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P.O. BOX 5271 AUCKLAND 1 NEW ZEALAND
PH: 64-9-377 1865 FAX 64-9-307 0265

REPORT FOR RESEARCH PROJECT NO.91/60

EARTHQUAKE AND WAR DAMAGE COMMISSION

EVALUATION OF DYNAMIC CONSOLIDATION FOR N.Z. CONDITIONS

Report prepared by:

T.J.E. SINCLAIR

TONKIN & TAYLOR LTD

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1.0 INTRODUCTION

1.1 General

This report presents an evaluation of Dynamic Consolidation (DC) as a means of mitigating liquefaction potential, particularly in relation to its application in New Zealand.

The work has been carried out under EQC Research Grant No.91/60, under agreement dated 6 June 1991 for a research project entitled "Evaluation of Dynamic Consolidation (DC) for NZ Conditions."

1.2 Objectives

The objective of the research is to evaluate the effectiveness of Dynamic Consolidation (DC) for alleviating the potential for liquefaction to the range of New Zealand soils.

In this respect, the following issues have been addressed:

- Experience and availability of expertise in NZ
- General effectiveness and limitations
- Effectiveness of technique in particular soil types
- Cost comparison with alternative methods
- Constraints due to disturbance and "nuisance"

1.3 Relevance of RESEARCH

There are several population and industrial centres (e.g. Wellington Region, Christchurch, Tauranga, Napier, Taupo) where liquefaction risk is significant. In many parts of the world, DC has proved very effective in densifying certain soil types to depths up to 30 metres and at a cost of about half that of other methods. However, despite this significant cost-benefit, very little is known about the technique in New Zealand. It is expected, though that some NZ soils (e.g. pumice sands) may behave very differently.

This research will:

(i) bring to the notice of other NZ Engineers the benefits of DC

(ii) enable evaluation of the technique with respect to NZ conditions

(iii) enable disturbing effects to be evaluated leading to assessment of suitability of DC in population centres

1.4 Proposed Programme and Methodology

It was considered at proposal stage that the benefits of the research would be most evident from a two-stage work programme, as follows:

Stage I **Office Work**

(1) Comprehensive literature review

(2) Review of NZ experience

(3) Identify suitable sites for full scale trials and develop trial programme

(4) Report for Stage I

Stage II	**Field Trials (to be planned and costed in detail in Stage I)**
(5)	Carry out fully instrumented field trials at sites characterised by silica sands and pumice sands
(6)	Monitor disturbing effects: - vibrations - subjective perceptions
(7)	Report for Stage II

The work approved for funding included only Stage I. This report, therefore, covers Tasks (1) to (4) above.

2.0 BACKGROUND

2.1 General

The phenomenon of "liquefaction" is well known to most New Zealand engineers in the civil and building engineering environment, and requires little explanation. However, for the benefit of wider readership, a brief commentary on liquefaction is presented in the following sub-sections, giving a somewhat simplistic definition of the terms and illustrations of this specific earthquake hazard.

Dynamic Consolidation is not well known in New Zealand, with very few cases of the technique put into practice. A brief explanation of the method is presented, together with some background information on history and the extent of experience elsewhere in the world.

2.2 Liquefaction

The phenomenon known as "liquefaction" occurs when a soil is essentially transformed into a fluid as a result of ground shaking caused by earthquakes. Liquefaction, therefore, may result in a temporary loss of bearing capacity, causing complete failure or significant settlement of foundations. This was dramatically demonstrated during the 1963 Niigata earthquake where the ground under a number of large apartment blocks liquified and the structures, in effect, floated out of the ground and experienced catastrophic rotation.

More recently, our own Edgecumbe earthquake (1987) and the Loma Prieta earthquake (1989) provided numerous examples of liquefaction which have been extensively reported and researched. For the latter, liquefaction was the primary cause of damage to buildings and "lifelines" in the Marina district of San Francisco.

The requirements for liquefaction occurring are various but can be summarised essentially as follows:

- Loose to medium dense sand or silt soils
- High groundwater
- Earthquake of sufficient duration and stress level

As earthquake waves propagate vertically from below a soil deposit, shear strains are imparted to the soil particles, encouraging them to reorganise into a more compact structure. However, because the time it takes for this to happen is not sufficient for water to drain through the pores of the soil structure, the tendency to reduce in volume is manifested as an increase in pore water pressure. If this pore pressure increases to equal the total confining pressure (e.g. the overburden pressure) there is then no inter-particle stress (i.e. "effective stress") and therefore no friction between particles and no shear strength in the soil; a fluid by definition.

Following ground shaking, the excess pore water pressures dissipate at a rate dependent on the lengths of drainage path and the permeabilities of the various soils in the sub-surface profile, and the liquefied soil consolidates accordingly. It should be noted that a deposit of soil which has experienced complete liquefaction and subsequent pore pressure dissipation will be more resistant to further liquefaction due to the higher density of the deposit.

The obvious way to reduce the potential for liquefaction, therefore, is to densify the soil, possibly even by causing liquefaction artificially. It is in this respect that Dynamic Consolidation is being considered.

2.3 Dynamic Consolidation (DC)

In 1970, Techniques Louis Ménard introduced a technique initially known as "heavy tamping". It involved the simple and fundamental concept of improving ground to great depth by repeated application of very high intensity impacts to the surface. The high energy impacts are achieved simply by dropping heavy weights (tens of tonnes) from great heights (tens of metres), usually using a crane but sometimes purpose-made gantries.

The initial applications of heavy tamping were for densification of granular materials. Since then, much effort has gone into convincing the engineering community that it is also applicable to the improvement of saturated cohesive soils. The hypothesis is that the impacts cause localised shock pore pressures which dissipate rapidly because of the high pore pressure gradients, and the presence of fissures created by the mechanical disturbance and/or hydraulic fracture; in effect, rapid, forced consolidation. Louis Ménard, therefore, proposed the term "Dynamic Consolidation" which has gained reasonably widespread acceptance. However, the terms "Heavy tamping", "Dynamic Compaction", "Deep Compaction" and "High energy compaction" are still in common use, particularly in areas where some aspects of the technique were protected by patents and trade names.

For the purpose of this report, the term Dynamic Consolidation (DC) will be maintained, principally because it is now widely accepted that the process of ground improvement, (at least for non-cohesive soils) involves the creation of high pore pressures (liquefaction) and subsequent consolidation in the normal soil mechanics sense.

It is self evident, therefore, that the soils most susceptible to liquefaction will also be the most suited to improvement by DC.

3.0 LITERATURE REVIEW

3.1 General

The SATIS Information and Search facility identified 76 references relating to "dynamic consolidation", "dynamic compaction" or "heavy tamping". The list provided by SATIS is attached as Appendix A of this report. Some other references, not on the SATIS list, have been obtained from other sources within New Zealand and during various visits overseas, and these are also listed in Appendix A.

Copies of the references considered relevant to the study have been obtained and compiled into a project dossier. One copy of this dossier has been made available to EQC but does not form part of this report because the bulk of the report would be unmanageable and because the quality of some of the copying is not up to report standard. However, a few key papers are included with this report in Appendix B.

The references have been reviewed systematically (see Standard Form and examples included in Appendix B), particularly with regard to the following topics:

- Degree of improvement
- Depth of improvement
- Energy:
  - (a) Energy/drop
  - (b) Energy/unit area
- Drop Arrangement:
  - (a) No.of drops/point
  - (b) Spacing
  - (c) No. of passes
- Weight properties

- Monitoring/evaluation criteria

- Time between passes

- Time and Rate of Progress

- Costs:
  (a) Actual
  (b) Comparative

- Vibrations

- Miscellaneous aspects

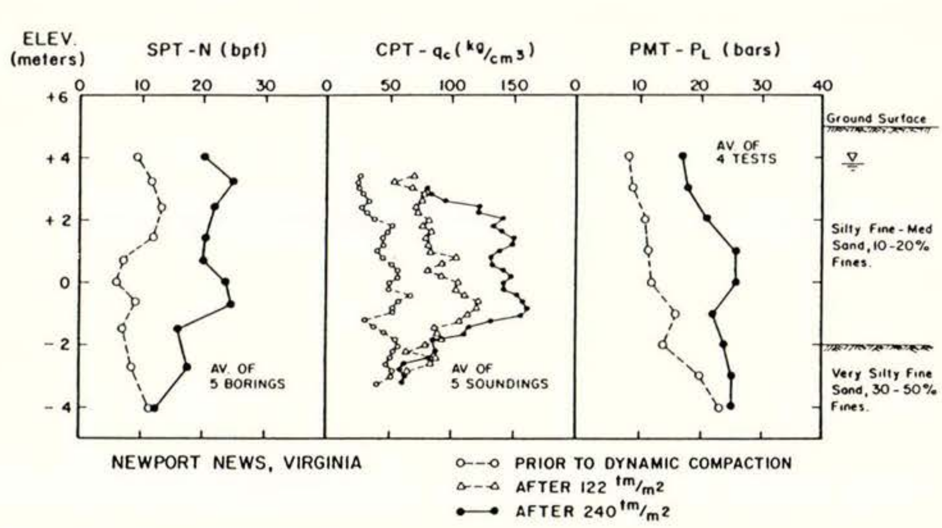
3.2 Degree of Ground Improvement

The degree of ground improvement depends on many factors, including:

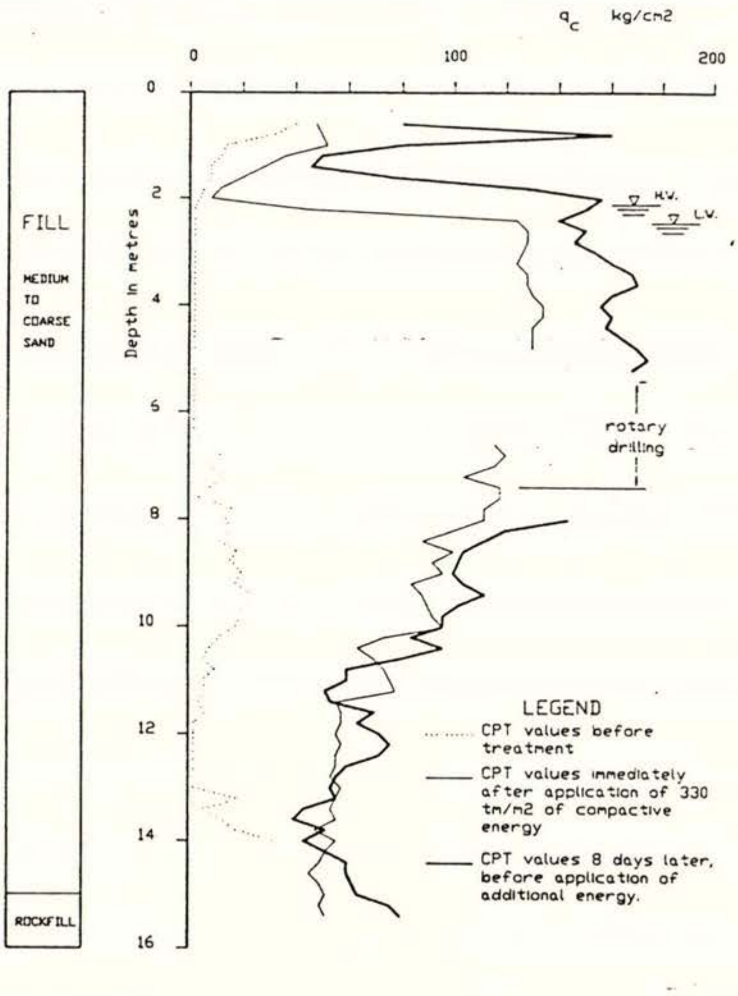
- nature of soils
- level of groundwater
- energy and arrangement of compaction

All these factors will be discussed under other headings. Figure 3.1 gives examples of the degree of improvement obtained in some case studies but, without reference to the background information, they should not be used directly for assessing achievable improvement.

However, a view of the case studies indicates that there is likely to be an upper limit to the degree of improvement. Leonards, et al., (1980) suggest that the "average cone penetration resistance in zone of maximum densification" is unlikely to exceed 15 MPa, but concede more data are needed for verification. The peaks in cone resistance exceed 15 MPa in several case histories but generally do not exceed 20 MPa.



(a) Mayne et al., 1984



(b) Dumas, 1986

Figure 3.1: Examples of degree of improvement with DC

Mayne et al., (1984), in their analysis of 124 case histories, indicate that all the usual monitored parameters (N value, cone resistance, pressuremeter modulus and limit pressure) increase as applied energy level increases but with diminishing returns at the high energy level (i.e. a linear relationship on a log-linear plot). Only one of their data points (18.5 MPa) lies outside the limit proposed by Leonards et al., (1980).

Ramaswamy and Yong (1982) point out that the upper limit is clearly governed by the 100 percent relative density (D_r) limit, the actual values depending on whose correlation between "N" and D_r one believes (see Fig 3.2). The data seem to indicate that the Gibbs and Holtz relationship is the more appropriate (i.e. that it is unlikely that average "N" values will be greater than 30 to 40).

3.3 Depth of Ground Improvement

It is now generally accepted that the depth of "effective" improvement is given by:

$$D = k\sqrt{Wh} \qquad -(1)$$

where: D = "effective" depth of improvement (m)
W = weight of Weight (tonnes)
h = height of drop (m)
k = empirical coefficient

Original work by Ménard (Ménard and Broise, 1974) was based on this relationship with k=1.0. However, as data have accumulated, several researchers have shown convincingly that this is too optimistic (e.g. Leonards et al., 1980; Lukas, 1980; Mayne at al., 1984; Fan et al., 1988). Examples of data from these references are given in Figure 3.3. It is generally agreed by these same researchers that, for sands, k is between 0.5 and 0.6 and that a value of 0.5 would be sufficiently accurate and conservative.

However, the meaning of the term "effective depth" is vague throughout the literature. The degree of improvement is generally not uniform; the improvement tending to diminish with depth. The proposed relationship (i.e. $D = 0.5\sqrt{Wh}$) appears to provide a depth for which most density requirements can be achieved by appropriate total energy input and spacing of drop points (see later discussion).

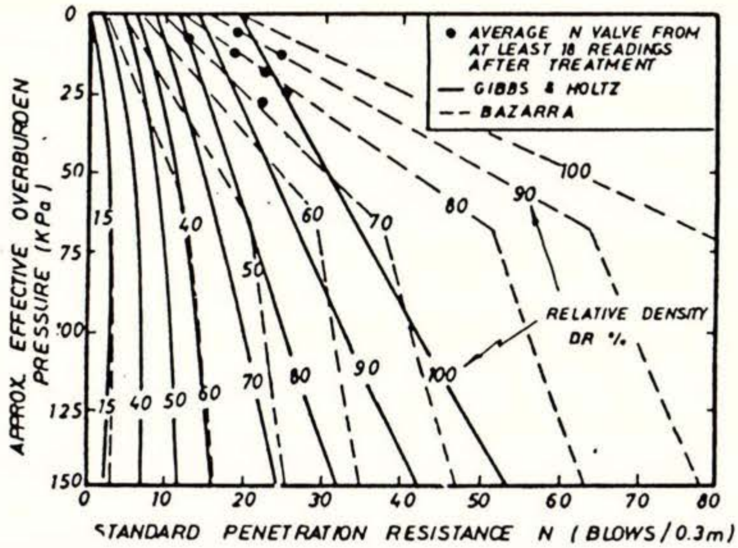


Figure 3.2: Correlation between relative density, effective overburden pressure and standard penetration resistance. (Ramaswamy & Yong, 1982)

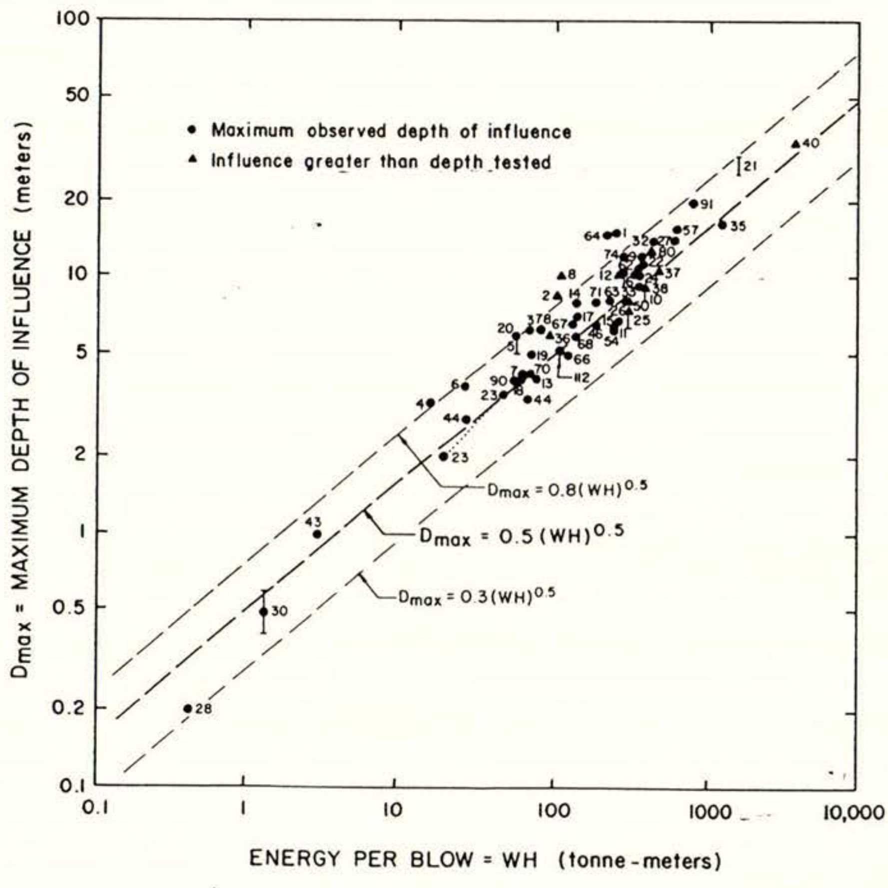


Figure 3.3: Trend between apparent depth of influence and energy per blow (Numerali refer to sites listed in Mayne et al., 1984)

3.4 Energy Application Criteria

3.4.1 General

The literature gives many examples of the two typical energy application criteria:

- energy/drop
- energy/unit area

However, there is often confusion with regard to the latter. It is not always clear if the energy is per pass or total energy (i.e. all passes) and sometimes the area is taken as that of the prints rather than the total area of the site. Unless otherwise stated, it will be assumed that energy/unit area means total energy per unit area of the site.

3.4.2 Energy/Drop

As discussed above, the energy/drop is governed by the required depth of improvement. The lower end of this scale tends to be above about 16 t-m because conventional means of densifying deposits less than 2 m deep would be more efficient than DC. The upper limit is dictated by practical considerations (e.g. size of lifting crane or gantry). Standard crawler cranes can lift weights of about 25 tonnes through heights of about 30 m, giving typical upper energy limits for standard equipment of about 750 t-m/blow. However, the Ménard special purpose "Megatripod" and "Gigamachine" managed drop energies of 1,600 t-m and 4,000 t-m respectively.

3.4.3 Energy/Unit Area

Whereas the required energy/drop is governed by the required depth of improvement, the energy/unit area is linked to the overall degree of improvement. Both theory and practice suggest that there would be a "saturation energy intensity" (a term introduced by Ménard), above which there would be no measurable improvement. Clearly there would be no advantage in applying more energy once the entire volume of the ground had achieved a relative density of 100 percent. Lo et al., (1990) have suggested a relationship between this saturation energy, the energy/blow and some parameter which characterises the initial conditions of the ground (e.g. pressuremeter limit pressure). Such a relationship would be very useful for design purposes but obviously more research is required in this respect.

Leonards et al., (1980) have used a parameter which is the energy/unit area times the energy/drop, and have shown that ground improvement (as characterised by average cone resistance) increases linearly with this parameter up to a limiting value (see Figure 3.4). This again is a useful relationship but is based only on seven case histories and also depends on the energy/blow being constant throughout the ground treatment.

Mayne et al., (1984), in their study of 124 case histories, show that unit energy values are usually between 150 t-m/m^2 and 400 t-m/m^2, with the commonest at around 200 t-m/m^2. Fan et al., (1988) state the Chinese practice for densifying liquefiable sands is as follows:

Depth of Treatment	Energy/unit area (t-m/m^2)
Up to 6 m	170 - 220
6 m to 8 m	270 - 320

3.5 Drop Arrangement

3.5.1 Number of Drops/Point

The literature gives many case histories, with the drops/point varying from 2 to 20. Mayne et al., (1984) present information from nine case histories in the form of crater depth plotted against number of drops and then crater depth normalised by square root of energy/blow plotted against number of drops (see Figure 3.5). This latter plot gives a relatively narrow band for the data.

The case histories indicate that the process is most efficient with the number of drops/ point within the range 4 to 8. However, it should be remembered that the number of drops/point will be governed by other factors such as:

- Energy/blow
- Energy/unit area
- No. of passes
- Grid spacing

3.5.2 Drop Point Spacing

Despite a lack of hard scientific data, it is now generally accepted that the drop point grid spacing is related to the required depth of improvement (e.g. Gambin, 1984 and Mayne et al., 1984).

The initial grid spacing should be at least equal to the thickness of the layer to be densified. It is important that the spacing is not too close during the first passes because this could result in a raft of dense material at an intermediate level, making it impossible to treat loose materials below.

Subsequent passes tend to have closer spacings and lower energies per blow, until the final "ironing" pass which typically would have slightly overlapping prints.

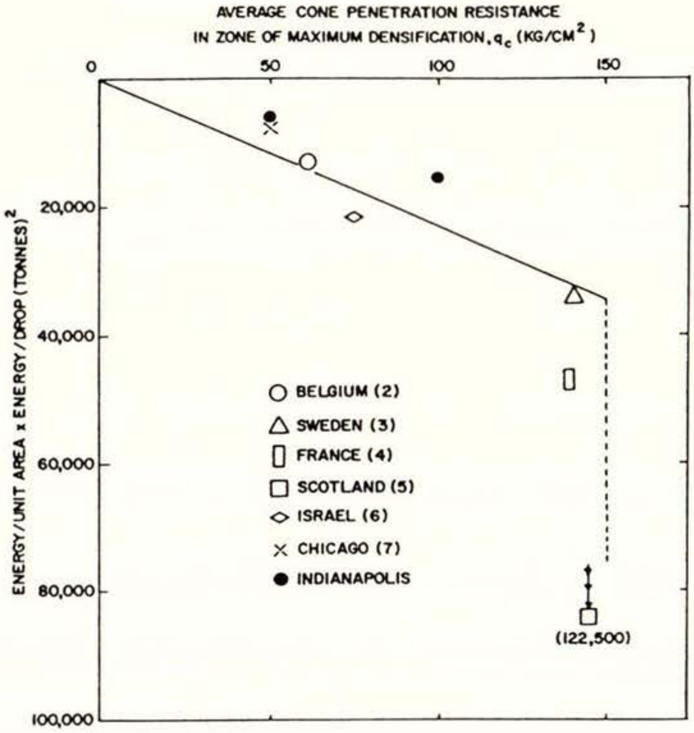


Figure 3.4: Limits of achievable ground improvement (Leonards et al., 1980)

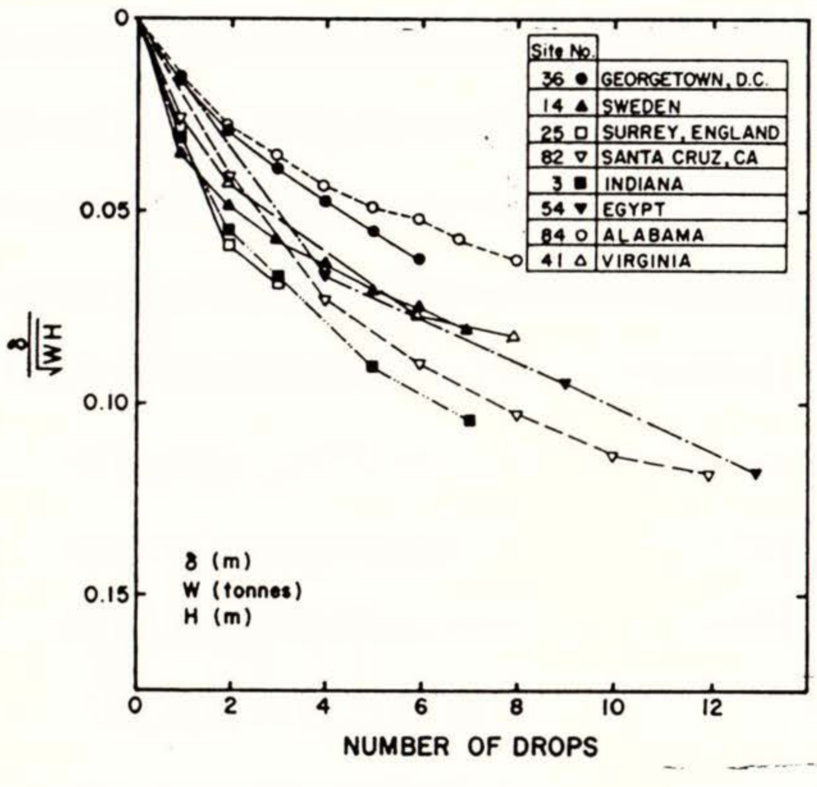


Figure 3.5: Drops per point and normalised crater depths (Mayne et al., 1984)

The literature gives several examples of initial grid spacing (e.g. Elson & Greenwood, 1986 : 5 to 10 m). However, no publication reviewed for this study has detailed the complete drop point arrangement for an entire project. The information available is all in terms of generalised statements similar to those above.

The exception to the practice of relating initial grid spacing to treatment depth is that of Chinese practice (Fan et al., 1988). There they adopt a spacing for treatment of sands of 4b, where b is the characteristic dimension of the weight. However, they also consider that just one pass is often sufficient for sands, which is contrary to the experience and research of almost all other practitioners.

3.5.3 Number of Passes

The literature is very vague on the number of passes considered necessary or used in case histories. Apart from the Chinese (Fan et al., 1988), who maintain that sands require only one to three passes, no one else makes a clear statement on this issue.

However, the number of passes will depend on the required total energy/unit area and other factors such as depth of improvement and grid spacing. There are several generalised statements to this effect in the literature (e.g. Gambin, 1984 and Mayne et al., 1984).

3.6 Weight Properties

Weights are usually square, circular or octagonal in base shape (Mayne et al., 1984). They are normally made of concrete or steel shells filled with concrete up to 8 tonnes, and of steel plates bolted together to 170 tonnes (Gambin, 1984).

Jessberger and Beine (1981) attempted to derive a theoretical relationship for the most effective base area of the weight, but this is not of much practical use yet.

Various case histories give some weights properties, e.g.:

- Charles et al., (1981) : 15 t weight of 4 m^2 base area and 13.5 t weight of 6.5 m^2 base area for final passes
- Ghosh & Tabba (1988) : 16 tonne weight of 2 m x 2 m
- Hartikainen & Valtonen (1983) : 12 tonne cylindrical weight of 1.9 m dia x 1.55 m height: concrete encased in 8 mm steel shell
- Kummerle & Dumas (1988) : 13.5 tonne octagonal weight
- Mori (1977) : 12 tonne weight of 3 m^2 base area

Chinese practice (Fan et al., 1988) uses a "thumb rule" of 2.5 to 4 tonnes per unit base area of weight. Elson and Greenwood (1986), however, state that normal western practice results in a mass per unit base area of 4 to 5 tonnes/m^2. This means that 15 to 20 tonne weights tend to be about 4 m^2 in base area (typically 2 m x 2 m square).

3.7 Monitoring and Evaluation Criteria

The effectiveness of DC is usually monitored in one or more of the following ways:

(A) Measurements of soil properties with depth, before and after treatment, using:

(i) Pressuremeter tests to give modulus (E_{PM}) and limit pressure (P_L)
(ii) Standard Penetration tests (SPT's)
(iii) Cone Penetration tests (CPT's)

(B) Measurement of average overall settlement of site

Because Techniques Louis Ménard developed DC into a marketable construction technique and also introduced the pressuremeter as a working instrument, a considerable amount of the initial monitoring data are in terms of the pressuremeter modulus and limit pressure. However, as others adopt the DC method, SPT's and CPT's are tending to become the preferred methods of monitoring.

Although the pressuremeter produces both deformation and strength properties, SPT and CPT methods tend to be favoured for projects with liquefaction considerations because:

(a) the criteria for liquefaction potential are developed in terms of SPT or CPT data

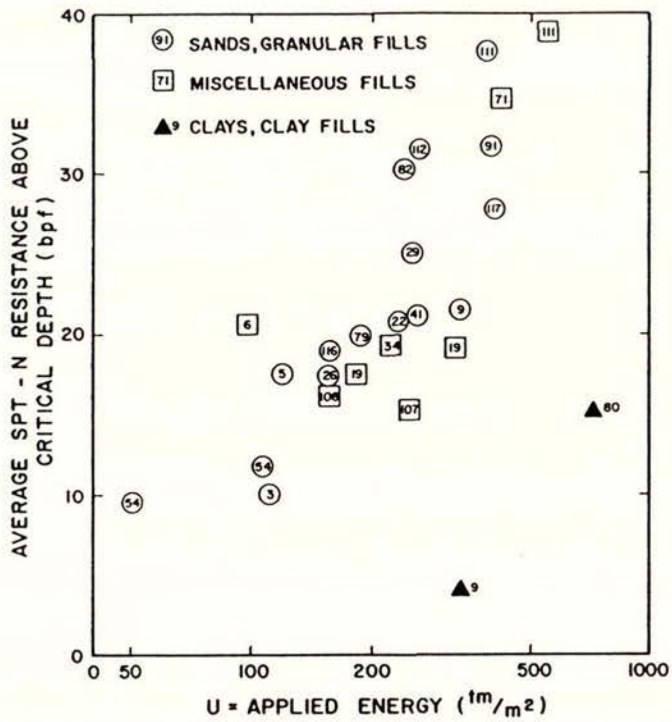
(b) the SPT and CPT are simpler and quicker than pressuremeter tests and provide a more continuous profile

Mayne et al., (1984) provide summaries of data from their review of 124 case histories and show that all parameters tend to improve with increasing applied energy per unit area (see Figure 3.6). These plots would provide a useful guide for upper and lower bands of achievable values.

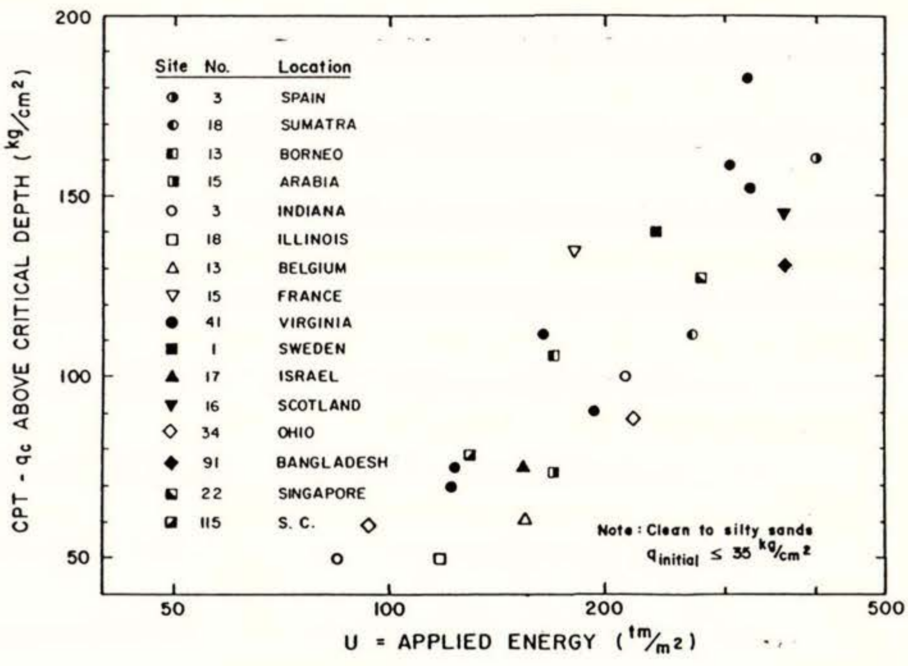
Ramaswamy and Yong (1982) provide a useful discussion on use of both SPT and CPT methods and conclude that the latter is preferable but a method of relating cone resistance to relative density and depth must be included.

Piezometers are sometimes installed to measure pore pressure response during DC (Ménard & Broise, 1975). However, this is now not usual because the instruments, which must necessarily have a very quick response time and are therefore expensive, are easily damaged.

The measurement of overall site settlement can give a useful guide to the achieved average densification. This can also be done per pass to give an "efficiency" of each pass (Juillie, 1980). Total volume change depends on the initial state of the soil but 5% to 10% is usual (Elson and Greenwood, 1986 and Gambin, 1984).



(a) SPT results



(b) CPT results

Figure 3.6: Applied energy - degree of improvement relationship (Mayne et al., 1984)

3.8 Time Between Passes

This aspect is not well covered by the literature. There are several references to the need to wait "... several weeks" (e.g. Ménard & Broise, 1975; Gambin, 1984 and Fan et al., 1988) for pore pressures to dissipate when DC is applied to cohesive soils. The implications (by omission) is that there is no time issue with sands. Fan et al., (1988) indicate that the "rest period" between passes for treatment of sands would be "several minutes".

3.9 Time and Rate of Progress

Despite many detailed case histories now available in the literature, very few of these give details of times and rates of progress for DC treatment.

Gambin (1984) makes a generalised statement that "... production rates can be as high as one hectare per month", but it is not clear if this is for sands or includes fine grained soils.

Mayne et al., (1984) give one example for a loose silty sand site for which 13 hectares (33 acres) were treated in 6 months using 3 cranes (i.e. approximately 0.75 hectares/month per crane).

Meynard and Broise (1975) mention briefly a case history with a "sandy silt" site for which a rate of 2.2 hectares/month was achieved, though details of plant are not given.

One very useful case history is described by Yarger (1986) for a major highway project in the USA. The relevant time and rate of progress information can be summarised as follows:

	Westbound Carriageway	Eastbound Carriageway	Total
Total time:			
• (days)	41	33	
• (weeks)	6	5	
No. of working days	30	25	55
Treated area:			
• (m^2)	45,600	47,200	92,800
• (ha)	4.6	4.7	9.3
No. of drops	29,366	31,146	60,512
Rate per week (ha/week)	0.78	1	
Rate per working day (m^2/day)	1,520	1,888	1,687
Drops per working day	979	1,246	1,100

These rates were achieved using two rigs and two 15 t weights; being representative of a reasonably high energy case.

For such conditions, therefore, an average figure of 550 to 600 drops per rig per working day would be a reasonable guide to the achievable rate of working.

Another example is given by Cleaud et al., (1983) who describe a large project with predominantly sand soils to be improved to about 10 m depth. Work was carried out at 2,500 m^2/day, using 6 rigs, working two shifts and 6 days per week. The 15 hectares of the site were treated within 2.5 effective work months (60 working days) with a total of 145,000 drops. This case therefore suggests an achievable rate of working of about 420 m^2/day per rig and 400 drops/day per rig.

3.10 Costs

Actual costs are not of much use to this study as they depend very much on location, time, size of project and many other factors. However, costs in comparison with other methods of treatment would be very relevant.

Yarger (1986) presents cost data for the large project described above, carried out in USA in late 1984/early 1985 (all costs in US$):

		Westbound	Eastbound	Total
Treated area (m^2)		45,600	47,200	92,800
Total drops		29,366	31,146	60,152
Extra drops (special provision)		1,236	1,718	2,954
Cost of extra drops	($10/drop)	12,360	17,180	29,540
Cost of regular DC	($8/drop)	366,450	379,140	745,590
Total Cost	($)	378,810	396,320	775,130
Final cost per drop	($)	12.90	12.72	12.81
Final cost per sq.m	($)	8.31	8.40	8.36

The author compares these actual costs with an alternative option for subexcavation and replacement as follows:

	Engineer's Estimate US$	Actual bid or cost (US$)
Subexcavation to 6 m	3,078,800	3,393,840
DC + Surface compaction	1,129,030	808,510
TOTAL SAVINGS	**1,949,770**	**2,585,330**

For this case, therefore, DC proved to be less than a quarter of the cost of subexcavation to 6 m depth with replacement and recompaction.

We have found only one other reference which discusses costs of DC. Lukas (1980) describes eight case studies between 1971 and 1979 in the USA; comparatively low energy examples with depths of treatment generally between 3 and 5 m. For these, the costs of treatment ranged from US\$5.4/m^2 to US\$10.75/m^2. Alternative designs were priced much higher. Removal and replacement with compacted fill to a depth of 3 m was priced at 3 to 5 times the DC cost. The cost of deep foundations at one of the sites was priced at 10 times the DC cost and at another site at 3.5 times the DC cost.

3.11 Vibrations

Many researchers have been concerned with ground vibrations due to DC and the potential damage or nuisance effects (e.g. Ménard and Broise, 1975; Leonards et al., 1980; Lukas 1980; Gambin 1984). Most of these have measured the vibrations and compared their results with damage criteria proposed by Wiss (1967) for pile driving.

Mitchell and Katti (1981) point out that for a given energy, peak particle velocities (ppv) tend to be less for DC than for pile driving. Most authors conclude that ground vibrations due to DC are of little consequence because ppv at the site boundary is generally very much less than the 50 mm/sec damage threshold suggested by Wiss (1967).

However, the key paper on this subject is that of Mayne (1985) who has collected data from 12 sites and given a number of important comments, warnings and recommendations, including:

(a) Vibrations from DC are characterised by low frequency waves (2 to 20 Hz) which are potentially more damaging than high frequency waves

(b) The DC frequencies may be below the frequency range of commercially available measuring equipment

(c) Vibration measurements are taken in three mutually orthogonal directions simultaneously but damage criteria should be compared with either the maximum single value on the "true vector sum" (TVS) and not the "pseudo vector sum" (PVS), (which assumes that each orthogonal component is a maximum simultaneously)

(d) Data showing attenuation of ppv with distance from source are presented in various ways (see Figures 3.7)

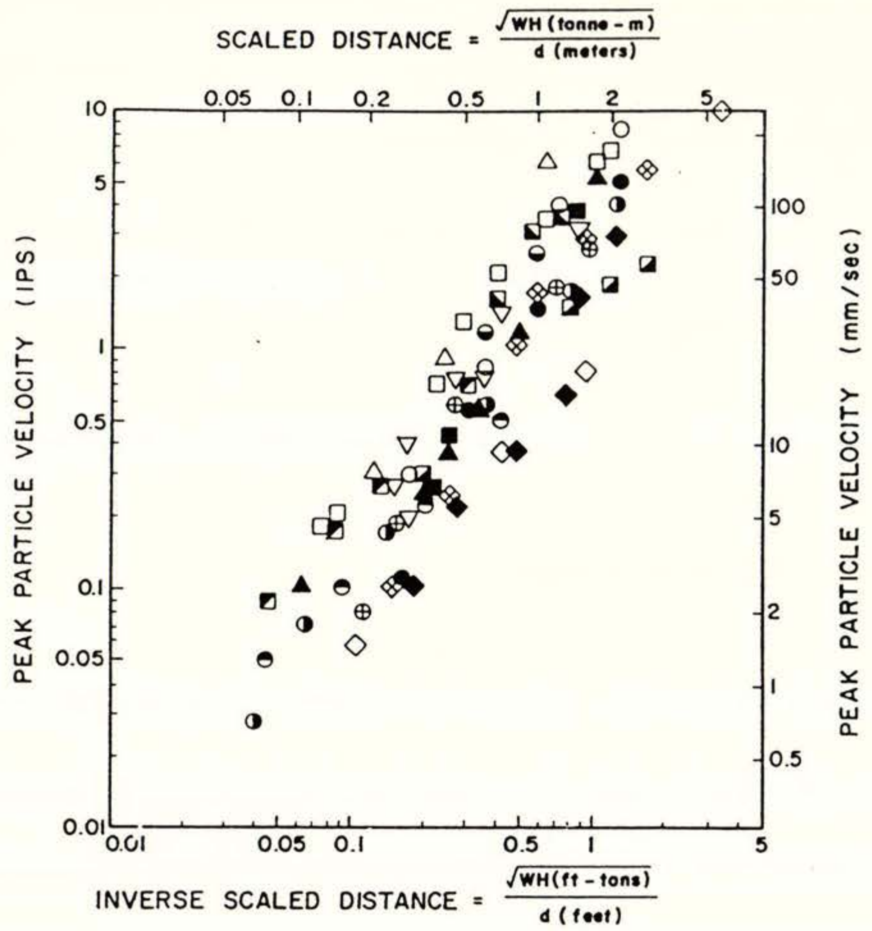
Since Mayne's paper (1985), others have found the same trends. Kummerle and Dumas (1988) measure ppv at site boundaries "... many times lower than the accepted safe limit ..." and Byongmu Song (1988) provides the attenuation relationship shown in Figure 3.8. For this latter study, the author investigates the effects of an open ditch on vibration amplitudes. It can be seen that the ditch does not have a very significant effect.

Chinese experience (Fan et al., 1988) suggests that vibrations are not so much a problem, even very close to buildings, as the effects of induced settlement. This, however, may reflect such factors as type of housing, uses and finishes.

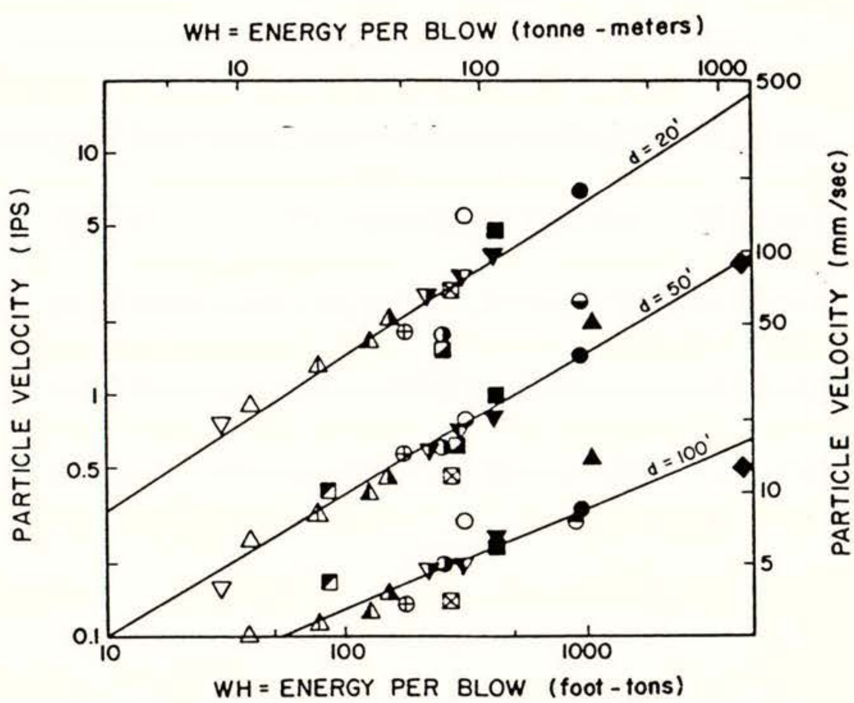
3.12 Miscellaneous Aspects

The literature provides information on various other aspects of DC, including:

- Practical details of equipment
- Effects of time on ground improvement
- Comparisons with other ground improvement techniques
- Required lateral extent of treatment



(a) Attenuation for normalised distance



(b) Relationship between PPV, energy per blow and distances

Figure 3.7: Vibration attenuation data (Mayne, 1985)

3.12.1 Equipment

Information on lifting and compacting equipment is not given in detail. Table 1 summarises some of the published information:

Table 1
Summary of Compaction Equipment

Reference	Equipment details	Tamping Weight (tonnes)	Max. Height of drop (m)
Kummerle & Dumas (1988)	Manitowac 3900 crawler with 100 foot boom	15	21
Hartikainen & Valtonen (1983)	Terasmies 43E; 43t weight increased to 50t	12	12
Ghosh & Tabba (1988)	120 tonne Manitowoc - crawler mounted	16	20
Juillié (1980)	Special purpose "Megatripod"	45	23
Yarger (1986)	Special purpose "compactors" weighing 103 tonnes moved by tracked base transporters	15	21
Gambin (1983)	Special purpose "Gigamachine"	170	22
Cleaud et al., (1983)	200 t crawler cranes	18	28

3.12.2 Time Effects

Several authors have noted the time effects. To quote Mitchell and Katti (1981):

> "More and more evidence is becoming available to indicate that time-dependent increases in strength and decreases in compressibility develop after densification by any of the deep compaction methods. Because these effects continue over periods of many weeks or months, they cannot be explained in terms of pore pressure dissipation, which continues only for periods of several minutes at the most in the case of clean sand. The aging effect has been shown to give substantial increase in the strength of sands under cyclic loading.

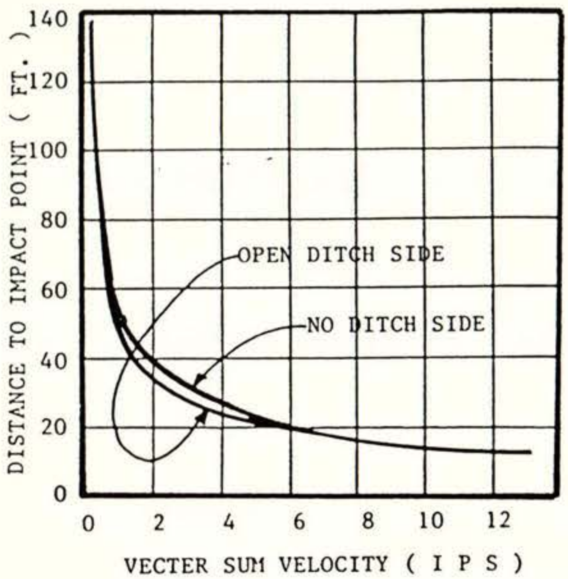


Figure 3.8: Effect of trench on vibration attenuation (Byongmu Song, 1988)

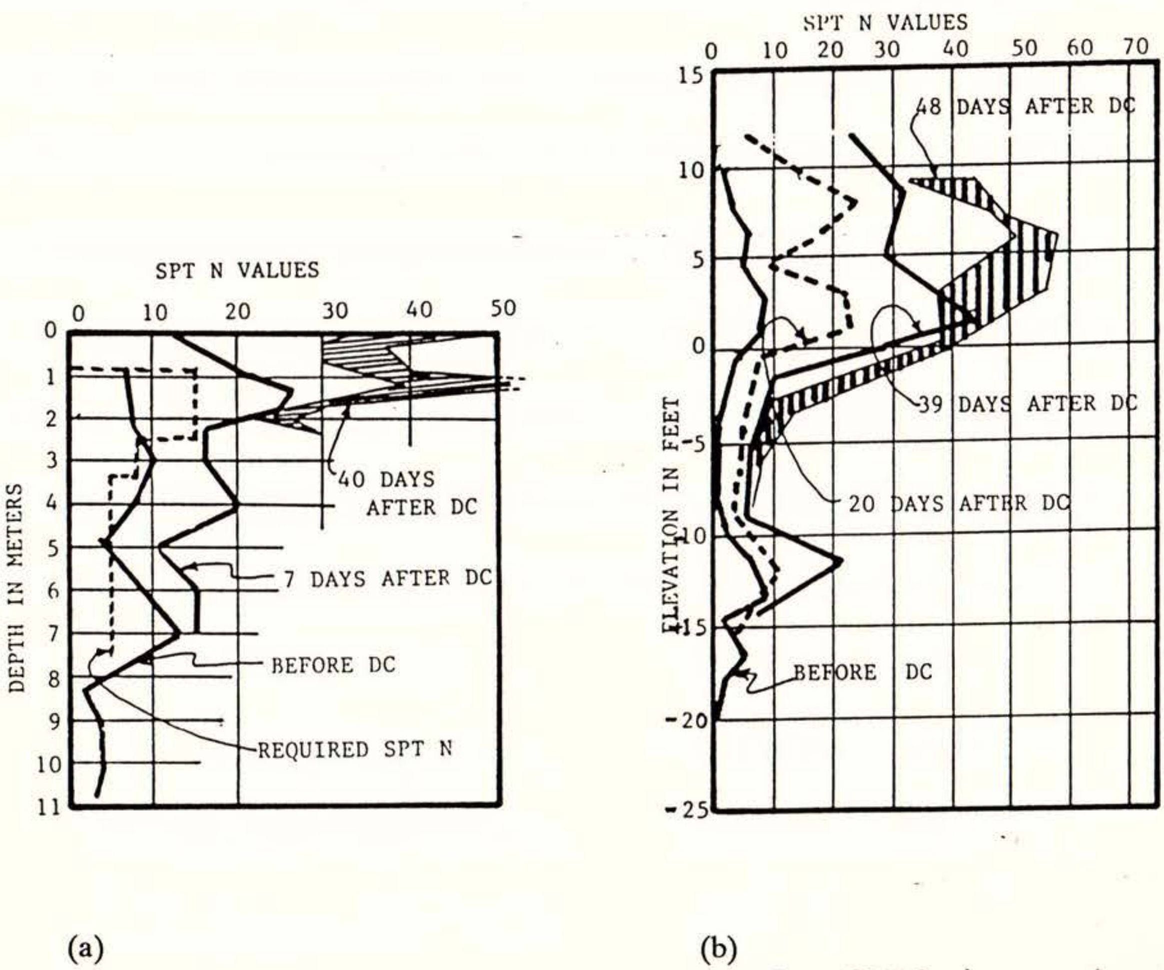


Figure 3.9: Examples of improvement with time (Byongmu Song, 1988)

> Although a number of hypothesis have been advanced to explain this behaviour (e.g., thixotropic hardening, chemical cementation, the effects of dissolved gases), the mechanism is not yet completely clear. From a practical standpoint, however, it would be reasonable to conclude that evaluations of the ground shortly after the completion of deep densification will give conservative results."

Solymar and Reed (1986) note that a "very substantial" increase in densification occurred after two months after treatment by blasting. DC also produced a "substantial but smaller" effect. These authors point out that ... "This strength gain should not be confused with the observed phenomena of pore pressure dissipation ..."

A dramatic example of degree of improvement with time is given by Dumas (1986), as shown earlier in Figure 3.1. Another example is given by Byongmu Song (1989), shown in Figure 3.9.

3.12.3 Comparisons With Other Techniques

Solymar and Reed have compared the technical characteristics of several different methods of deep densification of sandy soils. The following summarises some of their conclusions:

(a) Impact compaction (DC):

> "A characteristic of the method is the non-uniform improvement in the vertical direction ..." . However, the applied energy was constant for all passes and not reduced for later passes with closer spacing.

(b) Deep blasting:

> Only suitable method for densifying sands at depths between 30 m and 45 m but specialist expertise and experience is very limited.

(c) Vibrocompaction:

Shows relatively low compaction at point of penetration, improving laterally to maximum effect at about 2 m distance and decreasing to no effect at 3 m distance.

(d) Compaction piling:

Tests showed that the compaction within the same piles was very good and that densification elsewhere in the liquefiable soils was uniform.

The authors do not comment on cost effectiveness and their overall preferences are not clear, though there is some hint that they favour the compaction piling.

3.12.4 Lateral Extent of Treatment

Mitchell and Katti (1981) address the question of what happens if, during an earthquake, soil surrounding a densified zone liquefies. They recommend that the densified zone should extend laterally from the foundation, a distance at least equal to the thickness of the layer being densified.

4.0 NEW ZEALAND EXPERIENCE

4.1 General

Experience with DC in New Zealand is extremely limited. At time of writing of this report, only two known applications of the technique have been attempted in this country. The author's company has also designed and supervised a DC project in Western Samoa using expertise and contractors from New Zealand. There are several other prospective projects in planning stages for which DC is currently under consideration, notably the proposed Museum of New Zealand in Wellington and the proposal Naval Museum in Auckland.

The following sections present brief summaries of the knowledge gained from the experience to date. The details of the case studies are presented in the appendices to this report.

4.2 Case Study: Waiwhetu Terminal

The only known successful application of DC in New Zealand to date relates to the ground improvement works carried out for BP Oil NZ Ltd at Waiwhetu, Lower Hutt. The details of this case history are presented in Appendix D and the essential facts and points of interest are summarised as follows:

- Location : Waiwhetu, Seaview, Lower Hutt
- Facility : Oil storage terminal
- Owner : BP Oil NZ Ltd
- Purpose of DC : To reduce potential for liquefaction of sands
- Improvement Depth : 5 m
- Energy/drop : 100 t-m (max.)
- No. of drops/point : 6
- Initial grid spacing : 6 m
- No. of passes : 4

• Weight properties	:	Total weight =	8 t & 9.14 t
		Base area =	3.06 m^2 & 2.25 m^2
• Monitoring methods	:	- SPT	
		- CPT (mechanical and electrical)	
		- crater depth	
		- overall average settlement	
• Time between passes	:	1 day (typically)	
• Rate of progress	:	550 m^2/day or 68 m^2/hr	
• Costs	:	\$17/m^2 - \$29/m^2	

Figure 4.1 summarises the degree of improvement achieved in the initial trial in terms of average cone resistance for each metre. Also shown are the target cone resistance values for both the Design Basis Earthquake (DBE - 150 year return period) at the Maximum Credible Earthquake (MCE). Clearly, on the basis of these results, the potential for liquefaction under even the MCE can be substantially eliminated.

Figure 4.1 also shows that the relationship for depth of improvement (d):

$$d = k\sqrt{Wh}$$

for a weight "W" (tonnes), falling through a height "h" (metres), gives a value of k=0.5 for this site, which agrees well with published literature.

The time effects noted in the literature (Mitchell and Katti, 1981) were noted at this site. Tests carried out 52 days after compaction were substantially more favourable than the tests carried out after 8 days. On average, cone resistance after 52 days was about 15 percent higher than for the 8 day tests.

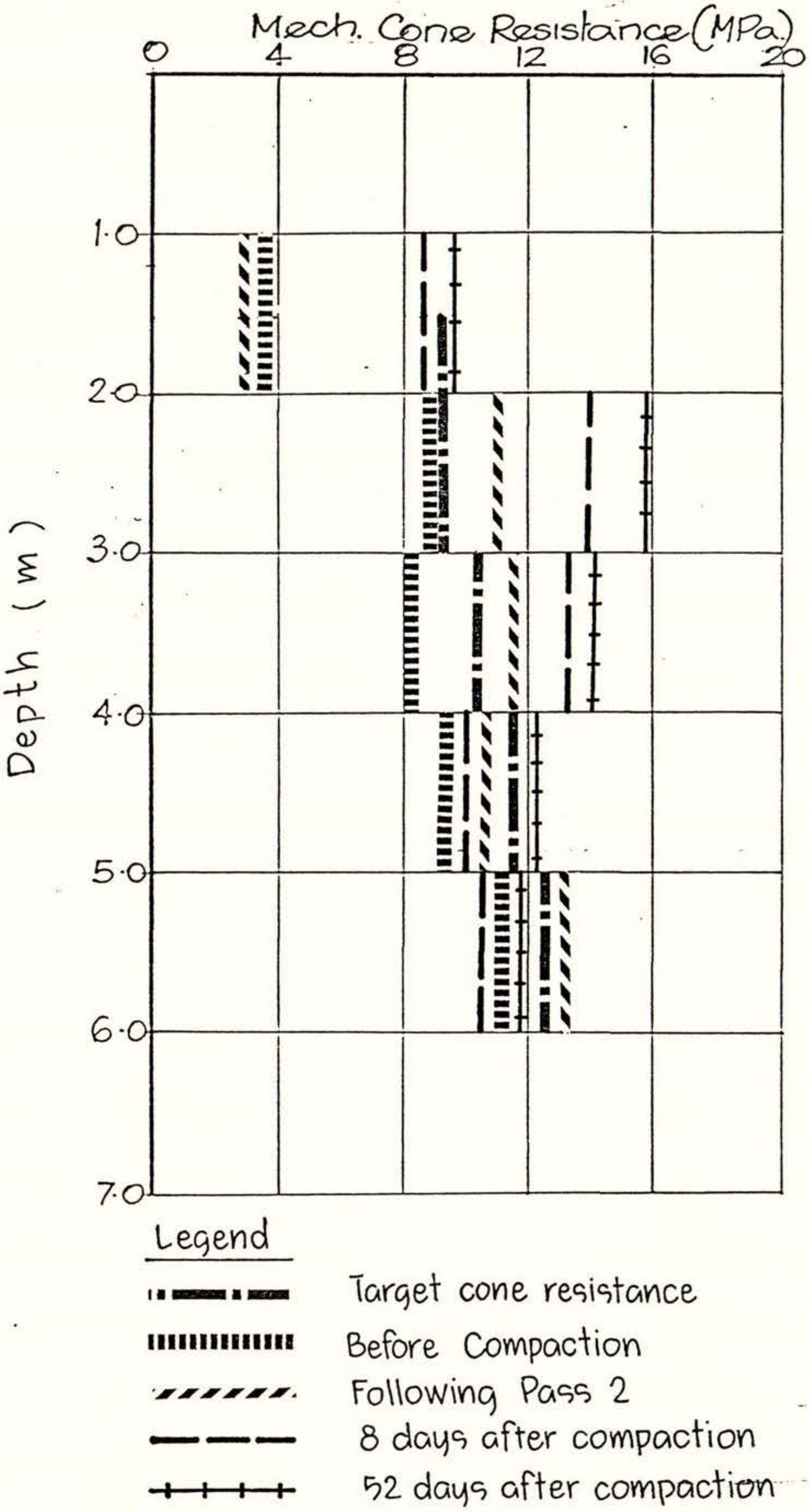


Figure 4.1: Degree of improvement: Waiwhetu case study

At planning stage, various alternative methods of ground treatment were investigated and costed as follows;

Method of Treatment	Estimated Total Cost $M	Estimated Unit Cost $/m^2
DC	0.6	30
Vibrocompaction	1.5	75
Excavation and backfilling	> 2.0	> 100

The estimated cost for DC of \$30/m^2 should be compared with the actual costs of \$17/m^2 to \$29/m^2. Estimates for the alternative methods of treatment were based either on quotations from specialist contractors or on actual rates for work in the immediate vicinity at Seaview, and would therefore carry a high degree of reliability.

A vibration study was carried out during the trial for which results are summarised on Figure 4.2. These show that peak particle velocities (PPV) had reduced to less than 5 mm/sec at a distance of about 10 m from the point of impact. This value of PPV may be compared with the commonly accepted (though not necessarily valid) value of 50 mm/sec as being the limit before damage to buildings would result.

4.2 Case Study: Apia, Western Samoa

A small scale DC project was carried out recently in Apia, Western Samoa to improve the site for a proposed 7-storey building. Details are given in Appendix D and the main points of interest can be summarised as follows:

(a) Materials to be improved were loose coral sands of 3 to 4 m depth, overlying coral.

(b) DC involved dropping 8-t weight through 8 m with 6 drops/point and 4 passes

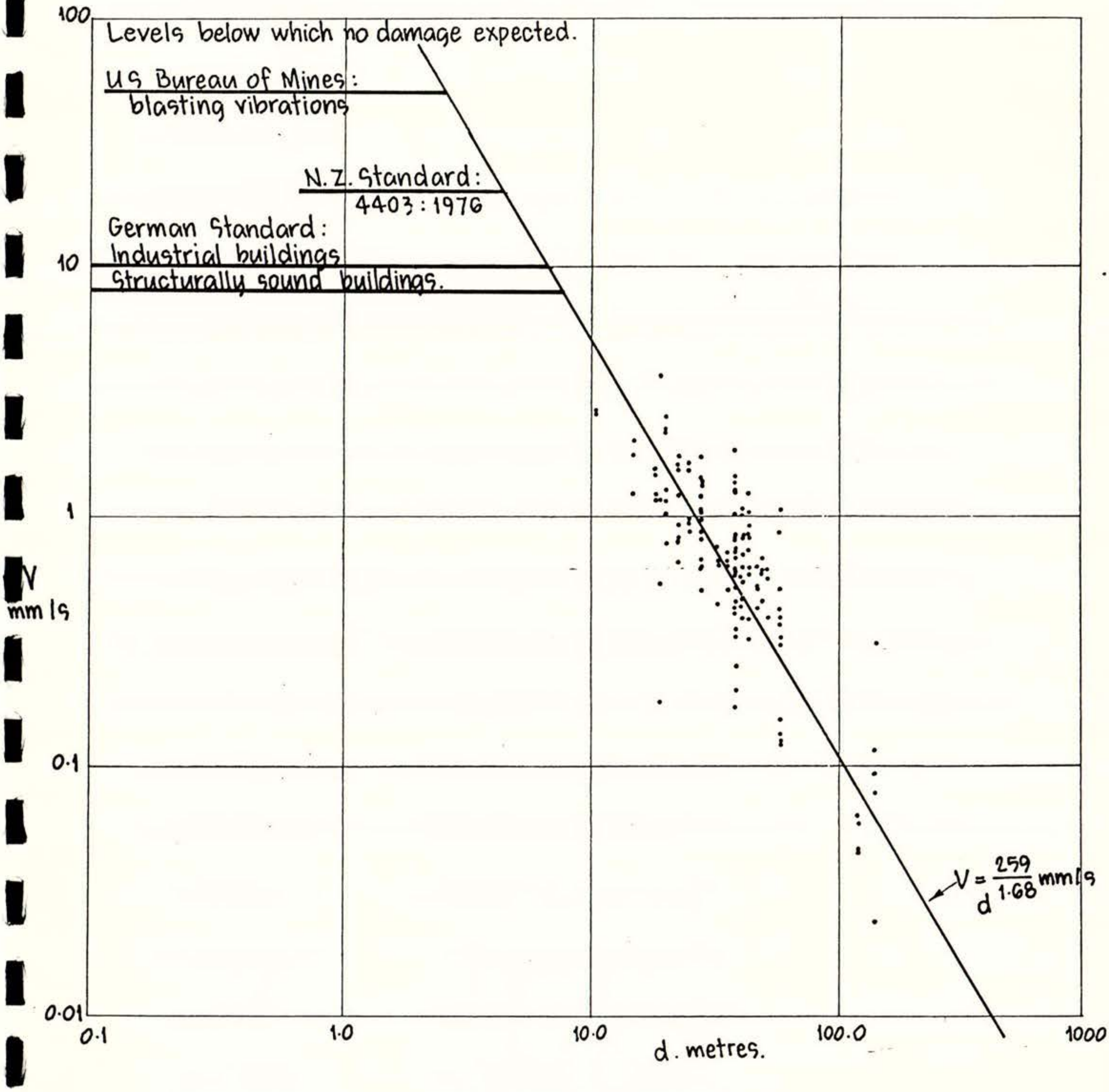


Figure 4.2: Vibration measurements : Waiwhetu case study

(c) Total time to complete the 50 m x 30 m site was 6 working days, giving an average working rate of 250 m^2/day.

(d) The primary method of compliance testing was by Scala penetration testing.

(e) The total cost of the DC treatment, including construction of two weights, was NZ$162,000, giving an average unit cost of NZ$108/$m^2$.

A small scale project, therefore, results in a substantially higher unit cost, mainly because of the cost of establishment and construction of the weights. It should be noted that two weights were constructed for this project, the second merely as a backup which was never used. However, if a weight breaks up and no backup is available, then there would be considerable extra costs for standing time of plant or demobilisation/remobilisation whilst waiting for a new weight to be constructed and cured for at least 28 days.

4.3 Case Study: Ohaaki

The first known attempt at deep compaction using DC was carried out by Ministry of Works at Ohaaki. This was essentially a research project and a trial of one option for improving foundation conditions for a large cooling tower.

The soils were pumice sands, loose in pockets but welded to rock strength in places. The initial favoured option for site improvement was for excavation of uncemented sands and backfilling with controlled compacted fill. This work had already started at the time the DC trial was carried out, and was proving troublesome due to the very irregular nature of the hard welded materials. The primary purpose of the DC trial, therefore, was to determine if the process would be effective in producing a dense, high strength foundation of relatively uniform stiffness.

A concrete weight of 9.14 t was dropped through a height of 12 m. Results were monitored using cone penetration tests (CPT's), down-hole density testing with a Gamma-Gamma geologger probe and cross-hole seismic tests. Only moderate improvements were observed and were not considered sufficient for the intended purpose and the objectives mentioned above. However, it is now recognised that the testing had probably been carried out too soon after the treatment for maximum effects to be detected.

At the time of writing, the full test data had not been located for inclusion in this report. Although the objectives of the DC work were not specifically for liquefaction mitigation, it would be of interest to pursue this case history further as the application of DC to pumice soils is of particular relevance to many of New Zealand's liquefaction prone regions.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Relative Merits of DC and Other Methods of Ground Improvement

The following conclusions can be drawn from to published literature and experience gained in New Zealand:

(a) DC is most effective in soils which are prone to liquefaction. The converse is also likely to apply, namely: If soils are prone to liquefaction, then DC will be effective in reducing the potential for liquefaction.

(b) DC is not likely to be cost effective with depths of treatment less than 3 to 4 m. It is not good for compacting the top 1 to 2 m for which more conventional methods would be necessary.

(c) The top end of the range is limited by practicalities. Very high energies have been applied to densify soils to 30 m depth, using special purpose lifting gantries. Clearly, this would only be cost effective for large areas. For most situations, commercially available cranes would tend to limit the practical depths of treatment to about 15 m. However, the risk of damage due to liquefaction below this depth is normally very low.

(d) For treatment within the depth range 15 to 25 m, the costs of DC should be evaluated in relation to other methods of ground treatment such as vibro-compaction, vibro-replacement, grouting or piling.

(e) For depths exceeding 30 m, compaction by deep blasting appears to be the only feasible technique. This requires specialist expertise and there is no known experience base in New Zealand.

(f) The cost effectiveness of DC in relation to other methods of ground improvement depends on the area to be treated as well as depth of treatment. For areas greater than about one hectare, unit costs for DC would be about one third to one half that for vibro-compaction, which would be the next cheapest option.

(g) Vibro-compaction tends to be most effective when performed with special-purpose, high energy probes which vibrate in a horizontal plane. Such equipment is no longer available in New Zealand. For large projects it may be cost effective to bring in the equipment from Australia or Singapore. However, it would not be practical to import the equipment for a trial only. Vibro-compaction equipment which produces vibrations in a vertical direction is readily available in New Zealand but the effective radius of treatment is much reduced and hence more compaction points would be necessary. Unit costs, therefore remain high, typically twice the cost of DC.

(h) DC clearly has an advantage over vibro-compaction in situations where obstructions such as boulders or rubble (or even hard clay fill) overlie the target soils.

(i) If liquefaction is considered possible in very silty, fine sands, then DC may not be appropriate. Some researchers claim that DC will consolidate fine-grained soils but the process takes considerably longer than for relatively clean sands. When conducting trials in such conditions it is necessary to wait for sufficient time after the treatment before carrying out the compliance tests.

5.2 Recommendations for Design Procedures

On the basis of published information and the experienced gained in New Zealand, the following procedures for design are recommended:

(a) Energy/drop:

The energy per drop should be selected on the basis of the formula:

$$d = 0.5\sqrt{Wh}$$

where:
Wh = energy/drop
W = weight of weight in tonnes
h = drop height in metres
d = required depth of treatment

(b) Size of Weight:

The weight should have a mass per unit area of base within the range 4 to 5 t/m^2. Weights of 15 to 20 t would therefore have a base area of about 2 m^2.

(c) Energy/unit area:

The total energy to be applied to the site should be determined from Figure 5.1 on the basis of target values for one or more of the following parameters:

- Cone penetration resistance
- SPT "N" value
- Pressure "Limit Pressure
- Pressuremeter modulus

The energy/unit area is generally within the range 150 to 400 $t\text{-}m/m^2$.

(d) Initial grid spacing:

The spacing of compaction points for the first pass should be equal to the required depth of treatment.

(e) Number of passes and grid spacing for each pass:

The number of passes will depend on the initial grid spacing (i.e. depth of treatment) and, to some extent, on the base area of the weight. Figure 5.2 shows the recommended number of passes and Figure 5.3 gives the recommended arrangement of drop points for the various passes.

For most liquefaction problems, it is likely that 6 to 7 passes would be sufficient.

(f) Number of drops per point:

Having established the required energy/drop, energy/unit area and number of passes, the required minimum number of drops follows. However, it is often advantages to monitor the depth of penetration of the weight for each drop to determine if an optimum number of drops/point is apparent.

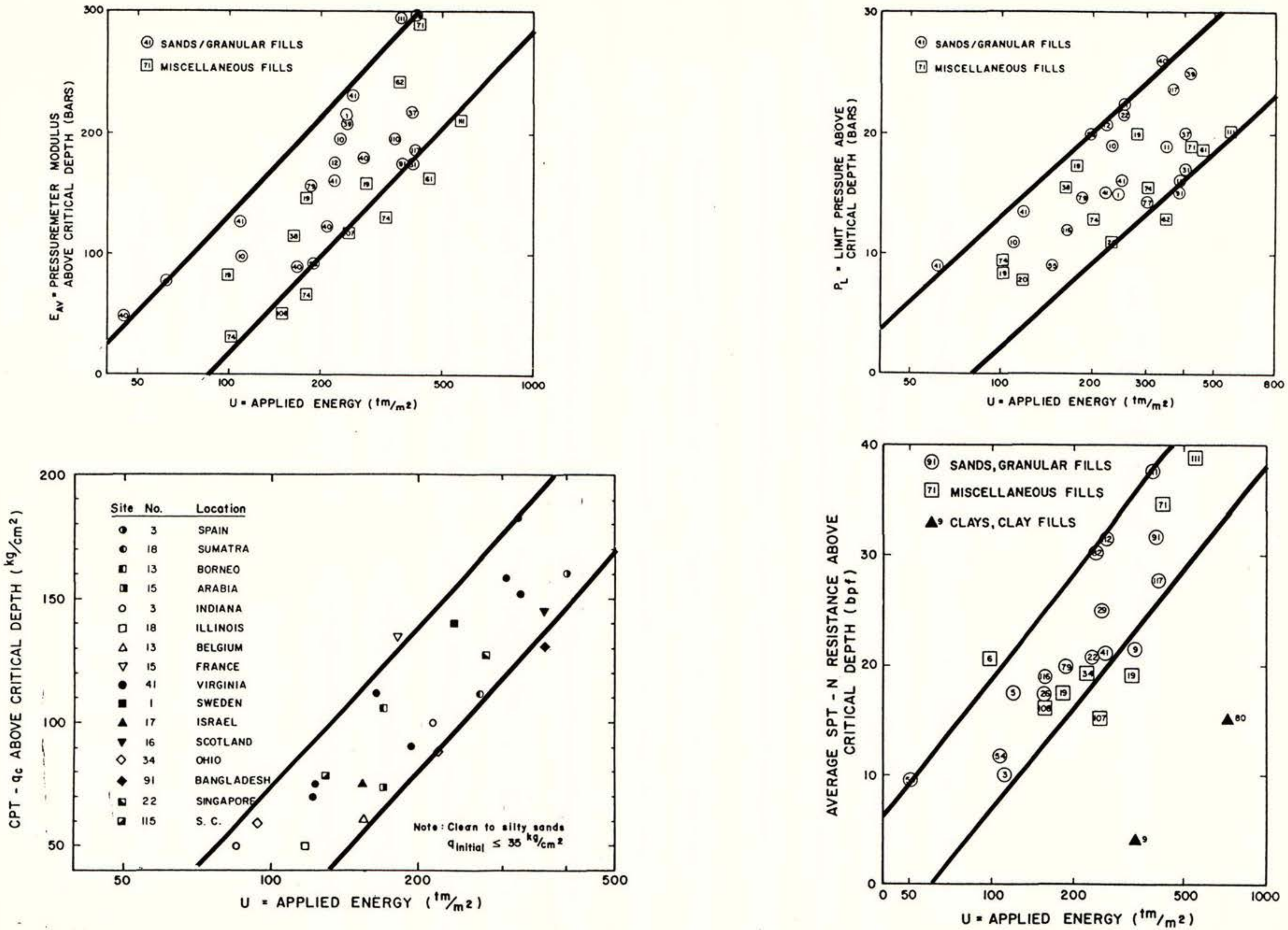


Figure 5.1: Applied energy and target values for pressuremeter, CPT and SPT parameters (after Mayne et al., 1984).

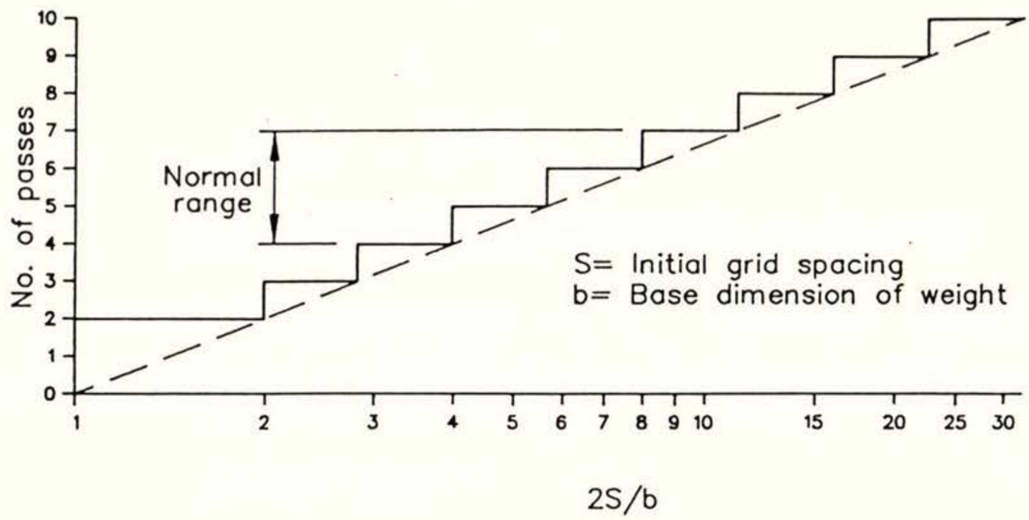


Figure 5.2: Recommended number of passes

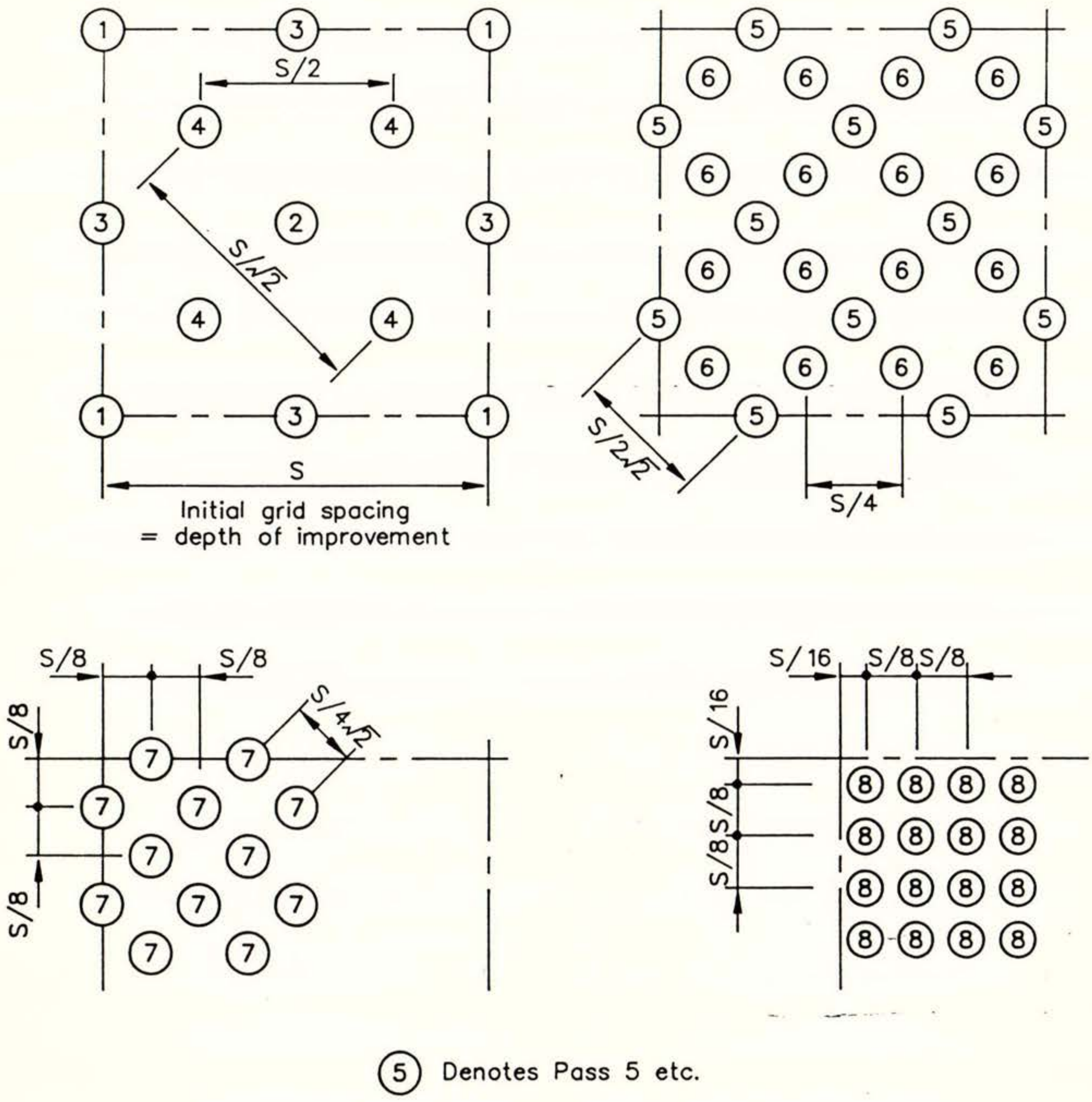


Figure 5.3: Recommended arrangement of drop points

(g) Extent of treated area:

The treated area should be greater than the envelope of facilities to be protected by a margin of width equal to the depth of treatment.

(h) Time between passes:

A period of several hours (e.g. 8 hours) should elapse between compaction at adjacent points in consecutive passes.

(i) Make-up fill:

The average depression or "subsidence" of the site will depend on the initial conditions as well as the depth of treatment and energy input. However, in general, the average subsidence tends to be within the range of 5 percent to 10 percent of the treatment depth.

5.3 Monitoring Procedures and Compliance Testing

It is normal practice to monitor the effectiveness of the DC by the following methods:

(a) "Before and after" tests with one or more of the following:

- Cone penetration tests (CPT's)
- Standard Penetration tests (SPT's)
- Pressuremeter tests

(b) Measurement of crater depths per blow

(c) Average "subsidence" of site per pass

With regard to (a) above, it is advantageous to allow as great a time as would be feasible after treatment, before completing the final compliance testing. A minimum period of one week is recommended but results can be expected to improve steadily up to two months after treatment.

5.4 Vibration Monitoring

If there are buildings or any facilities sensitive to vibrations within at least 100 metres of a compaction point, then monitoring of vibrations is strongly recommended; preferably during a pre-treatment trial.

In carrying out and evaluating the vibration monitoring, the following points should be noted:

(a) Vibrations from DC are characterised by low frequency waves (2 to 20 Hz). Measuring equipment often has non-linear magnification/frequency relationships at frequencies below about 5 to 10 Hz. It is essential that the frequency dependence characteristics for the equipment are known.

(b) Vibrations should be measured in three mutually orthogonal directions simultaneously (vertical, longitudinal and transverse). Results should be presented in terms of the maximum single value of the three directional components (MSV) or the true vector sum (TVS). Typically, the TVS is 10 to 40 percent higher than the MSV.

(c) Low frequency vibrations are potentially more damaging than high frequency vibrations typical of those caused by blasting or pile driving. It is recommended that damage criteria should be assessed on the basis of recommendations by the US Bureau of Mines (1980) summarised in Figure 5.4.

(d) To determine the probable attenuation and peak particle velocity (PPV) relationship with distance at the planning stage, the data presented by Mayne (1985) may be used, as summarised in Figure 5.5.

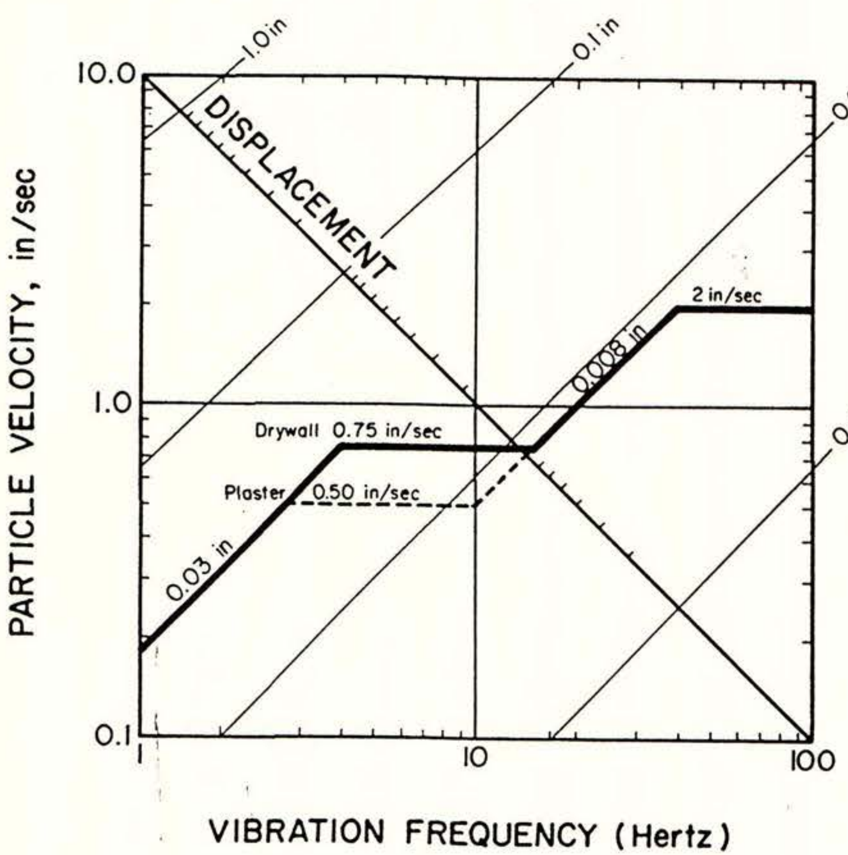


Figure 5.4: Suggested damage criteria (US Bureau of Mines, 1980)

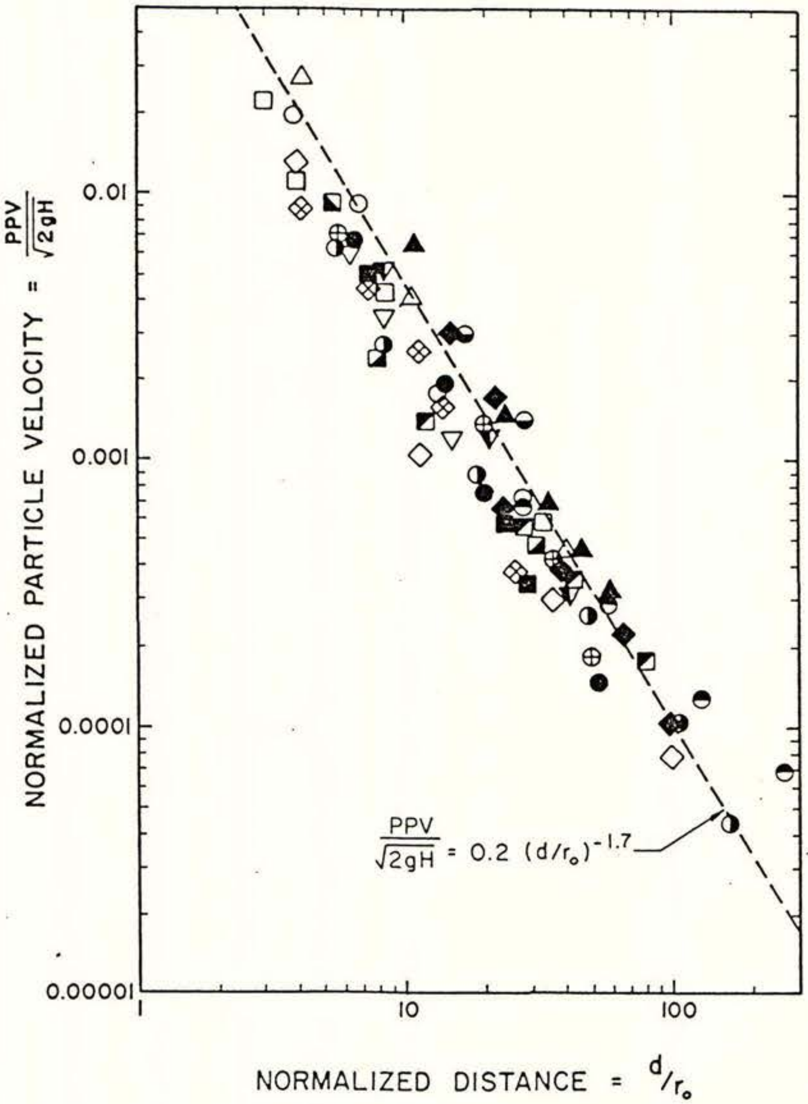


Figure 5.5: Vibration attenuation design guide (Mayne, 1985)

5.5 Further Research and Development

The purpose of this study has been to determine the effectiveness of DC for New Zealand conditions. The main point of difference between New Zealand and the published information arises with volcanic deposits, in particular pumiceous sands and silts. The review of literature and experience in New Zealand has shown that there is no available information on either the susceptibility of pumiceous deposits to liquefy or the potential for densification by DC. The Edgecumbe earthquake (1987) provided evidence of both liquefaction (in the form of sand boils) and densification of pumice sands and gravels (in the form of areal settlement), but it is not clear if the two phenomena are related. More research with regard to the whole subject of liquefaction in pumiceous soils is required. Even the common in-situ tests such as CPT and SPT may require special interpretation in such soils due to the crushable nature of the particles and the very low densities yet high values of internal friction. It is recommended, therefore, that any further research relating to DC on pumiceous sands should be combined with more general research with regard to liquefaction potential.

Stage 2 of this research programme for evaluation of DC for New Zealand conditions would involve a full scale trial for DC on volcanic deposits.

The following possible locations for future DC trials have been identified:

- Rotorua : Pumice sands
- Huntly : Pumice sands
- Whakatane : Pumice gravels overlying fine sands
- New Plymouth : Lahar sands

We have had tentative agreement from the owners of some of these sites to carry out trials. Others prefer to hold formal agreement until written proposals are received detailing exact scope of work and timing.

We have developed costs for a trial assuming that a new weight would be constructed at the site. However, some savings might be possible by transporting an existing weight from Wellington.

Estimated costs for a trial amount to $32,000 and include for the following:

- Investigation before trial:
  - Drilling
  - CPT
  - Laboratory tests

- DC trial:
  - mob/demob of crane and hire
  - construction of weight
  - dozer hire for regrading site
  - stripping topsoil and reinstating site

- Monitoring and testing:
  - CPT after DC
  - level surveys

- Management and report:
  - co-ordination of activities
  - supervision
  - analysis and report

Because of the relatively high cost of such a programme, it would be advantageous to include this trial in the wider research project as discussed above, possibly co-ordinating with other researchers (e.g. Universities).

The following attachments complete this report:

APPENDIX A: BIBLIOGRAPHY AND REFERENCES
APPENDIX B: KEY PAPERS FOR DC
APPENDIX C: NZ CASE STUDY : DC AT WAIWHETU TERMINAL
APPENDIX D: WESTERN SAMOA CASE STUDY : 7-STOREY BUILDING, APIA

APPENDIX A

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TITLE: Unified approach to ground improvement by heavy tamping
AUTHOR(S): Lo, Kwang Wei; Ooi, Peng Lee; Lee, Seng-Lip
CORPORATE SOURCE: Natl. Univ. Singapore, Dep. Civ. Eng., Singapore
SOURCE: Journal of Geotechnical Engineering vol. 116 no. 3 p. 514-527
DATE: 1990
DESCRIPTORS: IMPROVEMENT OF SOIL CONDITIONS

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Discussion of LUTENEGGER, AJ: Dynamic compaction in friable loess
BALLY, RJ; CULITZA, C
J Geotech Eng [New York] 113/11 P1416-1418 1987 JRNL CODE: JGENGI
DESCRIPTORS: ROCK MECHANICS; EOLIAN DEPOSITS

3
TITLE: Ground response to dynamic compaction
AUTHOR(S): Mayne, P. W.; Jones, J. S., Jr.; Dumas, J. C.
CORPORATE SOURCE: Law Eng., McLean, VA, United States; GeoSystems, United States, Geopac, Canada
SOURCE: Journal of Geotechnical Engineering vol. 110 no. 6 p. 757-774
DATE: 1984
LANGUAGE: English
NOTE: J. Geotech. Eng., Paper No. 18932
DESCRIPTORS: ENGINEERING PROPERTIES OF SOIL; IMPROVEMENT OF SOIL CONDITIONS

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Application and limits of soil improvement by dynamic compaction demonstrated on a foundation for a housing compound
WERNICK, E
Geotechnik (Essen) 5/3 P125-138 1982 JRNL CODE: GEOESI
DESCRIPTORS: FOUNDATION ENGINEERING; WATER TABLE

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A field study of the use of the dynamic consolidation ground treatmen t technique on soft alluvial soil
CHARLES, JA; WATTS, KS
Ground Eng (Brentwood) 15/5 P17-22+1 1982 JRNL CODE: GENGII
DESCRIPTORS: SOIL MECHANICS; ENGINEERING GEOLOGY; METHODS

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Dynamic compaction of granular soils
LEONARDS, GA; CUTTER, WA; HOLTZ, RD
J Geotech Eng Div Proc Am Soc Civ Eng (New York) 106/1 P35-44 1980 JRNL CODE: JGEDPI
DESCRIPTORS: ENGINEERING PROPERTIES OF SOIL; FOUNDATION ENGINEERING

7
Dynamic consolidation at Surrey docks
Ground Eng (Brentwood) 12/1 P32-34 1979 JRNL CODE: GENGII
DESCRIPTORS: IMPROVEMENT OF SOIL CONDITIONS; ENGINEERING PROBLEMS OF SOIL

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Dynamic consolidation of soil by a falling weight
HANSBO, S
Ground Eng (Brentwood) 1978 JRNL CODE: GENGII
11/5 P27-30
DESCRIPTORS: SOIL ENGINEERING PROBLEMS; SOIL ENGINEERING PROPERTIES

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Dynamic consolidation
PEARCE, RW
Consult Eng (London) 1977 JRNL CODE: CENGLH
38/4 P43-47
DESCRIPTORS: IMPROVEMENT OF SOIL CONDITIONS

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Sand without footprints (dynamic compaction of foundation sands)
WHITE, TC
Consult Eng (London) 1977 JRNL CODE: CENGLH
41/7 P34-35
DESCRIPTORS: IMPROVEMENT OF SOIL CONDITIONS; SANDSTONES & QUARTZITES

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Dynamic consolidation of rockfill at Uddevalla shipyard
HANSBO, S
In Proceedings of the 9th International Conference o CONF. DATES: 11 JULY TO 15 JULY NO. 77-0129
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P236-241
DESCRIPTORS: ROCK ENGINEERING PROPERTIES; IMPROVEMENT OF ROCK CONDITIONS

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Theoretical and practical aspects of dynamic consolidation
MENARD, L; BROISE, Y
Geotechnique (London) 1975 JRNL CODE: GEOTE
25/1 P3-18
DESCRIPTORS: IMPROVEMENT OF SOIL CONDITIONS

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TITLE: Some engineering properties of stabilized salina soil
AUTHOR(S): Stipho, A. S.
CORPORATE SOURCE: Coll. Eng., Civ. Eng. Dep., Riyadh, Saudi Arabia
SOURCE: Engineering Geology vol. 26 no. 2 p. 181-197
DATE: 1989
Language: English
Descriptors: salina soil; portland cement; lime; dynamic consolidation; sand pillar; Saudi Arabia

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Construction on dolomite in South Africa.
von M. Wagener, F.; Day, P. W.
Jones & Wagener Inc., Rivonia, South Africa.
Environmental Geology & Water Sciences 1986. v 8(1-2) pp 83-89.
Language: English

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LUTENEGGER, A. J.
Dept. of Civ. and Environmental Engrg., Clarkson Univ., Potsdam, NY 13676, USA.
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Language: English

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ZHANG, K. C.; HAO, Z. Z.; ZHOU, K. S.
Survey & Geotechnical Investigation-Research Inst., Ministry of Machine-Building Industry, Xian, China.
1986. pp 777-782.
Language: English

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SPARKS, R. S. J.; HUPPERT, H. E.; KERR, R. C.; McKENZIE, D. P. ; TAIT, S. R.
Dept. Earth Sciences, Univ. of Cambridge, CB2 3EQ, UK.
Geological Magazine 1985. v 122(5) pp 555-568.
Language: English

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Evaluation of densification of sandfill.
in: Improvement of ground. Proc. 8th European conference on soil mechanics and foundation engineering, Helsinki, 1983. Vol. 1
RAMASWAMY, S. D.; YONG, K. Y.
Ed.: Rathmayer, H.; Saari, K.
Nat Univ of Singapore, Singapore.
Publ.: (Balkema)
1983. pp 69-72 2 figs, 2 tables, 7 refs.
Language: English

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TITLE: Analysis of results obtained on a dynamic compaction site
AUTHOR(S): Cleaud, J. J.; Bourdon, L.; Karaki, P.
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering organized by the Finnish Geotechnical Society; Vol. 1
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 19-22
DATE: 1983
LANGUAGE: English

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RAMASWAMY, S. D.; YONG, K. Y.
Ed.: Verruijt, A.; et al.
Dept of Civil Eng, National Univ of Singapore, Singapore.
Publ.: (Balkema)
1982. pp 131-135 5 figs, table, 10 refs.
Language: English

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TITLE: Treatment of collapsing sands by dynamic compaction
AUTHOR(S): Liausu, P.; Varaskin, S.
CORPORATE SOURCE: Menard Soltrait., France
MONOGRAPH TITLE: French contributions to the XII I.C.S.M.F.E.; abstracts
AUTHOR(S): Salencon, Jean (prefacer)
CONFERENCE TITLE: XII international congress on Soil mechanics and foundation engineering
CONFERENCE LOCATION: Rio de Janeiro, Brazil
CONFERENCE DATE: Aug. 13-18, 1989
PUBLISHER: Ec. Natl. Pontes et Chaussees, Noisy le Grand, France p. 43
DATE: 1989
LANGUAGE: English

22
TITLE: Postdensification penetration resistance of clean sands
AUTHOR(S): Mesri, G.; Feng, T. W.; Benak, J. M.
CORPORATE SOURCE: Univ. Ill. Urbana-Champaign, Civ. Eng., Urbana, IL, United States; ATEC Assoc., United States
SOURCE: Journal of Geotechnical Engineering vol. 116 no. 7 p. 1095-1115
DATE: 1990
LANGUAGE: English

23
TITLE: Solution of equation of motion for dynamic compaction of sand using foam propellant
AUTHOR(S): Agarwal, K. B.; Ram, B. Siva
CORPORATE SOURCE: Univ. Roorkee, Dep. Civ. Eng., Roorkee, India
MONOGRAPH TITLE: Eighth symposium on Earthquake engineering
AUTHOR(S): Arya, Anand S. (chairperson)
CORPORATE SOURCE: Univ. Roorkee, Roorkee, India
CONFERENCE TITLE: Eighth symposium on Earthquake engineering
CONFERENCE LOCATION: Roorkee, India
CONFERENCE DATE: Dec. 29-31, 1986
PUBLISHER: Univ. Roorkee, Roorkee, India
vol. 8 Vol. 1 p. 213-220
DATE: 1986
LANGUAGE: English

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TITLE: Soil improvement by dynamic consolidation
AUTHOR(S): Dayal, Umesh
CORPORATE SOURCE: Indian Inst. Technol., Civ. Eng. Dep., Kanpur, India
SOURCE: Indian Geotechnical Journal vol. 18 no. 3 p. 289-297
DATE: 1988
LANGUAGE: English

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TITLE: Changes in ground-water levels due to dynamic compaction of subsoil by hidden underground explosions
AUTHOR(S): Dembicki, E.; Zadroga, B.; Bona, R.; Imiolek, R.; Kisielowa, N.; Semrau, I.
CORPORATE SOURCE: Tech. Univ. Gdansk, Gdansk, Poland
MONOGRAPH TITLE: Groundwater effects in geotechnical engineering--Les actions de l'eau souterraine en geotechnique
EDITOR(S): Hanrahan, E. T. (editor); Orr, T. L. L. (editor); Widdis, T. F. (editor); Maritime Inst., Poland
CONFERENCE TITLE: Proceedings of the Ninth European conference on Soil mechanics and foundation engineering--Comptes rendus du neuvieme congres Europeen de Mecanique des sols et des travaux de fondations
CONFERENCE LOCATION: Dublin, Ireland
CONFERENCE DATE: Aug. 31-Sept. 3, 1987
SOURCE: Proceedings of the European Conference on Soil Mechanics and Foundation Engineering = Comptes Rendus du Congres Europeen de Mecanique des Sols et des Travaux de Fondations vol. 9 Vol. 2 p. 599-602
DATE: 1987
LANGUAGE: English

26
TITLE: Dynamic compaction of loose and hydrocompactible soils on Interstate 90, Whitehall-Cardwell, Montana
AUTHOR(S): Yarger, Terry L.
CORPORATE SOURCE: Mont. Dep. Highw., Helena, MT, United States
SOURCE: Transportation Research Record vol. 1089 p. 75-80
DATE: 1986
LANGUAGE: English

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TITLE: Dynamic replacement and mixing of organic soils with sand charges
AUTHOR(S): Lo, Kwang Wei; Ooi, Peng Lee; Lee, Seng-Lip
CORPORATE SOURCE: Nat. Univ. Singapore, Dep. Civ. Eng., Singapore, Singapore; Singapore Polytech., Singapore
SOURCE: Journal of Geotechnical Engineering vol. 116 no. 10 p. 1463-1482
DATE: 1990
LANGUAGE: English

28
TITLE: Applied study of preventing structures from frost damage by using dynamic consolidation
AUTHOR(S): Han Huaguang; Guo Mingzhu
CORPORATE SOURCE: Heilungkiang Prov. Low Temp. Const. Sci. Res. Inst., Harbin, China
MONOGRAPH TITLE: Permafrost; Fifth international conference
EDITOR(S): Senneset, Kaare (editor)
CORPORATE SOURCE: Norw. Inst. Technol., Trondheim, Norway
CONFERENCE TITLE: Permafrost; Fifth international conference
CONFERENCE LOCATION: Trondheim, Norway
CONFERENCE DATE: Aug. 2-5, 1988
SOURCE: International Conference on Permafrost, Proceedings. vol. 5 no. 2 p. 1388-1392
DATE: 1988
LANGUAGE: English

30
TITLE: Experience in ground improvement by dynamic compaction and preloading at Half Moon Bay; Saudi Arabia
AUTHOR(S): Ghosh, N.; Tabba, M. M.
CORPORATE SOURCE: Sir William Halcrow and Partners, London, United Kingdom
MONOGRAPH TITLE: Second international conference on Case studies in geotechnical engineering; proceedings
EDITOR(S): Prakash, Shamsher (editor); Saudi Hotels and Resorts Areas Co., Saudi Arabia
CONFERENCE TITLE: Second international conference on Case histories in geotechnical engineering
CONFERENCE LOCATION: Rolla, MO, United States
CONFERENCE DATE: June 1-5, 1988
SOURCE: International Conference on Case Histories in Geotechnical Engineering vol. 2 Vol. 2 p. 1055-1061
DATE: 1988
LANGUAGE: English

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TITLE: Ten years of dynamic consolidation in China
AUTHOR(S): Fan, W.; Shi, M.; Qiu, Y.
CORPORATE SOURCE: Taiyuan Univ. Technol., Taiyuan, China
MONOGRAPH TITLE: Second international conference on Case studies in geotechnical engineering; proceedings
EDITOR(S): Prakash, Shamsher (editor)
CONFERENCE TITLE: Second international conference on Case histories in geotechnical engineering
CONFERENCE LOCATION: Rolla, MO, United States
CONFERENCE DATE: June 1-5, 1988
SOURCE: International Conference on Case Histories in Geotechnical Engineering vol. 2 Vol. 2 p. 1047-1054
DATE: 1988
LANGUAGE: English

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TITLE: Dynamic compaction; an unusual application
AUTHOR(S): Song, B.; Gambin, M.
CORPORATE SOURCE: U. S. Army Corps Eng., San Francisco, CA, United States
MONOGRAPH TITLE: Second international conference on Case studies in geotechnical engineering; proceedings
EDITOR(S): Prakash, Shamsher (editor); Solentanche, France
CONFERENCE TITLE: Second international conference on Case histories in geotechnical engineering
CONFERENCE LOCATION: Rolla, MO, United States
CONFERENCE DATE: June 1-5, 1988
SOURCE: International Conference on Case Histories in Geotechnical Engineering vol. 2 Vol. 2 p. 969-975
DATE: 1988
LANGUAGE: English

33
TITLE: Soil improvement using dynamic compaction for the Bristol Resource Recovery Facility
AUTHOR(S): Kummerle, R. P.; Dumas, J. C.
CORPORATE SOURCE: Tectonic Eng. Consult., United States
MONOGRAPH TITLE: Second international conference on Case studies in geotechnical engineering; proceedings
EDITOR(S): Prakash, Shamsher (editor); Geopac, Canada
CONFERENCE TITLE: Second international conference on Case histories in geotechnical engineering
CONFERENCE LOCATION: Rolla, MO, United States
CONFERENCE DATE: June 1-5, 1988
SOURCE: International Conference on Case Histories in Geotechnical Engineering vol. 2 Vol. 2 p. 921-927
DATE: 1988
LANGUAGE: English

34
TITLE: Analisis de los resultados sobre compactacion dinamica obtenidos en una obra del sur de Espana
TRANSLATED TITLE: Analysis of the results of dynamic compaction obtained in southern Spain
AUTHOR(S): Cleaud, J. J.; Bourdon, L.; Karaki, P.
SOURCE: Boletin de Informacion del Laboratorio de Carreteras y Geotecnia vol. 165 p. 15-19
DATE: 1984
LANGUAGE: Spanish

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TITLE: Dynamic consolidation of nearly saturated granular media
AUTHOR(S):   Vardoulakis, I.; Beskos, D. E.
CORPORATE SOURCE: Univ. Minn., Dep. Civ. and Miner. Eng., Minneapolis, MN, United States
MONOGRAPH TITLE: Dynamic behavior of foundations and buried structures
EDITOR(S):   Banerjee, P. K. (editor); Butterfield, R. (editor)
CORPORATE SOURCE: State Univ. N.Y. at Buffalo, Dep. Civ. Eng., Buffalo, NY, United States; Univ. Southampton, Dep. Civ. Eng., United Kingdom, Univ. Patras, Dep. Civ. Eng., Greece
SOURCE: Developments in Soil Mechanics and Foundation Engineering   vol. 3 p. 167-208
DATE: 1987
LANGUAGE: English

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TITLE: Dynamic compaction in friable loessdiscussion
AUTHOR(S):   Bally, Rene Jacques; Culitza, Cezar; Lutenegger, Alan J.
CORPORATE SOURCE: Sci. Dep. Hydraul. Eng. Res. Inst., Bucharest, Romania ; Constr. Enterp. Spec. and Tech. Insulation Works, Geotech. Dep., Romania
SOURCE: Journal of Geotechnical Engineering   vol. 113   no. 11   p. 1416-1418
DATE: 1987
LANGUAGE: English
NOTE: For reference to original article by Lutenegger, A. J., see J. Geotech. Eng., Am. Soc. Civ. Eng., Vol. 112, No. 6, June 1986

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TRANSLATED TITLE: Dynamic consolidation for blasted rock fill improvement
AUTHOR(S):   Yuasa, K.; Sakamoto, Y.; Yamazaki, H.; Okura, T.
SOURCE: Tsuchi-To-Kiso JSSMFE = Soil Mechanics and Foundation Engineering vol. 35   no. 5; Ser. No. 352   p. 39-44
DATE: 1987
LANGUAGE: Japanese   SUMMARY LANGUAGE: English

38
TITLE: Dynamic consolidation test on soft coastal sediments
AUTHOR(S):   Michalopoulos, Alex P.; Varosio, G.; Scrinzi, G. A.
CORPORATE SOURCE: D'Appolonia Consult., Madrid, Spain
MONOGRAPH TITLE: Proceedings of In situ '86, a specialty conference; Use of in situ tests in geotechnical engineering
AUTHOR(S):   Clemence, Samuel P.
CORPORATE SOURCE: Syracuse Univ., Dep. Civ. Eng., Syracuse, NY, United States; Ansaldo S.p.A., Civ. Eng. Dep., Italy
CONFERENCE TITLE: In situ '86; Use of in situ tests in geotechnical engineering
CONFERENCE LOCATION: Blacksburg, VA, United States
CONFERENCE DATE: June 23-25, 1986
SOURCE: Geotechnical Special Publication   vol. 6   p. 809-822
DATE: 1986
LANGUAGE: English

39
TITLE: Barrages en terre; compactage dynamique d'une tranchee d'etancheite
TRANSLATED TITLE: Earth dams; dynamic compaction of a sealing trench
AUTHOR(S): Degoutte, G.
CORPORATE SOURCE: CEMAGREF, Aix-en-Provence 13603, France
SOURCE: Informations Techniques - CEMAGREF vol. 60 p. 1-4
DATE: 1985
LANGUAGE: French

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TRANSLATED TITLE: Effects of dynamic consolidation on seismic response of stratified soils and earth dams
AUTHOR(S): Kyrgeorgiou, I.
SOURCE: Deltion Kentron Ereunon Demosion Ergon = Bulletin of the Public Works Research Center vol. 23 no. 3-4 p. 92-98
DATE: 1984
LANGUAGE: Greek SUMMARY LANGUAGE: English

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TITLE: Deep dynamic compaction ground improvement in karst terrain
AUTHOR(S): Myers, David A.; Munsey, John S.
CORPORATE SOURCE: Dunn Geosci. Corp., Latham, NY, United States
MONOGRAPH TITLE: Association of Engineering Geologists, 29th annual meeting; Better living through engineering geology
AUTHOR(S): Tryhorn, Alan D. (chairperson)
CORPORATE SOURCE: Geo/Resour. Consult., San Francisco, CA, United States
CONFERENCE TITLE: Association of Engineering Geologists, 29th annual meeting
CONFERENCE LOCATION: San Francisco, CA, United States
CONFERENCE DATE: Oct. 5-10, 1986
SOURCE: Annual Meeting - Association of Engineering Geologists vol. 29 p. 58
DATE: 1986
LANGUAGE: English

42
TITLE: Modified method for intensive dynamic compaction of weak soils
AUTHOR(S): Marinescu, K.
CORPORATE SOURCE: Inst. Des. and Plann. Highw., Water and Air Transp., Bucharest, Romania
SOURCE: Soil Mechanics and Foundation Engineering vol. 23 no. 2 p. 73-78
DATE: 1986
LANGUAGE: English

43
TITLE: Study of microstructures of loess deposit under dynamic consolidation
AUTHOR(S): Weiyuan Fan; Meiyun Shi; Yihui Qiu; Xiaohong Bai; Dongzuo Chen
CORPORATE SOURCE: Taiynan Univ. Technol., China
MONOGRAPH TITLE: International syposium on environmental geotechnology; Volume 1
AUTHOR(S): Fang, H.
CORPORATE SOURCE: Lehigh Univ., Dep. Civ. Eng., Bethlehem, PA, United States; Inst. Explor. and Des. Electr. Power Energy of Shaampi Province, China
CONFERENCE TITLE: International symposium on environmental geotechnology
CONFERENCE LOCATION: Allentown, PA, United States
CONFERENCE DATE: Apr. 21-23, 1986
PUBLISHER: Envo Publ. Co., United States p. 667-679
DATE: 1986
LANGUAGE: English

44
TITLE: Dynamic consolidation of flyash
AUTHOR(S): Ellis, G. W.
CORPORATE SOURCE: Princeton Univ., Princeton, NJ, United States
MONOGRAPH TITLE: International syposium on environmental geotechnology; Volume 1
AUTHOR(S): Fang, H.
CORPORATE SOURCE: Lehigh Univ., Dep. Civ. Eng., Bethlehem, PA, United States
CONFERENCE TITLE: International symposium on environmental geotechnology
CONFERENCE LOCATION: Allentown, PA, United States
CONFERENCE DATE: Apr. 21-23, 1986
PUBLISHER: Envo Publ. Co., United States p. 564-573
DATE: 1986
LANGUAGE: English

45
TITLE: Dynamic consolidation for a clay foundation with sand package
AUTHOR(S): Qian Jia-huan; Zhao, W. B.; Qian, Z.
CORPORATE SOURCE: East China Tech. Univ. Water Resour., Nanjing, China
MONOGRAPH TITLE: Numerical methods in geomechanics, Nagoya; in three volumes
EDITOR(S): Kawamoto, T. (editor); Ichikawa, Y. (editor)
CORPORATE SOURCE: Nagoya Univ., Nagoya, Japan; First Navig. Bur. Eng., China
CONFERENCE TITLE: Fifth international conference on numerical methods in geomechanics
CONFERENCE LOCATION: Nagoya, Japan
CONFERENCE DATE: Apr. 1-5, 1985
PUBLISHER: A. A. Balkema, Rotterdam, Netherlands p. 637-642
DATE: 1985
LANGUAGE: English

46
TITLE: Ground vibrations during dynamic compaction
AUTHOR(S): Mayne, P. W.
CORPORATE SOURCE: Law Eng. Test. Co., McLean, VA, United States
MONOGRAPH TITLE: Vibration problems in geotechnical engineering
EDITOR(S): Gazetas, G. (editor); Selig, E. T. (editor)
CORPORATE SOURCE: Rensselaer Polytech. Inst., Dep. Civ. Eng., Troy, NY, United States
CONFERENCE TITLE: Vibration problems in geotechnical engineering
CONFERENCE LOCATION: Detroit, MI, United States
CONFERENCE DATE: Oct. 22, 1985
PUBLISHER: Am. Soc. Civ. Eng., New York, NY, United States p. 247-265
DATE: 1985
LANGUAGE: English

47
TITLE: Development of heavy tamping of loess bases
AUTHOR(S): Minkov, M.; Donchev, P.
CORPORATE SOURCE: Bulg. Acad. Sci., Geotech. Lab., Sofia, Bulgaria
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering; organized by the Finnish Geotechnical Society; Vol. 2
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 797-800
DATE: 1983
LANGUAGE: English

48
TITLE: Ten years of dynamic consolidation
AUTHOR(S): Gambin, M. P.
CORPORATE SOURCE: Soletanche-Entrep., Nanterre, France
MONOGRAPH TITLE: Soil mechanics and foundation engineering; Vol. 1
EDITOR(S): Boyce, J. R. (editor); Mackechnie, W. R. (editor); Schwartz, K. (editor)
CORPORATE SOURCE: Loughborough Univ. Technol., United Kingdom
CONFERENCE TITLE: Eighth regional conference for Africa on soil mechanics and foundation engineering
CONFERENCE LOCATION: Harare, Zimbabwe
CONFERENCE DATE: 1984
SOURCE: Proceedings of the Regional Conference for Africa - Soil Mechanics and Foundation Engineering vol. 8 p. 363-370
DATE: 1984
LANGUAGE: English

49
TITLE: On dynamic consolidation
AUTHOR(S): Chien, S. T.; Chien, C. H.
CORPORATE SOURCE: East China Tech. Univ. Water Resour., Nanking, China
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering organized by the Finnish Geotechnical Society; Vol. 1
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 353-356
DATE: 1983
LANGUAGE: English

50
TITLE: Pack drain-dynamic consolidation for strengthening soft foundation
AUTHOR(S): Qian Zeng; Li Guan-wu
CORPORATE SOURCE: China Harbour Eng. Co., Beijing, China
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering organized by the Finnish Geotechnical Society; Vol. 1
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 293-296
DATE: 1983
LANGUAGE: English

51
TITLE: Heavy tamping of ground of Aeimaerautio bridge
AUTHOR(S): Hartikainen, J.; Valtonen, M.
CORPORATE SOURCE: Univ. Oulu, Oulu, Finland
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering organized by the Finnish Geotechnical Society; Vol. 1
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 249-252
DATE: 1983
LANGUAGE: English

52
TITLE: The Menard dynamic consolidation method at Nice Airport
AUTHOR(S): Gambin, M. P.
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering organized by the Finnish Geotechnical Society; Vol. 1
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 231-234
DATE: 1983
LANGUAGE: English

53
TITLE: Combination of the drain and surcharge method with dynamic compaction
AUTHOR(S): Cognon, J. M.; Liausu, P.; Vialard, R.
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering organized by the Finnish Geotechnical Society; Vol. 1
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 219-222
DATE: 1983
LANGUAGE: English

54
TITLE: Dynamic consolidation of rockfill at an oil refinery site
AUTHOR(S): Bjolgerud, O.; Haug, A. K.
CORPORATE SOURCE: NOTEBY, Oslo, Norway
MONOGRAPH TITLE: Improvement of ground; Proceedings of the Eighth European conference on soil mechanics and foundation engineering organized by the Finnish Geotechnical Society; Vol. 1
CONFERENCE TITLE: Improvement of ground; Eighth European conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Helsinki, Finland
CONFERENCE DATE: May 23-26, 1983
PUBLISHER: A.A. Balkema, Rotterdam, Netherlands p. 213-218
DATE: 1983
LANGUAGE: English

55
TITLE: Collapse and compaction of sinkholes by dynamic compaction
AUTHOR(S): Guyot, C. A.
CORPORATE SOURCE: Terra Firma, Murrysville, PA, United States
MONOGRAPH TITLE: Sinkholes; their geology, engineering and environmental impact
EDITOR(S): Beck, B. F. (editor)
CORPORATE SOURCE: Univ. Cent. Fla., Fla. Sinkhole Res. Inst., Orlando, FL, United States
CONFERENCE TITLE: The first multidisciplinary conference on sinkholes
CONFERENCE LOCATION: Orlando, FL, United States
CONFERENCE DATE: Oct. 15-17, 1984
PUBLISHER: A. A. Balkema, Rotterdam, Netherlands p. 419-423
DATE: 1984
LANGUAGE: English

56
TITLE: Mechanism of dynamic consolidation and its environmental effect
AUTHOR(S): Wang Zhong-qi; Deng Xiang-lin
CORPORATE SOURCE: Minist. Urban and Rural Constr. and Environ. Prot., China
MONOGRAPH TITLE: International conference on case histories in geotechnical engineering; Volume 3
EDITOR(S): Prakash, S. (editor)
CONFERENCE TITLE: International conference on case histories in geotechnical engineering
CONFERENCE LOCATION: St. Louis, MO, United States
CONFERENCE DATE: May 6-11, 1984
PUBLISHER: Univ. Mo.-Rolla, Rolla, MO, United States p. 1459-1465
DATE: 1984
LANGUAGE: English

57
TITLE: Improvement of a dumped rockfill foundation by dynamic consolidation
AUTHOR(S): Wightman, A.; Beaton, N. F.
CORPORATE SOURCE: Klohn Leonoff, Geotech. Div., Richmond, BC, Canada
MONOGRAPH TITLE: International conference on case histories in geotechnical engineering; Volume 3
EDITOR(S): Prakash, S. (editor); N. F. Beaton Consult., Canada
CONFERENCE TITLE: International conference on case histories in geotechnical engineering
CONFERENCE LOCATION: St. Louis, MO, United States
CONFERENCE DATE: May 6-11, 1984
PUBLISHER: Univ. Mo.-Rolla, Rolla, MO, United States p. 1365-1372
DATE: 1984
LANGUAGE: English

58
TITLE: In situ stabilization of two industrial sites by dynamic compaction
AUTHOR(S): O'Brien, J. F.; Gupton, C. P.
CORPORATE SOURCE: Dames & Moore, Boca Raton, FL, United States
MONOGRAPH TITLE: International conference on case histories in geotechnical engineering; Volume 3
EDITOR(S): Prakash, S. (editor)
CONFERENCE TITLE: International conference on case histories in geotechnical engineering
CONFERENCE LOCATION: St. Louis, MO, United States
CONFERENCE DATE: May 6-11, 1984
PUBLISHER: Univ. Mo.-Rolla, Rolla, MO, United States p. 1259-1264
DATE: 1984
LANGUAGE: English

59
TITLE: Stabilization of open pit mine spoil by dynamic compaction
AUTHOR(S): Wade, N. H.
CORPORATE SOURCE: Monenco Consult., Calgary, AB, Canada
MONOGRAPH TITLE: Geotechnical research and its application to Canadian resource development
EDITOR(S): McCreath, D. (editor)
CORPORATE SOURCE: Can. Geotech. Soc., Montreal, PQ, Canada
CONFERENCE TITLE: 36th Canadian geotechnical conference; Geotechnical research and its application to Canadian resource development
CONFERENCE LOCATION: Vancouver, BC, Canada
CONFERENCE DATE: June 22, 1983
SOURCE: Canadian Geotechnical Conference = Conference Canadienne de Geotechnique vol. 36 p. 1.2.1-1.2.8
DATE: 1983
LANGUAGE: English

60
TITLE: Dynamic compaction of Penitas Dam foundation
AUTHOR(S): Moreno, E.; Santoyo, E.; Fuentes de la Rosa, A.
CORPORATE SOURCE: Fed. Comm. Electr., Mexico City, Mexico
MONOGRAPH TITLE: Technical papers, Energy and transportation
CORPORATE SOURCE: Canadian Geotechnical Society, Montreal, PQ, Canada
CONFERENCE TITLE: Geotechnical engineering in resource development ; Seventh Panamerican conference on soil mechanics and foundation engineering
CONFERENCE LOCATION: Vancouver, BC, Canada
CONFERENCE DATE: June 1983
SOURCE: Memorias del Congreso Panamericano de Mecanica de Suelos e Ingenieria de Fundaciones = Proceedings of the Panamerican Conference on Soil Mechanics and Foundation Engineering vol. 7 no. 1 p. 123-133
DATE: 1983
LANGUAGE: English

61
TITLE: Impact stresses during dynamic compaction
AUTHOR(S): Mayne, P. W.; Jones, J. S., Jr.
CORPORATE SOURCE: Law Eng., Washington, DC, United States
SOURCE: Journal of Geotechnical Engineering vol. 109 no. 10 p. 1342-1346
DATE: 1983
LANGUAGE: English

62
MONOGRAPH TITLE: Proceedings of the Fourth international conference on numerical methods in geomechanics
AUTHOR(S): Eisenstein, Z.
CORPORATE SOURCE: Univ. Alberta, Dep. Civ. Eng., Edmonton, AB, Canada
CONFERENCE TITLE: Fourth international conference on numerical methods in geomechanics
CONFERENCE LOCATION: Edmonton, AB, Canada
CONFERENCE DATE: May 31-June 4, 1982
PUBLISHER: A. A. Balkema, Rotterdam, Netherlands
DATE: 1982 2 vol. p.
LANGUAGE: English
NOTE: Individual papers are cited separately

63
TITLE: Dynamic consolidation of liquefiable sands
AUTHOR(S): Bhandari, R. K. M.
MONOGRAPH TITLE: International conference on recent advances in geotechnical earthquake engineering and soil dynamics; Vol. II
EDITOR(S): Prakash, Shamsher (editor)
CONFERENCE TITLE: International conference on recent advances in geotechnical earthquake engineering and soil dynamics
CONFERENCE LOCATION: St. Louis, MO, United States
CONFERENCE DATE: Apr. 26-May 3, 1981
PUBLISHER: Univ. Mo. at Rolla, Rolla, MO, United States p. 857-860
DATE: 1981
LANGUAGE: English

64
TITLE: La consolidation dynamique; une technique permettant de diminuer les risques de liquefaction de sols fins satures en cas de tremblement de terre
TRANSLATED TITLE: Dynamic consolidation; a technique permitting a reduction of liquefaction risk in fine, saturated soils in the case of earthquakes
AUTHOR(S): Gambin, M. P.; Capelle, J. F.; Dumas, J. C.
CORPORATE SOURCE: Menard, Pittsburgh, PA, United States; ROCTEST, Canada , Geopac, Canada
CONFERENCE TITLE: Third Canadian conference on earthquake engineering
CONFERENCE LOCATION: Montreal, PQ, Canada
CONFERENCE DATE: June 4-6, 1979
SOURCE: Proceedings - Canadian Conference (on) Earthquake Engineering=Compte Rendus - Conference Canadienne (du) Genie Sismique
no. 3 p. 117-146
DATE: 1979
LANGUAGE: French SUMMARY LANGUAGE: English

65
TITLE: Dynamic consolidation of refuse at Cwmbran
AUTHOR(S): Downie, A. R.; Treharne, G.
CORPORATE SOURCE: Menard Tech., United Kingdom
MONOGRAPH TITLE: The engineering behaviour of industrial and urban fill; proceedings of the symposium
AUTHOR(S): Anonymous; Ove Arup and Partners, United Kingdom
CONFERENCE TITLE: Symposium on the engineering behaviour of industrial and urban fill
CONFERENCE LOCATION: Birmingham, United Kingdom
CONFERENCE DATE: April 23-25, 1979
PUBLISHER: Midl. Geotech. Soc., Birmingham, United Kingdom p. E15-E24
DATE: 1979
LANGUAGE: English

66
TITLE: Field observations of a trial of dynamic consolidation on an old refuse tip in the East End of London
AUTHOR(S): Charles, J. A.
MONOGRAPH TITLE: The engineering behaviour of industrial and urban fill; proceedings of the symposium
AUTHOR(S): Anonymous
CONFERENCE TITLE: Symposium on the engineering behaviour of industrial and urban fill
CONFERENCE LOCATION: Birmingham, United Kingdom
CONFERENCE DATE: April 23-25, 1979
PUBLISHER: Midl. Geotech. Soc., Birmingham, United Kingdom p. E1-E13
DATE: 1979
LANGUAGE: English

67
TRANSLATED TITLE: Application of dynamic consolidation to oil storage tank foundations
AUTHOR(S): Sakaguchi, A.; Nishiumi, H.; Hattori, M.; Sumiyoshi, M.
SOURCE: Tsuchi-To-Kiso vol. 27 no. 9 p. 5-11
DATE: 1979
LANGUAGE: Japanese SUMMARY LANGUAGE: English

68
TITLE: Compaction of clay fills in-situ by dynamic consolidation
AUTHOR(S): Thompson, G. H.; Herbert, A.
MONOGRAPH TITLE: Clay fills
AUTHOR(S): Vaughan, V. R. (chairperson)
CONFERENCE TITLE: Clay fills
CONFERENCE LOCATION: London, United Kingdom
CONFERENCE DATE: Nov. 14-15, 1978
PUBLISHER: Inst. Civ. Eng., London, United Kingdom p. 197-204
DATE: 1979
LANGUAGE: English

69
TITLE: Solution of equation of motion for dynamic compaction of soil
AUTHOR(S): Agarwal, K. B.; Siva Ram, B.
MONOGRAPH TITLE: Numerical methods in geomechanics; Volume two, 4, Rock behavior; 5, Underground openings; 6, Embankments and slopes; 7, Dynamics
EDITOR(S): Wittke, W. (editor)
CONFERENCE TITLE: Third international conference on numerical methods in geomechanics
CONFERENCE LOCATION: Aachen, Germany, Federal Republic of
CONFERENCE DATE: April 2-6, 1979
PUBLISHER: A. A. Balkema, Rotterdam, Netherlands p. 811-816
DATE: 1979
LANGUAGE: English

70
TITLE: Dynamic consolidation in urban environment
AUTHOR(S): Capelle, J. F.; Dumas, J. C.
CORPORATE SOURCE: Roctest, Ltd., Canada; Geopac, Canada
CONFERENCE TITLE: Genie geotechnique en milieu urbain
CONFERENCE LOCATION: Montreal, Quebec, Canada
CONFERENCE DATE: Oct. 8-10, 1975
SOURCE: Can. Geotech. Conf. no. 28 p. 231-243
DATE: 1975
LANGUAGE: English

71
MONOGRAPH TITLE: Les techniques pressiometriques et la consolidation dynamique des sols; Ce qu'elles sont? Ce que l'on peut en attendre?
TRANSLATED MONOGRAPH TITLE: Load-testing techniques and dynamic consolidation of soils; what are they and what does one look for?
AUTHOR(S): Van Wambeke, A.
SOURCE: Liege, Univ., Cent. Etud., Rech. Essais Sci. Genie Civ., Mem. no. 50
DATE: 1974 83 p.
LANGUAGE: French

72
TITLE: Consolidation dynamique par pilonnage intensif; aire d'essai d'Embourg
TRANSLATED TITLE: Dynamic consolidation by intensive piling; experimental region of Embourg
AUTHOR(S): De Beer, E.; Van Wambeke, A.
SOURCE: Ann. Trav. Publics Belg. no. 5 p. 295-318
DATE: 1974
LANGUAGE: French SUMMARY LANGUAGE: Dutch

73
TITLE: Theoretical and practical aspects of dynamic consolidation
AUTHOR(S): Menard, L.; Broise, Y.
CORPORATE SOURCE: Menard Tech. Ltd., Aylesbury, United Kingdom
MONOGRAPH TITLE: Ground treatment by deep compaction
AUTHOR(S): Anonymous
PUBLISHER: Thomas Telford Ltd., London, United Kingdom p. 3-18
DATE: 1976
LANGUAGE: English SUMMARY LANGUAGE: French

74
TITLE: Theoretical and practical aspects of dynamic consolidation
AUTHOR(S): Menard, L.; Broise, Y.
CORPORATE SOURCE: Menard Tech. Ltd., Aylesbury, United Kingdom
SOURCE: Geotechnique vol. 25 no. 1 Symposium on ground treatment by deep compaction p. 3-18
DATE: 1975
LANGUAGE: English SUMMARY LANGUAGE: French

75
TITLE: Dynamic phenomena of sediment compaction in Matagorda County, Texas
AUTHOR(S): Myers, Robert L.; VanSiclen, DeWitt C.
SOURCE: Gulf Coast Assoc. Geol. Socs. Trans. v. 14, p. 241-252, illus., tables
DATE: 1964
LANGUAGE: English

76
TITLE: Engineering properties and applications of nuclear excavations
AUTHOR(S): Circeao, Louis J., Jr
SOURCE: California, Univ., Livermore, Lawrence Radiation Lab. Rept.
UCRL-7657 46 p., illus.
DATE: 1964
LANGUAGE: English

LITERATURE SEARCH FOR DC

REFERENCE:

BHANDARI, R.K.M. (1981): Dynamic Consolidation of liquefiable sands. Proc: Int.Conf. on recent advances in geotech. earthquake eng. and soil dynamics; Vol.II, St.Louis, Mo.

Topics

Topic		Notes
1) Degree of improvement	✓	Based on Nishiyama (1977) $N = 25\ Dr^2$
2) Depth of improvement	✓	still using Menard $d = \sqrt{WH}$ but found only 51% appropriate
3) Energy: (a) /drop		
(b) /unit area		
4) Drop arrangement:		
(a) No. of drops/pt.		
(b) Spacing		
(c) No. of passes		
5) Weight properties		
6) Time between passes		-
7) Monitoring/evaluation criteria	✓	$N = 10$ at surface to $N = 25$ at 10 m depth
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations		
10) Other	✓	Overall settlement due to treatment = 144 mm after 1 coverage and 280 mm after 2 coverages

Summary of Main points

1) Found that $d = \sqrt{WH}$ over predicts by factor x2
2) Example of acceptance criteria
3) Treatment caused loosening in dense layer at 6 m - 10 m
4) Tanks filled to 8.5 m gave settlements between 19 mm and 23 mm and diff. of 1/3900

LITERATURE SEARCH FOR DC

REFERENCE:

BYONGMU SONG (1988): Dynamic Compaction - an unusual application. Proc.2nd Int.Conf. on Case Histories in Geotech.Eng. St.Louis, Mo.

Topics

Topic		Notes
1) Degree of improvement	✓	- requirements for footings - see Fig 9 for improvement in N values
2) Depth of improvement		
3) Energy: (a) /drop		
(b) /unit area	✓	300 t-m/m^2
4) Drop arrangement:		
(a) No. of drops/pt.		- 10 drops/pt for pass 1, 2 drops/pt for pass 2
(b) Spacing		
(c) No. of passes		
5) Weight properties		
6) Time between passes		-
7) Monitoring/evaluation criteria	✓	See above
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations	✓	
10) Other	✓	Time effects

Summary of Main points

1) Example of DC on sand overlying soft clay - main point : DC did not distress soft clay
2) Pore pressure in clay pockets dissipated in 36 hrs
3) Main point of interest : Improvements with time - recommends at least 20 days after completion
4) Vibrations - no problems but ditch not of much value

LITERATURE SEARCH FOR DC

REFERENCE:

CHARLES, J.A., BURFORD, D., and WATTS, K.S. (1981): Field studies of the effectiveness of Dynamic Consolidation. Proc. 10th. Int.Conf. on Soil Mech. and Found.Eng., Stockholm.

Topics

1)	Degree of improvement	☐	
2)	Depth of improvement	☐	
3)	Energy: (a) /drop	☐	
	(b) /unit area	✓	Several values: gen. 2500 kN-m/m
4)	Drop arrangement:		
	(a) No. of drops/pt.	✓	10 to 15 m
	(b) Spacing	✓	10 to 12 m
	(c) No. of passes	✓	Not clear
5)	Weight properties	☐	15 t of 4 m^2 or 13.5 t of 6.5 m^2 for final passes
6)	Time between passes	☐	10 days (soft alluvial soil)
7)	Monitoring/evaluation criteria	✓	Settlement borehole gauges - settlements, magnet extensiometers - and piezometers
8)	Costs:		
	(a) Actual	☐	
	(b) Comparative	☐	
9)	Vibrations	☐	
10)	Other	✓	Typical times

Summary of Main points

Mainly concerned with fills - e.g. refuse - not of much use except as examples of energy and wt dimensions, etc.

LITERATURE SEARCH FOR DC

REFERENCE:

CHIEN, S.T. and CHIEN, C.H. (1983): On Dynamic Consolidation. Proc. 8th Europ.Conf. on Soil Mech. and Found.Eng., Helsinki.

Topics

1) Degree of improvement ☐

2) Depth of improvement ☐

3) Energy: (a) /drop ☐
(b) /unit area ☐

4) Drop arrangement:
(a) No. of drops/pt. ☐
(b) Spacing ☐
(c) No. of passes ☐

5) Weight properties ☐

6) Time between passes ☐

7) Monitoring/evaluation criteria ☐

8) Costs:
(a) Actual ☐
(b) Comparative ☐

9) Vibrations ☐

10) Other ☑

Summary of Main points

Theoretical paper - gives some relationships for depth of treatment and lateral strains - ~~not~~ of much relevance.

LITERATURE SEARCH FOR DC

REFERENCE:

CLEAUD, J.J., BOURDON, L., and KARAKI, P. (1983): Analysis of results on a Dynamic Compaction Site. Proc.8th. Europ. Conf. on Soil Mech. and Found.Eng., Helsinki.

Topics

Topic		Remarks
1) Degree of improvement	✓	E, P_L, q_c & N given in tabular form
2) Depth of improvement		
3) Energy: (a) /drop	✓	
(b) /unit area	✓	variable - per pass given
4) Drop arrangement:		
(a) No. of drops/pt.		
(b) Spacing		
(c) No. of passes		
5) Weight properties		
6) Time between passes		
7) Monitoring/evaluation criteria	✓	CPT, SPT, Pressuremeter - gives nos of tests
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations		
10) Other		Rate of progress : 2500 m^2/day with 6 rings : 150,000 m^2 in 2 months

Summary of Main points

1) Useful practical information on time of treatment and type and size of cranes

2) Interesting correlations between E, P_L, q_c, N, etc.

LITERATURE SEARCH FOR DC

REFERENCE:

DUIMAS, J.C. (1986): Discussion on paper by MITCHELL, J.K. (1986): Practical problems from surprising soil behaviour. ASCE Geotech.Eng., Vol.112 No.9, Sept.

Topics

1) Degree of improvement	✓	
2) Depth of improvement	✓	7 m good - 14 m reduced improvement
3) Energy: (a) /drop		
(b) /unit area		
4) Drop arrangement:		
(a) No. of drops/pt.		
(b) Spacing		
(c) No. of passes		
5) Weight properties		
6) Time between passes		
7) Monitoring/evaluation criteria	✓	CPT
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations		
10) Other	✓	Improvements with time

Summary of Main points

Dramatic example of improvement in loose sands. Example of time effects (~~improvement~~ after 8 days)

LITERATURE SEARCH FOR DC

REFERENCE:

ELSON, K., and GREENWOOD, D.A. (1986): Deep compaction by heavy tamping - In Chapter 11 of "The Design and Construction of Engineering Foundations" edited by HENRY, F.D.C., 2nd Ed., Chapman & Hall.

Topics

Topic		
1) Degree of improvement	☐	
2) Depth of improvement	✓	Ref. Mitchell & Katti (1981) $D = 0.5\ (WH)^{1/2}$
3) Energy: (a) /drop	✓	Threshold of 1000 kJ/blow (i.e. 100 t-m/blow)
(b) /unit area	✓	
4) Drop arrangement:		
(a) No. of drops/pt.	✓	5-10 (Best with least!)
(b) Spacing	✓	5-10 m
(c) No. of passes	☐	
5) Weight properties	✓	2 m sq. or wt/unit area of 4-5 t/m^2 or 15 to 20 t wts are about 2 m sq.
6) Time between passes	☐	
7) Monitoring/evaluation criteria	✓	Imprint volumes/successive passes. Total vol.change of 5 to 10%
8) Costs:		
(a) Actual	☐	
(b) Comparative	☐	
9) Vibrations	✓	
10) Other	☐	

Summary of Main points - Key Paper for recommendations

1) Best suited to areas > 10,000 m^2
2) Frequency < 10 Hz
3) Threshold energy/blow 1000 kJ/blow for sands
4) Fundamental theory (p 911 - Jessberger & Beine, 1981)
5) Imprints of 0.3 to 2 m

LITERATURE SEARCH FOR DC

REFERENCE:

FAN, W., SHI, M. and QIU, Y. (1988): Ten years of Dynamic Consolidation in China. Proc.2nd Int.Conf. on Case Histories in Geotech.Eng., St. Louis, Mo.

Topics

1) Degree of improvement	✓	Requirements given for various objectives including liquefaction from Chinese Code
2) Depth of improvement	✓	$D=K\sqrt{WH}$ & values of K given for diff. soil types + alternative empirical formula
3) Energy: (a) /drop	✓	
(b) /unit area	✓	guideline on energy for depths
4) Drop arrangement:		
(a) No. of drops/pt.	☐	
(b) Spacing	✓	2.4 d (d= "dia" of tamper)
(c) No. of passes	✓	1-3 for sands
5) Weight properties	✓	Rule of thumb wt. of weight per unit - base area = 25-40 kN/m^2
6) Time between passes	☐	
7) Monitoring/evaluation criteria	☐	
8) Costs:		
(a) Actual	☐	
(b) Comparative	☐	
9) Vibrations	✓	
10) Other	☐	

Summary of Main points

1) Provides criteria - for liquefaction based on Chinese Code
2) Gives recommendations for effective depth - Sat. sands $d=0.5\text{-}0.6\sqrt{WH}$
3) Discusses lateral strengthening effect
4) Damage not due to vibration but settlement or heave at very close points (2 m)

LITERATURE SEARCH FOR DC

REFERENCE:

GAMBIN, M.P. (1983): The Menard Dynamic Consolidation Method at Nice Airport. Proc. 8th Europ.Conf. on Soil Mech.and Found.Eng., Helsinki.

Topics

1) Degree of improvement ✓

2) Depth of improvement ☐

3) Energy: (a) /drop ☐
   (b) /unit area ✓

4) Drop arrangement:
   (a) No. of drops/pt. ☐
   (b) Spacing ☐
   (c) No. of passes ☐

5) Weight properties ☐

6) Time between passes ☐

7) Monitoring/evaluation criteria ☐

8) Costs:
   (a) Actual ☐
   (b) Comparative ☐

9) Vibrations ☐

10) Other ☐

Summary of Main points

1) Example of large project - examples of settlement and E increase with unit energy---not of much value

LITERATURE SEARCH FOR DC

REFERENCE:

GAMBIN, M.P. (1984): Ten years of Dynamic Consolidation Proc. 8th Regional Conf. for Africa on Soil Mech. and Found.Eng., Harare, Zimbabwe.

Topics

Topic	Checked	Notes
1) Degree of improvement	✓	
2) Depth of improvement	✓	$D = \alpha \sqrt{WH}$ $0{:}5 < \alpha < 1.0$
3) Energy: (a) /drop	✓	
(b) /unit area	✓	1500 - 4000 kJ/m^2 = 150 - 400 $t\text{-}m/m^2$
4) Drop arrangement:		
(a) No. of drops/pt.		
(b) Spacing		
(c) No. of passes		
5) Weight properties	✓	How made : concrete -> 8 t; steel -> 170 t
6) Time between passes		
7) Monitoring/evaluation criteria	✓	Pressuremeter
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations	✓	Formula for limiting $E \leq 55\ B^2$ - not clear on units
10) Other	✓	Rates of progress - 1 hectare/month

Summary of Main points

1) Useful rules of thumb for rates of progress and induced settlement
2) Depth of improvement - modified Menard formula
3) Degree of improvement related to liquefaction - tests by Ecole Centrale "confirm that prior liquefaction is a means ... number of cycles required to create liquefaction again was much greater, although increase in density could be small"
4) Induced surface settlements typically: 5-10% layer thickness in most soils - more in fills

LITERATURE SEARCH FOR DC

REFERENCE:

GHOSH, N., and TABBA, M.M. (1988): Experience in ground improvement by Dynamic Compaction and preloading at Half Moon Bay, Saudi Arabia. Proc.2nd Int.Conf. on Case Histories in Geotech.Eng., St. Louis, Mo.

Topics

1) Degree of improvement	✓	
2) Depth of improvement		
3) Energy: (a) /drop	✓	drop = 20 m -> 25 m for production (trial)
(b) /unit area		
4) Drop arrangement:		
(a) No. of drops/pt.		10 -> 20 in places
(b) Spacing		10 m spacing
(c) No. of passes		
5) Weight properties	✓	2 x 2 m, 16 tonne
6) Time between passes		
7) Monitoring/evaluation criteria	✓	piezometers and CPT's
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations		
10) Other	✓	120 tonne Manitawac crawler

Summary of Main points

1) Specified relative density of 50% at 8 m below datum

2) Compaction at depth inhibited by intermediate silt layer (40 - 70% fines and 10% clay content)

LITERATURE SEARCH FOR DC

REFERENCE:

HANSBO, S. (1978): Dynamic consolidation of soil by a falling weight. Ground Engineering, July.

Topics

1)	Degree of improvement	✓	one example only
2)	Depth of improvement	✓	uses superseded relationship
3)	Energy: (a) /drop		
	(b) /unit area		
4)	Drop arrangement:		
	(a) No. of drops/pt.		
	(b) Spacing		
	(c) No. of passes		
5)	Weight properties		
6)	Time between passes		
7)	Monitoring/evaluation criteria	✓	Seismic method and plate load test
8)	Costs:		
	(a) Actual		
	(b) Comparative		
9)	Vibrations		
10)	Other		

Summary of Main points

One relevant example only:

- Not v.useful for liquefaction study
- "Ironing" pass: 1-2 m drop with overlapping prints

LITERATURE SEARCH FOR DC

REFERENCE:

HANZAWA, H. (1981): Improvement of a quicksand. Proc.10th ICSMFE, Stockholm

Topics

1) Degree of improvement ✓

2) Depth of improvement ✓ "Limited" to 5 to 6 m under seabed

3) Energy: (a) /drop

(b) /unit area

4) Drop arrangement:

(a) No. of drops/pt.

(b) Spacing

(c) No. of passes

5) Weight properties

6) Time between passes

7) Monitoring/evaluation criteria

8) Costs:

(a) Actual

(b) Comparative

9) Vibrations

10) Other

Summary of Main points

1) Improvement of silty sand: silt content = 20-40%
2) Very little improvement in N but significant change in P_L and E
3) Shows results of liquefaction tests (cyclic triax.) - before and after
4) Nothing of much direct relevance - see Mitchell & Katti report comments

LITERATURE SEARCH FOR DC

REFERENCE:

HARTIKAINEN., J. and VALTONEN, M. (1983): Heavy tamping of ground of Aimarautio Bridge. Proc.8th Europ.Conf., on Soil Mech. and Found.Eng., Helsinki.

Topics

1) Degree of improvement			
2) Depth of improvement		✓	6-7 m (0.5 √ WH)
3) Energy:	(a) /drop	✓	123 kN through 12 m
	(b) /unit area	✓	730 kN m/m^2 x 2
4) Drop arrangement:			
	(a) No. of drops/pt.		
	(b) Spacing		
	(c) No. of passes		
5) Weight properties		✓	123 kN, cylindrical (1.55 m x 1.9 m dia.) RC inside 8 mm steel shell
6) Time between passes			
7) Monitoring/evaluation criteria			
8) Costs:			
	(a) Actual		
	(b) Comparative		
9) Vibrations			
10) Other		✓	gravel bed: Depth of pit after 3 drops = 0.35 m

Summary of Main points

1) V.silty sand 30%-70% silt content. "The compaction process proved most effective when content of the fine material was close to lower limit" - site is "borderline case"

2) Main pt of interest : thickness of gravel bed 0.8 m insufficient for machine of 500 kN -> increased to 1.5 m

LITERATURE SEARCH FOR DC

REFERENCE:

JUILLIE', Y., (1980): Ashuganj Fertiliser Plant: A typical example of dynamic consolidation. Sols Soils No.32

Topics

Topic	Checked	Comments
1) Degree of improvement	✓	
2) Depth of improvement		
3) Energy: (a) /drop		
(b) /unit area		
4) Drop arrangement:		
(a) No. of drops/pt.		
(b) Spacing		10 m x 10 m & 7 m x 7 m -> for d=10 m 20 m x 20 m & 14 m x 14 m -> for d=20 m
(c) No. of passes		
5) Weight properties		
6) Time between passes		
7) Monitoring/evaluation criteria	✓	Pressuremeter & SPT/CPT - v.extensive testing
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations		
10) Other	✓	Crane & Tripod gantry - some details given

Summary of Main points

1) Useful case history - good figures
2) Illustrates time effects -> measurements up to 50 days after treatment
3) Settlement/unit surface area -> efficiency per pass
4) Example of acceptance criteria for sands and silts

LITERATURE SEARCH FOR DC

REFERENCE:

KOPONEN, H., (1983): Soil improvement by deep compaction at the site of a harbour storage. Proc. 8th European Conf. on Soil Mech. and Found.Eng., Helsinki.

Topics

1) Degree of improvement	☐	
2) Depth of improvement	✓	10 m - uses $d = \sqrt{WH}$
3) Energy: (a) /drop	☐	
(b) /unit area	☐	
4) Drop arrangement:		
(a) No. of drops/pt.	✓	2 drops/pt in 1 pass
(b) Spacing	☐	
(c) No. of passes	☐	
5) Weight properties	☐	
6) Time between passes	☐	
7) Monitoring/evaluation criteria	☐	
8) Costs:		
(a) Actual	☐	
(b) Comparative	☐	
9) Vibrations	☐	
10) Other	✓	Time 2500 m^2 in 10 days

Summary of Main points

1) Reference to Hansbo with relationship between induced settlement and total energy

2) Rate of progress = 250 m^2/day (but only 2 drops/pt & 2 passes)

LITERATURE SEARCH FOR DC

REFERENCE:

KUMMERLE, R.P. and DUMAS, J.C., (1988): Soil improvement using Dynamic Compaction for Bristol Resource Recovery Facility. Proc. 2nd Int.Conf. on Case Histories in Geotech. Eng., St. Louis, Mo.

Topics

1) Degree of improvement	☐	
2) Depth of improvement	☐	
3) Energy: (a) /drop	✓	falling 60-70 ft
(b) /unit area	☐	
4) Drop arrangement:		
(a) No. of drops/pt.	☐	
(b) Spacing	☐	
(c) No. of passes	☐	
5) Weight properties	✓	Octagonal 13.5 ton -> increases to 15 ton
6) Time between passes	☐	
7) Monitoring/evaluation criteria	✓	PMT favoured
8) Costs:		
(a) Actual	☐	
(b) Comparative	☐	
9) Vibrations	✓	2 in/sec criteria
10) Other	✓	Time: 141,900 ft^2 - completed in 30 days

Summary of Main points

1) Considerable compactive effort required to "punch" through dense layer
2) Practical info on crane size etc.
3) Correlations between various parameters
4) Induced settlement v.energy input
5) Vibrations - PPV_{max} = 0.146 in/s at site border

LITERATURE SEARCH FOR DC

REFERENCE:

LEONARDS, G.A., CUTTER, W.A., and HOLTZ, R.D. (1980): Dynamic compaction of granular soils. ASCE Journ.of Geotech.Eng. Vol.106, No.GT1, January.

Topics

1)	Degree of improvement	✓	
2)	Depth of improvement	✓	$D = \frac{1}{2}\sqrt{WH}$
3)	Energy: (a) /drop	✓	72 t-m
	(b) /unit area	✓	
4)	Drop arrangement:		
	(a) No. of drops/pt.	✓	7
	(b) Spacing	✓	5 ft
	(c) No. of passes		
5)	Weight properties	✓	6 ton
6)	Time between passes		
7)	Monitoring/evaluation criteria	✓	Increase N by 3 to 5 times
8)	Costs:		
	(a) Actual		
	(b) Comparative		
9)	Vibrations	✓	
10)	Other		

Summary of Main points

1) 2 Prelim.trials 37.6 t-m & 72 t-m -> Production : DC
2) Refutes original $D=\sqrt{WH}$ -> proposes $D = \frac{1}{2}\sqrt{WH}$ -
3) Degree of compaction correlates with: (energy/drop) x (total energy/unit area) -> Upper bound to densification ≈ Q_c = 15 MPa or N=30-40
4) Vibration measurements. 72 t-m -> 3 m dist.for PPV=50 mm/s - Longitudinal vibration most sig.

LITERATURE SEARCH FOR DC

REFERENCE:

LO, K.W., OOI, P.L. and LEE, S.H. (1989): Unified approach to ground improvement by heavy tamping. ASCE Journ. of Geotech.Eng., Vol.116, No.3, March.

Topics

1) Degree of improvement ☐

2) Depth of improvement ☐

3) Energy: (a) /drop ✓ }
} Saturation Energy
(b) /unit area ✓ }

4) Drop arrangement:

(a) No. of drops/pt. ☐

(b) Spacing ☐

(c) No. of passes ☐

5) Weight properties ☐

6) Time between passes ☐

7) Monitoring/evaluation criteria ☐

8) Costs:

(a) Actual ☐

(b) Comparative ☐

9) Vibrations ☐

10) Other ☐

Summary of Main points

Re-introduces concept of "Saturation Energy" - proposes relationship with P_L

LITERATURE SEARCH FOR DC

REFERENCE:

LUKAS, R.G., (1980): Densification of loose deposits by pounding. ASCE., Journ.Geotech.Eng. Vol.106, No.GT4, April.

<u>Topics</u>

1) Degree of improvement	✓	
2) Depth of improvement	✓	$0.75 \times \sqrt{WH}$
3) Energy: (a) /drop		
(b) /unit area		
4) Drop arrangement:		
(a) No. of drops/pt.	✓	7 - 9
(b) Spacing		3 ft
(c) No. of passes		- not significant ?
5) Weight properties	✓	wts but not size
6) Time between passes		-
7) Monitoring/evaluation criteria	✓	SPT & Limit Pressure
8) Costs:		
(a) Actual	✓	\$0.5 - \$1.0/ft^2 for d=3-5 m
(b) Comparative	✓	Rem. & replacement: 3-5 times
9) Vibrations	✓	Method of predicting critical dist.
10) Other		

<u>Summary of Main points</u> Key Paper for early case studies

1) Generally low energy case studies
2) Improvement not uniform
3) Energy/unit vol. approx. same as Standard Proctor
4) Brief comment on comparative costs

LITERATURE SEARCH FOR DC

REFERENCE:

MAYNE, P.W., JONES, J.S. and DUMAS, J.C. (1984): Ground response to Dynamic Compaction. ASCE Journ.Geotech.ENg. Vol.110, No.6 June

Topics

1) Degree of improvement	☐	
2) Depth of improvement	✓	$d_{max} = \frac{1}{2}\sqrt{\frac{WH}{n}}$
3) Energy: (a) /drop	✓	
(b) /unit area	✓	
4) Drop arrangement:		
(a) No. of drops/pt.	✓	
(b) Spacing	☐	
(c) No. of passes	☐	
5) Weight properties	✓	square, circular or octagonal - no base areas given
6) Time between passes	☐	
7) Monitoring/evaluation criteria	✓	
8) Costs:		
(a) Actual	☐	
(b) Comparative	☐	
9) Vibrations	✓	
10) Other	✓	Example: 6 months to complete 33 acres with 3 cranes

Summary of Main points

Key Paper: Gives summaries of data from 124 case studies

In particular:
- Induced subsidence
- Ground vibrations
- d_{max}
- CPT & SPT v. unit energy

LITERATURE SEARCH FOR DC

REFERENCE:

MAYNE, P.W. (1985): Ground vibrations during dynamic compaction. Proc.of Conference on Vibration problems in geotechnical engineering, Detroit, ASCE.

Topics

1) Degree of improvement ☐

2) Depth of improvement ☐

3) Energy: (a) /drop ☐

(b) /unit area ☐

4) Drop arrangement:

(a) No. of drops/pt. ☐

(b) Spacing ☐

(c) No. of passes ☐

5) Weight properties ☐

6) Time between passes ☐

7) Monitoring/evaluation criteria ☐

8) Costs:

(a) Actual ☐

(b) Comparative ☐

9) Vibrations ✓

10) Other ☐

Summary of Main points

Key paper on vibrations - recommendations for measuring (use max,. single component or TVS - not PVS):

- Low frequency more damaging - new criteria
- Use scaled distance graph <u>or</u> normalised graph

LITERATURE SEARCH FOR DC

REFERENCE:

MENARD, L., and BROISE, Y., (1975): Theoretical and practical aspects of dynamic consolidation. Geotechnique, Vol.XXV, No.1, March.

Topics

Topic		Comment
1) Degree of improvement	✓	} Not of much use
2) Depth of improvement	✓	}
3) Energy: (a) /drop		} Useful examples of actual projects
(b) /unit area		}
4) Drop arrangement:		
(a) No. of drops/pt.		
(b) Spacing		
(c) No. of passes		
5) Weight properties		
6) Time between passes		
7) Monitoring/evaluation criteria	✓	
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations	✓	Amplitude increases with area of weight
10) Other: Times of treatment - examples		110,000 m^2 in 5 months: sandy silt (2.2 hectares/month)

Summary of Main points

Some limited use: Information on times and actual energy used on projects: e.g. 5 months for 110,000 m^2.
Also wt. of tamping machine 60-200 t
General statements on vibrations

LITERATURE SEARCH FOR DC

REFERENCE:

MITCHELL,J.K., and KATTI, R.K., (1981): Soil Improvement - State-of-the-art report. Proc.10th Int.Conf. Soil Mech.and Found.Eng., Stockholm

<u>Topics</u>

1)	Degree of improvement	✓	One example given
2)	Depth of improvement	✓	See Figure : $d - \frac{1}{2}\sqrt{WH}$
3)	Energy: (a) /drop		
	(b) /unit area		
4)	Drop arrangement:		
	(a) No. of drops/pt.		
	(b) Spacing		
	(c) No. of passes		
5)	Weight properties		
6)	Time between passes		
7)	Monitoring/evaluation criteria		
8)	Costs:		
	(a) Actual		
	(b) Comparative		
9)	Vibrations	✓	For given scaled energy, PPV less than for pile driving
10)	Other	✓	

<u>Summary of Main points</u>

- Useful summary of experience to date
- Examples of degree of improvement
- General statements and criteria for vibrations
- Emphasises time effects - "aging", not pore pressure dissipation

LITERATURE SEARCH FOR DC

REFERENCE:

MITCHELL,J.K., and KATTI, R.K., (1981): Soil Improvement - General Report. Proc.10th Int.Conf. Soil Mech.and Found.Eng., Stockholm

Topics

1) Degree of improvement	☐
2) Depth of improvement	☑
3) Energy: (a) /drop	☐
(b) /unit area	☐
4) Drop arrangement:	
(a) No. of drops/pt.	☐
(b) Spacing	☐
(c) No. of passes	☐
5) Weight properties	☐
6) Time between passes	☐
7) Monitoring/evaluation criteria	☐
8) Costs:	
(a) Actual	☐
(b) Comparative	☐
9) Vibrations	☐
10) Other	☐

Summary of Main points

1) Comments on other DC papers in conference - nothing of much direct use.
   - Hanzawa) comments not of much use to
   - Jessberger & Beine) this study

LITERATURE SEARCH FOR DC

REFERENCE:

MORI, H. (1977): Compaction of the deep fill of boulder soils by impact force. 5th S.E. Asian Conf. on Soil Eng., Bangkok, Thailand

Topics

1) Degree of improvement	☐	
2) Depth of improvement	✓	7 m good, + another 7 m partial improvement
3) Energy: (a) /drop	☐	
(b) /unit area	☐	
4) Drop arrangement:		
(a) No. of drops/pt.	☐	5
(b) Spacing	☐	
(c) No. of passes	☐	
5) Weight properties	✓	12 t, 3 m^2 area, h=20 m -
6) Time between passes	☐	
7) Monitoring/evaluation criteria	✓	pressuremeter & geologger, - some N values
8) Costs:		
(a) Actual	☐	
(b) Comparative	☐	
9) Vibrations	☐	
10) Other	✓	use 100 - 150 ton crane

Summary of Main points

Bouldery soil only - good example of drop arrangement and crater sizes, etc.

LITERATURE SEARCH FOR DC

REFERENCE:

RAMASWAMY, S.D., LEE, S.L., and DAULAH, I.U., (1981): Dynamic Consolidation - dramatic way to strengthen soil. Civil Engineering, ASCE, April

Topics

Topic		
1) Degree of improvement	✓	range of N values
2) Depth of improvement	✓	6 m
3) Energy: (a) /drop	✓	
(b) /unit area	✓	62.5 t-m/m^2
4) Drop arrangement:		
(a) No. of drops/pt.		
(b) Spacing	✓	6 m x 6 m
(c) No. of passes		
5) Weight properties	✓	
6) Time between passes		
7) Monitoring/evaluation criteria	✓	SPT & surface settlement
8) Costs:		
(a) Actual		
(b) Comparative		
9) Vibrations		
10) Other		

Summary of Main points

Example of DC on loose sand fill - 30% fines

N from 5 -> 20 (low side of range) or
20 -> 40 (high side of range)

LITERATURE SEARCH FOR DC

REFERENCE:

RAMASWAMY, S.D. and YONG, K.Y., (1982): Assessment of in-depth densification of sandfill due to compaction in a reclamation area by cone penetration resistance. Proc.2nd European Symposium on penetration testing, Amsterdam.

Topics

1) Degree of improvement [✓]

2) Depth of improvement [ ]

3) Energy:
   - (a) /drop [ ]
   - (b) /unit area [ ]

4) Drop arrangement:
   - (a) No. of drops/pt. [ ]
   - (b) Spacing [ ]
   - (c) No. of passes [ ]

5) Weight properties [ ]

6) Time between passes [ ]

7) Monitoring/evaluation criteria [✓]

8) Costs:
   - (a) Actual [ ]
   - (b) Comparative [ ]

9) Vibrations [ ]

10) Other [ ]

Summary of Main points

1) Refers to Changi data

2) Useful discussion of SPT and CPT in relation to relative density - see Fig 2

LITERATURE SEARCH FOR DC

REFERENCE:

RAMASWAMY, S.D., and YONG, K.Y., (1983): Evaluation of densification of sandfill. Proc. Eighth European Conf. on Soil Mech. and Foundation Eng., Helsinki.

Topics

1) Degree of improvement ☑

2) Depth of improvement ☐

3) Energy: (a) /drop ☐
(b) /unit area ☐

4) Drop arrangement:
(a) No. of drops/pt. ☐
(b) Spacing ☐
(c) No. of passes ☐

5) Weight properties ☐

6) Time between passes ☐

7) Monitoring/evaluation criteria ☑

8) Costs:
(a) Actual ☐
(b) Comparative ☐

9) Vibrations ☐

10) Other ☐

Summary of Main points

1) Compares DC on 3 sites: Changi, Ayer Merbau, Ashuganj
2) Mainly concerned with correlating SPT, CPT and pressuremeter
3) Gives examples of improvement - see Fig 2

LITERATURE SEARCH FOR DC

REFERENCE:

SOLYMAR, Z.V., and REED, D.J., (1986): A comparison of foundation compaction techniques. Can Geotech.Journ. Vol.23

Topics

1) Degree of improvement ✓

2) Depth of improvement ☐

3) Energy: (a) /drop ☐
(b) /unit area ☐

4) Drop arrangement:
(a) No. of drops/pt. ☐
(b) Spacing ☐
(c) No. of passes ☐

5) Weight properties ☐

6) Time between passes ☐

7) Monitoring/evaluation criteria ☐

8) Costs:
(a) Actual ☐
(b) Comparative ☐

9) Vibrations ☐

10) Other ✓

Summary of Main points

1) Comparison of vibrocompaction, DC, blasting and others
2) Emphasises time dependent strength gain (ref. Mitchell & Solymar),
3) "A characteristic of the method is the non-uniform improvement in the vertical direction, and this cannot be completely eliminated" - but didn't use reduced energy for following passes
4) Can compact to 15-20 m depth to D_r = 65-75% and 75-85% at depths less than 10-12 m

LITERATURE SEARCH FOR DC

REFERENCE:

YARGER, T.L. (1986): Dynamic compaction of loose and hydrocompactible soils on Interstate 90, Whitehall-Cardwell Montana. Transportation Research Record 1089, National Research Council, USA.

Topics

1) Degree of improvement	✓	Examples of increase in N
2) Depth of improvement	✓	use $d=0.5\sqrt{WH}$
3) Energy: (a) /drop	✓	15 t @ 60 ft
(b) /unit area		
4) Drop arrangement:		
(a) No. of drops/pt.	✓	5 + 3 (provision for extra)
(b) Spacing	✓	12 ft
(c) No. of passes		
5) Weight properties	✓	15 ton, 60 ft
6) Time between passes		
7) Monitoring/evaluation criteria	✓	SPT & Crater depth
8) Costs:		
(a) Actual	✓	about US\$7/yd^2
(b) Comparative	✓	- general and actual US\$
9) Vibrations	✓	
10) Other	✓	55 working days for 111,300 yd^2, etc.

Summary of Main points

Useful paper, particularly for time and cost info.

APPENDIX B

KEY PAPERS FOR DC

Ground Response to Dynamic Compaction

By Paul W. Mayne,[1] A. M. ASCE, John S. Jones, Jr.,[2] A. M. ASCE, and Jean C. Dumas[3]

Abstract: Field measurements from over 120 sites have been collected to study current practice and determine if similarities exist in the response of the ground to site improvement by dynamic compaction. Data were obtained from published reports and files. Ground conditions at these sites were quite diverse, including natural sands, hydraulic fills, rubble, clay fills, and miscellaneous materials. General trends are presented which show that crater depths, ground vibrations, and the depth of influence increase with the energy per blow. The magnitude of induced subsidence, static cone resistance, standard penetration resistance, pressuremeter modulus, and limit pressure tend to increase with the applied energy per unit area.

Introduction

The densification of loose sands by falling weights dates back to antiquity. The first known published reference on the subject involved a site in Germany (46). Not until 1969, however, was the technique finally promoted by Louis Menard as a routine method of site improvement.

During the past decade, dynamic compaction (also referred to as impact densification, heavy tamping, and dynamic consolidation), has evolved as an accepted method of site improvement by treating poor soils in situ. The method is often an economically attractive alternative for utilizing shallow foundations and preparing subgrades for construction when compared with conventional solutions (pile foundations, excavation and replacement, surcharging, etc).

In general, the ultimate goals of dynamic compaction are to increase bearing capacity and decrease total and differential settlements within a specified depth of improvement. The method consists of systematically dropping large weights (often with standard equipment) onto the ground surface to compact the underlying ground. Dynamic compaction seems especially advantageous in treating reclaimed land and heterogeneous fill materials, although some unique applications include forming stone columns, displacing unsuitable materials such as peat, and collapsing sinkholes.

Dynamic compaction has been utilized on a wide variety of soil types and conditions, primarily sandy materials and granular fills, although a limited number of cohesive soils have also been treated. The method has been used for different types of civil engineering projects, including building structures, highways, airports, coal facilities, dockyards, and reducing the liquefaction potential of loose soils in seismically active regions. Furthermore, the degree of improvement resulting from dynamic

[1]Sr. Geotechnical Engr., Law Engrg., Drawer QQ, McLean, Va. 22101.
[2]Pres., GeoSystems, Inc., P.O. Box 618, Sterling, Va. 22170.
[3]Pres., Geopac Inc., 680 Birch, St.-Lambert, Quebec, J4P 2N3.

Note.—Discussion open until November 1, 1984. To extend the closing date one month, a written request must be filed with the ASCE Manager of Technical and Professional Publications. The manuscript for this paper was submitted for review and possible publication on June 7, 1983. This paper is part of the *Journal of Geotechnical Engineering*, Vol. 110, No. 6, June, 1984. ©ASCE, ISSN 0733-9410/84/0006-0757/$01.00. Paper No. 18932.

compaction has been measured by a variety of field testing methods, such as standard penetration tests (SPT), cone penetration tests (CPT), pressuremeter tests (PMT), and other means. Consequently, each particular project has been treated according to different criteria, depending upon the specific soil conditions, engineering purpose, and performance monitoring program employed.

This paper reviews, for the benefit of foundation and structural engineers, general trends in terms of current practice and ground response to the treatment.

Source of Data

A number of engineering projects have reported success with the dynamic compaction method. Field data from 124 different sites have been compiled during this study (see Table 1). In the following, numerals enclosed in brackets [ ] refer to the sites listed in Table 1. The information was obtained from published articles, marketing brochures, and private unpublished reports prepared by either the soils consultant or tamping contractor. Where feasible, the data have been sorted by soil type.

Since each reference was not complete in every detail, many important factors could not be evaluated during this study, including: (1) Effect of groundwater levels; (2) percent fines (material passing the No. 200 sieve); (3) waiting period for verification tests; and (4) weight shape, construction materials, dimensions, etc. Nevertheless, approximate trends were observed when the field measurements are compared with the energy per drop or applied energy per unit area.

With respect to soil type, approximately half of the sites were underlain by natural soils and half underlain by fills of varying composition. Over 50% of the sites contained sand, sand fill, or hydraulically placed silty sand. In contrast, about 27% contained silt, clay or silty clayey fill. Twelve sites were comprised of rubble fills [Sites No. 6, 7, 36, 83], miscellaneous refuse fills [18, 19, 20, 46, 102] and sanitary landfills [47, 50, 88]. Dynamic compaction has also been used to densify mine spoil [Sites No. 32, 38, 71, 74, 84, 93]. In some rather unique applications, the method has been applied to rockfills [11, 21, 61, 76, 100], sinkholes [79], peat [27, 80], collapsible soils [105], to strengthen potentially liquefiable soil deposits [12, 35, 45], to collapse abandoned coal mines [106], and to densify soils under water [11, 45, 76, 92].

Current Practice

Methodology of Compaction.—Dynamic compaction involves the use of heavy steel or concrete blocks weighing typically 5–20 ton (4.5–18 tonne) which are dropped in free-fall from heights of up to 100 ft (30 m) using heavy crawler cranes. This order of compactive energy will allow the improvement of compressible soils to depths of as much as 50 ft (15 m). With special equipment (14,24,25,34,35,57,63,69) it is possible to drop heavier weights and thus affect soils to depths of 100 ft (30 m). The total cumulative applied energy levels typically range from 30–150 ft-ton/sq ft (100–400 tonne-m/m^2) as shown by the histogram in Fig. 1. Several sites, however, have been subjected to energy levels in excess of 200 ft-

TABLE 1.—Sources of Data on Dynamic Compaction

Number (1)	Site (2)	Reference number (3)	Number (4)	Site (5)	Reference number (6)
[1]	Karlstad, Sweden	34, 59, 69	[48]	English Midlands, U.K.	64
[2]	Thryborough, U.K.	69, 82	[49]	Narbonne, France	78
[3]	Indianapolis, Indiana	45	[50]	Hertfordshire, U.K.	10
[4]	Floor Slab, Chicago	47	[51]	Guildford, U.K.	10
[5]	Refinery Tanks, USA	47	[52]	Ambes, France	70, 78
[6]	Parking Garage, USA	47	[53]	Chicago, Illinois	84
[7]	Floor Truck Terminal, USA	47	[54]	Alexandria, Egypt	23, 65
[8]	Riviera, France	24	[55]	Three-Rivers, Canada	19
[9]	Jacksonville, Florida	11, 24	[56]	Riviere-Au-Renard, Gaspe	19
[10]	Papenburg, Germany	24	[57]	Duke Point, British Columbia	19
[11]	Al-Jlayah, Kuwait	24	[58]	North Vancouver, B.C.	19
[12]	Dominican Republic	24, 26	[59]	Brampton, Ontario	19
[13]	Belgium	14, 17	[60]	Montreal Harbor, Quebec	19
[14]	Berne, Switzerland	57	[61]	Prince Rupert, B.C.	19, 22
[15]	Pont de Clichy, France	29, 70	[62]	Methil, Scotland	18, 69
[16]	Scotland	83	[63]	Port Clarence	18
[17]	Israel	16	[64]	Kirkcaldy	18
[18]	Rouen, France	14	[65]	Burghead	18
[19]	Public School, Indiana	32, 79	[66]	Heston	18
[20]	Shopping Center, Indiana	47	[67]	Thorpe	18
[21]	Uddevalla, Sweden	14, 24, 35	[68]	Ryston, United Kingdom	18
[22]	Singapore Airport	72, 73	[69]	Jubail, Saudi Arabia	18
[23]	Soviet Union	1	[70]	Tunbridge	18
[24]	Swedish plastic clay	34	[71]	Wabamum, Alberta	19
[25]	Lavender Dock	66, 78	[72]	Sandy Clay, Illinois	48
[26]	Al-Jubail, Saudi Arabia	43	[73]	Germany	41
[27]	Eben, Austria	7, 59	[74]	Willisville, Illinois	33, 42
[28]	Road Research Lab, U.K.	75	[75]	Asnieres, France	78
[29]	Air Terminal, Singapore	59, 72	[76]	Brest, France	14, 24
[30]	Arrow, U.K.	75	[77]	Thunder Bay, Ontario	19
[31]	Teesside, England	59, 68	[78]	Assam, India	5
[32]	Corby Snatchill, U.K.	8, 9, 68, 78	[79]	Bayonet, Florida	80
[33]	Mentor Project, Ohio	81	[80]	Singapore Peat	72, 73
[34]	Dayton, Ohio	76, 81	[81]	Drop Ball, Illinois	85
[35]	Long Beach, California	24, 28	[82]	Santa Cruz, California	14
[36]	Georgetown, Washington, D.C.	76, 81	[83]	Baltimore, Maryland	30, 32
[37]	Seal Sand, England	14	[84]	Birmingham, Alabama	53
[38]	Port Mellon, B.C.	19, 20	[85]	Embourg, Belgium	70
[39]	Souk, Sharjah, UAE	57, 69	[86]	Ogawara, Japan	70
[40]	Nice Airport, France	25, 63	[87]	Lisbon, Portugal	70
[41]	Newport News, Virginia	49, 51, 52	[88]	Springdale, Arkansas	4, 81
[42]	LaSalle, Quebec	19	[89]	Durban, South Africa	38
[43]	Berlin, Germany	46	[90]	South Bronx, New York	86
[44]	Sofia, Bulgaria	60, 61	[91]	Ashuganj, Bangladesh	28, 74
[45]	Arabian Gulf	36	[92]	Lagos, Nigeria	27
[46]	London East End, U.K.	10	[93]	Moberly, Missouri	81
[47]	Redditch, U.K.	10	[94]	Quebec City, Canada	19

TABLE 1.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
[95]	Lincolnshire, England	59, 69	[111]	Lonsdale, Vancouver, B.C.	19, 21
[96]	Santos, Brazil	70	[112]	Alexandria, Virginia	50
[97]	Aluminum Foundry, U.K.	82	[113]	Pulau Ayer Merbau	74
[98]	Hampshire, U.K.	15	[114]	Tampa, Florida	2
[99]	Rivera, France	24	[115]	Myrtle Beach, South Carolina	54
[100]	Hull, Quebec	19	[116]	Rahimah, Saudi Arabia	40
[101]	Toronto, Ontario	19	[117]	Cadiz, Spain	12
[102]	LaBaie, Quebec	19	[118]	Belawan, Sumatra	39
[103]	L'Assomption, Quebec	19	[119]	Mongstad, Norway	6
[104]	Laurel, Maryland	81	[120]	Borneo, Indonesia	13
[105]	Algodones, New Mexico	81	[121]	Oulu, Finland	37
[106]	Prittstown, Pennsylvania	77	[122]	Helsinki, Finland	44
[107]	Cwmbran, U.K.	69	[123]	Tianjin, China	71
[108]	Canterbury, U.K.	69	[124]	Charlottesville, Virginia	55
[109]	Boran, France	69			
[110]	Jeddah, Saudi Arabia	69			

Note: Brackets [ ] in table, text, and figures indicate site number.

ton/sq ft (600 tonne-m/m^2) in order to achieve the desired results from dynamic compaction [21, 27, 58, 80, 88].

The spatial distribution of the compactive energy and the chronological sequence of its application is critical in achieving successful compaction, particularly of the deeper zones to be treated. In the early stage of the work, impacts are spaced at a distance which is dictated by the depth of the compressible layer, the depth to ground water, and grain size distribution. Initial grid spacing is usually at least equal to the thickness of the compressible layer and up to 50 drops could be used at each impact point.

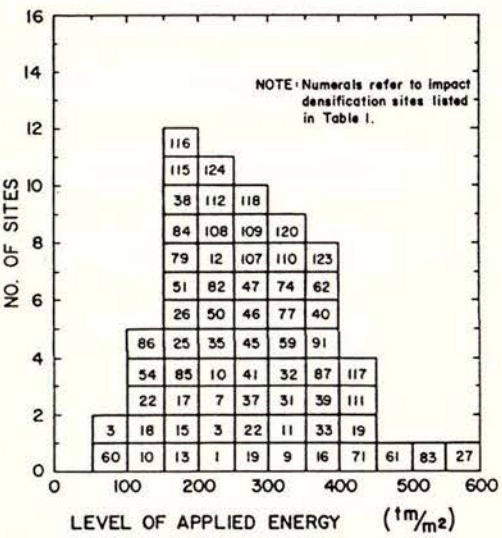

FIG. 1.—Histogram of Energy Levels per Unit Area Applied on Dynamic Compaction Projects

This first phase of the treatment with wide spaced impacts, normally called a pass, is designed to improve the deeper layers. Incorrect spacing and energy at this stage could result in the creation of a raft of dense material at an intermediate level, making it difficult, if not impossible, to treat loose materials below.

In saturated fine-grained soils the process is complicated by the creation of increased pore-water pressures during compaction, a phenomenon which will reduce the effectiveness of the subsequent compactive passes, unless it is properly recognized and sufficient delay is planned between succeeding passes to allow these pressures to dissipate. Consequently, varying degrees of success have been reported in the dynamic compaction of saturated cohesive materials (8,9,11,13,19,24,56,59, 63,64,67,71,72,78).

After each pass, the imprints are usually backfilled with the surrounding materials. In that case, the working platform is gradually lowered by an amount which is proportional to the densification achieved during each pass. In some circumstances, it may be necessary to maintain the working platform at a constant level throughout the work. Such would be the case, for instance, in a situation of a high water table, necessitating that the craters be backfilled with imported materials.

The initial passes are also called the "high energy phases," because the compactive energy is concentrated on points distant by at least 10 ft (3 m). The initial passes are followed at the end by a low energy pass, called "ironing," to densify the surficial layers in the interval from 0–5 ft (0–1.5 m).

Compaction Equipment.—The weights used on dynamic compaction projects have been typically constructed of steel plates, sand or concrete-filled steel shells, and reinforced concrete. In addition, several investigators used laboratory systems to investigate the impacts of falling weights [28,30,53]. Typically, weights range from 5–20 ton (4.5–18 tonne), although the Gigamachine [40] used a 190-ton (172-tonne) steel weight. Base configurations are square, circular, or octagonal. The latter two are better suited for primary phases of tamping since little energy is wasted in forming the eventually circular crater shape (19). Square weights are better used for ironing phases. For underwater applications, special hollow shapes have been designed to increase the fall velocity through water (36).

In practice, the weight and drop height are not independent parameters (Fig. 2). In order to maximize the effect of dynamic compaction, cranes are utilized to lift a given weight to the highest drop height possible, considering the structural and operational limitations of the system. Excluding special tripod rigs developed by Menard and his associates (25,36,57,69), standard crawler cranes have essentially been restricted to maximum weights of 25 ton (23 tonne) and drop heights of 100 ft (30 m). The Mega tripod [12,21,35] lifted 44 ton (40 tonne) to heights of 130 ft (40 m). The drop height of the Gigamachine [40] was limited to 75 ft (23 m) because of airport safety restrictions.

Control Testing.—Depending upon the type of engineering project, the aim of dynamic compaction is to improve the strength and compressibility characteristics of the underlying soils within some desired depth interval below the ground surface. To successfully improve ground

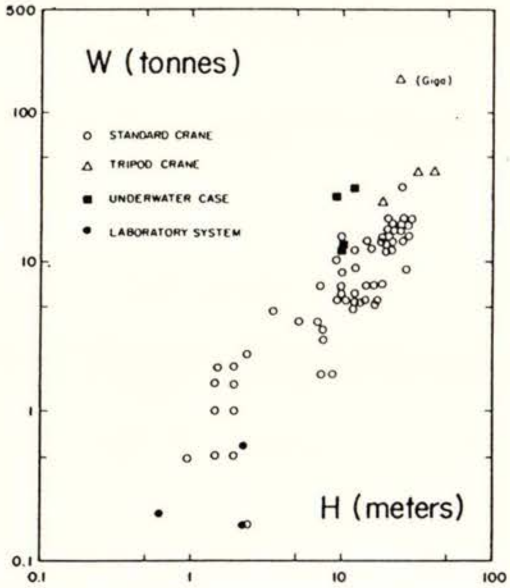

FIG. 2.—Relationship between Size of Weight and Drop Height

in situ, certain quality control measures must be undertaken to ensure that improvement does indeed occur. Adequate ground surface coverage is verified by field surveys of the crater locations and depths. The ironing pass, which is carried out at the end of the treatment to densify the near surface materials loosened by the high energy passes, is also particularly useful for determining areas where high pore pressures still prevail.

Control testing may be divided into three types: production, environmental, and specification. Production control testing includes the quality assurance aspects such as logging of imprints, elevation survey measurements, and in situ geotechnical testing methods. Environmental controls consist of measuring ground vibration levels and boundary surveys to minimize the effects of the tamping operations on adjacent properties. When compacting in close proximity to existing structures, it may include instrumentation such as inclinometers or subsurface settlement points. Specification controls establish the minimum required goals needed to certify an allowable bearing pressure or allowable differential settlement criterion.

The depth and degree of improvement are often evaluated by comparing field measurements before and after dynamic compaction. Reviewing the sites in Table 1, a variety of field testing methods have been employed for monitoring the performance. The most common geotechnical tests used on these projects include SPT, CPT, and PMT. Other types of field measurements include pore pressure monitors, peak particle velocities, subsurface settlement points, geophysical surveys [21, 74,84,98], dynamic cone tests [32,59,71,77], loss point or Becker probes [34,74], surcharge/plate load tests [2,21,88,95], field vane tests [9,59,95], and dilatometer tests [105].

The degree of improvement due to dynamic compaction has also been evaluated by comparing soil properties before and after heavy tamping, as determined from conventional laboratory tests [9,12,29,32,41,45,49, 51,80].

Ground Response

Induced Subsidence.—Dynamic compaction causes an areal subsidence to occur within the area treated. In unsaturated materials above the ground-water table, this occurs relatively quickly, whereas in saturated soils below the ground-water table, the subsidence occurs more slowly, as the cyclic pore pressures dissipate with time.

Since the energy is applied to points on a preselected grid, the most obvious manifestations of this subsidence are the relatively large craters induced at each impact point. Typically, the craters are 3–6 ft (1–2 m) deep. A summary of crater depths as a function of number of blows is presented in Fig. 3 for several sites. Generally, negligible heave outside the point of impact was reported for these sites. Soil types included silty sands [41,82], sand fills [3,54], rubble [36], rockfill [14], and coal spoil [84]. Crater depths reported by Ramaswamy, et al. (72,73) for a site with peat were not included in Fig. 3 since mixing of different soil materials apparently took place at that site and not simply a reduction of void ratio.

When the crater measurements are normalized with respect to the square root of energy per blow, as shown in Fig. 4, the data fall within a rather narrow band. Crater measurements may be used for selecting the optimal number of blows per pass (45) and estimating the average areal subsidence caused by the dynamic compaction process. In addition, a summary of the crater depths plotted on a site plan helps in recognizing anomalous areas requiring additional treatment or possible undercutting.

After each pass of dynamic compaction, the surface of the site is relevelled by bulldozing surface materials into the craters. The settlement caused by each pass of compaction can then be measured by topographic survey. Several sites are reported to have subsided as much as 6 ft (2 m) or more [21,27,83,88]. The magnitude of ground surface subsidence depends upon the applied energy per unit area (14,21,25, 31,43,79). Reviewing the data base, a comparison of induced ground settlements is made in Fig. 5 for different soil types and a similar trend is apparent for all sites considered. Although not evaluated in this study, the thickness of the compressible layer is probably another important factor governing this relationship.

Ground Vibrations.—The impact of falling weights causes ground surface vibrations. Peak particle velocities (PPV) are generally used to define damage criteria for building structures and annoyance levels to persons, especially in urban environments. The PPV are measured in the field with velocity recorder seismographs. The attenuation of PPV is site dependent and is related to the scaled distance (horizontal distance, *d*, divided by the square root of the energy). A compilation of available PPV data from several dynamic compaction projects is presented in Fig. 6. Soil types at these sites included silty sands [35,39,41,82], sandy fills [3,15,112,114], sandy clay [72], rubble [6], coal spoil [84], and debris fill [19]. For preliminary estimates of ground vibration levels, a

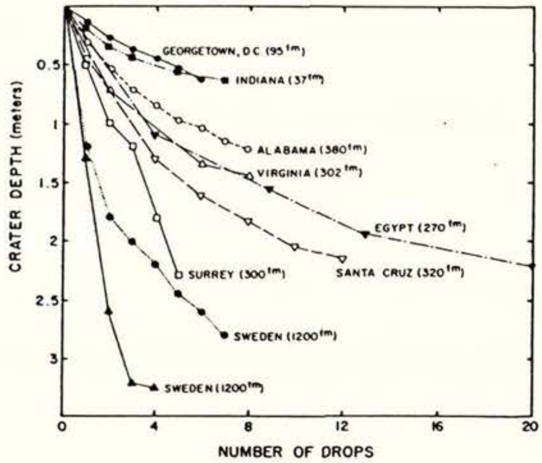

FIG. 3.—Crater Depths as Function of Number of Blows for Several Different Sites

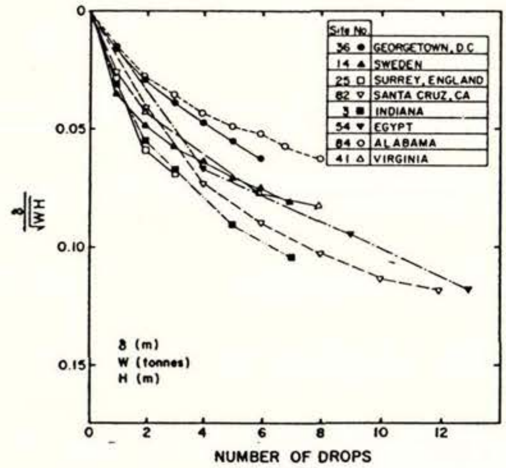

FIG. 4.—Normalized Crater Measurements

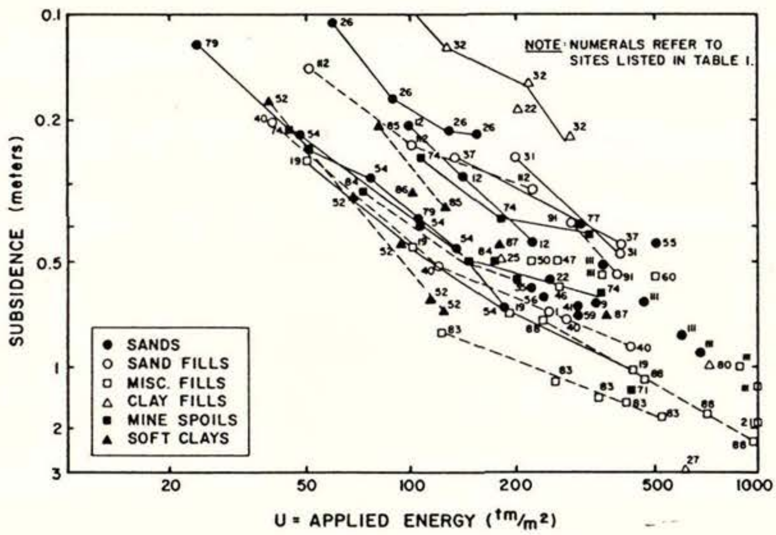

FIG. 5.—Observed Magnitude of Ground Subsidence with Level of Applied Energy per Unit Area

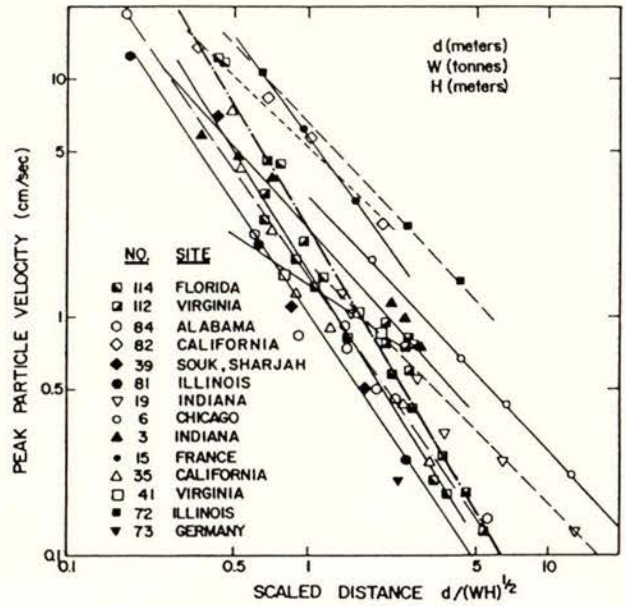

FIG. 6.—Attenuation of Ground Vibrations Measured on Different Dynamic Compaction Projects

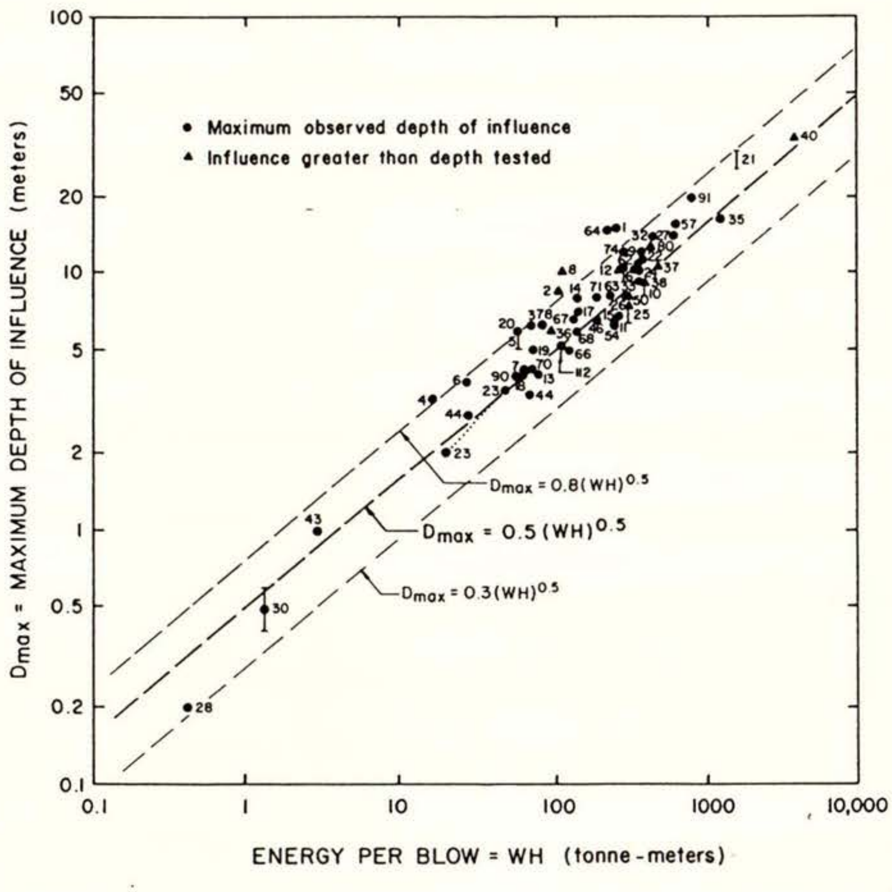

FIG. 7.—Trend between Apparent Maximum Depth of Influence and Energy per Blow (Note: Numerals Refer to Sites Listed in Table 1)

conservative upper limit appears to be

$$\text{PPV (cm/s)} \leq 7\left(\frac{\sqrt{WH}}{d}\right)^{1.4} \quad \text{(1)}$$

in which d and H are in meters; and W in tonnes. It should be noted that, within the treated area, PPV measurements tend to increase with the number of blows as the materials become more dense (19,22,69). Recommendations concerning safe vibration levels during dynamic compaction are given by Mitchell (62,63) and Wiss (85).

Depth of Influence.—The application of dynamic compaction at the ground surface is limited in its effect on the subsurface soils. Menard and Broise (59) suggested that the depth of influence, d_{max}, is as great as the square root of the product of weight, W, times drop height, H, or energy per blow. Several investigators have modified this expression for soil type, crane efficiency, and energy level (10,18,33,45,47,62). The compilation of data presented in Fig. 7 indicates that a conservative estimate is approximately

$$d_{max} = \frac{1}{2}\sqrt{\frac{WH}{n}} \quad \text{(2)}$$

in which n = units factor = 1 tonne/meter = 672 lb/ft. A sufficient number of drops and adequate coverage of the site area must be made, of course, so that the subsurface soils "remember" the dynamic stresses imposed on them by the compaction process. The greatest depth of influence achieved to date has been at the Nice Airport (>33 m) using the Giga tripod machine [40], which delivers approximately 3,900 tonne-m of energy per blow (28,000,000 ft-lb).

The degree of soil improvement has been observed to achieve a maximum at a critical depth, d_c, and then diminish with depth until reaching d_{max}, below which the soil properties remain unchanged. An example of the improvement in loose silty sand obtained at a 33-acre coal handling facility in Newport News, Virginia, is shown in Fig. 8 in terms of SPT, CPT and PMT data. Believed to be the largest area yet treated in the United States, the site was dynamically compacted using 3 cranes (WH = 400 to 480 tm) and took 6 months to complete (49,51,52). The depth of influence and critical depth were approximately 33 and 16 ft (10 and 5 m), respectively, for this site. A cursory review of the available data by the writers indicated that the critical depth is roughly one-half the maximum depth of influence.

The apparent depth of influence, d_{max}, of the dynamic compaction program is paramount to the proper selection of the crane and weight beforehand, since the mobilization of these items has a significant impact on the total cost. Although the relationship previously presented as Fig. 7 indicates that, on the average, the depth of influence is often conservatively estimated to be one-half $\sqrt{WH}$, a closer examination reveals that the depth of influence may be as low as one-third or as high as one times $\sqrt{WH}$.

In this respect, it is useful to point out that optimization of results in terms of depth of influence depends not only on the appropriate selec-

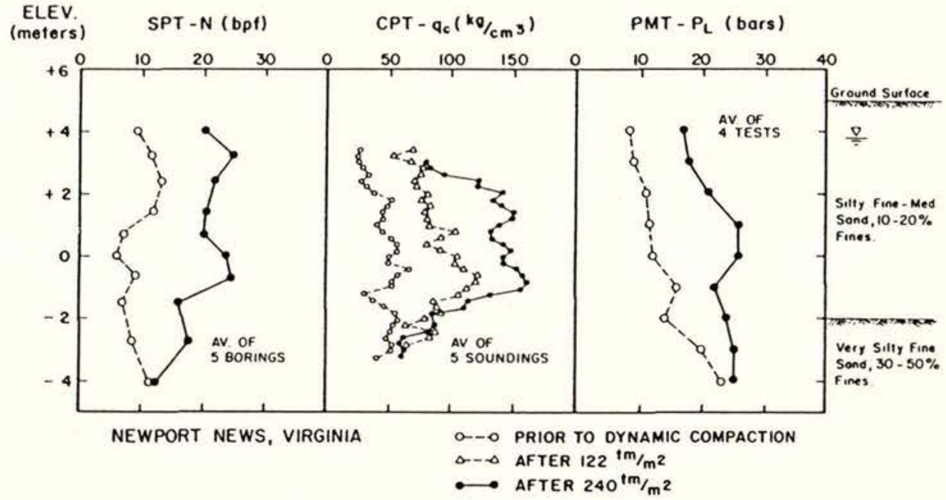

FIG. 8.—Degree of Improvement from Dynamic Compaction at Massey Coal Terminal, Newport News, Virginia [Site No. 41]

tion of *W* and *H*, but also on the proper assessment of the variable and given parameters characterizing each project. In addition to *W* and *H*, the variable parameters include the surface area of weight, the initial and final grid spacing, number of passes, time delay between passes, etc. The nonvariable or given parameters include the existing soil types (sand, clay, fill), initial soil conditions (loose, soft), ground-water levels, etc.

Pressuremeter Tests.—Because Menard, et al. (56,57,59) developed "dynamic consolidation" into a marketable construction technique, and since Menard also introduced PMT equipment to the geotechnical community, a considerable amount of data before and after compaction exists in terms of pressuremeter modulus and limit pressure. An advantage of PMT over CPT and SPT is that an entire stress-strain-strength relationship is developed. The PMT modulus is a measure of soil compressibility and the limit pressure is an indicator of shear strength.

As shown by Gambin (25) and Guyot and Varaksin (33), the limit pressure above the critical depth tends to increase with the level of applied energy per unit area. This is evident from the summary of PMT data collected from various sites and presented in Figs. 9–10. Initial limit pressures for sandy soils before improvement are typically between 4 and 8 bars, and for clays between 1 and 3 bars.

A similar trend is apparent between the pressuremeter modulus after heavy tamping and the applied energy per unit area, as shown by Fig. 11 for sands, granular materials, and miscellaneous fills. Generally, the initial PMT moduli for these sites were less than 50 bars before site improvement.

The degree of improvement in terms of PMT data appears more significant for sands, granular fills, and rubble fills than for clayey soils. The actual degree of improvement for a particular site should be determined by field verification tests conducted before and after heavy tamping. In addition, a sufficient waiting period should be provided for the development of thixotropic effects and the dissipation of excess pore pressures in the case of soils below the water table.

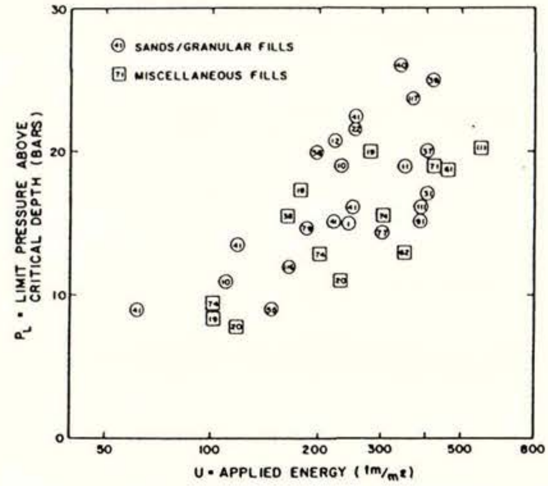

FIG. 9.—Observed Trend between Limit Pressure and Applied Energy Level for Granular Soils

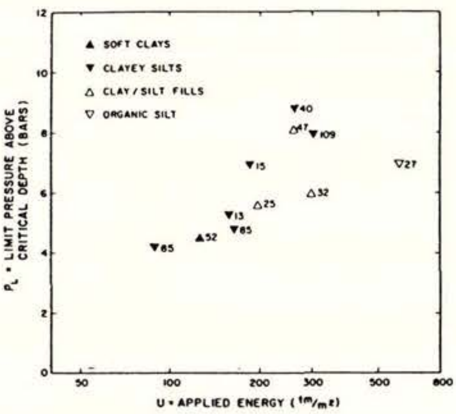

FIG. 10.—Observed Trend between Limit Pressure and Applied Energy Level for Cohesive Soils

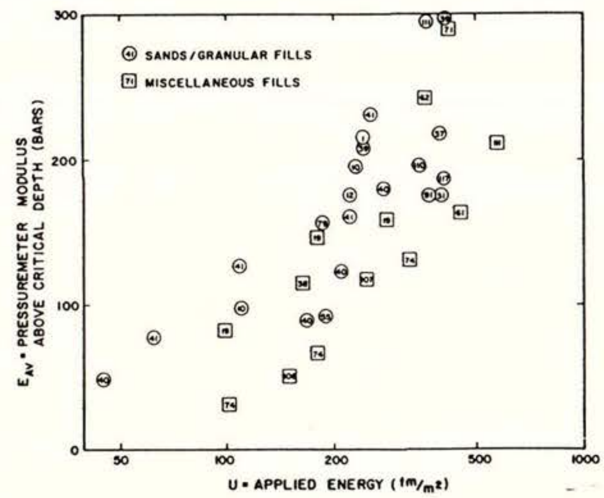

FIG. 11.—Increase in Pressuremeter Modulus in Granular Soils and Fills with Applied Energy per Unit Area

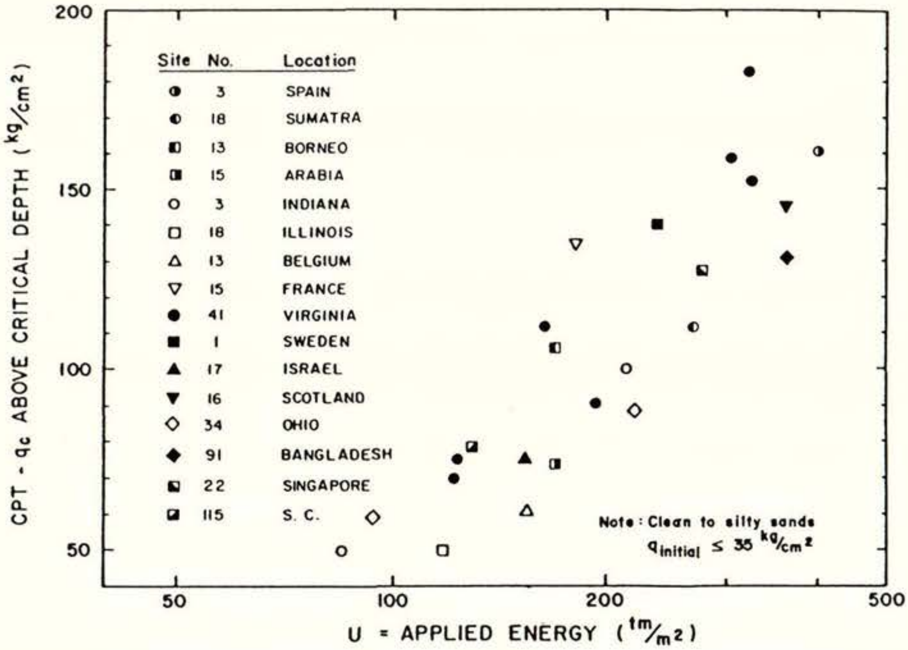

FIG. 12.—Relationship between Static Cone Resistance and Applied Energy Level for Granular Soils

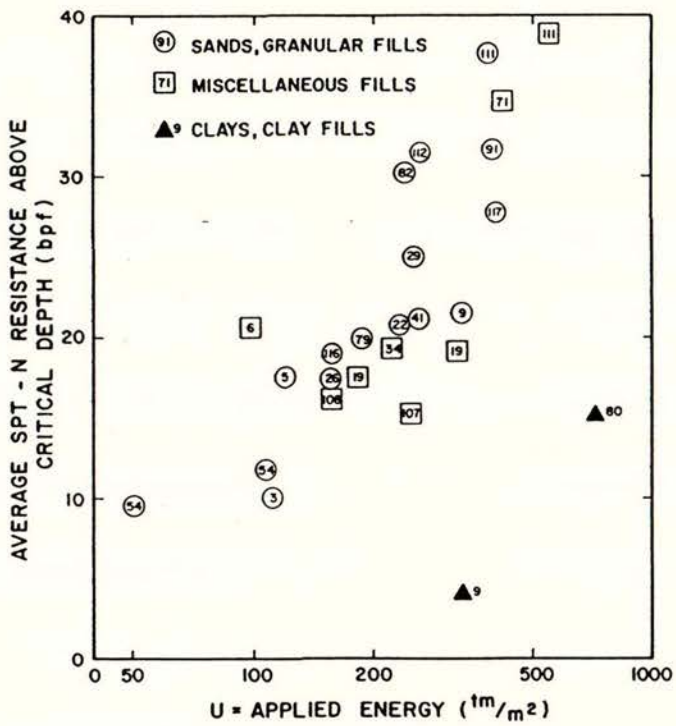

FIG. 13.—Observed Trend between SPT-N Value and Applied Energy per Unit Area

Penetration Tests.—Standard penetration tests (SPT) and cone penetration tests (CPT) are easier, quicker, and more economical to perform than pressuremeter tests. Consequently, over 300 CPT soundings were used at the large coal handling facility in Newport News [41] to verify the dynamic compaction program. Consistent with the PMT trends, the cone penetration resistance after compaction also appears to be related to the applied energy per unit area (Fig. 12). Data are from granular soils only. A similar trend in SPT resistance after compaction has also been developed (53), and is presented as Fig. 13. As apparent from the few data on clays, the trend is much less significant than for sandy soils.

Conclusions

The development of a ground improvement technique into an effective construction method depends, among other factors, on the awareness by potential users of its capacities. By establishing similarities that exist in the response of ground at more than 120 sites improved by dynamic compaction, this paper underlines some of the relationships linking the effectiveness of the process to the level of compactive energy applied, thus providing a database useful for the evaluation of the applicability of the process. Most of the data reviewed were obtained at sites underlain by granular soils and various types of fill materials. The relatively limited data available on cohesive materials have also been collected during this study.

Based on a review of the field data, general trends are developed which show that the size of the craters, ground vibration levels, and depth of influence increase with the level of energy per blow. The induced ground subsidence, static cone resistance, standard penetration resistance, limit pressure, and pressuremeter modulus after dynamic compaction are shown to increase with the applied energy per unit area.

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JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Densification of Loose Deposits by Pounding

By Robert G. Lukas,[1] M. ASCE

Introduction

Certain types of marginal sites can be improved to the point where one-story to four-story structures can be supported by conventional spread footing foundations. The improvement consists of densifying loose soil or fill deposits by means of pounding. The pounding process used to date has consisted of dropping a weight of 2 tons–6 tons (1.8 metric tons–5.4 metric tons) from heights of 30 ft–35 ft (9.2 m–10.7 m) to impact into the soil thereby causing densification to depths ranging from 10 ft–20 ft (3.1 m–6.1 m).

This process has been successfully used on eight project sites under the writer's supervision. At five of the sites, the subsurface conditions consisted of building rubble and miscellaneous fill overlying a medium strength natural clay or clayey silt deposit. A natural loose fine sand was present at two sites and one project consisted of a shopping center constructed on a former garbage dump. This paper examines the ground subsidence observed during the pounding process, the degree of densification achieved with depth below grade, ground vibrations associated with pounding and the performance of the structures which were supported upon the densified soils.

Description of Projects

The pertinent features of each of the eight projects are summarized in Table 1. Typically, the pounding was used to densify the upper loose deposit thereby enabling the structural loads to be supported at grade. In the case of buildings, spread footings were normally supported within the densified deposit utilizing a bearing pressure of 3,000 psf (144 kPa). In Projects 1 and 3, the site densification was undertaken to minimize future maintenance problems associated with constructing a pavement or slab-on-grade over a loose deposit.

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[1] Principal Engr., Soil Testing Services, Inc., Northbrook, Ill.

The earliest densification projects were undertaken with whatever weight was available and in most cases this consisted of a 2-ton (1.8-metric tons) wrecking

TABLE 1.—Description of Projects

Project number (1)	Project description (2)	Date (3)	Substrata densified (4)	Weight, in tons (5)	Drop, in feet (6)	Average ground depressions, in inches (7)	Coverage (8)
1	Freight terminal truck parking area	1971	10 ft of rubble with large voids	2	30	Tight areas—6 Loose areas—12	Grid at 3 ft center to center
2	Four-Story building	1975	9 ft of cinders and sand over 3 ft of rubble	2	25	3-9	Entire footing areas
3	Floor slab loaded to 2,300 pounds per square foot	1976	13 ft of loose fine sand	4.8	12	4-6	Grid at 10 ft center to center
4	Two-Story shopping center	1976–1978	Upper 15 ft–20 ft of 60 ft of miscellaneous fill	6	35	Typical—12 to 18 Occasional—18-36	Grid at 7 ft Spacing plus footings
5	Two-Story parking garage	1977	15 ft of building rubble	3.4	25	12–15	Grid at 7 ft center to center
6	Refinery racks	1978	Loose fine sand from 5 ft–13.5 ft	6	30	Typical—12 Occasional—18	Grid at 6 ft center to center
7	Two-Story parking garage	1978	7 ft of rubble fill over 6 ft of sandy silt	4	25	Typical—12 Occasional—32-47	Grid at 5 ft center to center
8	One-Story truck terminal building	1979	5 ft–16 ft of miscellaneous fill over saturated clayey silt	6	40	Typical—12-15 Occasional—24-36	Grid at 7 ft plus footings

Note: Sites 1, 2, 5, 7, and 8 underlain by medium strength clay; and sites 3 and 6 underlain by medium dense to dense sand. 1 ft = 0.305 m; 1 ton = 907 kg; 1 in. = 25.4 mm.

ball. This weight worked reasonably well on building rubble formations but the rounded shape was not suited for densification of other fill or natural soil deposits. The densification conducted after 1976 was generally done with a heavier weight such as a 3.5 ton–6 ton (3.2 metric tons–5.4 metric tons) size which has a flat bottom.

The amount of ground depression as a result of pounding is listed in Col. 7 of Table 1. This value represents an average range throughout the project and the individual craters were many times this amount. The depth of ground displacement by itself does not indicate the degree of improvement being attained but it does serve as a practical field guide.

The coverage applied to each site is shown in Col. 8 of Table 1. At some projects, the primary concern was improvement only at concentrated load points so pounding was undertaken at the footings. The weight was dropped on a grid basis throughout the footing area plus a short distance beyond the edges. On other projects, there was concern that the floor slab as well as the footings could settle so the entire building area plus a short distance beyond the edges was pounded on a grid basis with the distance between impact points being about 4 ft–10 ft (1.2 m–3.1 m). At two projects, the grid pounding was followed by pounding at individual footing locations thereby effecting double coverage at these locations. At each pounding location, the weight was dropped seven to nine times.

Improvement versus Depth

The primary purpose for using the pounding procedure is to achieve a significant densification at depths greater than can normally be achieved by compaction equipment or heavily-loaded proofrolling devices. Menard and Broise (1) have proposed the following formula as the first-step indicator of the required energy to achieve densification to a predetermined depth:

$$W \times H = D^2 \qquad (1)$$

in which W = weight, in metric tons; H = height of drop, in meters; and D = depth of improvement, in meters.

To investigate the amount of densification, borings were made at the project sites before and after the pounding process had been undertaken. These borings included Standard Penetration Resistance tests or pressuremeter tests, or both. The Standard Penetration Tests were generally used in the relatively uniform deposits such as natural sandy soils and the pressuremeter tests in the nonhomogeneous deposits such as miscellaneous fill materials. At four of the project sites, sufficient borings and tests were made to measure the degree of improvement as a function of depth. The results of these tests performed before and after pounding are shown on Figs. 1–3.

For the hammers in the range of 3.5 tons–6 tons (3.2 metric tons–5.4 metric tons) falling through a distance of 25 ft–40 ft (7.6 m–12.2 m), the improvement in soil properties was found to extend to levels on the order of 15 ft–20 ft (4.6 m–6.1 m) below grade. At and below this level, either the Standard Penetration Resistance Value or the limit pressure was found to be approximately the same after densification as it was initially. A comparison of the depth of improvement by Eq. 1 to the depth of improvement as measured from Standard Penetration or pressuremeter tests is presented in Table 2. These data indicate that the depth to which improvement occurs is only on the order of 65%–80% of the depth predicted by Eq. 1. The improvement of the soil properties was not uniform throughout and was greater at the upper levels diminishing to slight improvements at the deeper level.

One of the most beneficial effects of pounding is to collapse voids or to

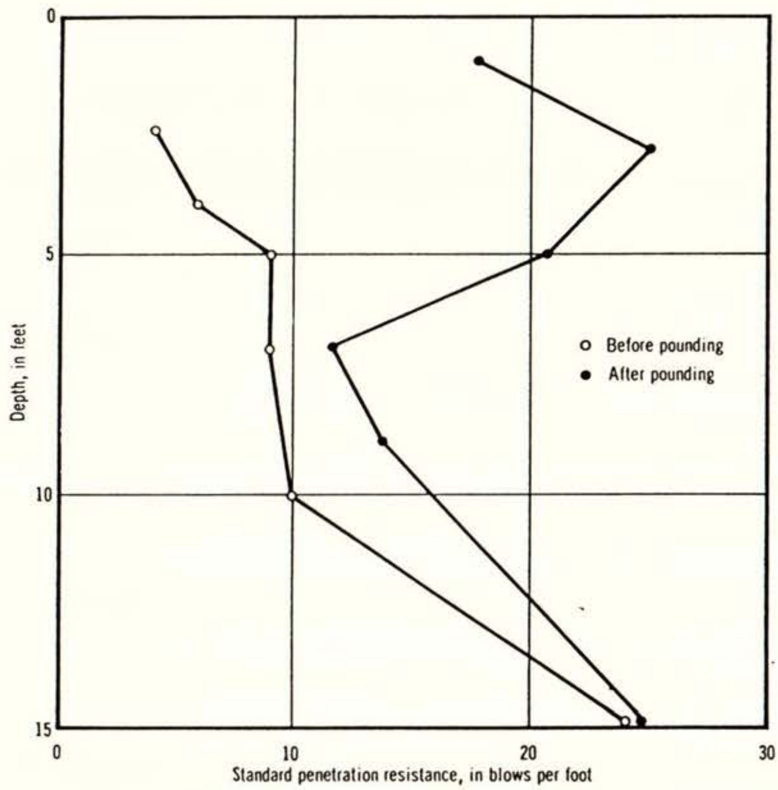

FIG. 1.—Standard Penetration Resistance Versus Depth: Site 3 (1 ft = 0.305 m)

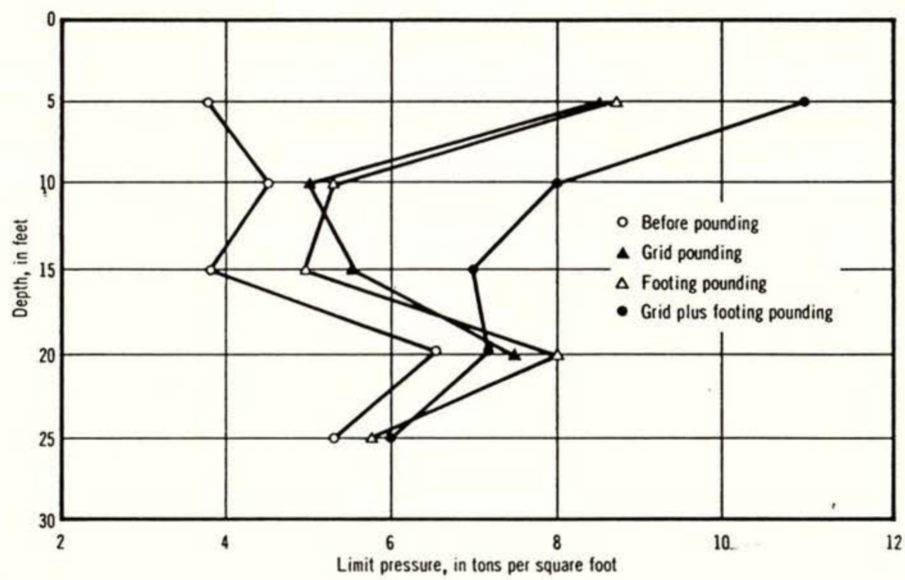

FIG. 2.—Limit Pressure versus Depth: Site 4 (1 ft = 0.305 m; 1 ton/sq ft = 95.8 kPa)

densify very loose layers. This is shown in Fig. 3(*a*). Within the depth range of 8 ft–14 ft (2.4 m–4.3 m) below ground surface, the initial investigation indicated an extremely loose deposit of sand with the Standard Penetration Resistance Value on the order of one blow per foot. After site densification, the Standard Penetration Resistance Value near the center and edge of this deposit increased

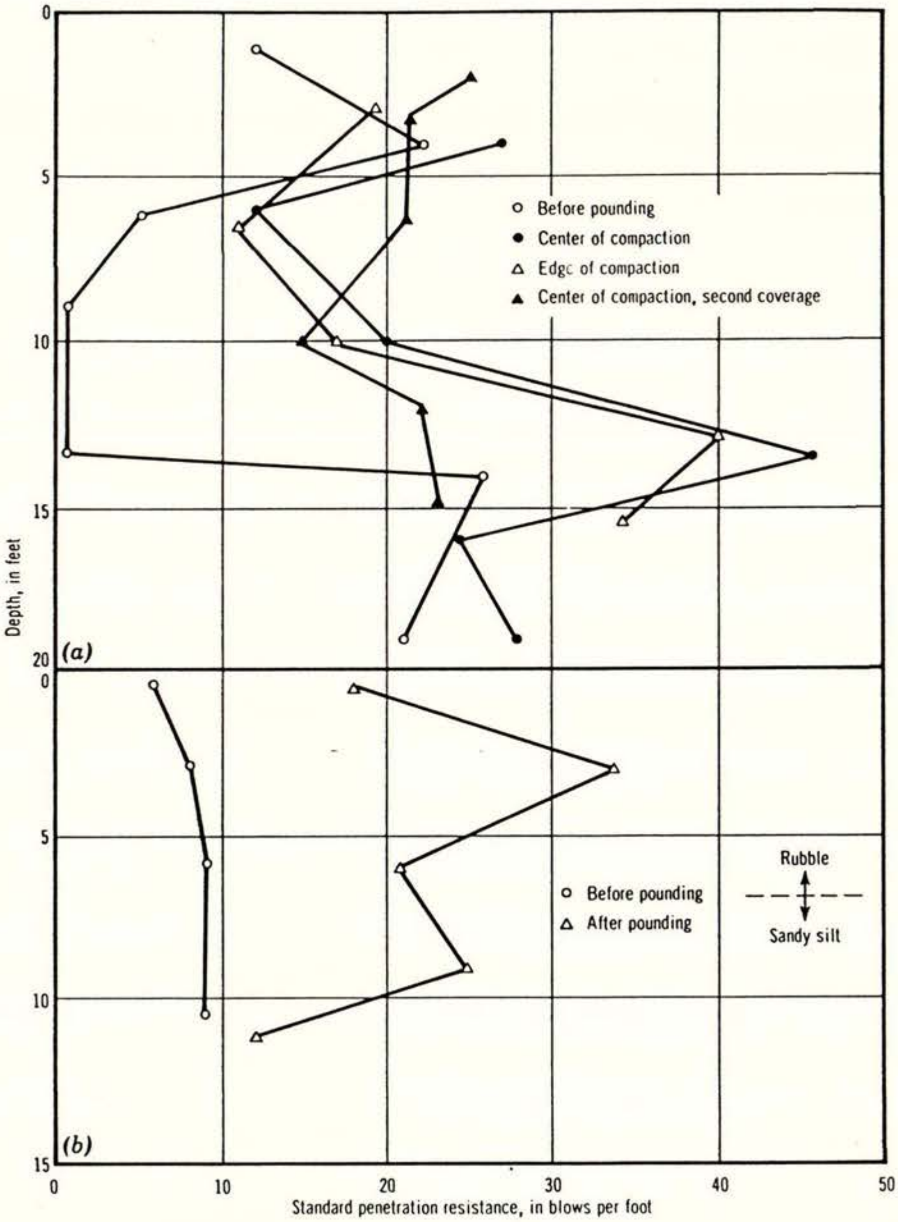

FIG. 3.—Standard Penetration Resistance Versus Depth: (*a*) Site 6; (*b*) Site 7 (1 ft = 0.305 m)

to 15 blows per ft–20 blows per ft. Within the upper portion of this deposit where the Standard Penetration Resistance Value was initially on the order of 22 blows per foot, the Standard Penetration Resistance Values after pounding were still only on the order of 20 blows per ft–27 blows per ft.

This data also indicates that the depth of improvement does not appear to increase with additional coverages. At Site 6, which is represented by Fig. 3(*a*), the depth of improvement was approx 15 ft (4.6 m) for single and double coverage. A similar phenomena occurred at Site 4 as represented by Fig. 2. In both areas the additional coverage improved the degree of compaction achieved in the upper levels.

Densification Mechanism

At seven project sites, the densification was undertaken in materials that were relatively free draining and not fully saturated. Densification of the deposit was due to compaction wherein the air within the void spaces was compressed

TABLE 2.—Improvement with Depth

Site (1)	Depth of improvement, in feet: Predicted from Eq. 1 (2)	Depth of improvement, in feet: Measured (3)	Ratio: measured/predicted (4)
3	13	10	0.77
4	25	20	0.80
6	24	16	0.67
7	17	11	0.65

Note: 1 ft = 0.305 m.

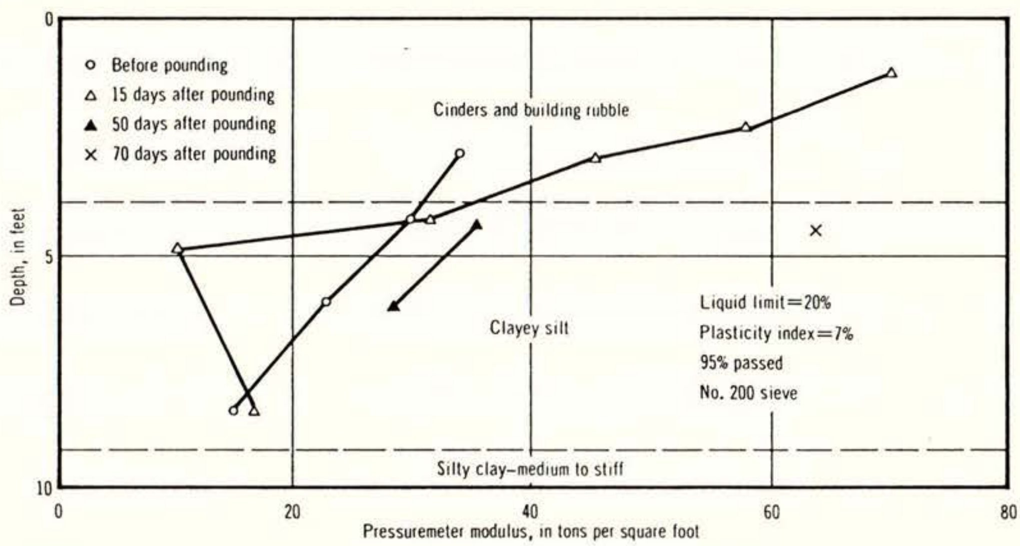

FIG. 4.—Pressuremeter Tests in Clayey Silt: Site 8 (1 ft = 0.305 m; 1 ton/sq ft = 95.8 kPa)

or expelled and large voids were collapsed. The improvement was immediate and the process could be described as dynamic compaction.

At two projects, double coverage was applied with a 6-ton (5.4 metric tons) weight dropping 35 ft–40 ft (10.7 m–12.2 m) with 9 tamps per impact point.

The data obtained from this study indicate that the depth of soil affected by pounding was approx 75% of the value computed from Eq. 1. The energy applied was assumed to be a maximum at ground surface and decrease hyperbolically to zero at this depth. Dividing the total energy applied at ground surface by the volume of soil computed as described, the unit energy applied at ground surface computes to 12,343 ft-lb/cu ft (60,311 kg-m/m^3) which is almost identical to Standard Proctor ASTM D-698 energy.

At Site 8, densification was required in areas where the saturated clayey silt soils were present below 5 ft (1.5 m) of rubble fill. Ground depressions occurred after impact but the ground surface behaved in a spongy manner and the soils liquefied. Shortly after pounding, water was observed to partially fill the craters. Measurements indicated that the ground water rose to a level 3-ft higher than normal and a period of 6 weeks elapsed before the water returned to the original position.

The liquefaction that occurred at Site 8 within the saturated clayey silt, is illustrated in Fig. 4. The pressuremeter modulus 15 days after pounding was lower than tests performed before pounding. After 50 days, the pressuremeter modulus improved to about 25% higher than the initial value. After 70 days, the modulus at the surface of the clayey silt was about double the original value. Samples taken 50 days after pounding indicated that the average water content of the silt dropped from 22%–19%. At Site 8, densification of the clayey silt deposit appears to be due to consolidation following liquefaction.

Ground Induced Vibrations

During the pounding process, a considerable amount of the energy is transmitted into the ground directly below the point of impact to densify the soil. However, some of the energy is transmitted through the ground to locations off the site. One of the concerns with regard to the pounding process is whether any damage could occur to buildings or utilities located beyond the edges of the site being densified.

At Sites 2, 4, and 5, the densification process took place in a relatively congested area adjacent to occupied structures. At Site 2, densification took place immediately adjacent to a one-story auto repair building and within 40 ft (12.2 m) of a 20-story high rise structure. The ground vibrations could be felt in both of these buildings but they were not of significant magnitude to cause damage even though the auto structure was a 50-yr old building. At Site 4, densification took place 20 ft (6.1 m) behind the retaining wall. The wall was measured to laterally deflect 1/8 in.–1/4 in. (3.2 mm–6.3 mm) at the top, but rebounded after each impact. In a restaurant building located about 75 ft–100 ft (22.8 m–30.5 m) away, the chandeliers were observed to swing for a period of about 5 sec–10 sec after impact, but no other adverse conditions were observed.

Site 5 is located in a downtown business district area adjacent to a 40-story high rise and across a city street from a three-story old railroad terminal building. For this project, the particle velocity was measured with two Sprengnether seismograph units. One unit was stationed on the sidewalk at the property line at a point 30-ft (9.2-m) distant from the point of impact and the second unit was moved to different locations around the site and included readings taken within the three-story railroad building where the tenants were complaining of vibrations. The results of the readings obtained and the distances from the point of impact are summarized in Table 3. All of the readings are below 2 in./sec (51 mm/s) which for low frequencies has been generally accepted as the level above which damage to residential structures could occur (2). After each impact, the wave frequency was measured in the range of 10 Hz–20 Hz but there was a complete decay before the next impact.

The seismic velocity readings have been plotted on Fig. 5 which relates scaled energy factor to particle velocity. The scaled energy factor is defined as the square root of the energy applied to the ground in foot pounds divided by the distance from the point of measurement to the point of impact. The chart was prepared by Wiss (2) to predict particle velocity resulting from pile driving operations when the subsoils consisted of wet sand, dry sand, and clay. At Site 5, the subsurface profile consists of building rubble. Points 1 to 4 of Table 3 define a new line on Fig. 5 which can be labeled as rubble fill. These 4 points were observed measurements made at ground surface at various distances from the point of impact. All the measurements taken within the building fall within the range that indicates perceptible vibration and this agrees with the reaction of the tenants. No damage occurred within this building even though the site densification took place over a period of about 3 weeks. The readings taken immediately adjacent to the area being densified indicates an objectionable range of ground vibrations but this instrument was located at the property line where no structures or permanent facilities were located.

TABLE 3.—Record of Vibration Measurements

Location (1)	Distance from point of impact, in feet (2)	Particle velocity, in inches per second (3)	$\sqrt{\text{Energy, in foot-pounds}}$/Distance, in feet (4)
1. Sidewalk at north property line	30	0.696	13.8
2. Sidewalk adjacent to railroad building	108	0.174	3.8
3. Street on west side of project	70	0.270	5.9
4. At grade beyond south edge of property	200	0.085	2.0
5. First floor of railroad building	155	0.051	2.7
6. Basement of railroad building	120	0.070	3.4
7. Second floor of railroad building	115	0.040	3.6
8. Roof of railroad building	115	0.055	3.6

Note: 1 ft = 0.305 m.; 1 in. = 25.4 mm; 1 lbf = 4.45 N.

One of the advantages of plotting the seismograph readings on a plot like Fig. 5 is that the data can be extrapolated to determine the distance from the point of impact where damage to a structure could occur. The generally accepted safe level for particle velocity to prevent damages to residences is 2 in./sec (51 mm/s) (2). Extrapolating the line labeled building rubble to an intersection with 2 in./sec (51 mm/s) would result in a scaled energy factor of about 41. For the amount of energy applied at this site, the anticipated distance at which the particle velocity would be 2 in./sec (51 mm/s) computes to be 10 ft (3.1 m). If the impact energy were changed to 6 tons (5.4 metric tons) and a drop

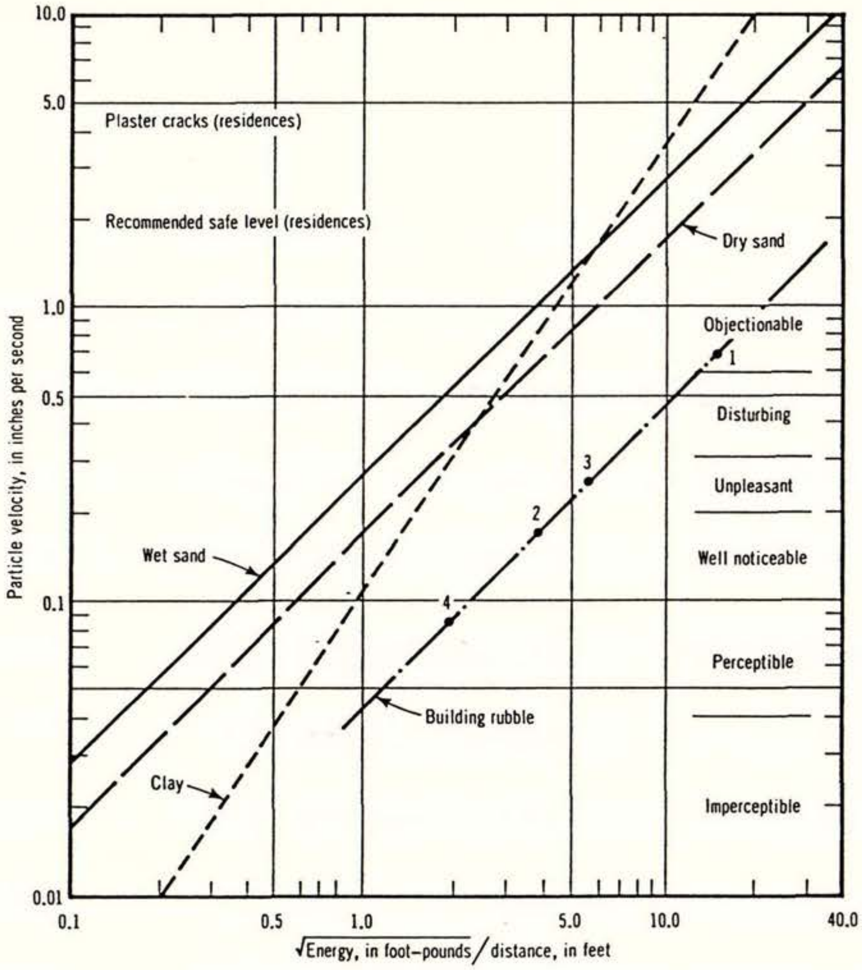

FIG. 5.—Scaled Energy Factor Versus Particle Velocity: Site 5

height of 40 ft (12.2 m), the distance beyond which the particle velocity is predicted to be less than 2 in./sec (51 mm/s) computes to be 17 ft (5.2 m).

For future projects, it would appear worthwhile to take measurements with a portable seismograph at varying distances from the point of impact during driving and then to plot the data on a chart such as Fig. 5 to develop the relationship between particle velocity and scaled energy for that particular site. This data could then be extrapolated to determine the appropriate distances that the points of impact should be kept from nearby structures to prevent damage.

Limitations

Some fill deposits were found to resist densification and had to be removed and replaced with a better material. This included fill containing a high proportion of wood, pockets of sawdust, and high water content organic soils.

Most materials respond to the pounding process by an improvement of the properties. However, there is a limitation as to how much property improvement can be achieved and this must be kept in mind when designing structures to be supported on these deposits. As an example, a localized area of Site 4 consisted of cinders, glass and clay fill. This deposit was densified by pounding and achieved a pressuremeter modulus of only 40 tsf–50 tsf (3.8 MPa–4.8 MPa), whereas, a pressuremeter modulus of 70 tsf–90 tsf (6.7 MPa–8.6 MPa) was typically achieved in other areas of the project. Additional pounding was undertaken in this area to further improve the properties of the soils but the modulus could not be improved. At Site 6, represented by Fig. 3(*a*), the Standard Penetration Resistance of the sandy soil at the 4 ft (1.2 m) level remained at about 20 blows/ft after single and double coverage.

Pounding of fine grained saturated soils should be approached with caution. The experience gained at one project site indicates that the degree of improvement attained is limited and occurs at a slow rate.

Structural Performance

The performance of Sites 1–3 and Sites 5–7 have been checked only by visual inspections. In all of these sites, no visual effects of settlement were observed. At Site 4, two of the shopping center buildings were monitored for settlement. At this site, there is 60 ft (18.3 m) of fill consisting primarily of an old refuse dump and miscellaneous fill that had been dumped over the years. Within Building A, 23 columns were monitored and within a period of 6 months after completion of the structure, the maximum observed settlement was 1/2 in. (13 mm). The typical column settlement was 1/4 in. (6.3 mm) or less. Within Building B, 67 columns were monitored and within 2 months after completion of the building, the average settlement ranged from 1/4 in.–9/16 in. (6.3 mm–14.5 mm). The settlement occurred as the column loads were applied and stopped when the structures were completed. The buildings were designed to take additional settlement due to potential future decomposition of the underlying organic matter within the fill deposits. Fortunately, the refuse which had been placed at this site was deposited 30 or more years ago and most of the organic decomposition has already occurred. In addition, open burning was undertaken at this pit when the refuse was dumped so a large part of the fill consisted of ashes and decomposed material. The column loads for both of these buildings are on the order of 300 kips (1,335 kN). Building A was completed in 1977 and Building B in 1978 and to date, the performance has been satisfactory.

At Site 8, footing settlement of 1 in.–2 in. (25 mm–51 mm) was recorded in an area where saturated clayey silt soils were present at footing level. This settlement occurred before any structural loads were applied to the footings and the settlement is attributed to dissipation of pore pressures following pounding. The footings were constructed 2 weeks–4 weeks after pounding and settlement continued until 6 weeks–8 weeks after pounding. The structural loads were applied 12 weeks after pounding and no measureable settlement occurred when these loads were applied.

Practical Applications

There are many marginal sites especially in urban areas. These marginal sites frequently consist of land that has been filled to raise the grade over soft ground deposits or where buildings have been wrecked and the rubble has been left in place to fill the former basement areas. The pounding process has proven to be practical and economical for improving these sites to support structures of one story–four stories in height. The costs of pounding wherein the site was improved to levels of 10 ft–15 ft (3.1 m–5.3 m) below grade have ranged from $0.50/sq ft–$1.00/sq ft. Alternative designs have been priced as more expensive. Removal and replacement with compacted fill assuming 10 ft (3.1 m) of existing fill depth has been priced at three to five times site improvement costs. The cost for extended foundations depends upon the length of foundation required. At Site 4, deep foundations were priced ten times site improvement costs while at Site 7, the cost ratio was 3.5.

Conclusions

On the basis of the data obtained in conjunction with the construction of eight projects, plus the performance of the structures afterwards, it can be concluded that:

1. The pounding process which consists of dropping a heavy weight through a predetermined distance to impact into the soil is a practical way of densifying certain marginal sites. In partly saturated materials above the water table densification is due to compaction plus a collapse of any large voids which may be present therein. In saturated clayey silt, densification is due to consolidation following liquefaction.
2. The depth of improvement was observed to levels of 10 ft–20 ft (3.1 m–6.1 m) below ground surface for weights on the order of 2 tons–6 tons (1.8 metric tons–5.4 metric tons) which were dropped through distances of 30 ft–35 ft (9.2 m–10.7 m) with 7 tamps per location–9 tamps per location. The depth of improvement was approx 65%–80% of the square root of the product of the weight in metric tons and the drop in meters. The number of coverages applied to the area does not appear to affect the depth of improvement.
3. The energy imparted to the improved zone is approximately equal to the Standard Proctor (ASTM D-698) energy when a 6-ton (5.4-metric ton) weight is dropped a height of 35 ft–40 ft (10.7 m–12.2 m) with 9 tamps per impact point.
4. When the weight impacts into the soil, ground vibrations are transmitted off the site. A method of estimating the particle velocity at a distance from the point of impact is described in the main text. For building rubble, the distance from the point of impact to the location where the particle velocity will be 2 in./sec (51 mm/s) computes to 17 ft (5.2 m) for a weight of 6 tons (5.4 metric tons) falling 40 ft (12.2 m).
5. Each material that is densified achieves a maximum or limited improvement.

This must be kept in mind when designing structures to be supported on these deposits.

Acknowledgments

The writer is indebted to the field engineering personnel who monitored the pounding operations and assisted in making field adjustments as this method of densification was developed and improved. In particular, Norman Seiler who was involved with five projects and Sylvio Pollici who was involved with one project, are deserving of special mention.

Appendix I.—References

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Appendix II.—Notation

The following symbols are used in this paper:

D = depth of improvement, in meters;
H = height of drop, in meters; and
W = weight of hammer, in metric tons.

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Dynamic Compaction of Granular Soils

By Gerald A. Leonards,[1] F. ASCE, William A. Cutter,[2]
and Robert D. Holtz,[3] Members ASCE

Introduction

Dynamic compaction was used to densify granular fill prior to the construction of a warehouse at National Starch and Chemical Corporation's Indianapolis plant. The original ground surface within the building area was approx EL 675 (206 m). During the 1930's, embankments of granular material were placed along the northern property line and through the central portion of the development area. The fill used to construct the embankments was a sand spoil from an adjacent gravel pit operation. The two embankments merged on the east side of the property, enclosing a triangular-shaped tract of land over which new construction was initially proposed.

Original plans called for the construction of a warehouse, approx 200 ft × 250 ft (61 m × 76 m) in plan, to be founded on controlled granular fill located entirely between the two spoil embankments. Grade for the structure was established at approx EL 693 (211 m). Subsequent to the filling and grading operations, it was decided to enlarge the warehouse to a plan area of approx 370 ft × 440 ft (113 m × 134 m) and to shift its location eastward. As a result of these changes, both the northeast and southeast corners of the warehouse structure were situated over the old spoil embankments, which had been constructed simply by end-dumping. As the project was being constructed by "fast-tracking," many of the footings for the enlarged plan had already been placed. This series of events led to the need for improvement of the old spoil embankments as expeditiously as possible.

Additional borings were made, and some typical results are shown in Fig. 1. Basically, the spoil materials were loose, fine to medium sand (with thin

Note.—Discussion open until June 1, 1980. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 106, No. GT1, January, 1980. Manuscript was submitted for review for possible publication on June 11, 1979.

[1] Prof., School of Civ. Engrg., Purdue Univ., West Lafayette, Ind.
[2] Sr. Project Engr., ATEC Associates, Inc., Indianapolis, Ind.
[3] Assoc. Prof., School of Civ. Engrg., Purdue Univ., West Lafayette, Ind.

gravelly seams) covered by well-compacted sand whose thickness increased with increasing distance from the crest of the old spoil piles. The amount of fines passing the U.S. No. 200 sieve ranged from 2%–10% and was typically 5%–6%. The depth to the underlying original ground surface varied from about 16 ft–20 ft (5 m–6 m), and the ground-water table was 30 ft–35 ft (9 m–10.5 m) below the present ground surface. A variety of ways for dealing with the problem was considered and densification was considered to be the cheapest and most expedient. Estimates were made of comparative costs and time to completion for excavation and replacement with controlled compacted backfill

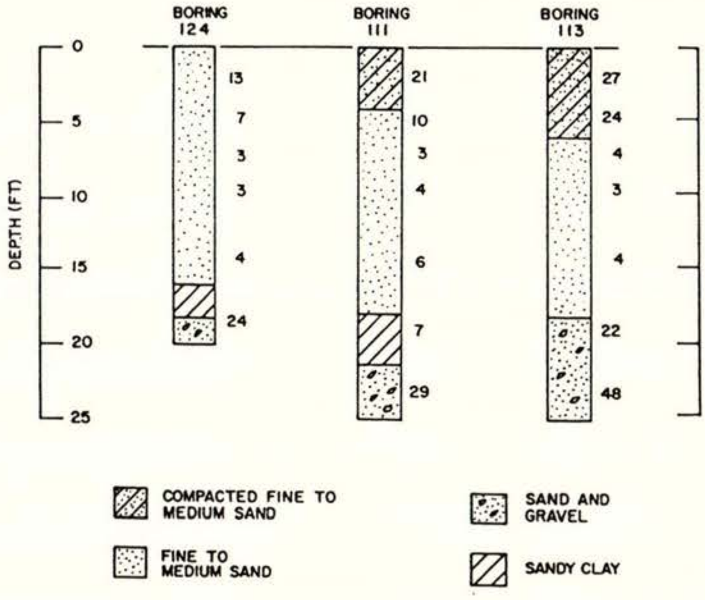

FIG. 1.—Typical Results of Additional Soil Borings [Numbers to Right of Boring Logs are SPT "N" Values (1 m = 3.28 ft)]

versus deep compaction in situ. Deep compaction with a heavy falling weight was selected on a trial basis.

Preliminary Trials

The requirements set for successful deep compaction were N values ≥ 15 to an effective depth of 18 ft (5.5 m), and it was decided to conduct a preliminary trial using a 4-1/2-ton (4.1-tonne) weight dropped 30 ft (9 m) in a pattern shown in Fig. 2(a). At drop point No. 1 the crater depth (at its center) was measured after successive drops (Fig. 3), whence it was decided to limit the number of drops to seven. Standard penetration, N, and Dutch cone penetration, q_c, tests were obtained before and after completion of the pattern, and the results are shown in Fig. 4. Three conclusions were drawn from this figure: (1) The effective depth of compaction was about 9 ft (3 m); (2) the N values were increased from values around 4 to as high as 10; and (3) the cone penetration resistance, q_c (in kilograms per square centimeter) $\simeq$ 4–5 N. While compaction requirements were not achieved, the results were sufficiently promising to justify another trial.

A 6-1/2-ton (5.9-tonne) weight dropped 40 ft (12 m) was selected for the second trial, in the pattern shown in Fig. 2(*b*), with the results shown in Fig. 5. Except for the first 2 ft–3 ft (0.6 m–1 m), the required compaction was achieved down to the underlying clay layer. Note that compaction is concentrated immediately below the drop coverage. It was also apparent that the clay layer absorbed energy remarkably well and prevented deeper densification. It was

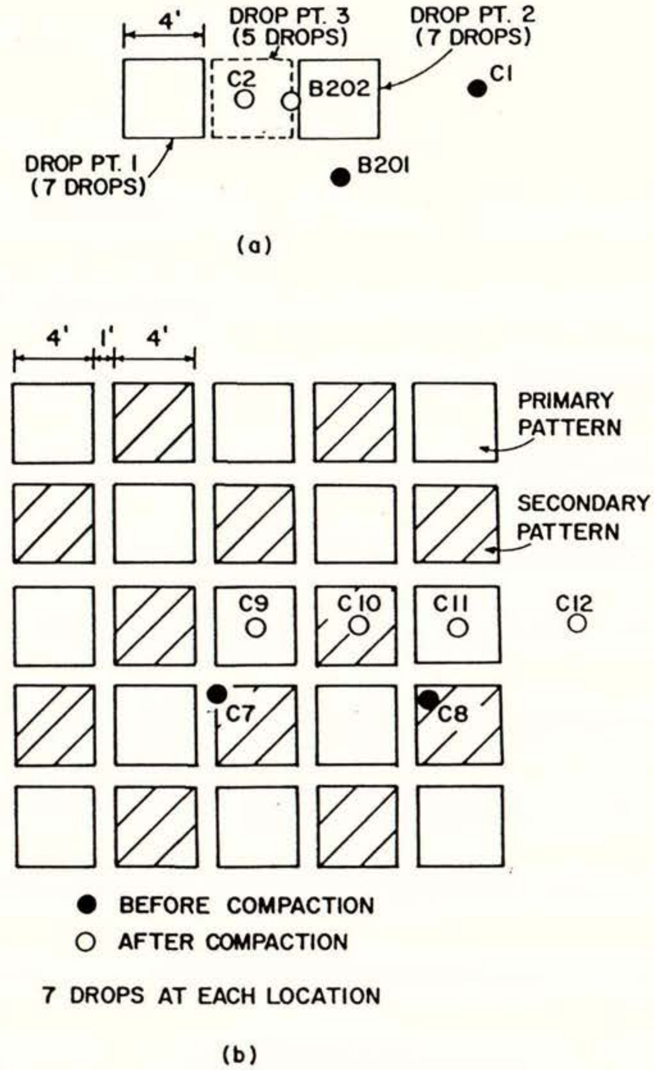

FIG. 2.—Number of Drops and Drop Pattern: (*a*) Trial No. 1—4.1 tonne Dropped 9 m; (*b*) Trial No. 2—5.9 tonne Dropped 12 m (1 m = 3.28 ft)

concluded that, with the clay layer at a greater depth in the area to be improved, dynamic compaction in the pattern shown in Fig. 2(*b*) at each footing location should meet the requirements.

Results of Dynamic Compaction

A grid was outlined at each footing and compaction was commenced. Figs. 6 and 7 are typical of the results achieved. In all cases, sufficient compaction

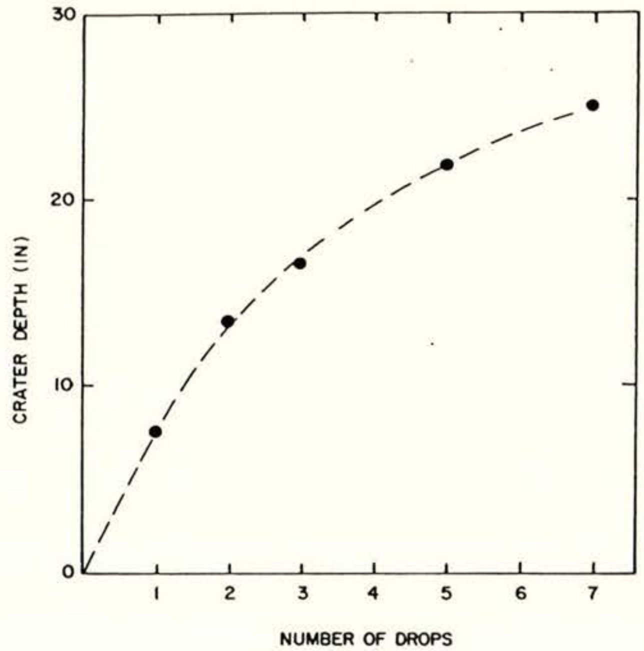

FIG. 3.—Crater Depth Versus Number of Drops—Trial No. 1, Drop Point No. 1 (1 in. = 25.4 mm)

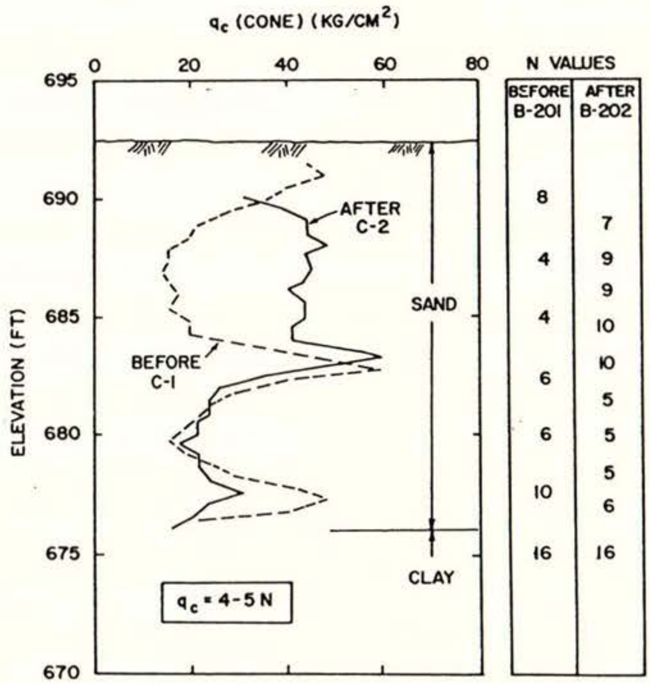

FIG. 4.—Cone Penetration Resistance Versus Depth Before and After Dynamic Compaction, Trial No. 1 (1 m = 3.28 ft)

was obtained to the desired depth and the footings were proportioned using a contact pressure of 3.5 ksf (168 kPa). The warehouse has been in service for 2 yr: measurements on brass plugs embedded in the columns showed that the maximum total settlement was less than 0.2 in. (5 mm). Area compaction of lesser intensity was applied between footings to support the thin slab on

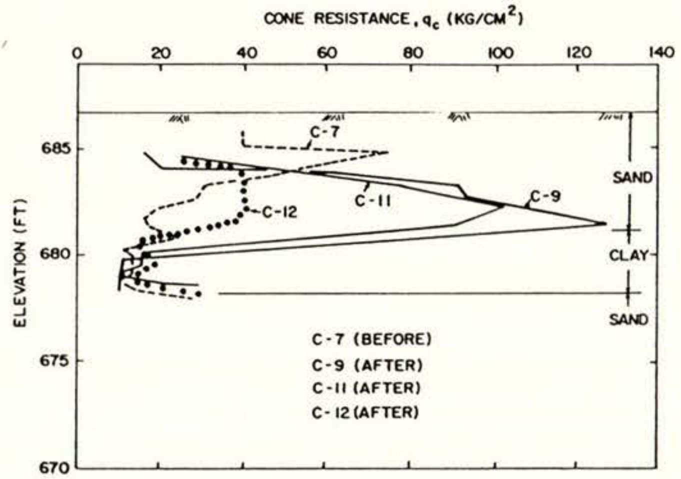

FIG. 5.—Cone Penetration Resistance Versus Depth Before and After Dynamic Compaction, Trial No. 2 (1 m = 3.28 ft)

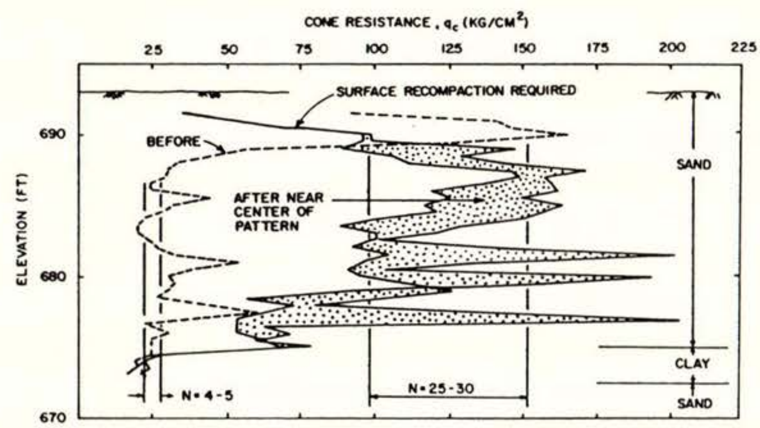

FIG. 6.—Cone Penetration Resistance Versus Depth Before and After Dynamic Compaction, Footing $H-1$ (1 m = 3.28 ft)

ground used for the warehouse floor. Although measurements were not made on the floor slab, its settlement has not been noticeable.

Comparison with Published Data

As a guide for future work, compare the experience in Indianapolis with those available in the literature. Fig. 8 shows the relation obtained between the energy/drop and the depth to which significant densification took place. A common rule of thumb (1) is that

$$D \simeq \sqrt{Wh} \quad (1)$$

in which D = depth of influence, in meters; W = falling weight, in metric tonnes; and h = height of drop, in meters. (A suitable criterion for the depth of influence would depend on the soil type and its initial state of compaction; for the purpose of this paper, the criterion was an increase in N value of more than 3 to 5.) It appears that the use of this rule tends to overestimate

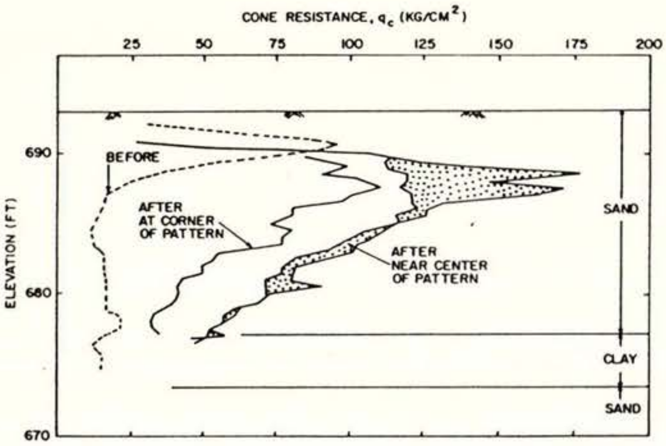

FIG. 7.—Cone Penetration Resistance Versus Depth Before and After Dynamic Compaction, Footing $H-3$ (1 m = 3.28 ft)

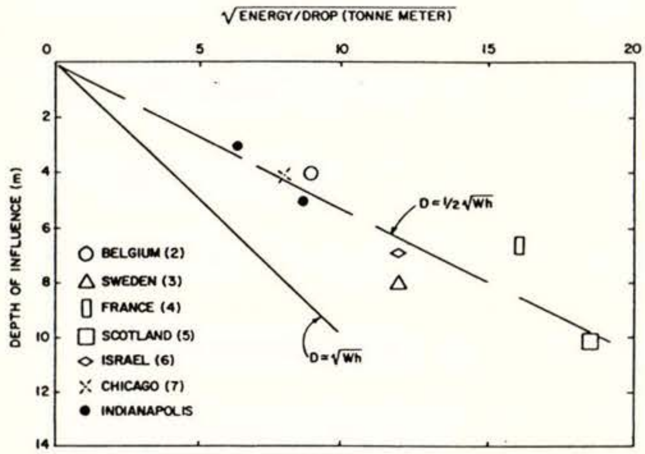

FIG. 8.—Depth of Compaction Influence Versus Square Root of Energy Per Drop

the effective depth of compaction substantially, and that $D \simeq (1/2)\sqrt{Wh}$ more nearly reflects available experience.

The degree of compaction attained depends not only on the energy per drop but also on the sequence of drop points and the number of drops at each point. Available data suggest that degree of compaction (as measured by the cone penetration resistance) correlates best with the product of the energy per drop times the total energy applied per unit of surface area (Fig. 9). It appears there may be an upper bound to the densification that can be achieved,

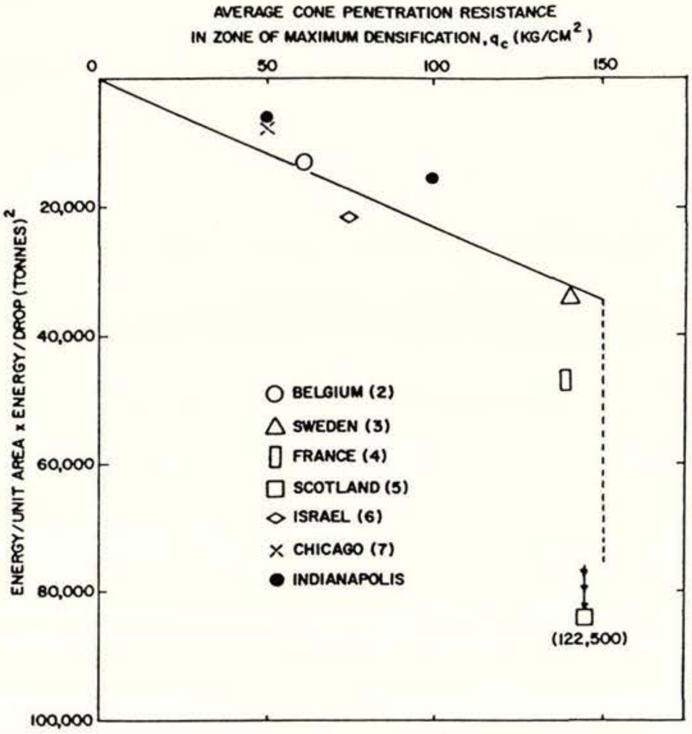

FIG. 9.—Cone Penetration Resistance Versus Product of Energy/Unit Area Times Energy/Drop

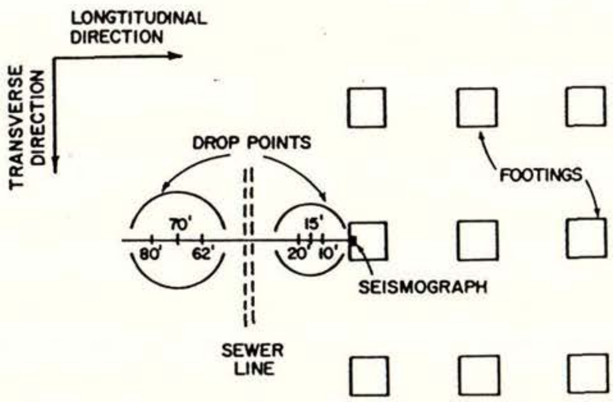

FIG. 10.—Plan Layout of Drop Points for Vibration Measurements, 5.9 tonnes Dropped 12 m Twice at Each Location (1 m = 3.28 ft)

corresponding approximately to q_c = 150 kg/cm^2, but more data are needed to verify this result.

Vibration Effects

At Indianapolis, the possibility of further extensions to the plant made it desirable to measure the relation between the distance of a drop point from an existing structure and the induced vibrations. A plan of the experiment

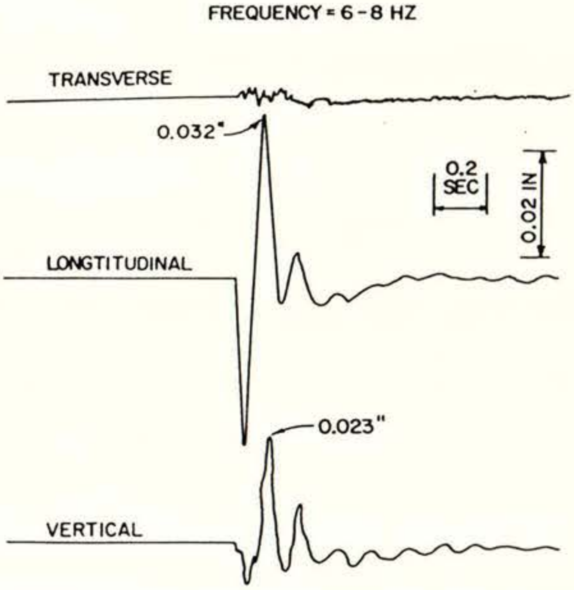

FIG. 11.—Facsimile of Seismograph Records, 5.9 tonnes Dropped 12 m at Point 3 m from Footing (1 in. = 25.4 mm)

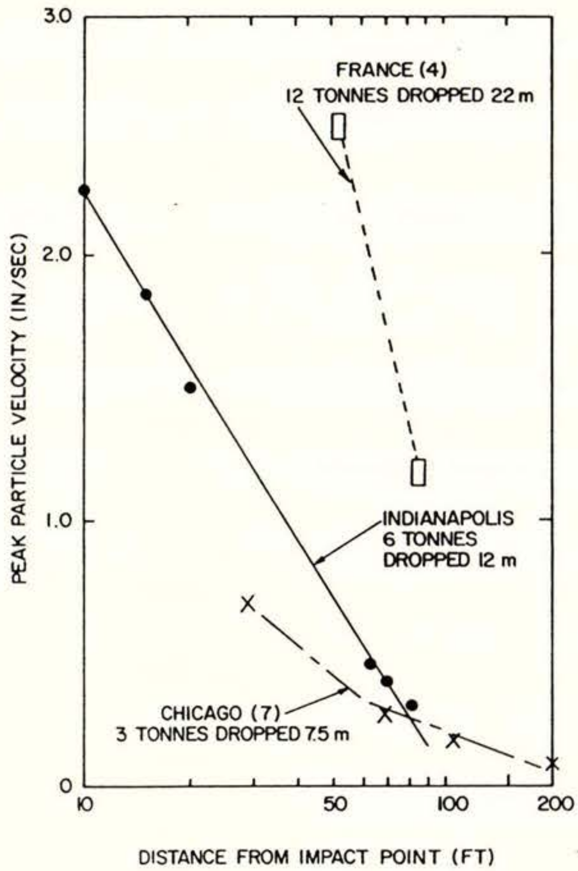

FIG. 12.—Peak Particle Velocity Versus Log Distance from Point of Impact (1 m = 3.28; 1 in. = 25.4 mm)

conducted is shown in Fig. 10. A seismograph was placed on an exterior footing (before the columns were cast) and the 6-1/2-ton (5.9-tonne) weight dropped 40 ft (12 m) at locations ranging from 10 ft–80 ft (3 m–24 m) away from the footing. Two drops were made at each location with essentially the same result. Fig. 11 shows the displacement versus time measured in the vertical, longitudinal, and transverse directions for a drop point 10 ft (3 m) from the footing. The frequency of vibration was approx 7 cycles/sec. The peak resultant particle velocity calculated for each drop point is shown as plotted in Fig. 12, and appears to vary inversely with the log of the distance from the drop point. The measured velocities are essentially *ground motions*. The data indicate that a 6-1/2-ton (5.9-tonne) weight dropped from a height of 40 ft (12 m) at a distance of 10 ft (3 m) from a sound structure founded on drained granular soils would cause little, if any, damage (particle velocities $\leq$ 2 in./sec). However, heavier weights dropped from greater heights may produce damaging vibrations.

Conclusions

The following conclusions may be drawn from this paper:

1. In granular soils the depth to which densification is significant is controlled mainly by the energy per drop. The relationship $D \simeq (1/2)\sqrt{W h}$ is recommended as a guide for preliminary trials. The presence of clay layers, or seams, will greatly attenuate the effective depth of compaction.
2. The upper meter of soil is usually left in a relatively loose state and surface recompaction is required.
3. The degree of compaction achieved depends on the energy per drop as well as on the sequence of drop points and number of drops per point; the total energy applied per unit surface area is a reasonable measure of the latter effects. The product of the energy per drop times the total energy applied per unit surface area correlates well with the degree of compaction achieved; it appears that an upper bound may exist to the compaction that can be attained corresponding to a cone penetration resistance of $q_c \simeq 150$ kg/cm^2 ($N = 30$ to 40).
4. Peak particle velocities on the order of 2 in./sec (51 mm/s) were measured 10 ft (3 m) away from the impact of a 6-1/2-ton (5.9-tonne) weight dropped from a height of 40 ft (12 m) on a drained granular soil, which should cause little, if any, damage to a sound structure. Heavier weights dropped from greater heights may produce damaging vibrations.

Appendix I.—References

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Appendix II.—Notation

The following symbols are used in this paper:

D = depth influenced by dynamic compaction (L);
h = height of drop of falling weight (L);
N = standard penetration resistance ($1/L$);
q_c = cone penetration resistance (F/L^2); and
W = weight of falling block (F).

GROUND VIBRATIONS DURING DYNAMIC COMPACTION

by Paul W. Mayne[1]

ABSTRACT

Ground vibration data taken at twelve different dynamic compaction sites are reviewed for comparative purposes. Weight sizes ranged from 3 to 45 tons and drop heights varied from 5 to 100 feet. Soil types were generally granular materials (silty sands to rockfill). Scaled distance graphs based on the square root of the applied energy per blow appear applicable, yet possibly conservative for the high energy systems reviewed in this study. The sites also show very similar attenuation of particle velocities normalized to the impact velocity versus distance normalized by the radius of the weight. Since the observed frequencies of vibration are in the low range of 2 to 20 hertz, an important consideration in the measurement of vibrations due to dynamic compaction is that many commercial seismographs have transducers with nonlinear responses below 6 hertz. In addition, a lower threshold velocity than the commonly accepted 2 ips is warranted since recent studies have shown that low frequency transient vibrations are potentially more damaging than high frequency vibrations.

INTRODUCTION

During the last several years, dynamic compaction has become popular as an effective method of improving loose sands and granular fills insitu (4, 10, 11, 12, 15, 16). The procedure involves systematically dropping a large steel or concrete weight onto the ground surface to densify the underlying soils. One undesirable side effect is the generation of ground vibrations which emanate from the point of impact (5, 12, 15). Since dynamic compaction is an attractive economical solution, its use is seen increasingly in urban and suburban communities where real estate costs are high.

Ground vibrations can be potentially damaging to nearby building structures and sensitive equipment, as well as annoying to people. Consequently, careful and proper monitoring of ground vibration levels and vibration frequencies must be made in order to protect all interested parties. Ground vibrations caused from dynamic compaction operations are unique from other types of construction activity, such as blasting, pile driving, and traffic. In this regard, vibrations from dynamic compaction are characterized by low-frequency waves which are (1) potentially more damaging than high-frequency waves and (2)

1. Senior Geotechnical Engineer, Law Engineering Testing Company, Post Office Drawer QQ, McLean, VA 22101

below the frequency range of many commercially available vibration monitor seismographs.

The purpose of this paper is to discuss the field measurement and analysis of ground vibrations during dynamic compaction. This includes a review of seismograph equipment, measurement limitations, threshold vibration criteria, and a summary of ground vibration data which were previously obtained at 12 different dynamic compaction sites.

VIBRATION MEASUREMENT

The magnitude of ground vibration levels may be measured in terms of displacement(s), velocity (v), or acceleration (a). If the time history of the waveform is known, then numerical or digital integration and differentiation may be used to relate s, v, and a as functions of time t:

$$a = \frac{dv}{dt} = \frac{d^2s}{dt^2} \tag{1}$$

Often, for simplicity sake, harmonic motion is assumed in converting from one mode to another. Real motions are almost always more complex, irregular, and variable than simpler sinusoidal waveforms. However, since one often deals with orders of magnitude and logarithmic scales in vibration measurement, an approximate analysis may in fact be sufficient for many purposes.

The relationships among peak values of harmonic waves may be expressed by:

$$a = 2\pi f v = (2\pi f)^2 s \tag{2}$$

where f = frequency of vibration.

For most construction-related vibrations, the velocity at a point on the ground (the particle velocity) has been shown to be the best indicator of damage potential and annoyance levels (2, 6, 13, 19, 21, 22, 25). For certain situations, a combination of velocity and displacement measurements may be appropriate. Possibly, the choice of particle velocity is related to the observed frequency range of transient vibrations occurring in construction, which are typically between 5 and 200 Hz. For comparison, seismologists studying earthquakes, which have frequencies of about 1 to 2 Hz or less, use accelerometers. Also, for alignment and performance monitoring, mechanical engineers often use spectrum analyzers to measure dynamic displacement levels of machines (typically f > 100 Hz).

Routine field measurements are taken using a vibration monitor seismograph. Usually, the seismograph package includes a triaxial component transducer or geophone, electronic signal conditioners, and a recording mechanism. A review of several leading commercial units which are available has been prepared by Stagg and Engler (21). Most, commonly, the ground vibration records are written on oscillographic paper or magnetic tape, although a few units provide an electronic digital display output or ticker-tape summary of the vibration levels. Most units record the complete waveform of the measured vibration. Often, the wave is believed to be of the Rayleigh-wave type, although compression, shear, and Love waves also exist. One recent seismograph unit on the market also has a built-in microprocessor to provide fast-Fourier transforms and spectral analysis. Data recorded on magnetic tape also allows a spectrum analysis. However, routine field measurements have not yet developed to this stage and a discussion of frequency spectral methods are beyond the scope of this study. The seismograph unit currently used by the author is reportedly accurate to within + 10 % for vibration amplitudes at 30 hz. Timing marks are claimed accurate within 3 %.

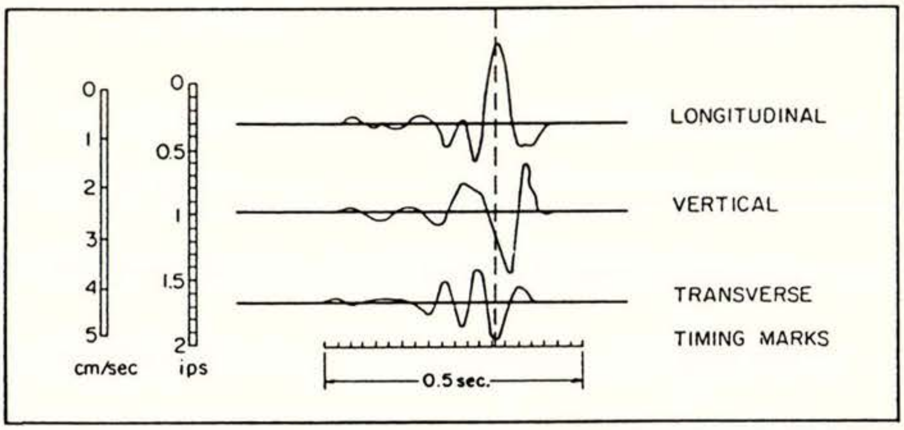

Fig. 1 Example trace of vibration record from dynamic compaction in Morris, Alabama (W = 20.9 tonne, H = 18.3 m, distance = 12.2 m).

Vibration measurements are taken in three mutually orthogonal directions simultaneously (vertical, longitudinal, and transverse axes). An example vibration recording taken during dynamic compaction is presented in Fig. 1. Often, the peak value of each directional component is sought. Beyond this, unfortunately, data are presented in a variety of ways by different individuals. Damage criteria developed by the Bureau of Mines (19) for blasting have been based upon the maximum single value of the three directional components (x_{max}, y_{max} or z_{max}). Since real waves are three-dimensional and the transducer axes may not be exactly in line with the source of vibrations, some engineers (21, 22) prefer to calculate the true vector sum (TVS) of the triaxial components:

$$TVS = \sqrt{(x_t)^2 + (y_t)^2 + (z_t)^2} \qquad (3)$$

where all values are obtained at the same time t. Some seismograph equipment presents the vibration data directly in the TVS format. Mistakenly, several individuals (3, 20) have expressed the vibration levels in terms of the pseudo vector sum (PVS):

$$PVS = \sqrt{x_{max}^2 + y_{max}^2 + z_{max}^2} \tag{4}$$

It is noted, however, that x_{max}, y_{max}, and z_{max} rarely, if ever, occur at the same time. At most, the PVS could be 73% higher than the maximum single component velocity. For the example vibration in Fig. 1, the peak single component, TVS, and PVS are 0.62, 0.69, and 0.83 ips, respectively (16,17, and 21 mm/sec). Typically, the TVS values are about 10 to 40% higher than the maximum single component velocity, as shown by Fig. 2.

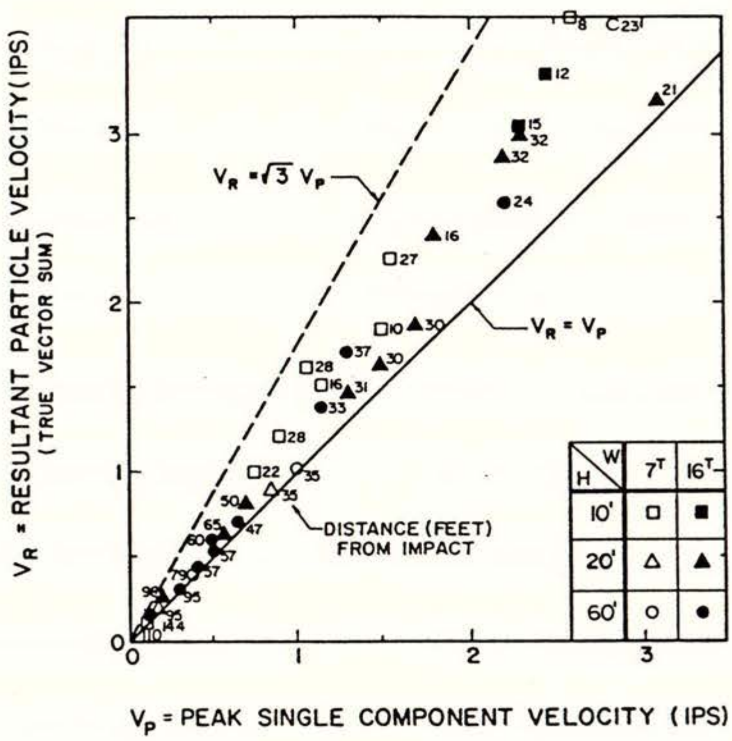

Fig. 2 Observed trend between measured true vector sum velocity and peak single component velocity from Tampa, Florida.

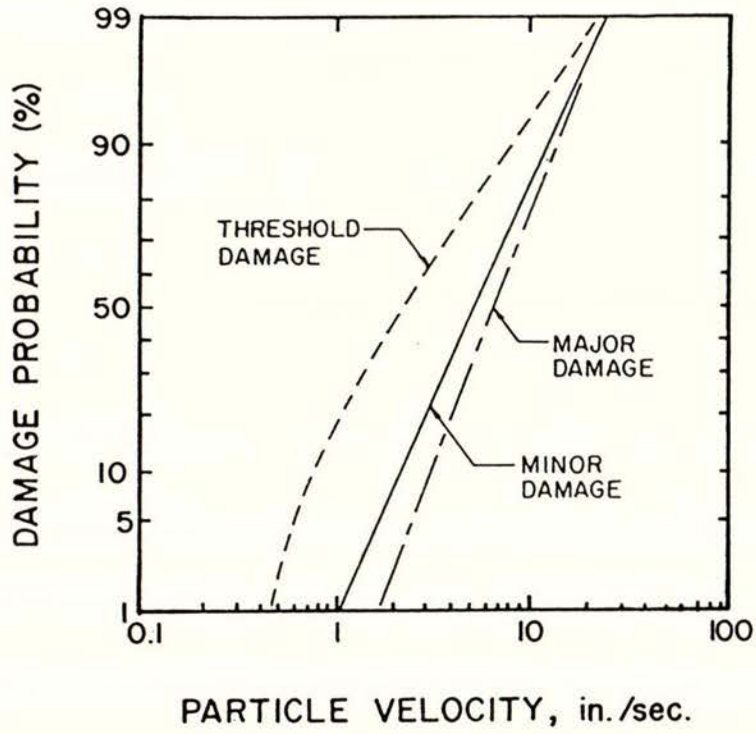

Fig. 3 Probability analysis of damage potential from transient ground vibrations (data from surface mine blasting, U.S. Bureau of Mines, 1980, ref. 19).

DAMAGE CRITERIA

For many years, a limiting peak particle velocity of two inches per second (50 mm/sec) has been considered the structural damage criteria for one and two-story buildings. The primary sources of data for this basis came from blasting records from surface mining operations near residential communities. Higher and lower limits were proposed for larger structures and older sensitive structures, respectively (2). Despite the use of a 2 ips criterion, numerous litigation claims and complaints were filed in the courts. Consequently, a re-evaluation study of vibration damage was performed by the Bureau of Mines and published in 1980 (19). Previous data and new data were analyzed using three different methodologies: (1) statistical mean and variance, (2) probability theory, and (3) observational. Damage was classified according to three types: threshold, minor, and major categories, as indicated by the probability of damage graph shown in Fig. 3. The extensive review culminated in a combined particle velocity-displacement criterion, shown as Fig. 4.

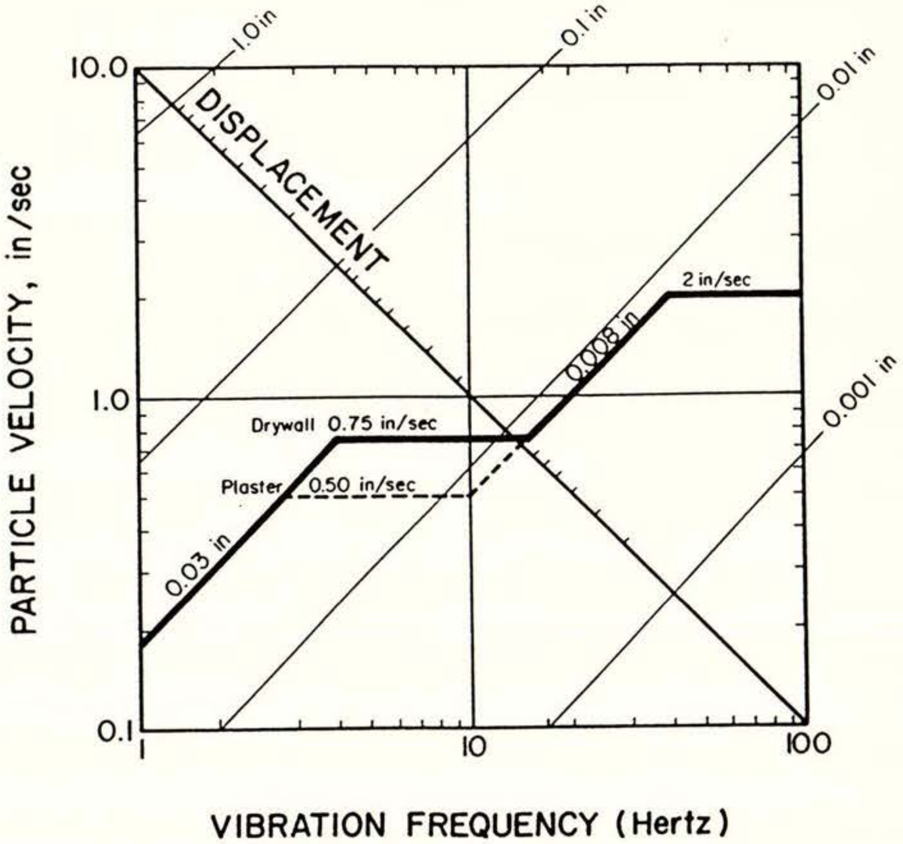

Fig. 4 Combination velocity-displacement criterion (U.S. Bureau of Mines, 1980, ref. 19).

Several commercial seismographs are capable of measuring either displacement, velocity, or acceleration, although not simultaneously. Alternatively, displacements could be estimated using Eq (2), especially since the axes in Fig. 4 assume harmonic motion. The use of the damage criterion in Fig. 4 for dynamic compaction operations could be questioned since it was developed for blast induced vibrations. However, a separate and independent study of vibration damage from blasting, pile driving, and machine sources resulted in similar criteria where limiting particle velocities depend upon frequency (22).

The significant point relevant to dynamic compaction is that low-frequency transient vibrations are potentially more damaging to structures than higher frequency vibrations. Most construction vibrations, blasting operations, and pile driving cause vibrations with frequencies between 5 and 200 Hz. For dynamic compaction, however, Mitchell (16) has indicated that frequencies are typically between 2 and 20 Hz.

FREQUENCY CONSIDERATIONS

Ideally, the vibration frequencies of a waveform should be determined from spectral analysis on Fourier transforms. Practically, however, most equipment available today does not provide this information, especially for field work requiring immediate decisions. Vibration frequencies may be approximately determined by scaling the individual periods from the waveform (21) or by an averaging method by counting the major peaks within a specified time duration of the waveform (22).

Histograms of vibration frequencies obtained by the author at two sites are presented in Fig. 5. The Tampa site was underlain by loose sands with a high groundwater table. For distances between 6 and 57 feet, the mean vibration frequency was 7.5 Hz with standard deviation of + 4.2. At Birmingham, dynamic compaction was used to densify coal spoil material with groundwater over 100 feet deep and D_{50} = approximately 2 inches. The mean and standard deviation of observed frequencies were 10.5 and 2.8 Hz, respectively. Pearce (17) and Leonards et al (10) have also indicated typical vibration frequencies of 5 to 8 Hz for dynamic compaction operations.

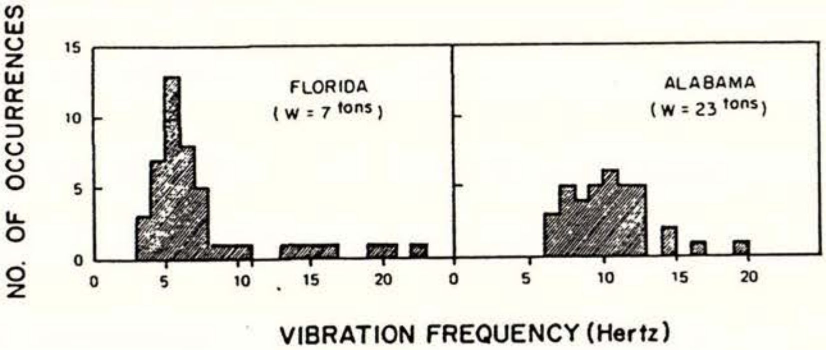

Fig. 5 Histograms of vibration frequencies from dynamic compaction operations at two sites.

Low-frequency vibrations present another problem for those responsible for monitoring them. Many commercial seismographs cannot directly measure vibration levels when the frequency of vibration is less than 5 or 6 Hz. The restriction is primarily due to the resonant frequency and damping characteristics of the transducer. Several manufacturers provide a magnification factor for determining the vibration amplitude when the vibration frequency falls below the specified frequency range of the equipment. The gain factors of two commercial units shown in Fig. 6 indicate that the measured particle velocities may be wrong by a factor of 5 or more unless the vibration frequency is known. The author knows of at least one seismograph unit available which does not measure vibration frequency at all! Readers are cautioned to check the manufacturer's specifications regarding the applicable range of the equipment used.

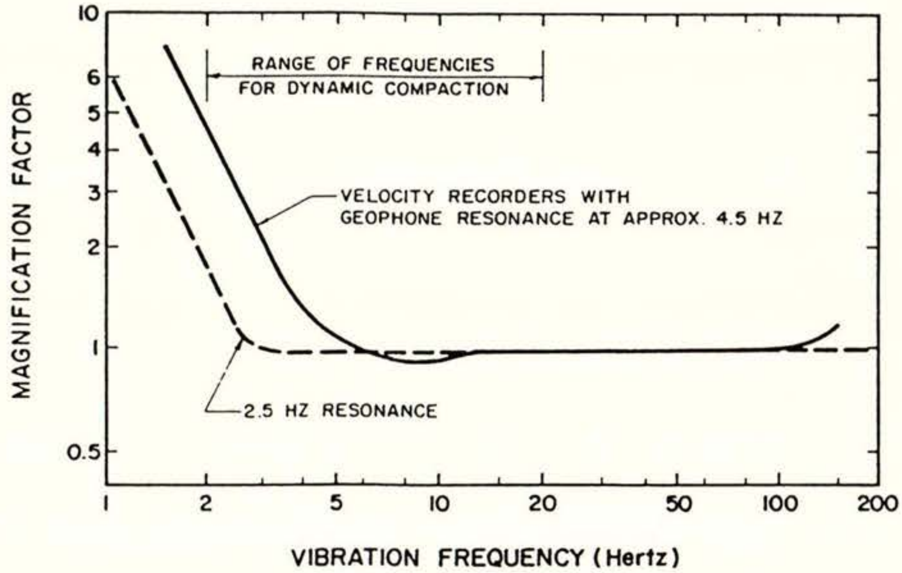

Fig. 6 Frequency dependence of magnification factor for commercial seismographs. Transducer resonance may result in unconservative measurement unless vibration frequency is known and vibration amplitude corrected accordingly.

As a first order approximation (14, 18) the vibration frequency (f_n) from dynamic compaction operations may be estimated as:

$$f_n = \frac{1}{T} = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \qquad (5)$$

where T = period of vibration

$k = \dfrac{4\,G\,r_o}{1-\nu}$ = vertical stiffness of the system (18)

G = shear modulus

r_o = radius of the mass

ν = Poisson's ratio

m = mass of weight = W/g

g = gravitational constant = 32 ft/sec^2 = 9.8 m/sec^2

Eq (5) indicates that low frequency vibrations are associated with loose soils (with low shear moduli) and for larger weights. The deceleration-time histories of two impacts during dynamic compaction are shown in Fig. 7. The decelerations were measured by mounting an accelerometer at the center and top of a 23-ton steel/concrete weight (14). The accelerometer output was transmitted to an oscilloscope by cable and the image recorded on polaroid film. With groundwater at considerable depth, each successive blow of the weight densified the sandy gravelly soils. The observed half-period is approximately 50 msec or, T = 0.1 sec, indicating a frequency of vibration of about 10 hertz. This is consistent with the observed mean frequency from particle velocity monitoring previously presented in Fig. 5 and taken at distances of 40 to 350 feet from impact.

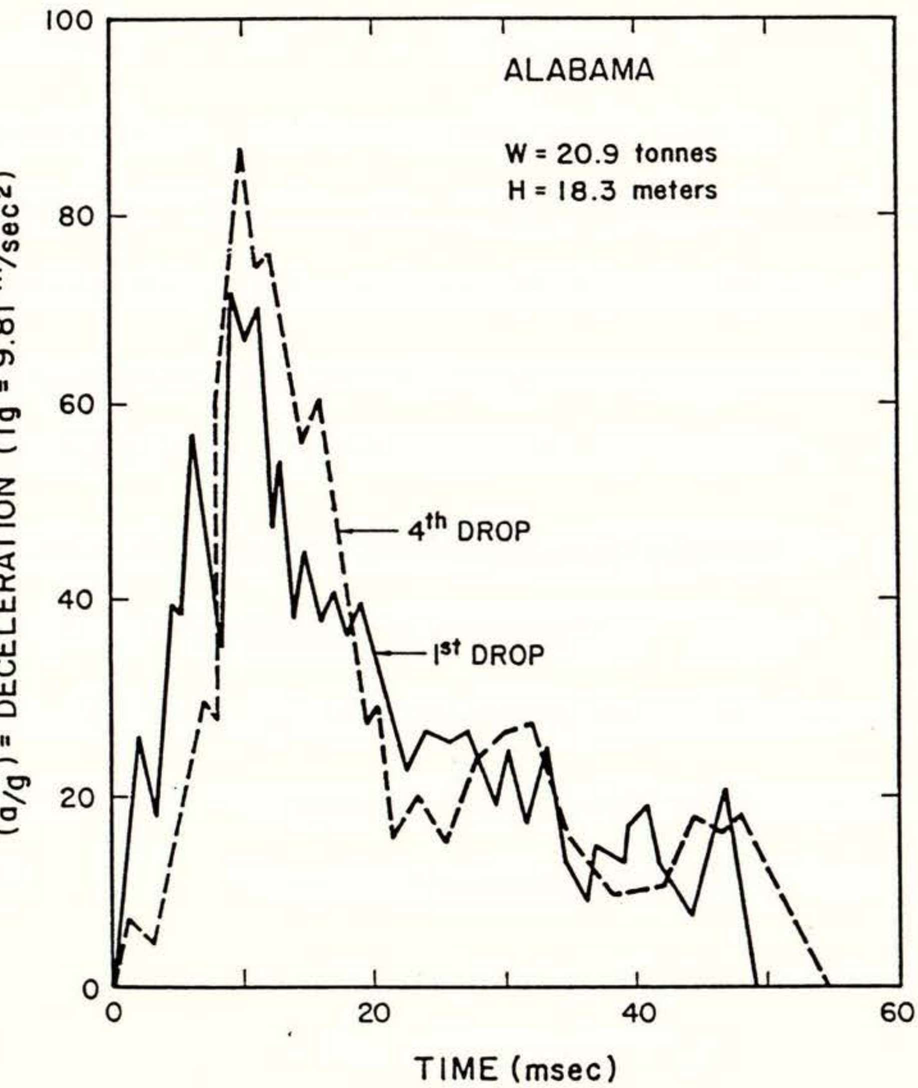

Fig. 7 Accelerometer measurement during impact of a falling weight.

For sites with a high groundwater table, localized liquefaction may occur around the point of impact. Consequently, excess pore pressures develop, effectively reducing the soil stiffness and causing low-frequency vibrations. This may explain the observed low-frequencies at Tampa (see Figure 5). The potential for damage increases at such a site since the level of shear strain may be high. Shear strain amplitudes may be measured in the field as the ratio of peak particle velocity (PPV) to shear wave velocity (Vs) of the soil medium. Unfortunately, since shear wave velocity and shear modulus are related, Vs also decreases with higher levels of shear strain.

TABLE I.

DYNAMIC COMPACTION SITES WITH GROUND VIBRATION DATA

Site Location	Soil Type	Weight in Tons (tonnes)	Drop Height in Feet (m)	Reference Source
◇ Birmingham, Alabama	coal spoil	23 (20.9)	60 (18.3)	This study
△ Alexandria, Virginia	clayey sand and gravel fill	7.8 (7.1)	5 (1.5)	This study
◬ (as above)			10 (3.0)	
◭ (as above)			20 (6.1)	
◮ (as above)			30 (9.1)	
▼ (as above)			40 (12.2)	
▼ (as above)			54 (16.4)	
◑ Baltimore, Maryland	sand fill with brick	4.8 (5.3)	45 (13.7)	This study
● Tampa, Florida	loose sand with high water	16 (14.5)	60 (18.3)	This study
○ (as above)			20 (6.1)	
■ (as above)		7 (6.3)	60 (18.3)	
□ (as above)			10 (3.1)	
⊠ Charlottesville, Virginia	silty sand rockfill	6 (5.4)	45 (13.7)	This study
⊕ (as above)			20 (6.1)	
▽ (as above)			5 (1.5)	
◆ California	silty sand fill	45 (40.5)	100 (30.5)	Gambin (7)
▲ United Kingdom	rubble fill	16.5 (15)	66 (20)	Pearce (17)
◪ Illinois	granular fill	6 (5.4)	25 (7.6)	Lukas (12)
◩ Indianapolis	granular fill	6.7 (6.0)	40 (12)	Leonards et al. (10)
◒ Seine, France	unknown	13.3 (12)	72 (22)	Leonards, et al.(10)
⊖ Indiana	granular fill	15 (13.6)	60 (18.2)	Varaksin(24)
◩ Chicago, Illinois	rubble fill	3.4 (3.1)	25 (7.6)	Lukas (11)

GROUND VIBRATION DATA

Ground vibration data obtained from 12 different dynamic compaction sites were compiled during this study. Five sites were monitored by the author. The data from the other seven sites were obtained from papers and reports prepared by others (see Table I). Primarily, the sites were underlain by natural sands and granular fill materials. It is believed that the groundwater level was relatively shallow on only two of these sites: Tampa (4 feet) and Long Beach (15 feet). Data obtained by the author are expressed in terms of the true vector sum (TVS). Particle velocities reported by others are believed to be primarily in terms of single peak component or TVS. Thus, some error is introduced when comparing data of different format.

The size of weights in Table I range from 3.4 to 45 tons (3.1 to 40.5 tonnes). Drop heights varied from 5 to 100 feet (1.5 to 30.5 meters). Total theoretical energy levels per drop (WH) range between 30 to 4500 ft-tons (8 to 1235 tonne-meters). More technically correct, energy levels up to 12 MN-m were applied, however, the industry commonly uses units of tonne-meters in reporting energy levels.

Conventional crawler cranes were used to hoist and drop the weights on all 12 sites, except the Long Beach, California site, where a special tripod crane was erected. Weights were constructed of either steel or composite steel/ concrete.

The amplitude of ground vibrations attenuate with distance from the point of impact. Figure 8 presents a summary of peak particle velocity data from all sites considered. Distances as close as 7 feet and as far away as 400 feet were monitored. Based on the available data from these sites, a safe conservative upper limit (neglecting special tripod equipment) may be estimated for preliminary purposes from:

$$\text{PPV (ips)} = \left(\frac{75}{\text{d (feet)}}\right)^{1.7} \tag{6a}$$

$$\text{PPV (mm/sec)} = \left(\frac{153}{\text{d (meters)}}\right)^{1.7} \tag{6b}$$

Eq (6) does not consider the level of energy applied during dynamic compaction. Furthermore, the data is derived soley from a few sites, all underlain by granular materials. Extrapolation of these trends to sites underlain by clayey soils, variable fill materials, complex stratigraphy, shallow rock, or other dissimilarities may result in unconservative results.

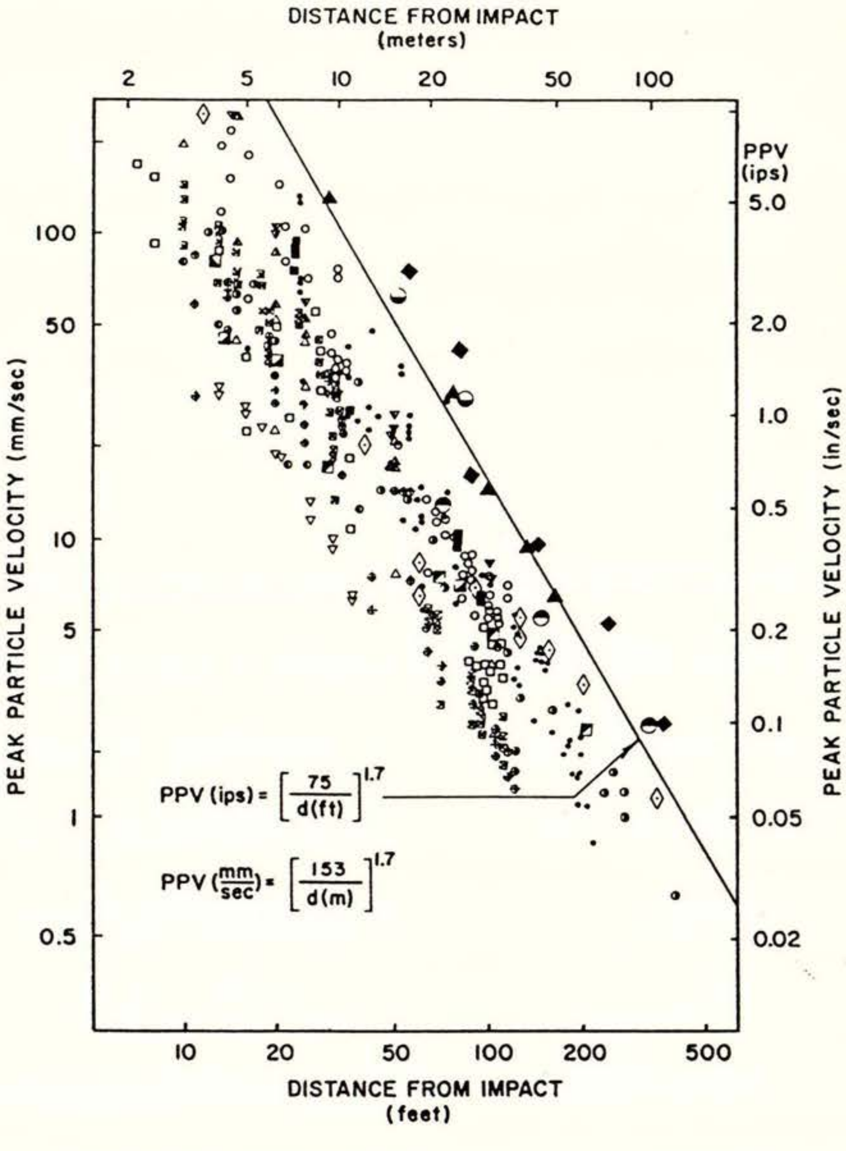

Fig. 8 Summary of peak particle velocity attenuation with distance from impact. Data from 12 dynamic compaction sites listed in Table I.

Within the dynamic compaction limits, it has been observed that vibration levels increase as the treated area becomes densified (4, 17). Generally, a maximum level of particle velocity is achieved after one or two passes of heavy tamping or about 150 tm/m^2.

SCALED DISTANCE DATA

Scaled distance graphs are often used to present particle velocity data (10, 11, 15, 16, 25). Most commonly, the scaled distance axes is defined as the distance from the source to the ratio of the square-root of the applied energy. Cube-root scaling is advocated by others (1, 9). Based on the peak velocity values observed at the author's site after densification and the available supplementary data, a summary of particle velocity attenuation with inverse of square-root scaled distance is presented in Figure 9. Expressions for the upper limit of the observed trend are:

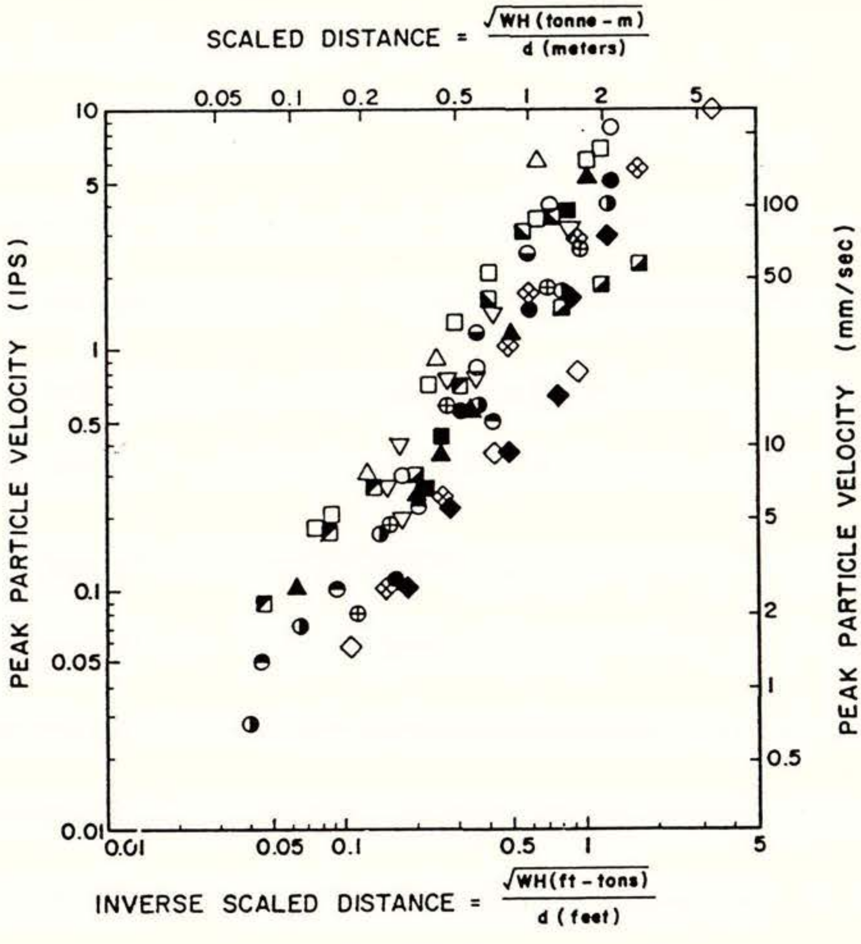

Fig. 9 Particle velocity attenuation for scaled distance according to square root of energy per blow. Data from sources given in Table I.

$$\text{PPV (in/sec)} = 8\left(\frac{\sqrt{WH}}{d}\right)^{1.7} \tag{7a}$$

where d and H are in feet and W in tons;

and
$$\text{PPV (mm/sec)} = 92\left(\frac{\sqrt{WH}}{d}\right)^{1.7} \tag{7b}$$

where d and H are in meters and W in tonnes.

A close examination of Figure 9, however, indicates that the derived upper limit expression in Eq (7) appear conservative for the largest weights and highest drop heights (Tampa, Long Beach, U.K., and Alabama), possibly because of site specific differences. The effect of energy level on particle velocity was studied as suggested by Wiss (25). At several distances, the log of PPV was graphed as a function of WH, as shown in Fig. 10. Apparently, the exponent term decreases with distance away from the point of impact. At distances of 20, 50,

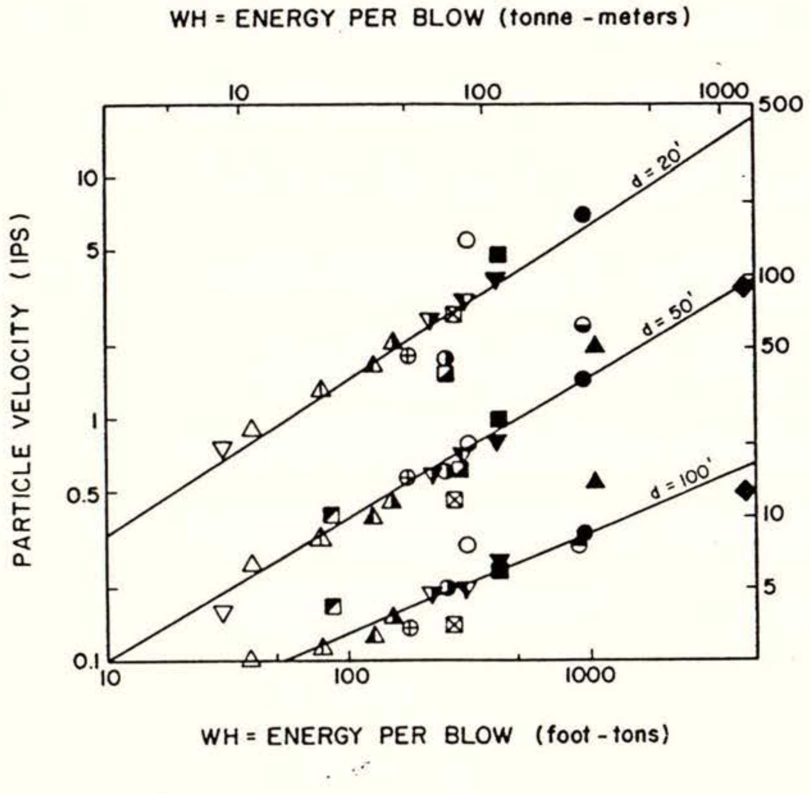

Fig. 10 Observed relationship between particle velocity and energy per blow at distances of 6, 15, and 30 meters.

and 100 feet, the observed effect of energy level is $(WH)^{0.6}$, $(WH)^{0.5}$, and $(WH)^{0.4}$, respectively, as determined from linear regression analyses. Such variations may be explained due to factors such as plastic deformation, material damping, geometrical damping, stratification, and other phenomena (23).

In actuality, friction in the system prevents a true free fall of the weight upon release of the clutch. The total energy per blow is somewhat less than WH. Deceleration measurements have indicated the efficiency to be on the order of 80% (8). Considering all factors involved, a more involved expression for vibration attenuation may be:

$$PPV = A_o \; W^a \; H^b \; / \; d^c \tag{8}$$

where A_o, a, b, and c are all parameters and not necessarily constants for a given site and the specific equipment utilized.

In an effort to discern the effects of different weight sizes on particle velocity, particle velocities were measured during dynamic compaction with a 7-ton weight and 16-ton weight in Tampa, Florida. If square root scaling applied, then the particle velocities from the 16-ton weight would be $\sqrt{16/7}$ or 1.51 times those from the 7-ton weight. For this site, the observed ratio of particle velocities averaged about 1.35, implying $(W)^{0.4}$.

The effects of drop height were investigated at a dynamic compaction site in Alexandria, Virginia. Drop heights of 5, 10, 15, 20, 30, 40, and 54 feet were monitored. At this site, the effect of drop height varied approximately as $(H)^{0.6}$ at a distance of 20 feet to $(H)^{0.4}$ at a distance of 100 feet. Based on the limited data obtained at Tampa and Alexandria, it is postulated that drop height is slightly more influential than weight size in determining the magnitude of particle velocities. Weight size may affect vibration frequency, however, as implied by equation (5).

Using an entirely new approach, the same data base from Figure 9 was re-graphed in the form of a normalized vibration level (particle velocity divided by the theoretical impact velocity of a falling weight) versus distance normalized to the weight radius (d/r_o), (see Fig. 11). Apparently, a close trend is obtained with this empirical approach, although maybe fortuitous. It would seem intuitive that the maximum possible particle velocity would occur on the weight during impact. For a free falling body, the impact velocity (v_i) is:

$$v_i = \sqrt{2 g H} \tag{9}$$

where g = gravitational constant. In addition, for a rigid mass, the size of the weight is related to the mass radius. Referencing Figure

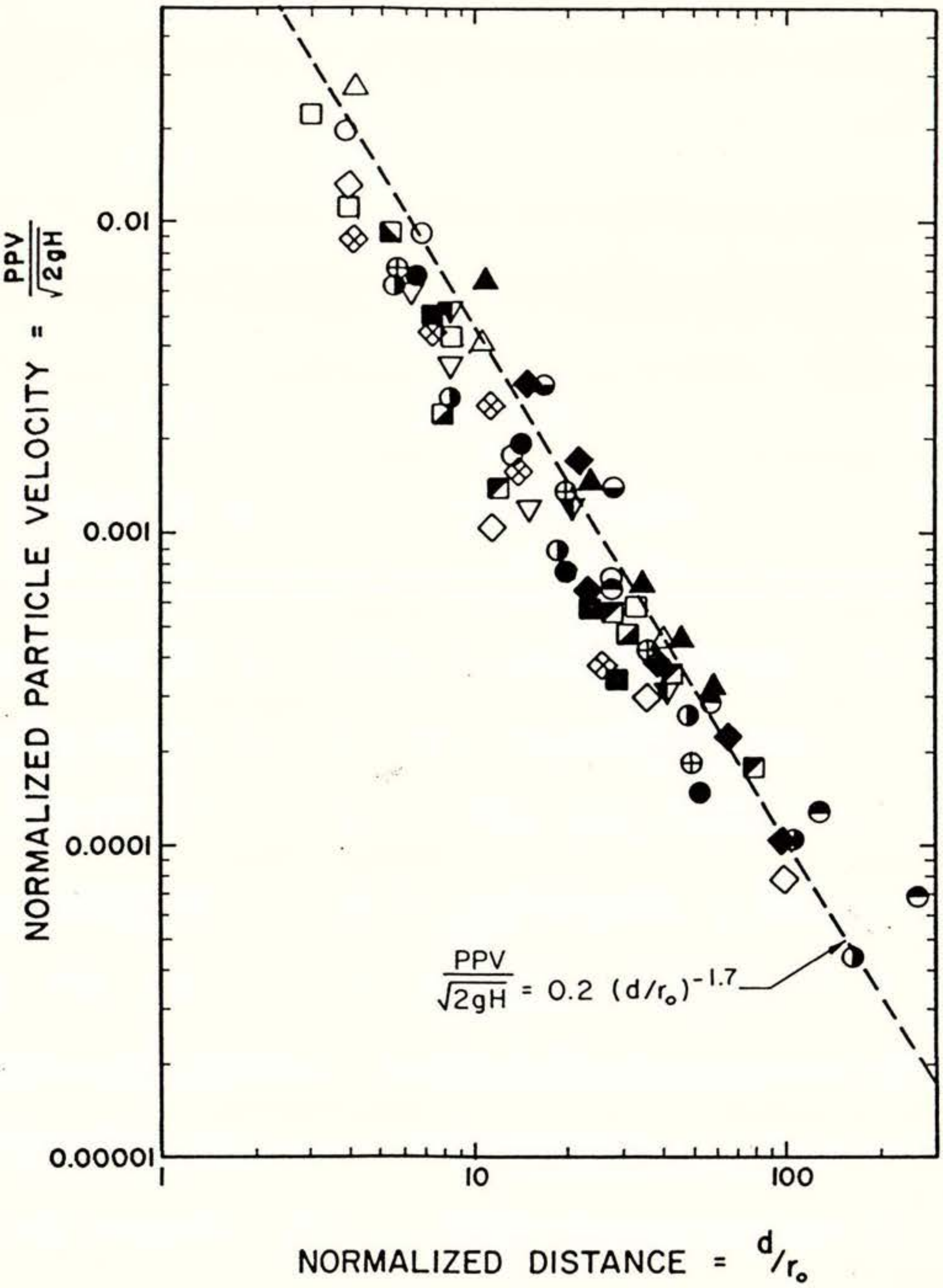

Fig. 11 Observed attenuation of normalized particle velocity with normalized distance. Data from sources given in Table I.

9, a linear approximation of the particle velocity attenuation using this approach would be:

$$PPV = 0.2 \sqrt{2\ g\ H}\ (d/r_o)^{-1.7} \tag{10}$$

where PPV and v_i are in consistent units and distance d is normalized to the radius of the weight (r_o). Since the equivalent radii of typical weights lie in the narrow range of 2 to 3.5 feet (0.6 to 1.1 m), a similiar trend is observed between normalized particle velocity and distance.

CONCLUSIONS

The findings of this study on ground vibrations caused by dynamic compaction of granular soils indicate that:

(1) Transient low-vibration frequencies resulting from dynamic compaction may require a combination velocity and displacement criterion, (or frequency-dependent velocity criterion), as recommended by two recent studies on vibration-induced damage (19, 22).

(2) It is extremely important to measure vibration frequencies of the waveform to determine whether these are within the range of the seismograph operating range. Many units are nonlinear below 6 Hz and thus, a magnification factor may be required.

(3) Vibration levels should be reported in terms of maximum single component amplitude or true vector sum (TVS), not pseudo vector sum (PVS).

(4) Particle velocity attenuation from square-root scaled-distance graphs appear applicable for data from 12 different sites underlain by granular soils and reviewed in this study. Significant trending was also observed for particle velocities normalized to impact velocity and attenuated with distance normalized to the weight radius.

(5) Additional research and data are needed on the use of spectrum analyses for vibration monitoring during dynamic compaction. In addition, a damage criterion based on dynamic compaction data is warranted.

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APPENDIX C

NZ CASE STUDY :

DYNAMIC CONSOLIDATION AT WAIWHETU TERMINAL, SEAVIEW, LOWER HUTT

APPENDIX C

NZ CASE STUDY :

DYNAMIC CONSOLIDATION AT

WAIWHETU TERMINAL, SEAVIEW, LOWER HUTT

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APPENDIX C

NZ CASE STUDY : WAIWHETU

C1 INTRODUCTION

An investigation was undertaken at a site for a proposed redevelopment of a tank farm facility at Waiwhetu Terminal, Seaview, Lower Hutt.

The objective of the site investigation work was to assess the nature of subsoils across the site, make an assessment of subsoils to support the proposed facilities, assess the liquefaction potential of the site and to recommend ground improvement techniques which may be necessary to reduce the risk of liquefaction to an acceptable level.

The site was investigated by putting down six machine drilled boreholes to depths ranging between 10.4 to 20.2 m below existing ground level. The boreholes were advanced by auger and wash drilling techniques with frequent in-situ testing and sampling to the depth explored. Special drilling procedures were undertaken in the 20 m borehole, which penetrated the Hutt Valley aquiclude, to prevent the possible development of artesian flow.

In addition to the boreholes, 40 cone penetration tests (CPT) were put down at the site, these tests were well spread to give good coverage across the site and extended to depths of between 10.0 and 12.0 m below existing ground level.

The site is situated on reclamation fill over an alluvial plain near the mouth of the Hutt River. The alluvial plain comprises fluviatile marine gravels and sequences of sand and silt.

A typical soil model based on borehole information is as follows:

Layer	Depth (m)	Description
1	0.0 - 2.2	FILL:GRAVEL/SAND very loose to loose
2	2.2 - 10.2	GRAVELS/SANDS, loose to moderately dense, some firm grey silt layers
3	10.2 - 19.4	SILTS/CLAYS, firm to stiff, grey
4	19.4 +	GRAVEL, very dense, grey

The investigation showed that under static conditions the proposed tank farm components could be supported on the existing soils provided some form of subgrade preparation was carried out.

Under earthquake conditions, the subsoils would be susceptible to loss of strength due to liquefaction. Liquefaction of site subsoils could cause significant ground surface displacements and damage to the proposed development. A site specific seismic hazard study was undertaken to provide design information, assess the peak ground accelerations and define an acceptable level of site protection for a given earthquake.

To achieve the necessary level of site protection against liquefaction under earthquake conditions, the investigation showed that subsoils extending to depths of 6.0 m below ground level needed to be densified.

Various forms of subsoil pre-treatment were considered, but the investigation showed that, with a high ground water level, the most appropriate form of pre-treatment was by dynamic compaction. Target CPT values to prevent liquefaction under earthquake were assessed and a field trial of dynamic compaction by high energy tamping was recommended.

C2 FIELD TRIAL

C2.1 Description

A dynamic compaction field trial by high energy tamping was undertaken at the Waiwhetu Terminal, Seaview, Lower Hutt. The purpose of the trial was to:

(a) determine the effectiveness of dynamic compaction in densifying the existing site subsoil

(b) evaluate the disturbing effects of dynamic compaction upon neighbouring structures and people

(c) assess rate of progress and cost per unit area of treatment

Dynamic compaction by high energy tamping involves dropping a heavy weight through specified heights for a given impact energy. The energy level also depends on the depth of influence of ground treatment. At the Waiwhetu Terminal the object was to densify subsoils to a depth of 5.0 m below ground level requiring a drop energy of 100 t-m.

The degree of compaction/densification depends on the number of drop points and the uniformity of compaction depends on the number of passes and the configuration of the drop points. Four trial conditions were established to optimise the ground treatment. This involved varying the number of passes and the number of drops as follows:

(1) 3 passes x 4 drops
(2) 4 passes x 4 drops
(3) 3 passes x 6 drops
(4) 4 passes x 6 drops

Two trial areas were established:

Trial Area A consisted of a 12 x 24 m area and trial Area B consisted of a 6 x 6 m area. Trial Area A was subjected to surface dynamic compaction (top of reclamation fill) and Trial Area B subjected to dynamic compaction below the reclamation fill (i.e. with top layer of reclamation fill removed).

C2.2 Specific Subsoil Conditions

The dynamic compaction trial was carried out close to an investigation borehole. The subsoil log of the borehole is summarised below.

Depth (m)	Soil Description
GL - 1.3	RECLAMATION FILL, SAND, gravelly, loose, yellow brown
1.3 - 4.1	SAND (med. to coarse) gravelly, loose to mod.dense, grey
4.1 - 9.2	SAND (med.) slightly silty, moderately dense, grey
9.2 - 10.4	SILT, slightly sandy, stiff, grey

A cut-off trench adjacent to the Trial area indicated that the reclamation fill depth varied from 1.4 to 1.7 m below ground level.

C2.3 Trial Procedures

Quantitative assessment of ground improvement was undertaken by subsurface testing carried out before, during and after the dynamic compaction trial. Testing comprised 38 cone penetrometer tests to depths of 8.0 m below ground level using a truck mounted and ballasted CPT rig.

Level survey work was carried out before and after the trial to check the magnitude of ground displacement. Monitoring of vibrations due to the dynamic compaction was undertaken using three accelerometers and a seismograph for various configurations around the points of impact.

To monitor potential damage to surrounding buildings, a condition survey comprising a systematic visual and photographic record was compiled. Visual assessment was carried out before and after the trial.

Human response to vibration and disturbance caused by impact was observed and recorded at strategic locations around the trial area. Felt vibration and perceived reaction at each point was recorded immediately following impact.

C2.4 Trial and Equipment Details

The required energy of impact was delivered by a 9.14 t reinforced concrete block, of dimensions 1.5 x 1.5 x 1.7 m high, free falling specific heights using a NCK 605 crane. The crane had a capacity of 40 t, a 21-m jib and used a 26 mm wire cable having a breaking strain of 25 tonne.

Prior to dynamic compaction, survey control has established outside areas A & B along two sides of each area. Survey control comprised two offsets adjacent to each compaction set out point. This control enabled quick and accurate setting out of all compaction points for the four passes.

The schedule for drop heights for each of the four passes and the associated energy was as follows:

Table C1

Schedule of Drop Heights 9.14 tonne Tamping Weight

Pass No.	Height of Drop (m)	Energy Per Blow (t-m)
1	11	100
2	11	100
3	5.25	48
4	2.5	24

The arrangement of the four trial conditions is shown in Fig C1 and the configuration of the drop points for each of the four passes is shown in Figure C2.

The trial in Area B was carried out over a 6 x 6 m grid system and comprises 2 passes only of the 100 t-m energy/drop.

A total of 5 CPT tests were put down prior to the DC trial in Area A. The DC trial commenced following a spot level check over the test area. Upon completion of Pass 1, the trial area was regraded using a Case 1100 dozer and rolled using a steel wheeled roller to provide an even surface for subsequent passes.

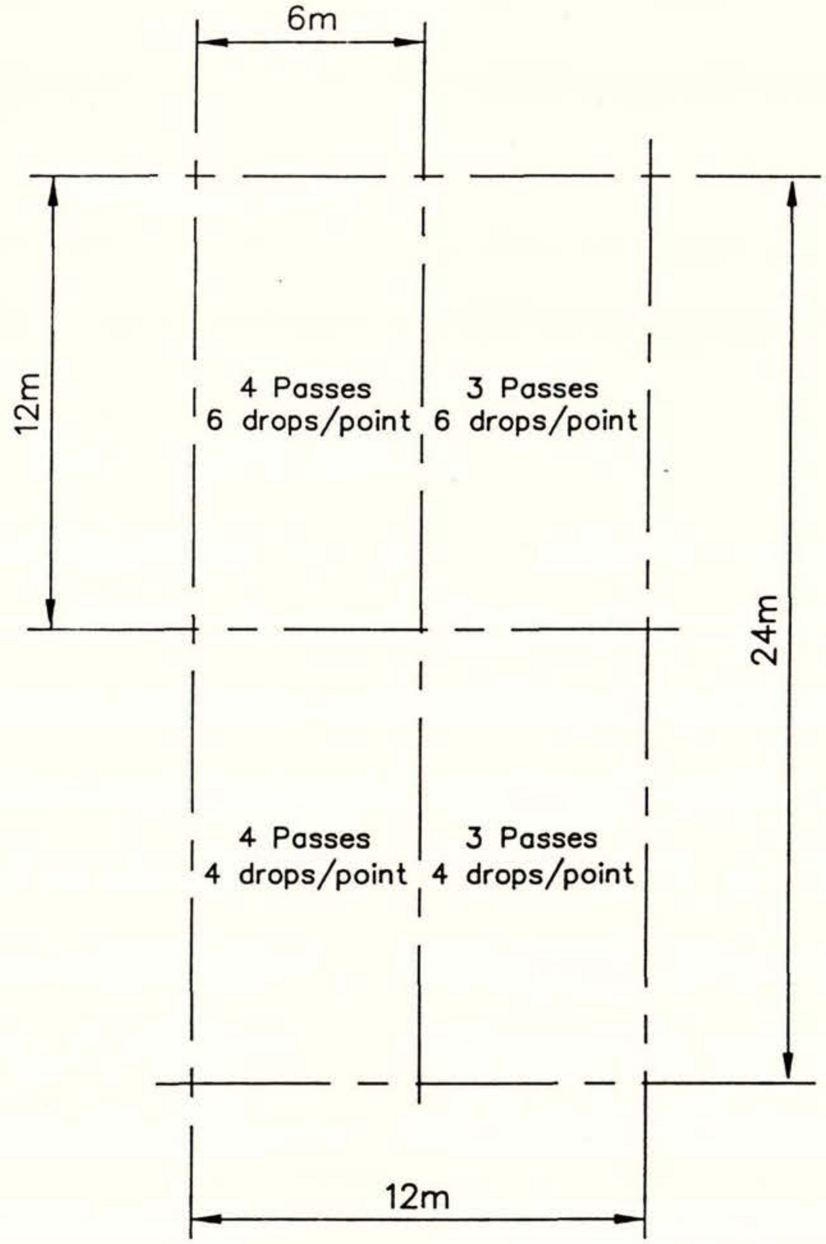

Figure C1: DC Case Study: Waiwhetu Case Study

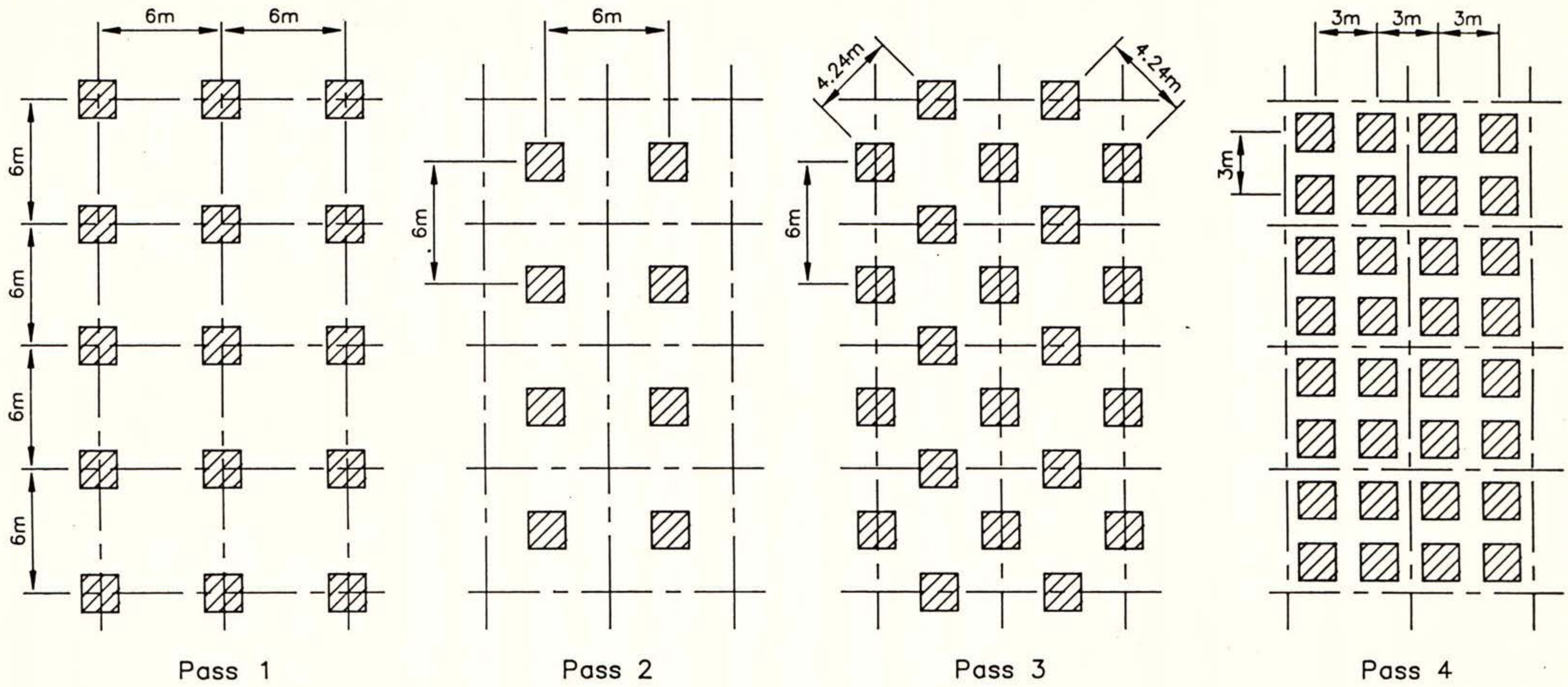

Figure C2: DC Trial Drop Arrangement: Waiwhetu Case Study

C2.5 Results of Trial

Ten CPT tests were put down over the trial area to measure the effect of Pass 1. The general ground improvement is tabulated below in Table C2.

Table C2

Pass 1: General Ground Improvement

Depth Below Ground Level (m)	AVERAGE CONE RESISTANCE (MPa)		
	Before Compaction	After Compaction	
		1 Pass	1 Pass
		6 Drops	4 Drops
	13.9		
1.0			
	5.1	7.2	5.8
2.0			
	7.9	9.2	8.6
3.0			
	9.1	10.9	9.5
4.0			
	8.8	11.9	8.7
5.0			
	11.8	10.6	11.5
6.0			
	8.2	8.6	8.9
7.0			

Evaluation of specific ground improvement was undertaken by considering ground improvement directly on compaction points, in line between two compaction points and diagonally between two compaction points. The results of this analysis for the first pass at both 6-drop points and 4-drop points is shown in Table C3 below.

Table C3
Specific Pass 1 : Improvement

No. of drops per point	Depth below ground level (m)	AVERAGE CONE RESISTANCE (MPa)			
		Before Compaction	After Compaction		
			On Compaction point	In line between comp. point	Diagonally between comp. point
6		13.9			
	1.0				
		5.1	11.0	7.4	4.5
	2.0				
		7.9	6.9	13.0	8.9
	3.0				
		9.1	11.4	11.9	11.6
	4.0				
		8.8	12.0	11.8	10.6
	5.0				
		11.8	10.0	9.6	13.3
	6.0				
		8.2	10.1	7.5	8.4
	7.0				
4		13.9	-	-	-
	1.0				
		5.1	5.3	5.7	6.3
	2.0				
		7.9	8.2	7.8	9.5
	3.0				
		9.1	8.9	9.7	9.8
	4.0				
		8.8	8.4	8.6	9.1
	5.0				
		11.8	10.5	12.7	11.4
	6.0				
		8.2	8.4	8.5	9.7
	7.0				

The above test result summary indicates that the greatest improvement is achieved between 1.0 m and 5.0 m below existing ground level; thereafter improvement being markedly reduced. The greatest ground improvement has been attained directly in line between two 6-drop compaction points (6 m centres).

The greatest improvement following the Pass 1 with four drops/point is attained diagonally between two points. Other results are inconclusive and may reflect unevenness of compaction points and variable ground conditions.

Evaluation of post-trial (completion of Pass 4) was based on a total of 12 CPT tests, with 3 tests in each quadrant, (reference Figure C1 "Trial Areas"). The results of post trial ground improvement within each quadrant of the trial area are tabulated below:

Table C4

DC Area A : Post Trial

Depth Below Ground Level (m)	AVERAGE CONE RESISTANCE (MPa)				
	Before Compaction	After Compaction			
		3 Passes	4 Passes	3 Passes	4 Passes
		4 Drops	4 Drops	6 Drops	6 Drops
	13.9	9.7	9.3	10.6	9.4
1.0					
	5.1	7.8	9.2	9.0	11.1
2.0					
	7.9	11.2	12.7	13.2	15.0
3.0					
	9.1	10.3	10.4	11.5	13.0
4.0					
	8.8	8.8	10.8	10.5	12.2
5.0					
	11.8	12.2	11.2	12.2	10.9
6.0					
	8.2	9.3	9.6	8.4	9.7
7.0					

Ground improvement extends to depths of 7.0 m below existing ground level with significant improvement measured between 1.0 m and 5.0 m. No ground improvement was measured between 5.0 - 6.0 m and significantly reduced ground improvement between 6.0 - 7.0 m in the 6 d/p area. It appears that impact energy may have been attenuated between 5.0 - 6.0 m below ground level due to the presence of a silty sand or sandy silt layer. The borehole at this point indicates that the sandy stratum does become silty at a depth of 5.0 m below existing ground level.

There was a significant increase in ground improvement between the 6 d/p area and the 4 d/p area, which confirms published literature that an increased number of blows at a point increases the ground improvement.

The results clearly show that the greatest ground improvement occurs under the higher number of passes and higher number of drops per compaction point.

At the outset of the project and during the DC trial in Area 4, it was considered that the full effect of the DC was being attenuated by the presence of the reclamation fill overlying the alluvium subsoils. In an attempt to demonstrate the potential ground improvement through direct impact onto the surface of the alluvial subsoils, DC in Trial Area B was carried out at approximately 800 mm below existing ground level.

Pre-trial CPT tests were conducted at ground surface through moderately dense ground prior to subexcavation.

Post-trial tests were carried out following fill replacement to original ground surface. No specific compaction was given to replacement fill resulting in lower cone resistance in the surface layers.

The effectiveness of ground treatment in Trial Area B is summarised on Table C5.

Table C5
Area B : Post Trial

Depth Below Ground Level (m)	AVERAGE CONE RESISTANCE (MPa)			
	Before Compaction	After Compaction		
		General Improvement	In line Between Compaction Points	Diagonally Between Compaction Points
	14.9	2.4		
1.0				
	5.1	7.2	6.1	8.2
2.0				
	8.2	11.8	11.1	12.4
3.0				
	11.3	11.5	11.3	11.8
4.0				
	6.9	8.2	7.8	8.5
5.0				
	11.7	11.1	11.5	10.6
6.0				
	8.4	8.4	8.2	8.5
7.0				

C2.6 Rate of Progress of DC Trial Treatment

There is no useful information concerning the rate of progress for Pass 1 of the DC trial as vibration monitoring and human response survey were carried out at this time, which prolonged the duration of this pass.

Time rating performance checks were made during subsequent passes. Incremental time rates for the various passes are summarised in Table C6 below.

Table C6

Time Rating for DC Treatment

DC Trial	Time taken to complete pass (mins)	Time taken to complete 10 points x 6 drops x 5.25 m per drop (mins)	Time taken to complete 12 points x 4 drops x 5.25 m per drop (mins)	Average time taken to complete 1 comp. point (mins)
Pass 2	120	-	-	3
Pass 3	112	36	35	2
Pass 4	50	-	-	1

C3 PRODUCTION TREATMENT

The trial showed that the full energy input (4 passes, 6 drops/point) offered the greatest degree of densification as measured by the CPT test, and was therefore used in the production treatment.

DC production treatment was carried out in two stages over 17,000 m^2 of the site. Stage 1 comprised 9,300 m^2 of treatment on the surface of the reclamation fill in the southern half of the site. Stage 2 treatment comprised the balance of the area and actual DC treatment was undertaken below the reclamation fill on alluvium in order to achieve design elevation.

Within Stage 1, a separate area of treatment was undertaken for the administration block. The area of the administration block was about 800 m^2. CPT testing was undertaken before and 4 days after DC treatment. The test results did not achieve target resistance values and subsequently testing was carried out 44 days after completion of DC work in this area. The results of the DC treatment are tabulated in Table C7 below:

Table C7
Improvement with Time after 4 passes and 6 drops/point

Depth Below Ground Level (m)	AVERAGE CONE RESISTANCE (MPa)		
	Before Compaction	4 Days after Compaction	44 Days after Compaction
0			
	-	9.9	10.2
1			
	3.5	12.3	14.9
2			
	3.4	9.6	11.3
3			
	7.3	9.8	10.3
4			
	7.6	9.3	10.0
5			
	7.6	10.2	11.0
6			

Friction ratios indicate that there is a sandy silt layer in the vicinity of the administration building. This material is likely to experience significant pore water pressures under DC treatment and ground improvement will tend to increase with time as these pore pressures dissipate. Table C7 shows significant increase in cone resistance between depths 1.0 to 2.0 m where the subsoil was described as sandy silt.

Table C8 below shows the ground improvement measured specifically over the main part of Stage 1. The cone resistance tabulated are based on an average of values obtained from 7 CPT tests before DC treatment and 9 CPT tests after DC treatment.

Table C8

Production Treatment : Stage 1
(4 passes, 6 drops/point on surface of Fill)

Depth Below Ground Level (m)	AVERAGE CONE RESISTANCE (MPa)	
	Before Compaction	After Compaction
0		
	12.7	12.6
1		
	5.6	9.1
2		
	8.7	15.3
3		
	9.2	12.3
4		
	10.8	13.4
5		
	10.7	11.6
6		

The results shown in Table C8, with the exception of the top metre, show a marked increase in cone resistance after DC treatment. Input energy was designed to influence the subsoils to a depth of 5.0 m below ground level. The tabulated results show little average ground improvement beyond this 5.0 m depth.

As stated previously, Stage 2 DC treatment was undertaken on the surface of alluvial sand beneath the reclamation fill. The results of the DC treatment is tabulated in Table C9 below.

Table C9

Production Treatment : Stage 2
(5 passes, 6 drops/point, surface layer removed)

Depth Below Ground Level (m)	AVERAGE CONE RESISTANCE (MPa)	
	Before Compaction	After Compaction
0		
	6.1	8.6
1		
	4.9	11.2
2		
	8.0	17.1
3		
	11.2	17.7
4		
	13.7	15.6
5		
	9.6	9.3
6		

Whilst significant ground improvement was recorded between ground surface to 1.0 m depth, the greatest improvement occurs between 2.0 - 4.0 m below ground level. Overburden pressure appears to play an important part in the densification process.

C4 COST OF TREATMENT

The cost of production treatment is governed by the following factors:

(1) The stiffness of surface material on which the crane has to operate effects rate of progress. The tamping weight becomes partially buried on impact in soft ground and causes a lesser imprint in more competent surface soils, as was experienced with Stage 1 DC treatment.

(2) Crane operator skills required in applying brake at the critical moment following tamping weight impact, to limit cable overrun.

(3) The method of regulating ground after treatment. Maximum progress was achieved by filling tamping weight imprints as DC treatment progressed across a given area. It was found that time delays occurred when the treated area was regraded at the end of each pass.

(4) Higher energy input causes larger imprint on tamping weight impact with consequent time extension required for a given drop point.

The cost of unit area of treatment varied significantly between Stage 1 and Stage 2 and reflected varying surface ground conditions associated with factor (1) above. More importantly, a stop-start method of operation requiring mobilisation and demobilisation of plant and equipment lead to higher costs for the treatment. The cost of treatment is summarised below in Table C10.

Table C10

Cost of DC Treatment

Stage	Main Feature of Treatment	Approx. Cost per m^2
(1)	Competent surface to work on. Vacant site. No significant delays	$17.0
(2)	Loose, variable strength working surface. Stop-start method of treatment. Improved operator skills	$29.0

C5 RATE OF PROGRESS OF DC TREATMENT

Incremental time rates are implicitly associated with the costs per unit area of treatment and the factors mentioned in the previous section also effect rate of progress of DC treatment.

Incremental time rates for the various passes are summarised for Stage 1 in Table C11 and Table C12 for Stage 2 as follows:

Table C11

DC Treatment Rate of Progress : Stage 1

DC Production	Time taken to complete 50 compaction points (mins)	Average time taken to complete 1 compaction point (mins)	Individual measured time to complete 1 compaction point (mins)
Pass 1	430	8.60	} 2 - 3.0
Pass 2	430	8.60	
Pass 3	135	2.70	1.5 - 2.0
Pass 4	115	2.30	~ 1.0

There is a significant time differential between the average time to complete one compaction point and the individual measured time to complete one compaction point. The average time period reflects delays due to weather and wire cable repair.

Table C12

DC Treatment Rate of Progress : Stage 2

DC Production	Time taken to complete 50 compaction points (mins)	Average time taken to complete 1 compaction point (mins)	Individual measured time to complete 1 compaction point (mins)
Pass 1	526	10.50	} 2.5 - 3.0
Pass 2	526	10.50	
Pass 3	160	3.20	1.5 - 2.0
Pass 4	126	2.50	< 1.0
Pass 5	98	$\simeq$ 2.0	-

The results show the rapid increase in time to complete a given number of compaction points with increasing level of energy. A plot of time versus energy level shows that the rate of change of time with respect to change in energy level significantly increases above the 60 t-m level of energy.

C6 CONCLUSIONS AND GENERAL COMMENTS

The purpose of the DC trial was to optimise the DC energy level to achieve target subsoil density and to improve resistance to liquefaction for those soils most susceptible to liquefaction.

The trial was designed to improve the ground on the basis of the following relationship.

$$\textit{Depth of Treatment} = 0.5\sqrt{\textit{drop energy}}$$

The maximum input energy of 100 t-m provides for treatment to a depth of 5.0 m below ground level. The DC production treatment. Ground improvement at shallow test results confirm the accuracy of the above relationship.

It was found that the four passes did not show the expected ground improvement at shallow depth during the DC production treatment. Ground improvement at shallow depth was achieved by the introduction of a fifth pass. This fifth pass was a low energy pass (10 t-m), designed to densify subsoil to depths of 2 m below ground level.

The trial indicated that there are beneficial effects in leaving the reclamation fill in place. Some energy would be attenuated by the reclamation fill on impact, but this would appear to be offset by the beneficial confining effect of the stiffer layer. However, the results of the production treatment did not clearly confirm the benefits of having the reclamation fill in place. The test results showed greater percentage improvement (limited sample available) in Stage 2 where the reclamation fill was removed. One factor which could have contributed to this apparent reversal in the trend may be the magnitude of surcharge or preload that was previously in place over the Stage 2 area when in use as a tank farm. The Stage 1 area, where the trial was carried out, had not been surcharged to any significant degree.

The results of the DC trail may be summarised as follows:

- Significant ground improvement has taken place following DC treatment. The trial indicated that full energy input was required (4 passes, 6 drop/point) and this level of energy was used in the production treatment. The test results have shown that the biggest ground improvement occurred between depths of 3.0 to 5.0 m below ground level. Test results showed inconsistency of ground improvement reflecting changes in soil type and attenuation of energy because of the presence of fine grained soils interbedded between granular soils

- Densification and ground improvement has occurred equally as well in Stage 1 with the hardfill in place, as Stage 2 where the hardfill was removed to expose the natural alluvial sands

- The results of the vibration study showed that the DC could be carried out within 5.0 m from a building before the threshold limit of 50 mm/s is reached as detailed in NZS 4403:1976. Trenches were excavated on two sides of the trial area to determine if these would reduce the transmission of surface energy. The vibration study concluded that the trench had negligible effect on ground vibrations

- A condition survey was undertaken on buildings within distances ranging between 50 and 112 m from the trial area. A comparison of "before" and "after" trial condition showed that there was no apparent disturbance or distress to the buildings resulting from the vibration effects of the DC trials

- There were no alarming human responses to the DC vibrations. Eight strategic positions were monitored around the trial area. Positions ranged between 45 to 240 m away from the trial area and only two positions perceived DC vibration as "very slight tremor" but "insignificant". Vibrations close to point of impact were more pronounced. Descriptions ranged from "significant jolt" to "slight tremor". Significant vibrations were felt inside a parked car close to the DC trial area.

- It was found that a heavy duty crane similar to those used in piling work performed well. Repeated lifting and falling of the tamping weight lead to unravelling of the wire cable. Weak points quickly developed in the wire rope due to pinching of the cable on the drum where the wire rope became crossed on the drum during lifting. Good quality ropes proved more cost effective than cheap ropes. Repeated lifting and free-falling of the tamping weight causes severe wear on the brake and clutch linings of the crane. A skilled operator is required to limit rope over run. A single lifting eye on the tamping weight can lead to excessive motion on lifting. Four lifting eyes on the tamping weight reduces motion during hoisting of the weight. The tamping weight easily tilts and causes a sloping imprint with a single lifting eye. Four lifting eyes on the tamping weight assists in steadying the weight and ensures upright position of tamping weight on impact.

APPENDIX D

WESTERN SAMOA CASE STUDY :

DYNAMIC CONSOLIDATION AT APIA FOR

7-STOREY BUILDING

APPENDIX D

WESTERN SAMOA CASE STUDY:

DYNAMIC CONSOLIDATION FOR 7-STOREY BUILDING

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APPENDIX D

WESTERN SAMOA CASE STUDY:

DYNAMIC CONSOLIDATION FOR 7-STOREY BUILDING

D1 INTRODUCTION

The project involved DC of a layer of reclamation fill, comprising sands, overlying coral reef formation at the site of a 7 storey building in Apia, Western Samoa. The building is founded on a raft foundation. Western Samoa is situated in a highly active seismic area and DC was principally carried out to provide an "earthquake resistant" platform and also to limit foundation settlements.

D2 SITE DESCRIPTION

The site is situated on a foreshore reclamation fill, in the Central Business District of Apia. The near-surface soils, identified as reclamation fill, comprised loose silty sands. These extended to 3 to 4 m depth and overlay a weakly to moderately cemented 0.5 to 2 m thick layer of coral reef formation. The underlying soils were loose silts and sands, extending to approximately 20 m depth, where basalt rock was encountered.

Ground water, was encountered at 1 m depth i.e. within the reclamation fill.

D3 TRIAL

A trial was carried out to evaluate consolidation effects and set procedures for the contract production. For the trial, variations in the number of drops per grid point were tried. Field testing, by way of Scala penetrometer (ScPT) and Continuous Standard Penetration Tests (CSPT) showed 6 drops per point were required to achieve an acceptable level of densification. Table D1 below summarises the trial test results.

Other monitoring carried out comprised the following:

(i) Level Survey - which showed a consolidation of approximately 120 mm, after DC treatment.

(ii) Vibration monitoring - velocities of up to 7 mm/s were recorded within 20 m of the impact point.

Table D1
Summary of DC Trials
(ScPT Tests)

Depth Below Ground Level (m)	AVERAGE SPT "N" VALUE		
	Before Compaction	Trial "A" (6 drops x 4 passes)	Trial "B" (4 drops x 4 passes)
0.0 - 1.0	20	8	8
1.0 - 2.0	12	15	13
2.0 - 3.0	10	16	14

Note 1: SPT "N" Values derived from ScPT using the relationship:

$$\text{SPT "N"} = \frac{\text{ScPT}}{1.5} \text{ (Blows per 300 mm): Ref.3}$$

2: Target "N" value = 15 (ref. App.D1)

D4 PRODUCTION TREATMENT

The production treatment at the site was carried out using 4 passes and 6 drops on each grid. The compliance testing comprised ScPTs at regular intervals over the site. An acceptable level of consolidation was generally achieved across the site with a minimum equivalent SPT "N" value of 15 being used as the critical value. A typical profile of strengths achieved is attached with the target Scala blow count being approximately 4 blows per 50 mm. Compliance testing showed the presence of a few isolated loose layers.

D5 RATE OF PROGRESS AND COSTS

A relatively rapid rate of progress was achieved on site, due to the experience of the operator who achieved a high production rate. The duration of the treatment was very short due to the relatively small size of the site (i.e. 50 x 30 m). The first two passes were completed over half of the site in one day and the total job was completed in six days. Two days were lost due to mechanical breakdown.

Details of the contract price for the work carried out are as follows:

		$NZ
•	Establishment	60,000
•	2 No. 8 tonne Tamping Weights	7,000
•	DC work, incl. regrading site passes	95,000
	TOTAL	$162,000

This therefore results in an average unit cost of $108/m^2.

D6 COMMENTS

- Whilst target strengths were achieved, it would probably have enhanced results if there had been a time delay of approximately 1 day between passes in order to allow pore pressures to dissipate. Construction conditions became difficult at times during the high energy passes due to excess pore pressures and saturated surface conditions.

- A fifth low energy pass was found to be necessary to densify the upper 1 m of sands.

TJES:MP

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GEOTECHNICS LTD
P.O. BOX 5271 AUCKLAND NEW ZEALAND
19 MORGAN STREET NEWMARKET AKL 1
TELEPHONE (09) 793 067 FAX (09)370 265

Form No. F 17

Sheet ..1.. of ..1...

Operator A. BARI Date 19/12/90 Site CENTRAL BANK Job No. 97/10702

TEST METHOD USED: NZS4402: 1988 Test 6.5.2. Dynamic cone penetrometer.

Test No.

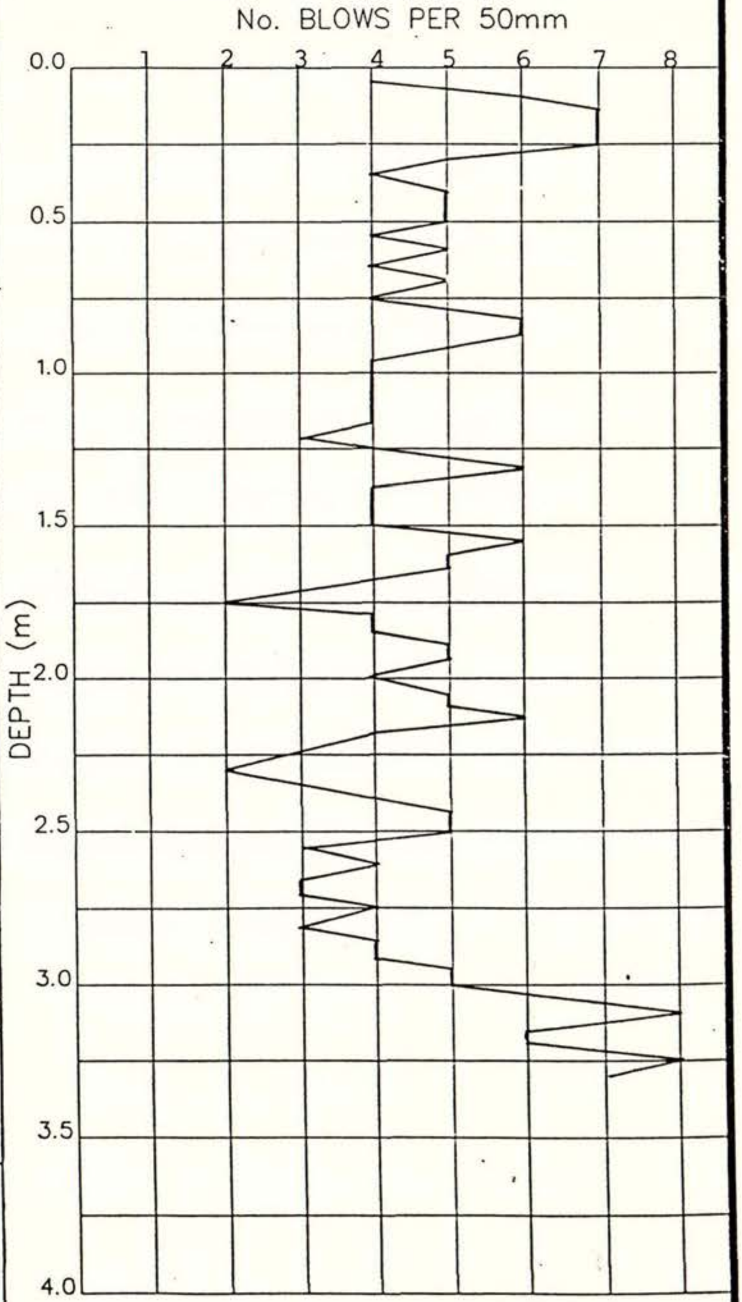
Depth from ground surface to excavated surface: ....0....... mm.

R.L. of ground surface: ....2.0...... m.

Location of Test

SPT 21

mm DRIVEN	No. BLOWS	mm DRIVEN	No. BLOWS
50	4	2050	5
100	6	2100	5
150	7	2150	6
200	7	2200	4
250	7	2250	3
300	5	2300	2
350	4	2350	3
400	5	2400	4
450	5	2450	5
500	5	2500	5
550	4	2550	3
600	5	2600	4
650	4	2650	3
700	5	2700	3
750	4	2750	4
800	6	2800	3
850	6	2850	4
900	5	2900	4
950	4	2950	5
1000	4	3000	5
1050	4	3050	7
1100	4	3100	8
1150	4	3150	6
1200	3	3200	6
1250		3250	8
1300	6	3300	7
1350	4	3350	(Refusal)
1400	4	3400	
1450	4	3450	
1500	4	3500	
1550	6	3550	
1600	5	3600	
1650	5	3650	
1700	4	3700	
1750	2	3750	
1800	4	3800	
1850	4	3850	
1900	5	3900	
1950	5	3950	
2000	4	4000	



DYNAMIC CONE PENETROMETER

APPENDIX C

NZ CASE STUDY :

DYNAMIC CONSOLIDATION AT
WAIWHETU TERMINAL, SEAVIEW, LOWER HUTT

APPENDIX C

NZ CASE STUDY :

DYNAMIC CONSOLIDATION AT

WAIWHETU TERMINAL, SEAVIEW, LOWER HUTT

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