Vibrations Measurement During Soil Dynamic Compaction.

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VIBRATIONS MEASUREMENT DURING SOIL DYNAMIC COMPACTION

THE CASE STORY OF THE OLYMPIC SAILING CENTRE IN ATHENS - GREECE

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1. WHAT DYNAMIC COMPACTION IS

Dynamic compaction essentially is lifting and dropping a heavy weight several times in one place. The process is repeated on a grid pattern across the construction site.

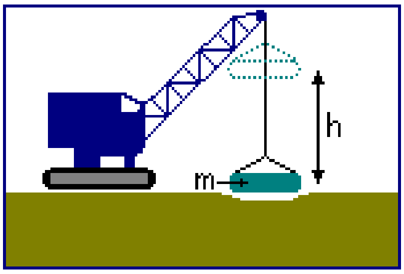
Soil densification by dynamic compaction (DC), also called "heavy tamping" is a well-known compaction method. The method was "rediscovered" by Menard, who transformed the crude tamping method into a rational compaction procedure. Soil is compacted by repeated, systematic application of high energy using a heavy weight (pounder). The imparted energy is transmitted from the ground surface to the deeper soil layers by propagating shear and compression waves types, which force the soil particles into a denser state. In order to assure effective transfer of the applied energy, a 1 to 2 m thick stiff layer usually covers the ground surface. Pounders can be square or circular in shape and made of steel or concrete. Their weights in the EU countries normally range from 5 to 25 tons and drop heights of up to 25 m have been used. Heavier weights and larger drop heights have been used for compaction of deep soil deposits, but are not very common.

Heavy compaction tends to annoy the neighbours, which limits its use in built-up areas.

The compactive energy per blow (W) is equal to:

$$\text{Energy} = m.\ g.\ h$$

where m = mass, g = gravitational constant, h = drop.



Dynamic compaction is carried out in several passes. During each pass, the weight is dropped repeatedly in a predetermined grid pattern. The distance between the compaction points is normally decreased in the subsequent passes and compaction is carried out in-between the previously compacted points. The final pass, also called "ironing pass", usually performed with low compaction energy, is carried out with a reduced drop height. The objective is to densify the superficial soil layers without remoulding the already densified deeper layers.

Although the dynamic compaction method appears to be very simple, it requires careful design of the compaction process. The densification effect is strongly influenced by the dynamic response characteristics of the soil to be compacted, but also by the underlying soil layers. Usually, extensive compaction trials are needed to optimize the compaction process with respect to the required energy for achieving specified densification criteria. A major limitation of dynamic compaction is the lack of monitoring and quality control during the production phase. However, for research purposes, the pounder can be equipped with sensors to monitor the applied energy and to record the dynamic response of the soil layer.

The maximum depth which can be achieved by dynamic compaction depends on several factors, such as the geotechnical properties of the soil layer to be compacted, the dynamic soil properties in and below the layer to be compacted (e.g. a soft clay layer below the layer to be densified can significantly reduce the compaction effect), the ground water level, the compaction grid, the number of compaction passes and the time interval between passes. As a general rule, the maximum depth D_{max} to which a soil deposit can be estimated from the following relationship:

$$\boxed{D_{max} = \alpha \sqrt{(m.h)}}$$

where h is the average drop height and m the mass of the pounder. The factor α should be determined for each site, but varies between 0,3 – 1,0 depending on the grain size distribution and degree of saturation (0.5-1 for sands, 0.3-0.5 for silts and clayey soils).The typical depth of compaction for drop height of 15 m and a pounder mass of 15 tons is 7 - 8 m.

2. WHERE DYNAMIC COMPACTION CAN BE APPLIED

During the past 15 to 20 years, dynamic compaction has become an increasingly popular technique for densifying uncontrolled fills and loose natural deposits. Dynamic compaction is highly cost effective in urban areas with relatively high land values.

A major geotechnical dilemma with development of a site in karst topography is judging whether the site is susceptible to sinkhole development. For those sites with known sinkholes or with conditions conducive to sinkholes, do you correct the existing sinkholes or subsurface cavities and build? .... or do you found the structure on deep foundations? Most sinkholes develop by raveling of the overburden soils into fissures or cavities in the carbonate rock, causing voids or "domes" to form within the subsurface. This process is graphically illustrated in Fig. 1. Several techniques are available to locate these domes, including indirect geophysical techniques such as ground probing radar, gravity surveys, etc.; and direct techniques such as borings or probings. The cost of geophysical techniques for most small to medium-sized projects prohibits their use; and the geotechnical engineer is forced to rely on limited borings, surface observations, and his knowledge of the general area. The resulting risk and uncertainties as to whether soil domes and soft clay-filled cavities are present have caused many projects to be founded on deep foundations for conditions that would otherwise warrant shallow foundations. In addition, there are many recorded case histories where the subsurface cavities were undetected during the exploration but caused failures during or after construction.

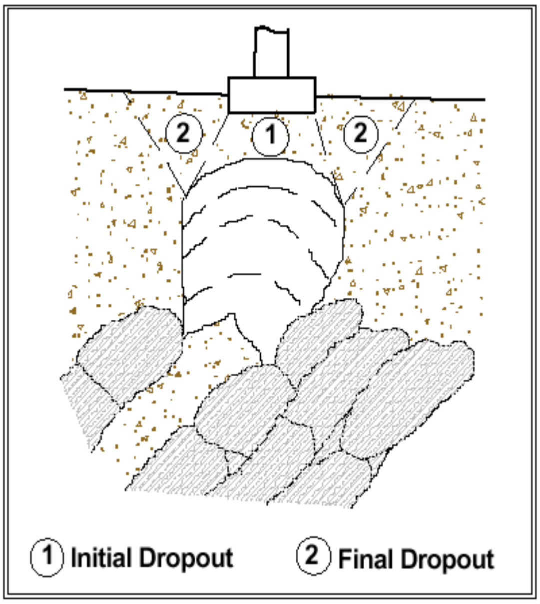


Fig. 1 Typical Sinkhole Development

Various techniques have been used to improve sites in sinkhole susceptible areas and reduce the probability of sinkhole development. These techniques have included preloading, various types of grouting, excavation of the overburden soils and sealing the fissures in the rock with grout, as well as dynamic compaction. In addition to improving the subsurface conditions by collapsing the soil domes, dynamic compaction serves as an exploration tool in locating the subsurface cavities. In essence, heavy weight dropping on a close grid pattern is used over the proposed building site to locate and collapse existing soil domes. The dynamic compaction technique allows the use of shallow foundations and can provide an owner tremendous savings. This process is illustrated in Figure 2.

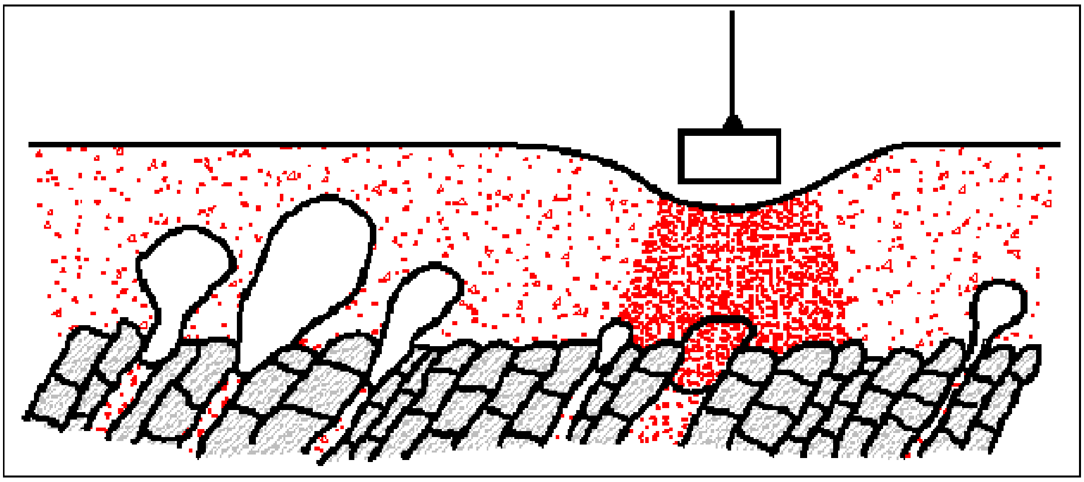


Fig. 2 – Dynamic compaction used to collapse subsurface voids

3. HOW MANY RESULTS CAN BE OBTAINED

Evaluation of ground improvement is accomplished using a variety of methods, from simple elevation surveys to document the amount of compression caused, to geotechnical in-situ and laboratory testing, to geophysical testing. The most common types of geotechnical testing used to evaluate the modulus of the soil after ground improvement are penetration tests such as the Standard Penetration Test (SPT) or the Static Cone Penetration Test (CPT).

Engineers occasionally ignore two significant factors when evaluating these test results: (a) continued increase of CPT or SPT values over time, and (b) an increased correlation between modulus and penetration resistance for overconsolidated soils, as compared to normally consolidated soils. During performance of ground improvement, ground surface settlements of one to three feet are not uncommon at many sites. Calculated settlements under foundation loads are generally much less than the observed settlement. If the post-improvement geotechnical testing indicates little improvement has occurred, the geotechnical engineer is faced with the following question. *"Was my observation that the soils have been prestrained incorrect or are the correlations between compressibility and the geotechnical testing parameter wrong?"*

Since the site settlement (i.e., prestraining) is an easily verifiable phenomenon, the answer is obvious. The next step is to determine how best to reconcile the known site settlement with the geotechnical testing. It has long been recognized that both strength and compressibility properties of soils improve with time in both natural soil deposits and soils improved by dynamic compaction and vibrocompaction or vibroreplacement. Improvement has been observed in sand, clay, and silt. Numerous articles have been published on this phenomenon, including the Twenty-Fifth Karl Terzaghi Lecture (Schmertmann, 1991). This strength gain and modulus increase occur well after excess pore water pressures dissipate, and thus can not be attributed merely to this occurrence.

Various investigators have attributed this aging improvement to thixotropy, secondary compression, cementation, dispersive particle movements and internal stress arching, as well as other explanations. Ground improvement practitioners, as well as geotechnical consultants involved in the ground improvement industry, are constantly faced with the question of how to evaluate the degree of improvement of a densified soil deposit. Quite often geotechnical testing by SPT or CPT falsely indicates a decrease in strength and compressibility shortly after ground improvement. However, after a short time period the strength and compressibility properties improve dramatically. Schmertmann (1986) presented data on a 10m layer of silty sand in Jacksonville, Florida that was improved by dynamic compaction. The static cone bearing capacity qc over time was compared to qco immediately after dynamic compaction.

This data is presented as Figure 3.

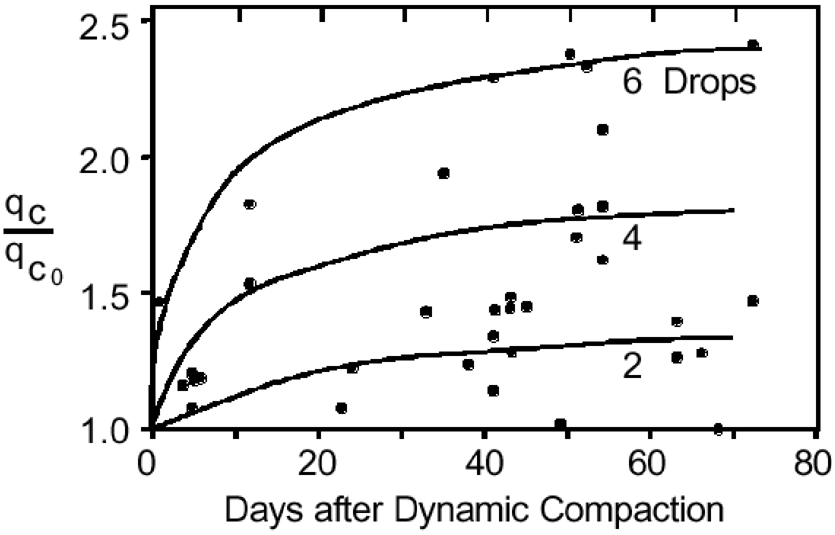


Figure 3 – Increase in Static Cone Resistance with Time Following Dynamic Compaction

Penetration test results are most commonly used to estimate the settlement behavior of the soils. Calculated settlement is inversely proportional to the soil modulus, either the elastic modulus, E, or the constrained modulus, M. The constrained modulus is the more commonly used parameter. Based on published studies from various sources, the general expression for constrained modulus from CPT data is:

$$M = \alpha \, . \, q_c$$

where α depends upon stress state, soil type, and degree of preconsolidation.

A review of calibration chamber tests on normally consolidated sands from Norway, Italy, UK and the US (Mitchell and Gardner, 1975, Lunne and Christoffersen, 1985) and presented as Figure 4 indicates that:

$3 < \alpha_{n.c.} < 8$

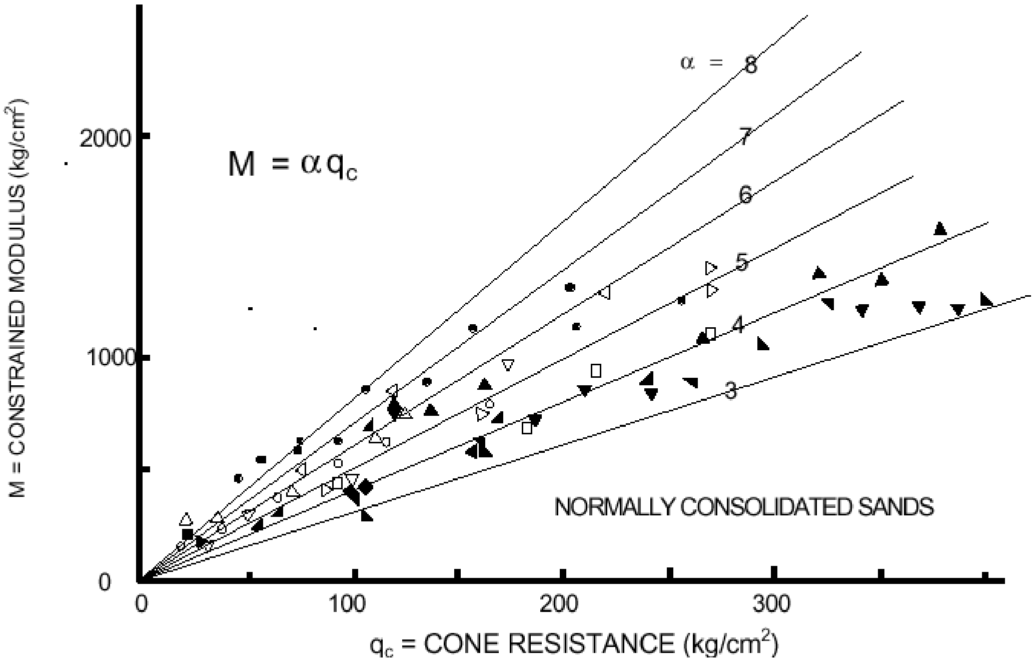


Figure 4 – Constrained Modulus vs. Cone Resistance for Normally Consolidated Sands

The application of dynamic compaction or vibrocompaction results in overconsolidated soil. Consequently, the ratio of $\alpha = M/q_c$ for dynamically compacted or vibrocompacted sand is much higher than for normally consolidated sand (Robertson and Campanella, 1983). Figure 5 summarizes data from overconsolidated sands indicating the range to be $7 < \alpha_{o.c.} < 36$, significantly greater than for normally consolidated sand. The use of SPT data may be similarly applied to calculation of foundation settlements. The SPT resistance may be converted into an equivalent CPT-q_c resistance and the foregoing approach applied to the particular foundation situation. In evaluating a soil modulus based on the equivalent CPT resistance, the same trends discussed previously apply. Schmertmann (1970) presented an approximate method for calculating foundation settlements in sand based on strain distributions from elastic theory, and presented revisions to his suggested calculation method in 1978. The 1978 work includes an acknowledgment that for the same CPT resistance, the modulus in an overconsolidated sand will be at least double that expected in a normally consolidated sand, resulting in half the settlement. As a lower bound estimate for clean to silty/clayey sands treated by dynamic compaction, the results of site correlations by Schmertmann et. al. (1986) indicate a conservative relationship:

$$M = 7\ q_c$$

This is significant in that for normally consolidated sands, the conservative bound for α is only 3. Thus, following soil improvement, even if the CPT or SPT test values do not show an increase, the literature indicates that the calculated settlement values would be less than half of what would be expected before ground improvement for those same CPT or SPT values.

In summary, two factors may lead engineers to underestimate the effectiveness of ground improvement procedures when reviewing post-construction test data. First, test values generally increase significantly for weeks, months, or even years after the ground improvement is completed. Second, test results may overestimate settlements unless care is taken to use the proper correlations for over-consolidated soils and to calibrate the correlations to the observed degree of soil prestraining from site observations.

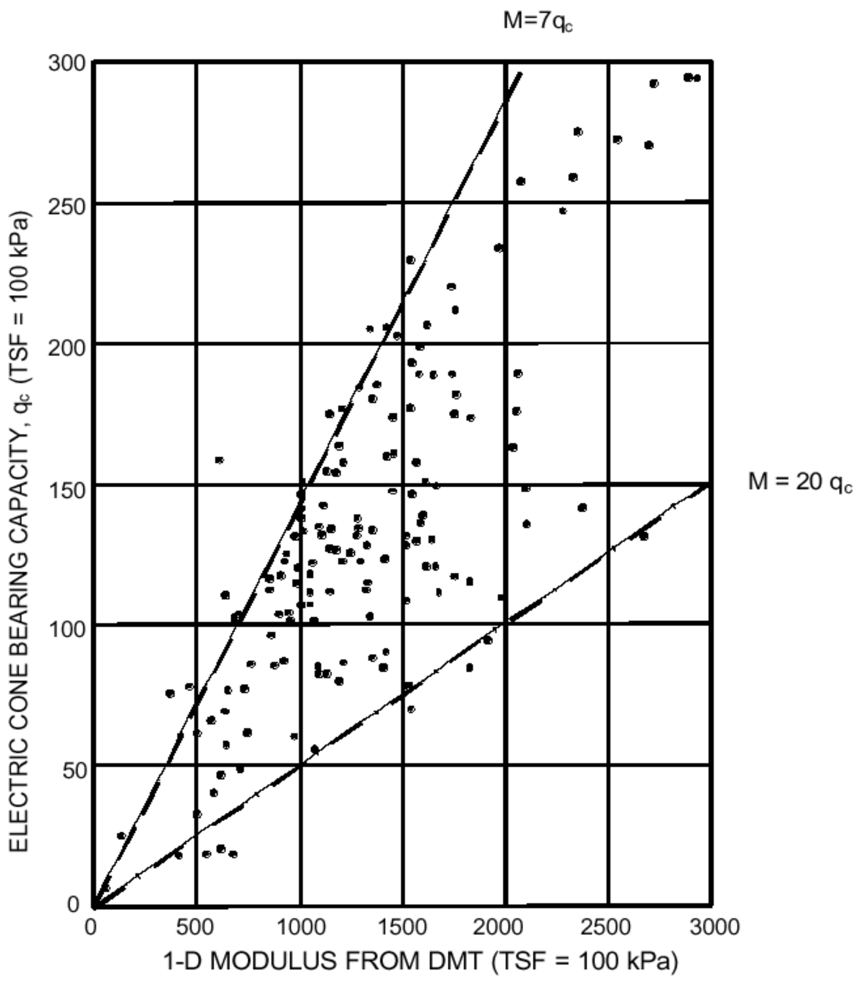


Figure 5 – Correlation of Static Cone Resistance and DMT Modulus for Overconsolidated Sands

4. GROUND VIBRATIONS DURING DYNAMIC COMPACTION WORKS

An undesirable feature of dynamic compaction, however, is the generation of ground vibrations caused by the dropping weight. Ground vibrations can be potentially damaging to adjacent structures in addition to being annoying to people. It is therefore important that vibration monitoring be performed whenever dynamic compaction is performed in close proximity to structures.

For many years the limiting peak particle velocity for damage threshold was considered to be 2 inches per second. In 1980 the U.S. Bureau of Mines re-evaluated the threshold values and revised the threshold values to be frequency dependent. It was found that the lower the frequency, the lower the damage threshold.

Figure 6, which was published by the Bureau, is now considered industry standard. The vibration levels generated by dynamic compaction are low frequency vibrations, generally in the range of 5 to 20 Hz. As Figure 6 illustrates, the peak particle velocities in this frequency range should be maintained below 0.5 inches per second to prevent damage to nearby structures with plaster walls and 0.75 inches per second for drywall construction. These limiting values may be overly conservative given that the duration of vibrations induced by dynamic compaction are very short. More damage occurs when steady state vibrations, such as are caused by vibratory pile drivers, are used.

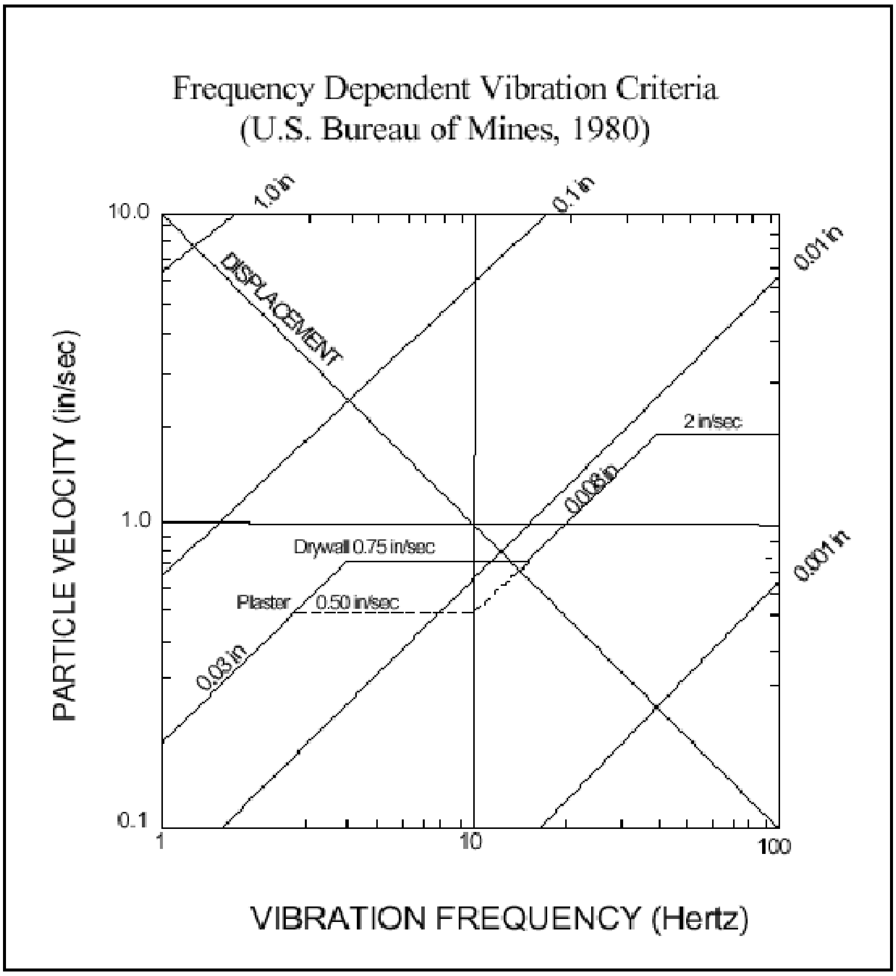


Figure 6: Frequency Dependent Vibration Criteria (U.S. Bureau of Mines, 1980)

Fortunately, the amplitudes of ground vibrations dampen with distance from the point of impact. Figure 7 is a summary of peak particle velocities from 12 dynamic compaction sites, using weights varying from 4 to 40 tons and drop heights of 5 to 100 feet. A safe upper limit from this data is: where PPV is in inches per second and d is in feet. The same data set was used to develop a scaled distance versus the peak particle velocity. The scaled distance is defined as the ratio of the square root of the energy per drop to the distance from the point of impact. The safe upper limit expression is:

$$PPV = \left(\frac{75}{d}\right)^{1.7}$$

where PPV is in inches per second and d is in feet.

The same data set was used to develop a scaled distance versus the peak particle velocity. The scaled distance is defined as the ratio of the square root of the energy per drop to the distance from the point of impact. The safe upper limit expression is:

$$PPV = 8\left(\frac{\sqrt{WH}}{d}\right)^{1.7}$$

where PPV is in inches per second, d and H are in feet and W is in tons.

If vibration levels are anticipated to cause off-site problems, isolation trenches can be dug between the point of impact and the area to be protected. The vibration levels can be reduced by factors of 2 to 10, depending upon such factors as the soil type, the depth of the trench and the position of the weight dropping to the trench.

In summary, it is important to consider the effect of vibration produced by dynamic compaction on off-site structures. However, even in built-up areas, it is generally possible to utilize dynamic compaction with a careful program of vibration monitoring, possibly in conjunction with isolation trenches.

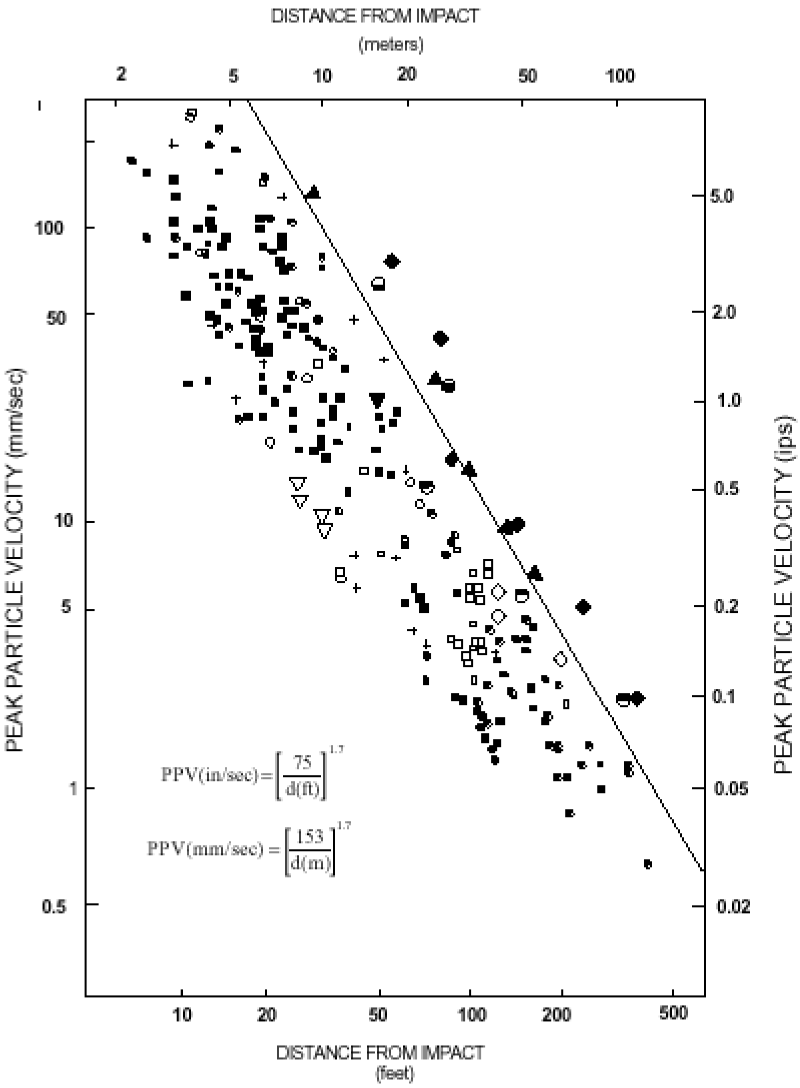


Figure 7: Summary of Dynamic Compaction Vibration Experience (Mayne, 1985)

5. CASE STORY OF THE OLYMPIC SAILING CENTRE IN ATHENS - GREECE

Two extensive programs of soil dynamic compaction were employed during the construction of the new Olympic Sailing Center of Athens to be used during the 2004 Summer Olympic Games.

A cylinder pounder was mainly used with 17 tonnes of weight and 24 meters drop.

During all blow sets, it was clearly seen that the values of particle velocity rised as the number of blows in the same point increased. The frequency spectra presented generally peaks in correspondence of 7 Hertz area for the two horizontal components, thus the peaks of the vertical component remained to the area of 10 Hertz.

The ground vibrations monitoring program allowed the uninterrupted progress of the dynamic compaction works, the safe documentation and also the prevention and the optimum management of any unpleasant situation.

Foto 1: Air view of the construction area (April 2002).

Foto 2: The cylinder pounder.

Foto 3: Dynamic Compaction works and the MR2002.

Foto 4: Close view of the crater after a blow.

Foto 5: Air view of the construction area (June 2003).

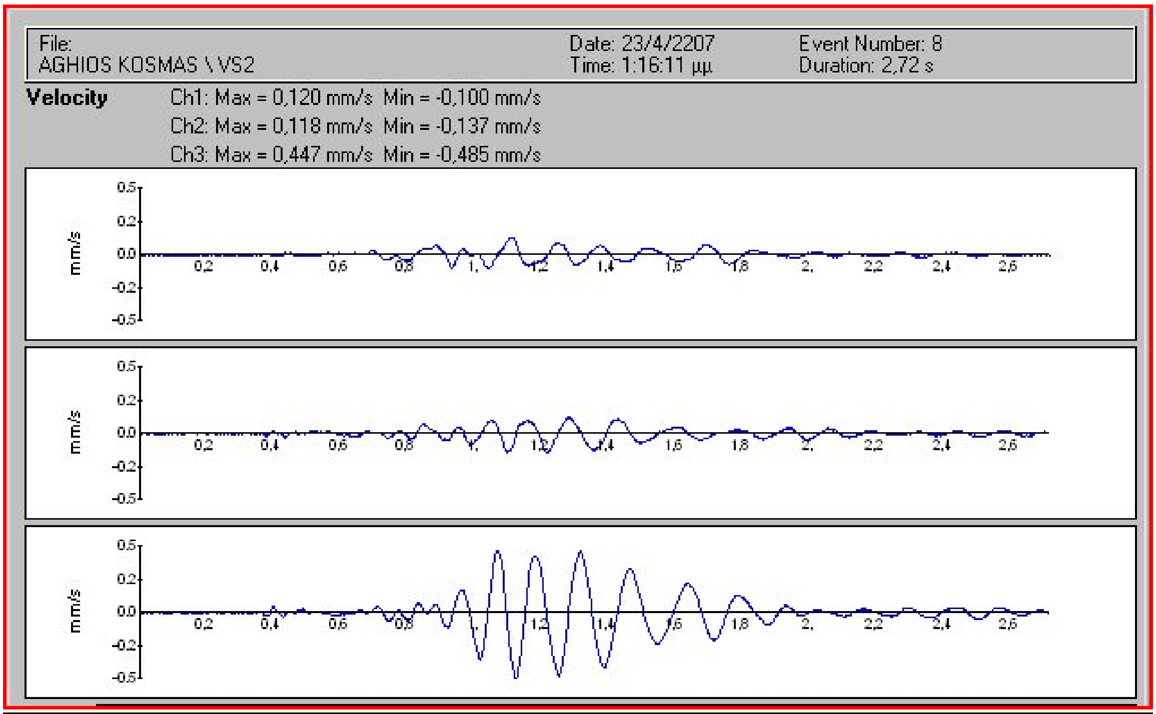


Foto 6: A typical MR2002 recording (April 2002).

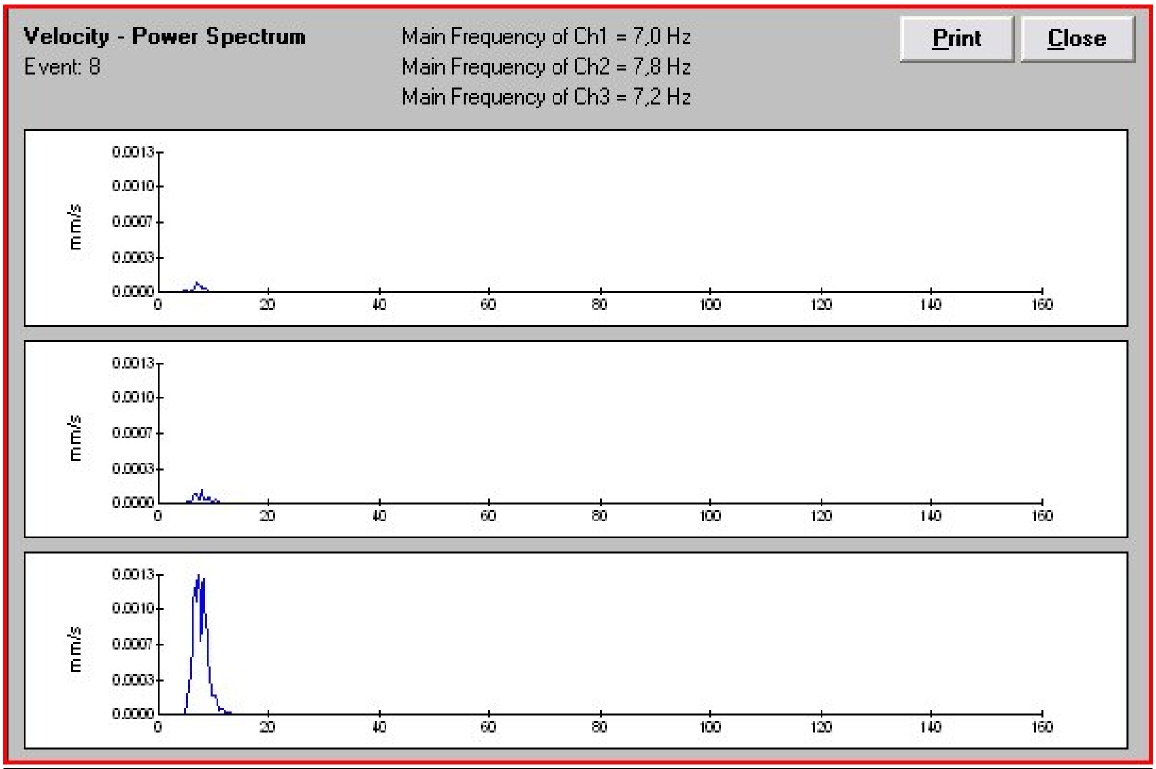


Foto 7: Power spectrum of the previous recording.

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