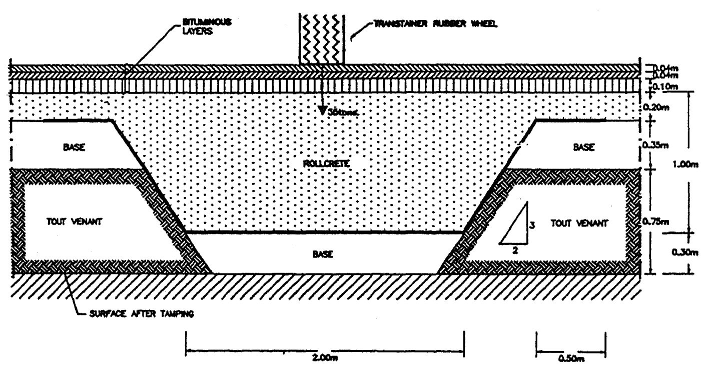


Fig. 5. Transtainer wheels foundations

DYNAMIC COMPACTION TEST

The test area is indicated in Figure 6, with the electropneumatic piezometers and CPT locations implemented for test monitoring. Only the 5 by 5 meters print grid was used in the test, with reference to the tamping results and to the indications of the analytical model.

The tamper weight and the drop height were set equal to 10 tons and 10 meters respectively.

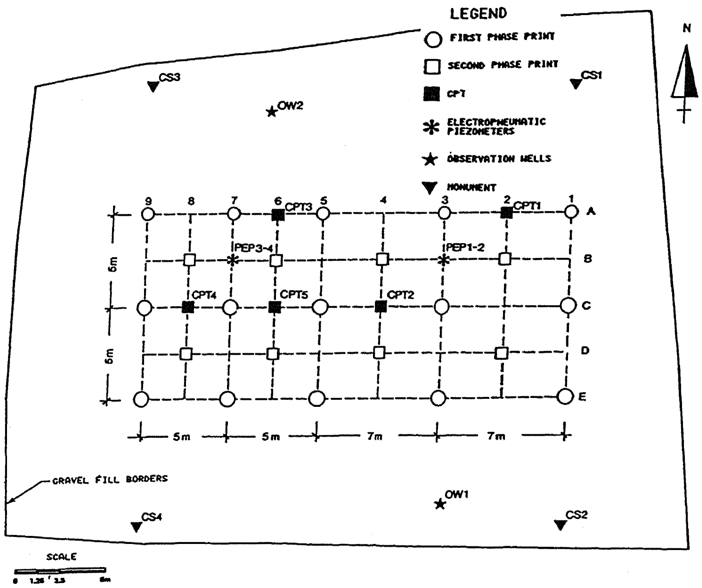


Fig. 6. Compaction test and instrumentation layout

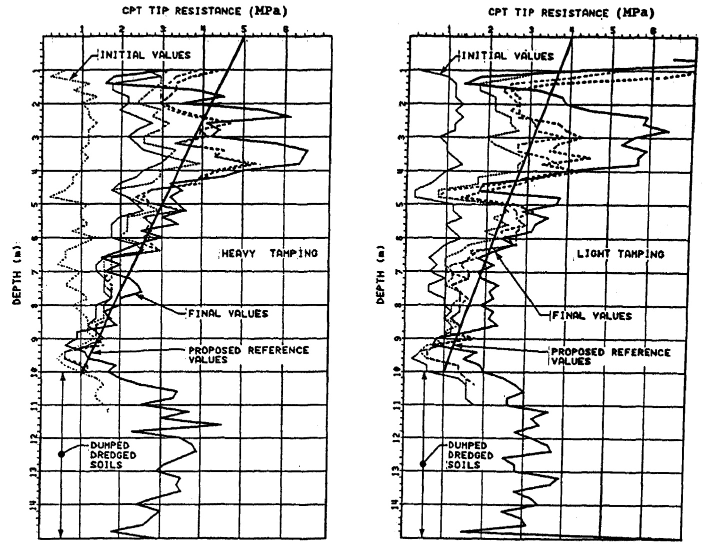


Fig. 7. Comparison of CPT tip resistances

Two alternative tamping energies were tested:

- o light tamping, with two passes of ten drops each, and a final ironing with ten drops over an area of 25 meters (120 tons per meter per square meter);
- o heavy tamping, with two passes of 30 drops each, and a final ironing with 10 drops over an area of 25 meters (280 tons per meter per square meter).

Cone Penetration and Pressuremeter Tests

The CPT tests performed to evaluate the tamping effects are presented in Figure 7. The reference average cone tip resistances are only slightly different for the two cases. The evaluation of gravel volumes used to fill the prints has however indicated settlements of about one meter induced by heavy tamping, which were approximately two times the settlements induced by light tamping. The CPT tests have also indicated that the tamping effects are felt down to a depth of ten meters, which approximately corresponded to the local thickness of the pipeline placed hydraulic fills.

The pressuremeter tests performed to evaluate the tamping effects have indicated the final moduli values shown in Table II. The low values of the ratio between moduli and tip cone resistances are also indicated in the table.

Pore Pressure Measurements

The electropneumatic piezometers have shown a quick dissipation of the pore pressures induced by tamping. The maximum induced excess pore pressures are lower than 1.5 meters (Figure 8). The water depth in the tamping area was at about 2.3 meters. No danger of soil liquefaction was indicated by the piezometric readings. No delays were hence predicted for the tamping program in order to avoid liquefaction.

Test Evaluation

The low vibration levels induced in the surrounding area particularly during the first tamper drops have suggested that most of the tamping energy was producing plastic soil strains.

The dynamic force induced by tamping was computed on the basis of the involved momentum, with reference to the impact velocity and to the penetration of the pounder. The computed dynamic force was equal to 2.85 MegaNewtons.

An estimate of the soil bearing capacity after the pounder penetration (of about 35 centimeters), with reference to the thickness of the gravel layer placed in the tamping area (70 centimeters) and to the soil properties of the hydraulic fills (Table I), has indicated a limit load of 2.75 MegaNewtons.

TABLE II. CPT and pressuremeter tests in the tamping area

| DEPTH (m) | TAMPING AREA 2.8 Drops/m$^2$ | | | TAMPING AREA 1.2 Drops/m$^2$ | | |
|---|---|---|---|---|---|---|
| | PRESSUR. MODULUS (MPa) | $q_c$ (1) (MPa) | E/qc | PRESSUR. MODULUS (MPa) | $q_c$ (1) (MPa) | E/qc |
| 1.5 | 3.9 | 3.4 | 1.16 | 3.4 | 2.8-3.6 | 0.94-1.20 |
| 3.0 | 6.3 | 4.4 | 1.43 | 4.0 | 4.4 | 0.90 |
| 4.5 | 2.5 | 2.6-2.8 | 0.90 | 1.7 | 1.4 | 1.23 |
| 6.0 | 3.5 | 2.6 | 1.35 | 4.6 | 2.8 | 1.63 |
| 8.0 | 1.1 | 1.6 | 0.80 | 0.9 | 1.4 | 0.65 |
| 10.0 | 2.8 | -- | -- | 1.2 | 1.4 | 0.90 |

Note:

(1) CPT Tip Resistance

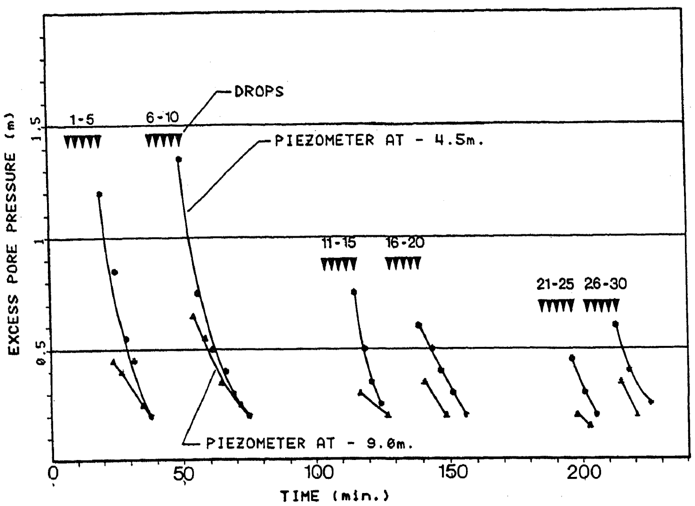


**Fig. 8. Pore pressure responses**

This simplified analysis of the physical model has confirmed therefore that, at least for the first drops, a complete soil failure was produced by tamping. After a number of drops and the subsequent print fills with gravel (Figure 9), the vibration level induced by tamping significantly increased with no apparent further soil overall plasticization.

The tamping effects were also evaluated with reference to the saturation energy concept proposed by Lo et al. (1990), who indicated hyperbolic correlations between the compaction energy and the induced soil properties. Meaningful increases of the enforced settlements could be predicted for the subject soils up to a total energy of about 300 tons per meter per square meter. Further tamping would only induce small additional settlements.

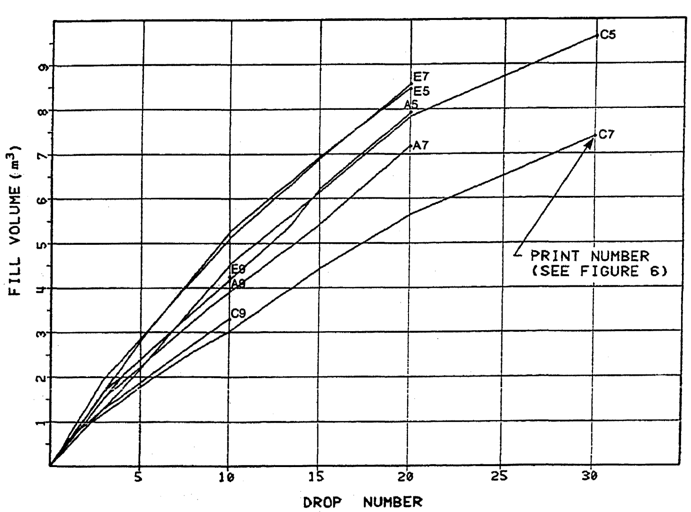


**Fig. 9. Fill volumes per print**

Lo also indicated that an energy increase of 2.3 times, from 120 to 280 tons per meter per square meter, could enforce total settlements that are approximately two times the settlement induced by the lower energy level, consistently with the compaction effects observed during the light and heavy tamping of the Voltri test.

## CONCLUSIONS

The test discussed in this paper has indicated that very loose hydraulic fills with a fine content of more than 50 per cent can be improved by dynamic compaction to a sufficient extent for the pavement foundations of a container storage area, on which heavy gantry cranes will operate.

The test results are on line with the recent unified approach for evaluating dynamic compaction, as proposed in the specialized literature. This approach can significantly simplify the planning of compaction tests, in general, indicating the energy range to be considered. For the time being, however, the method is still largely empirical and can be correctly evaluated only on the basis of carefully planned compaction tests.

## ACKNOWLEDGMENTS

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