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Dynamic Precompression Treatment - A Case History

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SYNOPSIS An unusual case history of a condominium apartment building, originally designed for eleven storeys, to which four additional floors were added after the footings had already been constructed and was successfully completed to fifteen storeys in height. The use of rather high soil-bearing values, from 7 ksf (350 kPa) in the original design to over 12 ksf (600 kPa). The project site, underlain by erratic soil profiles containing layers of soft fine-grained soils to about 20 ft (6 m) below the surface, had been effectively improved with an intense application of the Dynamic Precompression Treatment (DPT). A historical background of the DPT and extensive general and specific details of the implementation of this technique are presented together with selection of design parameters, results of conventional in-situ testing and non-conventional stress-strain tests for determination of soil compressibility moduli. Stress settlement analyses and settlement records are also provided.

INTRODUCTION

The project site of Ocean Village, an ocean-front community of condominium buildings within the city limits of Ft. Pierce, Florida, is underlain by compressible silty and organic soils to variable depths below the surface. In March 1977, the managers of this project decided to use shallow-type footings on corrected foundation soils for the construction of a first group of four 5-storey buildings.

The soil improvement technique adopted was the DPT, which resulted in such substantial reductions of cost that the managers extended their decision to use it in the construction of forty two buildings completed through 1981.

Three of the buildings were 11-storey towers with conventional spread footings designed for 7 ksf (300 kPa). The performance records of these three towers amply justified the recommendation given to the managers of using a higher design soil-bearing value for a proposed equal fourth 11-storey tower. However, there was an unusual rush in starting the construction of this fourth tower because of an impendent local moratorium on high-rises. Consequently, its construction was begun using the same construction plans as for the three previous ones.

When the construction of this fourth tower had reached the fourth floor level, the author was asked by the managers to assess the feasibility of increasing its number of storeys, following the previous recommendation of using a higher design soil-bearing value.

The approval by the lending institution and the City Building Officials of such an unusual building addition required extensive documentation by the author, which was based on existing soil records and actual behavior of the three previous buildings. An acceptable settlement prognosis for this heavier fourth building was established under the now much higher design values, some in excess of 12 ksf (600 kPa), without any modification whatsoever to the already constructed footings. However, the addition to this building required some structural modifications to the columns that had already been constructed below the fourth level, and redesign of the columns up to the roof and of the shear-wall system by the Project Structural Engineer, Raul Puig and Associates of Miami, Florida.

Fig. 1. Case History Building as of Sept. 1980.

Three independent groups of consulting engineers were called upon to review the proposed work before the addition was approved. Two firms: Reynolds, Smith and Hill, of Jacksonville, Fla. and McGlinchy-Pundt, of Miami, Fla.

were retained jointly by the owner and the lender. A third party, Victor J. Gerley, PE, of Jensen Beach, Fla. was retained by the local Building Department (after failing to engage two previous parties into this difficult commitment). Their favorable review made possible the erection of the 15-storey building. Mr. Gerley's detailed review coupled with his personal integrity were decisive at that stage for the issuance of the construction permit. Fig. 1 shows the case history building as of September 1980, when construction had been held up at the 11th storey level.

SOIL PROFILES

The project site is located in Hutchinson Island, 2 miles (3.2 km) south of the Ft. Pierce inlet, in an approximately 50 acre (20 ha) tract of land, limited on the east by the beach and the Atlantic Ocean. Hutchinson Island is a relatively narrow and low strip of land about ½ mile (0.8 km) wide, which runs parallel to, and about 3 miles (4.8 km) east of the main shoreline of the State of Florida.

A typical east-west vertical section across the island would show the following soil profiles:

East side: Natural beach underlain by a deep, loose to medium-dense deposit of fine silica sand intermixed with shelly calcareous sand, more shell fragments with increasing depth, becoming somewhat cemented below the 24 ft (7.3 m) level.

West Side: Low, marshy land adjacent to the Intracoastal Waterway (Indian River), covered by mangrove vegetation, underlain by layers of soft organic and silty soils interbedded with very loose, shelly fine sand to a depth of 16 to 18 ft (4.9 to 5.5 m). Below, the relatively dense, deep deposit of shelly sand is encountered again.

Intermediate sections: Man-placed sandy fill 2 to 4 ft (0.61 to 1.2 m) thick, underlain by erratically interbedded layers of soft organic and silty soils. The closer the location to the beach, the less erratic the profile, with a more favorable sand condition.

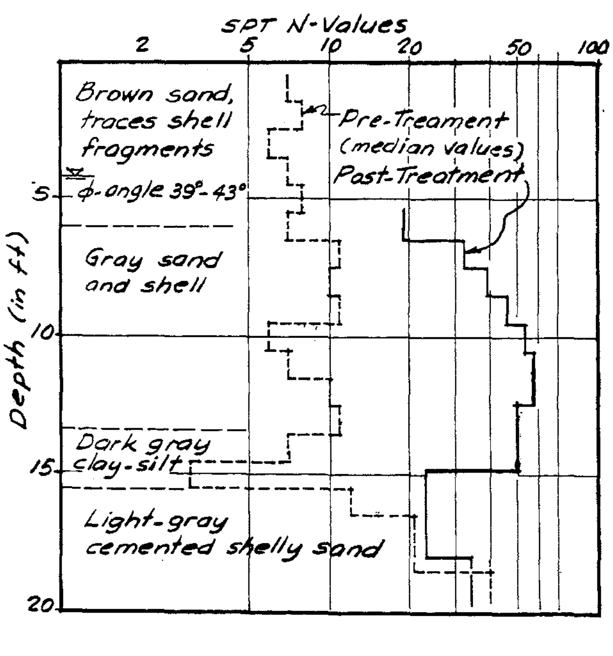


Fig. 2. Simplified Soil Profile.

The building under consideration is on the eastern side of the property, about 150 ft (45 m) from the ocean line. A simplified soil profile under this building is depicted in Fig. 2. Conditions were less erratic than at the locations of the other towers, although a layer of soft fine-grained soils was also present. Sieve analysis of these fine soils showed a fraction passing the US Sieve #200 ranging from 20% to 47%, while samples of the shelly sand had less than 10%. The sand had a mean particle size D_{50} , ranging from 0.3 mm to 1.5 mm. The water-table within the project site at any time almost coincided with the ocean level and was found 4 to 5 ft (1.2 to 1.5 m) below the ground surface.

DYNAMIC PRECOMPRESSION TREATMENT (DPT)

Historical Background

The DPT consists basically of making an effective use of the high-energy impacts that result from consecutively lifting and free-dropping a specific weight or pounder onto the ground surface, using a compatible self-propelled crane or untypically, a specially designed lifting device.

This technique was first applied in Germany in 1933, as reported by Wilhelm Loos (1936). The method was one of several included in an investigation carried out to determine the effectiveness of various compacting methods, in connection with the construction of then-new highways and other large projects.

Much more recently Menard (1972) reported the application of this method and became seriously interested in its development (1974, 1975) and its commercial application, to the extent that some engineers call it Menard's method.

The author has been independently associated with the implementation of the DPT since 1974, when he completed the first application in the USA in connection with two large 5-storey buildings constructed in North Ft. Myers, Florida. The original design called for 80-foot (24 m) piles which were successfully substituted by strip footings dimensioned for 4 ksf (200 kPa), with a dramatic net cost reduction on the order of 0.3 million dollars.

Since then, the author has been personally involved with the application of the DPT on more than 500 sites for a diversity of building types and miscellaneous structures, underlain by all sorts of unsuitable soil profiles except thick clay deposits. The projects have included commercial buildings, factories, shopping centers, multi-storey hotels, offices and apartment buildings; among them, a 21-storey tower presently being completed in the downtown area of West Palm Beach; and more than 50 warehouses constructed on waste disposal sites (dumps).

Other engineers have reported applications of the method in the USA: Leonards et al (1980), Lukas (1980), and Ramaswamy et al (1981).

Designation

The method has been named indiscriminately, Dynamic Compaction, Heavy Tamping, Dynamic Consolidation, High-Energy Impact, Pounding, Compaction Impact, Dynamic Impaction, Ground Bashing, etc. However, when the modifying actions to the treated foundation soils are taken into account, the most proper and comprehensive designation should be Dynamic Precompression. Though occurring simultaneously, these modifying actions can be related for simplicity, to three distinct mechanisms of soil improvement:

Densification: air or gas displacement.

Consolidation: water or fluid displacement.

Predeformation or Prestraining: structural rearrangement of constitutive particles, which may also result in actual modifications to the particles themselves.

The only term in soil mechanics that includes per se all those mechanisms is Precompression. Moreover, since it has been determined by experience that the most important effect on the treated soil is an extraordinary reduction in its compressibility, the designating name for the technique should convey an explicit meaning of such an improvement.

Practical Considerations

No theory has yet been published concerning this technique. The complexity of the product resulting from dynamic precompression is so extraordinary that the possibility of developing a general theory, valid for all practical purposes, is inconceivable. Perhaps, some acceptable theoretical frame may see the light for very specific cases of less complexity. An effort in this direction was made by Scott and Pearce (1975).

Based on his personal experience the author has developed several simple "Implementation Parameters" which provide some rational basis for the general application and control of the DPT. Some of these parameters are useful for a numerical determination of the expected extension and scope of the work, are readily adaptable to the specifications of the project, and serve to establish in advance a realistic cost estimate for the operation.

Following is a detailed list of these "Implementation Parameters" with symbols and notations defined when they first appear:

1. Available Energy $E = \alpha WH$
   α, efficiency coefficient to account for inertia losses, air compression and viscosity, shear friction between the sides of the pounder and the impinged soil, etc.

   W, weight of pounder.

   H, height of falling distance.

2. Average Dynamic Force Applied $F_i = \alpha WH_i/\Delta S_i$

   ΔS_i, impingement distance of the pounder into the ground for F_i.

3. Cumulative Average Dynamic Force Applied

   $$\Sigma F_i = \sum_1^n (\alpha WH_i/\Delta S_i)$$

   n, number of force applications or impact blows.

4. Intensity of Treatment $I = \Sigma F_i / \Sigma L_s$
   L_s, total design loads of the proposed structure. Based on the author's experience, practical applications limits are $100 < I < 1000$.

   Intensity of Treatment for a Footing Area

   $$I = (\Sigma F_i)_1 / L_1$$

   L_1, total design load on the footing.

5. Required 'n' to satisfy value of ΣF_i

   $$n = (\sum_1^n F_i)\Delta S_a/\alpha WH$$

   This is a conservative approach, since

   $$\Sigma F_i = \Sigma(\alpha WH_i/\Delta S_i) > (\alpha WHn/\Delta S_a)$$

   $\Delta S_a = \Sigma\Delta S/n$ and α, W, H, are constants.

6. Average DPT Soil Modulus $M_d = 2\alpha WH/B(\Delta S_a)^2$

   B, lateral dimension of pounder.

   M_d should be based on a value of ΔS_a resulting from a fixed predetermined value for $\Sigma\Delta S$ of not less, for example, than 3 ft (or about one meter). By correlating a range of values of M_d with measured values of the soil compressibility modulus M, it is possible to establish an approximate, but simple and very valuable relationship between M_d and minimum values of M. For example, for $\Sigma\Delta S$ = 3 ft, W = 17 kips, B = 5 ft, α= 0.9, the author has found that $M=M_d/n$ may be used as a conservative guideline.

7. Effective Depth of Treatment D_t

   The observed data related to this evasive parameter indicate that D_t is definitely dependent on the nature of the subsurface profile; and that the maximum attainable depth is a function of E, B, and the number of consecutive 'n'.

   Menard has suggested (1975) that the maximum D_t in meters is directly proportional to $E^{.5}$, for E in ton-m. The extensive experience of the author, using pounders of 7.7 and 16 metric tons, of B = 1.53 and 2.14 m respectively, and α= 0.9, is that for all ranges of energy, the maximum

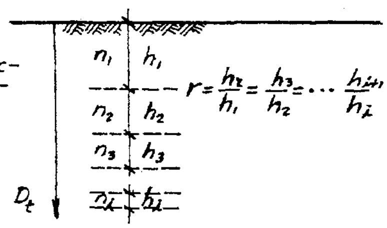
   $D_t = E^{.43}$(tons & m), or $E^{.47}$ (kips & ft).

Based on this experience, it also seems reasonable to anticipate that for all practical purposes, the maximum D_t will always be less than 6B. However, D_t may be much deeper, perhaps down to 10B when treating disposal sites (dumps, in the USA) formed of heterogeneous waste materials such as the product of building demolitions; car bodies and tires; domestic refuse like appliances, furniture, toys, etc.

It has been observed that consecutive equal-energy blows applied on the same impact point induce measurable soil improvements that tend to increase in depth following approximately a declining geometrical progression, viz., diminishing additional depth-intervals per blow. On this basis, the following empirical expression has been derived and may be used to determine the minimum 'n' required to attain D_t with a certain degree of improvement:

$D_t = \Sigma h_i$,

h_i, depth interval corresponding to 'n_i'.



$$D_t = h_1 \times \left[1 + \frac{r(r^n - 1)}{r - 1} \right] = h_1 R$$

But, $h_1 \approx 1.5$ B and the term in the brackets R tends to be an absolute maximum number when $n \to \infty$, thus $D_t \approx 1.5BR$

2

r	R	.99R	n
0.80	5.00	4.95	20
0.75	4.00	3.96	16
0.70	3.33	3.30	12
0.65	2.86	2.83	10
0.60	2.50	2.48	9
0.55	2.22	2.20	7
0.50	2.00	1.98	6

It is then possible to determine the minimum practical 'n' for 0.99R as given above to reach D_t with a certain degree of improvement, by just estimating 'r' from any h_{i+1}/h_i.

In general, a simple but very useful evaluation of the nature of the soil profile being treated can be made based on the type of shear-failure activated by the impingement of the pounder into the ground. Deeper treatments are always associated with a punching-shear failure; moderate effective depths with a local-type of shear-failure, while a general-type of shear-failure indicates a considerable reduction in the effective depth being attained with the treatment.

It is not unusual to observe modifications in the type of a shear failure during the application of the treatment, for example, from an initial punching-type failure to a local-type failure. These changes should be observed carefully because they reflect significantly not only on the effective depth of treatment, but also possibly more importantly, on the actual response of a soil profile to the treatment or the presence of profile variation.

8. Subsidence or Volumetric Reduction:

This is a very useful parameter for a simple but realistic evaluation of the DPT. Once an evaluation of the stress history is made for the extant conditions, specifically as related to a possible prestraining or overconsolidation effect from past loadings, a determination can then be made on the range of attainable volumetric reductions that may be expected for any specific practical purpose.

If the predominant characteristics of the original soil profiles are known to the practitioner, he should be in a position to correlate subsidences and type of failure with concomitant reductive soil compressibility, and on that basis, verify the actual precompression induced by the DPT. Furthermore, when other more elaborate means are not available or possible, the subsidence may enable the practitioner to ascertain or justify the suitability of the soil conditions after DPT for specific or predetermined design parameters.

The author has made extensive use of this resource in his professional practice. Actually, this parameter has been the most practical and meaningful one used in his evaluations of DPT's results on relatively deep waste disposal sites or 'dumps' where no standard testing technique is applicable.

9. Surficial Finishing Operation:

The DPT is a deep-type soil treatment; it is neither practical nor economical to attempt to improve the uppermost section of a profile (say down to 5 to 6 ft, or 1.5 to 1.8 m, below the surface) by an overextended application of low-energy blows. To complete a uniform prestraining operation within this depth interval, the most effective equipment is a heavy vibratory roller. Once the ground surface is leveled after the DPT, the roller should be applied uniformly over the entire area in successive overlapping passes, preferably in two perpendicular directions. The diesel-engine of the roller must be operated at full power, but the roller itself should be moved at a slow translational speed.

When the soil is predominantly granular the following simple and conservative empirical expressions, derived and used successfully over the last 12 years by the author, should serve as useful implementation parameters:

Minimum Cumulative Time-rate, Tr

$Tr = 0.08h(R/Ra)$ in minutes/sq yd

for $R \geqslant 2(h-1)$ and $h \geqslant 1.5$ ft h, thickness or depth to be treated in ft

R, total static weight in tons of required self-propelled vibratory roller, with 1.5R used for towed-type vibratory drums.

Ra, actual R of roller in tons, for Ra $\geqslant$ R. Equivalent expressions for metric tons and meters are:

Tr = 0.31 h(R/Ra) in minutes/sq m

for R $\geqslant$ 6(h-0.3) and h $\geqslant$ 0.45 m

The expression for Tr was deduced for self-propelled vibratory rollers of the present decade, using the following equipment parameters: total static weight of the equipment, total static weight transmitted by the drum, dynamic factor, amplitude and frequency of vibrations, and average contact area of drum based on its length and compatible width-prints. On this basis, the maximum output energy for the investigated range of parameters of vibratory rollers 1 to 15 tons in total static weight was calculated to be between 10.6 and 27.1 times the equivalent energy of the Standard Proctor expressed in tons - feet per square yard per foot of depth.

It should be noted that the modifying action on the sandy soils induced by a vibratory roller operation is a prestraining effect (creating a preloading or a stress-history), and not necessarily a densification effect. Experience and reliable experimental evidence published by Drnevich and Richart (1970) and Morgan and Gerrard (1971), clearly indicate that density is not a relevant parameter in relation to the stress-strain characteristics of sands once their fabrics have been effectively modified by vibrations or preloading. Consequently, the author's approach as briefly summarized above, emphasizes the procedural-type prescription instead of the end-result verification by density testing. As a corollary, he strongly recommends against the practice of total dependency on field-density tests, particularly in sands, where the results of such tests are irrelevant and may be seriously misleading.

Vibrations Control

When the DPT is to be applied on sites where there are existing buildings in the neighborhood, it is important to measure the vibrations being transmitted through the ground to determine the maximum range of values to be expected during the operation. This maximum range of vibrations must always be lower than certain predetermined criteria, selected either to reduce the annoyance to persons (the most frequent case) or to assure the non-possibility of actual damage (the most critical case).

In general, the impact forces set into motion several kinds of waves. In fact, the remaining energy after the abrupt mobilizations caused by the impact force is effectively dissipated through the underlying soil mass following bodies of compression and shear waves. Rayleigh waves and reflected Love waves, moving closer to the ground surface, are the most notorious to consider for the control of vibrations.

Vibration measurements are typically made on velocity, acceleration, or displacement, of particles. However, the particle velocity is the most common measure since it has been better correlated with the possibility of physical damage to the weakest of the building's components: the plaster. Numerous investigators have concluded that a particle velocity of 2 in/sec (5 cm/sec) can be taken as a safe threshold of damage, and some local ordinances or codes have established 50% of that value as the maximum vector-sum acceptable to reduce inconvenience to persons. Based on his experience, the author tries to maintain the level of vibrations below a vector-sum particle velocity V = 0.33 in/sec (0.8 cm/sec) when annoyance to persons is involved.

There are seismometers available with which the particle velocities are measured in a relatively simple operation. They may be just indicators, or registers; some give simultaneous readings along three perpendicular axes. In his practice, the author always conducts a seismographic survey at the beginning of the DPT operation, measuring the V at different distances from the point of impact along three or four different directions, while using the maximum energy available. With these data a curve specific for the site is prepared as shown in Fig. 3. Sometimes the V values must be taken in critical locations, particularly within multi-story buildings, where the readings may be much higher than at the ground level. As may be intuitively perceived, the looser or softer the soil-profile, the quicker the dampening of the ground vibrations with the distance from the point of impact. Typically, waste disposal sites show the largest reduction in the transmission of vibrations, followed by deep peat deposits. In general, for the strict criterion of 0.33 in/sec measured at the ground surface, the range of required minimum distance to existing buildings should be expected to fall between 80 ft (25 m) and 150 ft (45 m), for the more frequently used levels of energy.

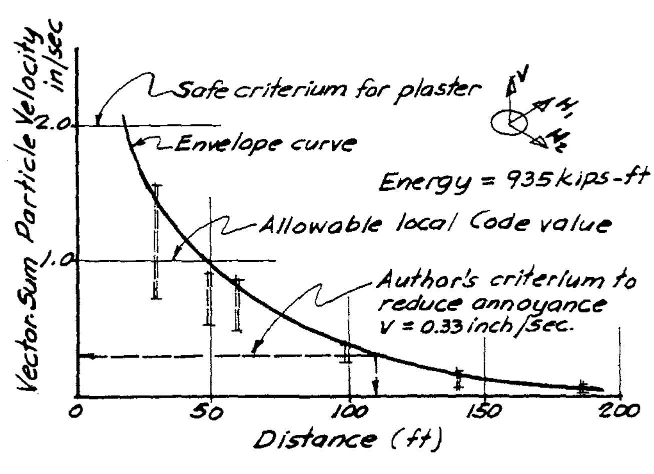


Fig. 3 Vector-Sum Particle Velocity vs Distance.

The excavation of an intercepting trench reaching the water table or as deep as it can be practicably made, excavated as close as possible to the existing building, is a very effective way to damp the transmission of vibrations, with reduction of up to 30% of the measured peak particle velocity.

DPT Application in This Case History

The DPT in this case history was programmed and implemented for the construction of an 11-story tower. Similar treatments had been applied previously in this same complex for three equal 11-story towers already constructed and occupied when the site preparation was started on this fourth tower in July, 1979.

The soil profiles underlying this new building site were actually less erratic, did not contain any peat, and had only one layer of unsuitable soft fine-grained soils. Consequently, the DPT operation had been limited to improve the soils under the footing areas, aiming at:

Correcting the typically extant erraticism in the sand deposit to assure a minimum degree of uniformity.

Reducing the compressibility of all participant soils in the profiles to permit the use of a safe design bearing value never below the previously recommended and proved value of 7 ksf (350 kPa).

The use of the implementation and control parameters as applied in this case are exemplified below for just one footing and only in English units for the sake of brevity:

- Footing No. 1, Type F-2, 6 ft x 6 ft
- Required D_t = 22 ft
- Required E to satisfy D_t

$$D_t = E^{.47} \therefore E = 723 \text{ kips-ft}$$

Since $E = \alpha WH$, selecting W = 17 kips and α= 0.9, $H = E/\alpha W$ = 47 ft, for which a minimum boom-length of 65 ft was required. A 70-ft boom was actually used, and the attainable

$$D_t = (\alpha WH)^{.47} = 24 \text{ ft} > 22 \text{ ft.}$$

- Required minimum 'n' to attain D_t = 22 ft (with certain minimum degree of improvement) for B = 5 ft and r = 0.70, 'n' is = 12 blows, giving D_t = 1.5BR = 24.7 ft > 22 ft.

To assure a satisfactory average degree of improvement to the required D_t, an intensity of treatment I = 400 had been conservatively established initially based on the results of the previous buildings. Since the initial design bearing value had been q = 7 ksf, then for Footing No. 1 the required (ΣF_i) was:

$$(\Sigma F_i) = I \times \text{Footing Area} \times q = 70{,}000 \text{ kips}$$

From this, a minimum conservative 'n' to be applied on the area of Footing No. 1 was calculated by trial and error, assuming different values of ΔS_a and checking for a compatible value of $\Sigma \Delta S = n\Delta S_a$.

In brief, assuming ΔS_a = 0.3 feet (conservative), and the use of the maximum energy available, the required

$$n = (\Sigma F_i)_1 \Delta S_a / \alpha WH = 25$$

During the application of the treatment the actual number of blows 'n' on each footing area was adjusted based on direct observations of the response of the soil. Sometimes, particularly for the first blows, 'H' was reduced as long as the mechanism of shear was evidently of the punching type. Some footing areas received less intensity of cumulative treatment than initially estimated as when, for example, the response was showing some obvious elastic behavior typical of a highly precompressed state in the sands.

Estimates of the range of the soil compressibility modulus being attained were made based on DPT moduli. For Footing No. 1, the established guideline indicated a range of values for M always in excess of 400 ksf, obtained by counting the applied 'n' required for a total impingement of 3 ft. For example, when 'n' was counted as being 15, for H =40 ft,

$$M_d = 2\alpha WH/B(\Delta S_a)^2 = 245/(\Delta S_a)^2$$

but $\Delta S_a = \Sigma \Delta S/n = 0.2$ ft

giving M_d = 6,125 kips-ft

and $M = M_d/n$ = 408 kip-ft

Most of the footing areas permitted the application of the treatment in a single phase, i.e., the prescribed or adjusted total 'n' was applied consecutively. However, a small group of footing areas had to receive the treatment in two phases after the first set of consecutive 'n' applied had developed an excess pore-water pressure condition which created a sudden inundation of the operating crater. Favorably, the dissipation of the pressure and the infiltration of the water back into the ground took only a few hours. The occurrence of this type of localized behavior was always associated with some silt content in the sand deposit.

The average net subsidence created by the DPT as measured during the application of the treatment in individual footing areas ranged from 3.5 ft to 5 ft. At the completion of the treatment the ground was leveled, forming a general depressed area (containing all the treated footings) with an average general subsidence of 1.6 ft. This surface was then uniformly prestrained with a 10-ton self-propelled vibratory roller, applied at a cumulative time-rate of ½ min/sq yd.

TESTING

In-situ testing should be a part of the essential program of field observations required for a successful application of the DPT. It furnishes the quantitative data for the verification and necessary judicious evaluations of the soil conditions pre- and posttreatment.

Generally, in Geotechnical Engineering practice the results of all soil testing or test parameters should be always the subject of interpretations and proper evaluations. Typically, there is no such thing as a clear-cut definite test result, but when the soil characteristics have been so intensively and extensively modified by the DPT the interpretative exercise becomes even more complex and difficult.

Design parameter, soil parameter, and test parameter, although inter-related very closely have not the same meaning. Moreover, none of the conventionally used in-situ tests: Standard Penetration (SPT), Cone (CPT), Vane (VST), and Pressuremeter (PMT) measures a soil parameter or models a specific foundation problem. It is now generally accepted that the successful application of such test parameters require pre-established empirical correlations between predicted and observed behavior of prototypes.

Early in his DPT practice, the author found that test values from SPT, CPT and VST do not reflect the extreme soil modifications (precompression effects: OCR, stress-anisotropy, fabric-anisotropy, stress-history) that are associated with the DPT. It was promptly realized that the only practical way for attempting to quantify the resulting soil modifications was by means of some sort of stress-strain measurements in-situ that would permit the determination of a soil modulus of compressibility, M, at different depths.

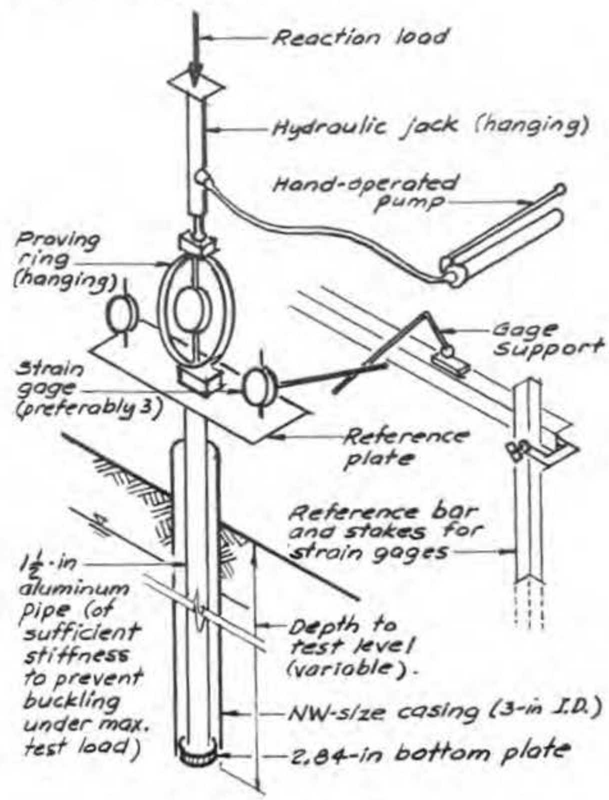


Fig. 4. Special LBT Features.

After an assessment of the equipment available for in-situ stress-strain testing (screw-plate and pressuremeter) with their intrinsic limitations and complexities coupled with their high cost, the author decided to use a rather unique testing technique which consists basically of a very small load bearing test (LBT) performed at preselected levels below the ground within lined boreholes carefully prepared for the purpose, and with the specific features shown in Fig. 4. Actual applications in the sites for the three previous towers are shown on Fig. 5.

(a) Side View

(b) Front View

(c) Aluminum Pipe and Plates for 20-ft Testing Depth. Note Crane and Pounder in Background.

Fig. 5. LBT Set-Up for Ocean Village Towers.

The most significant aspect of this type of LBT is the high precision for both the stress applications and the strain measurements. Stresses are applied with a hydraulic jack through a proving ring, giving a precision of ±0.4 lb/0.0001 inch (±1.8 N/2.5μ), and the strains are measured to ∓0.001 inch (±25μ). The combined weight of the aluminum pipe and plates is small and the initial overburden stress may be easily re-established while setting up the test by adding sufficient water inside the pipe.

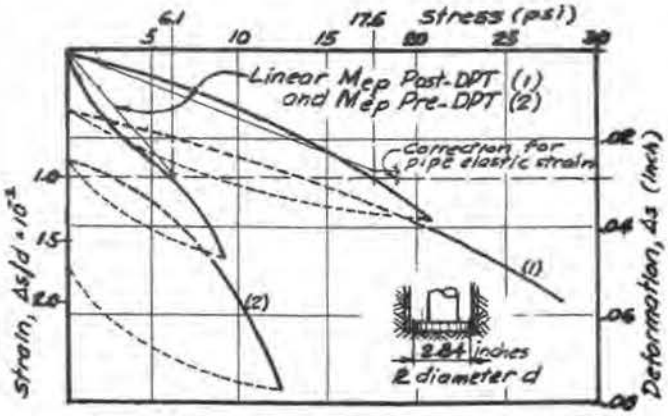
The jack and the proving ring screwed to it are installed hanging from one side of a specially designed but simple steel frame which supports, on the opposite side, the fender of an automobile acting as a reaction weight.

Since the diameter of the bottom plate is very small (2.84 in, 7.2 cm), any excess pore-water pressure induced by a load application is almost immediately dissipated in cohesionless soils and the test can be performed rather quickly. In fine-grained soils the testing may take considerably longer, although the set-up is readily adaptable to the installation of a sensitive piezometer-tip at the center of the plate through the aluminum pipe.

To prevent punching shear mobilizations under the small plate, the maximum stress to be applied should never exceed 0.25 (perhaps a very maximum of 0.3) of the bearing capacity calculated for the soil. Typically, the maximum testing stresses range from 30 psi (200 kPa) to about 100 psi (700 kPa). It should be noted that the reaction load required for the maximum stress would only be 635 lb (2.8 kN).

Typical stress-strain curves obtained with this testing technique, before and after DPT, are shown in Fig. 6. The depth of soil below the plate assumed to be contributing to the total deformation is taken as two plate diameters, as suggested by Burmister (1947, 1962). An extraordinary feature not obtainable with any other known in-situ testing device, is the determination of the complete hysteretic loop at any stress level and the clear determination of the elasto-plastic behavior of the soil.

From the stress-strain curve a simple linear value of secant modulus M_{ep} can be determined, representative of the real elasto-plastic soil compressibility, directly applicable to stres--settlement analyses by selecting an adequate point of strain on the curve and the corresponding direct applied stress. The author has been using the values of strain corresponding to 1% to 1.5% of the plate diameter, Fig. 6, which corresponds to the equivalent range of typical maximum tolerable settlement for prototype footings in terms of their breadth. This homological approach, coupled with model embedment factors, seem to provide compensating effects that allow the direct interpretation of a small-scale test parameter as a full scale design parameter, M . It should be of interest to mention that Osterberg (1947) after comprehensive evaluations of numerous LBT concluded that their results, plotted in terms of stress vs. ratio of strain to diameter fall on a line which appears to be independent of plate size, making thus possible to predict settlements and soil moduli of any size and shape area.



(2) Pre-DPT: Tower 2, Col 65, Depth=6 ft N=7
M_{ep}=(6.1×.144)(2×2.84)/(.0284-.0004)=178 ksf

(1) Post-DPT: Tower 2, Col 51, Depth=8 ft, N=28
M_{ep}=(17.6×.144)(2×2.84)/(.0284-.0014)=534 ksf

Alum. pipe elastic strain, 0.0004 and 0.0014 inch

Fig. 6. Typical LBT Stress-Strain Curves.

Further practical applications obtained directly from LBT results are possible. For example, if the structure is to be subjected to transient loads (wind, etc.), the corresponding additional plastic settlement can be determined from the hysteretic loop which provides the ratio between the elastic and the plastic strains of the soil.

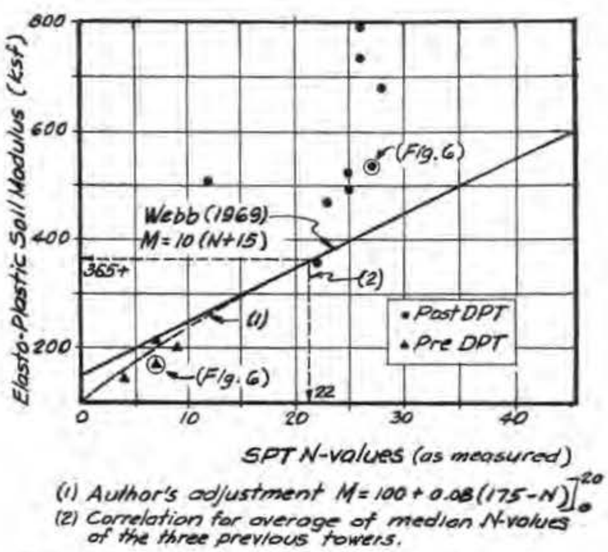


Fig. 7. LBT Soil Moduli vs. SPT N-values.

A general correlation of SPT N-values (as measured) with the soil moduli obtained under the three previous towers is presented in Fig. 7. The wide scatter, as could be expected, is mainly due to the intrinsic limitations of the SPT for the determination of the compressibility characteristics of the soils. In addition, the N-values used in this correlation were not necessarily obtained from the specific test intervals of the LBT. Obviously, the unexplained scatter of the values was coped with by a conservative approach, adopting Webb's (1969) suggested correlation, because it seemed to correspond with the very minimum values of M back-calculated from the measured settlements of the three first towers. Unfortunately, there was no opportunity to perform LBT under the tower of this case, due to the rush with which the construction was started.

* * *

The subsurface investigations typically carried out in this complex included SPT borings, CPT, and test pits. The author also made determinations of shear-strength angles with the Iowa borehole apparatus (BST), but most of the data were discarded as misleading because it was very difficult to keep a sustained diameter in the boreholes under the water-table even though they were filled with drilling mud.

STRESS-SETTLEMENT ANALYSES

The settlement of foundations may be generally regarded as consisting of three separate components giving:

$$\Delta S_t = \Delta S_{ep} + \Delta S_c + \Delta S_s \text{, where}$$

ΔS_t, total ultimate settlement.

ΔS_{ep}, almost immediate elasto-plastic settlement resulting from shear and volume distortions of the soil mass and some volume change.

ΔS_c, consolidation settlement resulting from the time-dependent flow of water from the soil mass under the influence of load-generated excess pore-pressure and concomitant volume reduction.

ΔS_s, secondary settlement or creep resulting from the time-dependent volume change of the soil mass at essentially constant effective stress.

Experience indicates that generally, the behavior of soils under loads after being properly treated with the DPT, is characterized by a significant reduction in the magnitude of the three components of settlements. Actually, in most soils excepting perhaps fat clays, ΔS_c and ΔS_s become negligible under the normal range of design bearing values. In sands, available records of settlement during and after construction point out almost conclusively this settlement behavior. Consequently, the only component of significance taken into account in this case was the elasto-plastic settlement.

Contrary to what may be perceived intuitively, the extraordinarily intense soil modification induced by the DPT does not seem to alter the basic elasto-plastic response observed in all soils, particularly in sands. Moreover, the ratio between the elastic and the plastic strains in sand at a given stress level seems to remain approximately the same after DPT, although the magnitude of the strains are definitely reduced within the same range of stress values.

Most methods to compute settlements require a knowledge of the stress distribution in the soils. This has been adequately resolved for vertical stresses by elastic theory. Other methods used an assumed strain distribution, such as Schmertmann's (1970, 1978). Each method is related to a particular procedure to determine the soil modulus of deformation M. Clearly, the success of any method rests crucially on the use of appropriate values of M, and the analytical sophistication of many methods implicitly presupposes unrealistically accurate values of M.

However, it is well known that nonlinearity, stress-dependency, and inelasticity, are always present and contribute to the difficulty of measuring accurate values of M, particularly in sand. Many authors have made significant contributions regarding the effects of different factors on the values of M, but the present state of the art compels to the use of, at best, semi-empirical methods. Just to mention one valuable publication, Lambrechts and Leonards (1978), after listing some relevant factors influencing the stress-deformation behavior of granular soils conclude that stress-history is the most important and present interesting findings.

Consequently, for his stress-settlement analyses the author opted for a simple approach using Schleicher's classical elastic expression and Janbu et al (1956) charts for the geometric factors; or Schmertmann's method, with a single conservative value of M representative of the entire significant depth and discarding the creep factor. It is unrealistic to attempt individual or layered characterizations of the complex soil mass after the DPT, unless either the time available and the testing budget be completely out of the typical, or there is ample technical and economical justifications for such academic endeavor.

In this case, the settlement prognosis was based on back-calculations of the average elasto-plastic M developed under the three previous 11-storey towers using the measured settlements and the corresponding approximate range of acting building loads. A total of 21 settlement points out of a much larger number initially set, were still useful at the completion of the towers, with total recorded settlements as shown on Fig. 8. A comprehensive

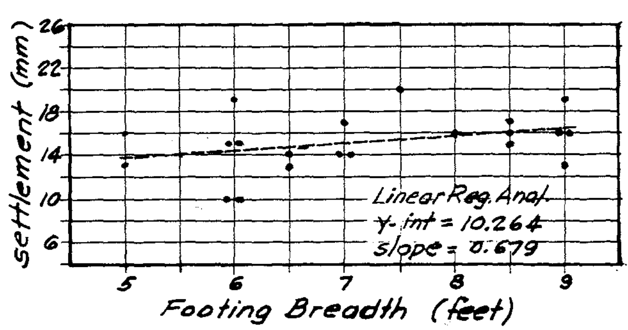


Fig. 8. Settlement Records of Previous Towers.

evaluation of these factual records, together with the corresponding footing sizes, and the relative stiffness of the interconnecting stuctural members, led to the conclusion that the soil-structure interaction in these buildings, after DPT, reduced significantly the footing size effect and the resulting differential settlements.

With a representative average settlement of 15 mm (0.59 inch) and a conservative average footing size of 8.0 ft (2.4 m) the elastic analysis, resulted in values of soil moduli (within the participant depth interval) represented by an average M of 365 kips-ft for a realistic range (at that stage) of 5.8 ksf (82.5% of the total design loads). For the same conditions and parameters, Schmertmann's analysis simplified as explained above, resulted in an average M=436 ksf, or 20% larger.

Following is a verbatim transcript of the specific paragraphs of the author's report (dated Sept. 30, 1980) for this case history, in which the justification for higher design bearing values was presented:

"The modified shearing strength and compressibility properties of the more competent soil fabric at this site, after the DPT, were then realistically characterized by an effective average ϕ-angle of 39°+ and by an average soil modulus M in excess of 350 ksf.

"That shearing strength parameter applied in Terzaghi's stability analysis for the same smallest footing-size (B = 5 ft) and same scour conditions as before, resulted in a soil bearing capacity in the order of 24 ksf at the critical incipient state of shear failure.

"Similarly, results of stress-settlement analyses can be now used with a higher degree of confidence to determine a prognosis of maximum total settlement to be expected. For example, on the very conservative basis of soil-structure interaction and foundation behavior equal to those of the three previous towers, the average soil contact stress required to induce the maximum value of total tolerable settlement (as per criterion $\Delta S \leqslant 0.01B$) would be in the 9.5 ksf range.

"However, it is important to point out that the criterion of tolerable maximum total settlement adopted before ($\Delta S \leqslant 0.01B$) has proven to be too conservative and not entirely adequate, particularly since it implies a direct linear relationship with respect to the size of the footings. A more comprehensive and realistic expression, derived and applicable to granular soils treated with the DPT, is as follows:

$$\Delta S \leqslant 0.20B(1-0.025B)K \quad \text{(in inches)}$$

In which B is the footing width in ft,
for $B \leqslant 10$ ft and K =
1.2 for insensitive structures (very rigid or very flexible).
1.0 for normal structures.
0.8 for sensitive structures.

"Using this settlement criterion for a normal-type structure (K = 1.0), and assuming the worse soil compressibility (M = 365 ksf, as derived from actual settlement records of the three towers), the following soil bearing values can then be considered safe, and consequently acceptable, for the present re-evaluation of the footings of this fourth tower:

B = 5 ft	q = 13.7 ksf
6	13.3
6.5	13.1
7	12.9
7.5	12.7
8	12.5
9	12.1
10	11.8

....."

"The range of average contact stresses to be imposed on the existing footings (due to the additional four storeys) by the maximum combination of dead + live + wind loads are given by the Structural Engineer as being between 8.7 and 12.1 ksf. ......"

"Actually, a conservative prognosis for the total maximum permanent settlement under maximum dead + live loads would be 0.9 inch (23 mm) for the most critical footing.

"A prognosis for the additional mostly elastic settlement for the most extreme stress condition (dead + live + wind loads) would be a conservative 0.4 inch (10 mm), of which about 75% or 0.3 inch (7 mm) should be expected to rebound or recuperate immediately after ceasing the short-term action of the transient load. ....."

SETTLEMENT RECORDS

Fig. 9 presents typical settlement records for two footings of the case building extended for a couple of months after the total dead load was acting (or approximately 90% of the total design vertical loads). The measurements are representative of the higher range of settlement values; actually, maximum total recorded settlements were 17 mm (0.67 in). Although

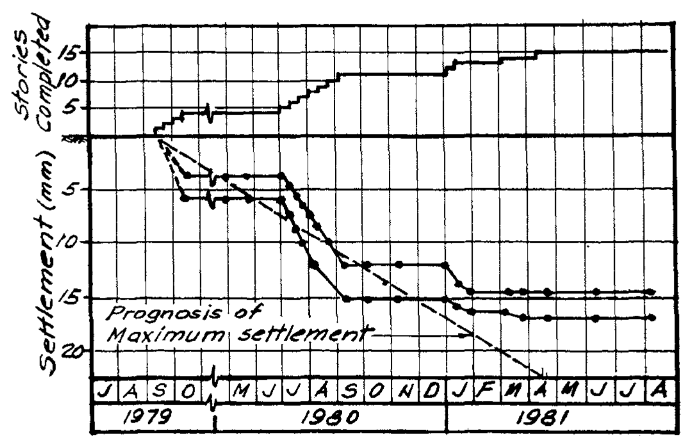


Fig. 9. Settlement Records of Case-Building.

numerous settlement markers were lost during the erection of the structure, the final records of 9 points indicated that the settlements were generally uniform, as under the previous towers, regardless of footing size; with differential settlements entirely negligible (6 mm, or 0.24 inch maximum).

The maximum measured settlement was 82% of the maximum anticipated total value [17/(0.9x23) = 0.82].

Significantly, no creep effects were measured either during the various construction delay periods (extending several months) or after the completion of the structure .

CONCLUSIVE REMARKS

The DPT is an extremely effective technique for the improvement or correction of foundation soils. Its proper application permits the safe use of very high design bearing values, even on originally poor profiles.

The implementation parameters presented in this paper allow a rational appproach to the application and control of the DPT, and may be used as a useful tool for the prognosis of the soil behavior after treatment.

The LBT, as applied by the author, seems to be a very effective in-situ testing technique.It provides a realistic and relatively simple approach for the study and investigation of the complex soil-compressibility phenomena. Some improvements are being introduced now to facilitate its application.

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