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Soil Response to Surface Impact Loads During Low Energy Dynamic Compaction

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Abstract: Low energy dynamic compaction has been developed as an efficient and rapid method of improving properties of foundation soils unsuitable for moderate building loads. A series of low energy dynamic compaction tests was undertaken on rigorously controlled model beds of sand at 1:20th scale in a 500 g tonne centrifuge at 20 g to prove the system and to validate an innovative real-time soil stiffness monitoring approach. The results of seven centrifuge tests are presented to demonstrate the effect of the impact on the performance of the sand bed with increasing number of blows. Attenuation monitored by the dynamic load cells and embedded accelerometers showed improvement of soil mechanical behaviour. The rate and efficiency of improvement was clearly demonstrated by the relationship of peak particle velocity and dynamic peak pressure with increasing number of blows. The soil response was monitored by real time signal processing and analysis using the WAK (Wave Activated Stiffness (K)) analysis which has been proved to be a very reliable indicator of process success.

Key words: Centrifuge modelling, dynamic compaction, impact, soil response, WAK test

INTRODUCTION

Dynamic compaction is the mechanical densification of a soil bed by the repeated impact of a heavy weight on the ground surface. Whilst used for many years it was described and developed in a more scientific manner by Menard and Broise (1975), Parvizi and Merrifield (2004) and Parvizi (2006a, b). Conventional dynamic compaction of a soil mass operates by allowing a mass typically of between 10 and 20 tonnes to fall between 10 and 40 m to impact the ground surface. Stress waves are propagated through the soil, disrupting dynamically particle packing and ultimately decreasing the soil voids ratio in the vicinity of the impact thus improving the mechanical characteristics of the soil. Current design methods for the use of dynamic compaction remain largely empirical and are based on previous field experience using high-energy impact. In comparison with other methods of soil foundation improvement, dynamic compaction is seen as a simple method of soil modification that is rapid, economical and more appropriate in many situations.

This study describes a modification of the conventional dynamic compaction method called Low Energy Dynamic Compaction (LEDC). This method has its genesis in the requirement for rapid improvement of the behaviour of soil to a relatively shallow depth over a limited area. This method of compaction is termed low energy because the energy input per blow is low compared with that imparted by traditional dynamic compaction techniques. Furthermore the frequency of blow application is considerably higher than that used in the conventional method. Originally developed for military purposes, it has now gained acceptance as an appropriate method of ground improvement in congested inner city areas where derelict land is being redeveloped. Given the relatively low level of energy being imparted to the ground, the advantage of such a system in a congested area is the rapid attenuation of signal radiating from the point of impact thus reducing the chance of damage to adjacent structures.

A comprehensive review of the development of LEDC is available in the literature (Watts and Charles, 1993; Allen *et al*., 1994; Allen, 1996; Davies, 1991, 1994; Parvizi, 1999, 2006a, b; Parvizi and Merrifield, 2000, 2002, 2004). This method has a major advantage over conventional dynamic compaction in that it produces potentially less damaging ground accelerations near the point of impact and as such it can be used closer to existing structures. Moreover, since the equipment is lighter and more compact than the conventional cranes and large drop weights, it can be brought into use economically on more restricted sites; however, the depth of soil improvement may be restricted (Parvizi, 2006a, b).

Experience has shown that the depth of improvement using this method is restricted to less than 5 m, which could be considered as a disadvantage in its use. Watts and Charles (1993) and Parvizi and Merrifield (2004) have shown through trials that improvement is effective to depths of approximately 4 m. Relatively recently a new type of low energy compaction apparatus has been developed. The apparatus was originally designed and developed for the rapid repair of bomb damaged airfield runways. However, the compactor was later adapted for civil engineering purposes and is claimed to be an ideal method of treating various types of fill and coarse-grained materials (Allen, 1996).

The research that was undertook as part of this project in the School of Engineering at the University of Manchester sought to develop a more comprehensive understanding of the effects of low energy compaction on the improvement of the soil bed at the point of impact, the extent of the improvement both vertically and radially and the rate of attenuation of the dynamic pressure signal away from the area being improved. To this end a model compactor was developed, which would simulate the action of the field scale compactor (7 t, 2 m drop, 2 m diameter impact anvil). A 1:20 linear scale was adopted as this provided a convenient magnitude of mass and associated energy, which could be monitored using appropriate instrumentation. Whilst the mass and geometry of the compactor was scaled correctly to 1:20 of the field scale the rate of blow application was not scaled. This was not considered important since all the research was undertaken on beds of dry Mersey River sand, obviating the requirement to consider the generation of transient water pressures in the soil due the impact (Parvizi and Merrifield, 2004).

CENTRIFUGE MODELLING

The experiments were undertaken on the large 500 g tonne geotechnical centrifuge in the University of Manchester School of Engineering. The centrifuge has a diameter of 6.4 m and is capable of carrying a soil package of the size 2×1 m in plan × 0.6 m deep (Fig. 1).

Soil beds of fine Mersey River sand (D_{50} = 0.22 mm, coefficient of uniformity (U_c) = 1.5) were constructed by dry pluviation into a rectangular rigid model container having sides of 560 mm and a depth of 460 mm, simulating a soil volume of sides 11.2 m and depth of 9.2 m at 20 g measured at the soil bed surface. Six of the tests reported were on beds of loose relative density (D_r = 39.8±3.4%) with one soil bed having a medium relative density (Table 1). The sand beds were instrumented with dynamic Earth Pressure Cells (EPCs) and accelerometers embedded in orientations as shown in Fig. 2, with the

Table 1: Initial relative density (D_r) of tests

Test	Initial relative density (D_r %)
1	62.8
2	42.5
3	37.3
4	41.3
5	43.2
6	33.6
7	36.4

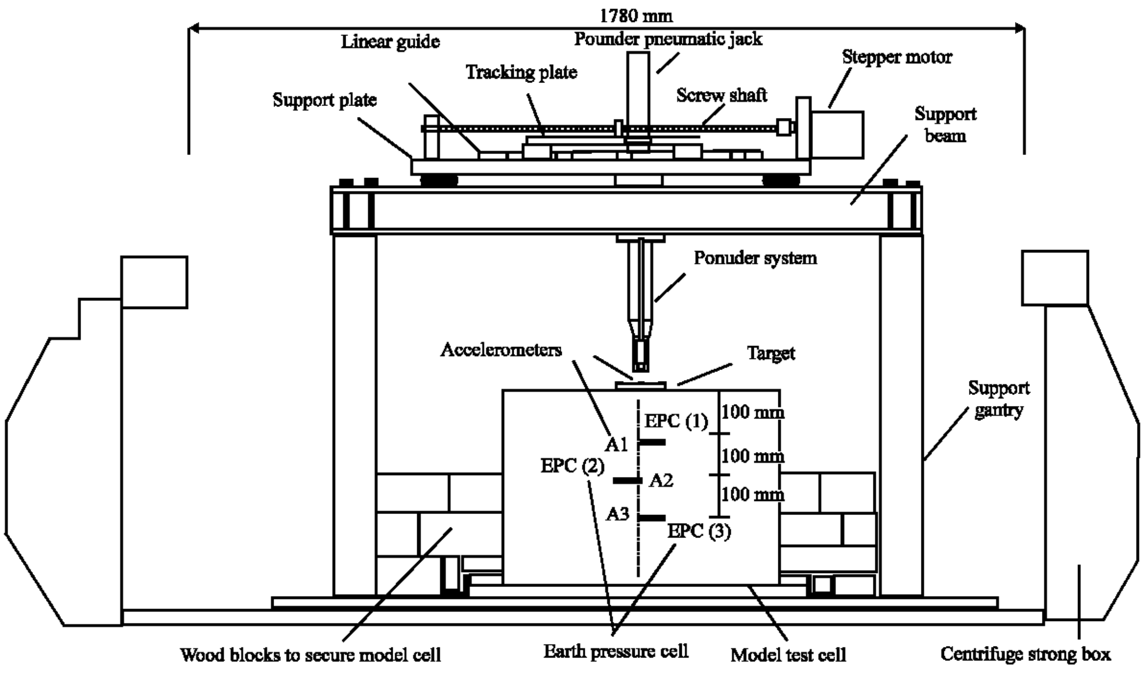

Fig. 1: Dynamic compaction test set-up

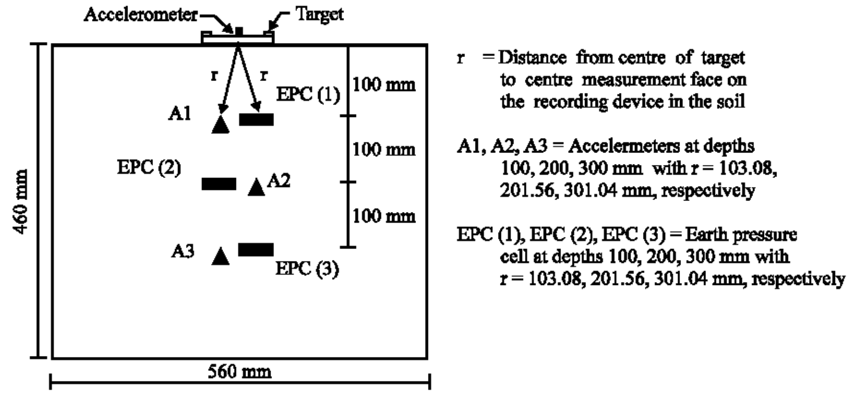

Fig. 2: Location of the instruments in the soil

active faces in the vertical and horizontal axes. A stiff target (diameter = 100 mm), instrumented with an accelerometer active in the vertical axis, was placed on the sand surface at the location of impact. The method and accuracy of placement are described by Parvizi (1999).

The model was accelerated to 20 g and allowed to stabilize for 20 min. The sand profile was monitored both prior to and after the test. The target was subjected to a maximum of twelve impacts (each impact having identical input energy i.e., constant height of fall of the drop weight with settlement of the soil bed) per compaction series. Since dry sand was being compacted and no pore pressures were being generated, no attempt was made to simulate a typical frequency of impact envisaged by Allen *et al.* (1994), (i.e., f = 0.5 Hz). The frequency of impact in these tests was of the order of 0.01 Hz (Merrifield *et al.*, 1998; Parvizi and Merrifield, 2004).

EXPERIMENTAL RESULTS

Instrumentation to demonstrate the response of the soil mass to the application of the dynamic compaction was embedded in the body of the soil, concentrated in the area below the target to a radial distance of 300 mm from the centre of the target. Preliminary analysis using the WAK test (Parvizi, 1999; Parvizi and Merrifield, 2004) showed that this would encapsulate the soil volume largely affected by the impact (Fig. 3). The instrumentation included miniature accelerometers and miniature dynamic earth pressure cells.

The ability to monitor internal soil response to both anthropogenic and natural phenomena through embedding miniature instrumentation within the soil mass without effectively modifying the characteristics of the bed is one of the significant advantages of centrifuge modelling over large scale field-testing. Not only are homogenous and essentially repeatable soil beds being modelled, but they are also instrumented in locations which would be extremely difficult to access in the field without substantial disruption to the soil mass being investigated.

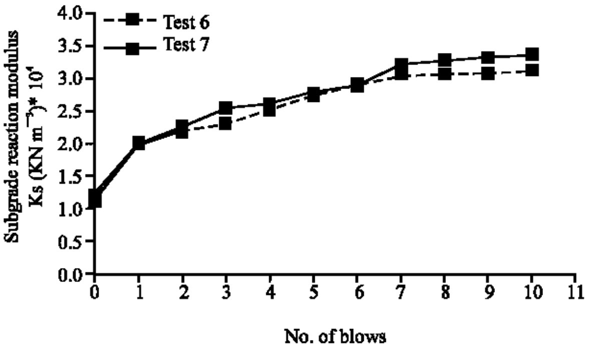

Fig. 3: Summary of centrifuge WAK test results

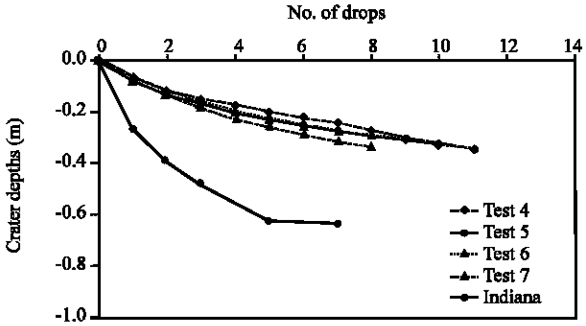

Fig. 4: Crater depths as function of number of blows for tests 4, 5, 6 and 7 (14 tm) with Indiana (37 tm) site

During dynamic compaction, both the accelerations and the earth pressure cells embedded in the soil recorded the signals generated by each impact. The vertical settlement of the target due to each blow was obtained by double integration of the signal from the accelerometers attached to the target (Fig. 4).

Time domain results: Two characteristics of the signals derived from the accelerometers and earth pressure cells in the time domain are:

- The maximum amplitude of each signal (peak acceleration and peak pressure, respectively)
- The arrival time of the signal

By integration of the acceleration, the particle velocities may be obtained in the time domain. Hence, both the magnitude and the arrival time of the Peak Particle Velocity (PPV) may be derived from any signal. Figure 5 shows a typical time domain signal of the dynamic pressure measured by dynamic earth pressure cells having the active faces in the vertical orientation at depths 100, 200 and 300 mm. The attenuation of the amplitude of the compressive stress wave as it travelled from the point of impact at the soil bed surface through the soil bed is clear. Both the magnitudes and arrival times of the peak pressure are also clear.

The change in soil properties due to each surface impact was measured in terms of the dynamic pressure response. For each blow the peak dynamic pressure and the time at which the peak occurred were plotted against the blow number in Fig. 6. As the soil became stiffer with increasing number of blows, the peak dynamic pressure

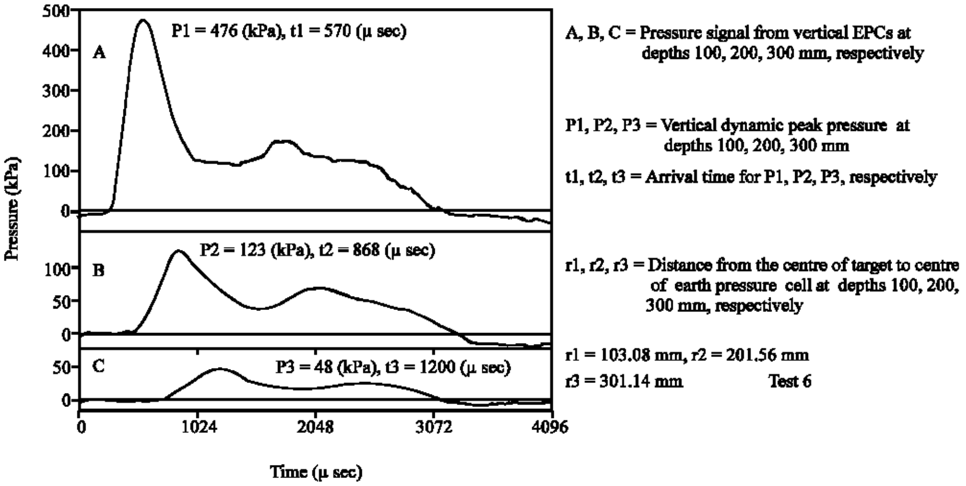

Fig. 5: Typical pressure signals in the time domain initiated by a dynamic load on the target

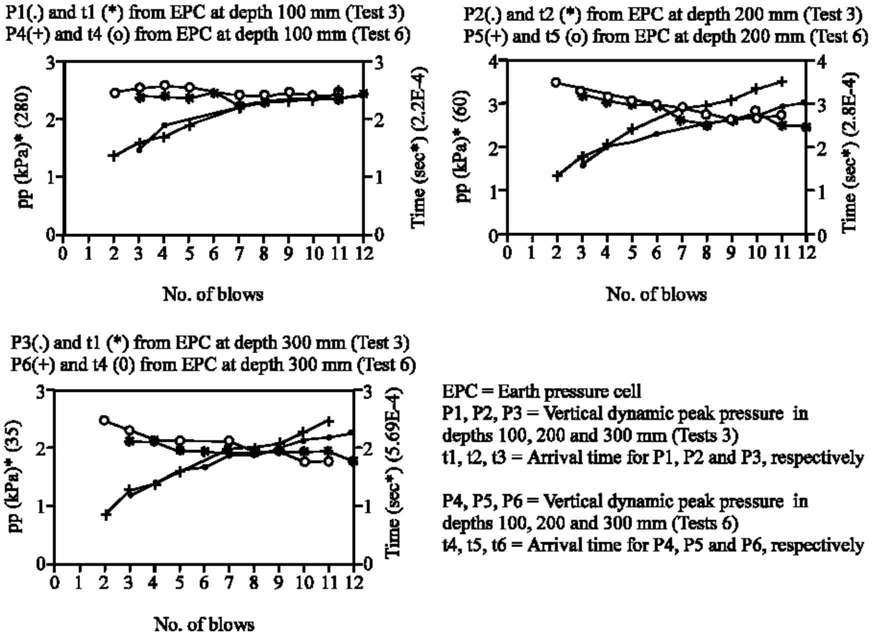

Fig. 6: Arrival times and magnitudes of peak dynamic pressure against blow number

increased at each of the measurement locations. Conversely, the arrival times for the signal, measured from the time of target impact, decreased.

Normalisation of results: In order to evaluate the soil response both in terms of peak particle velocity and peak dynamic pressure for all the tests undertaken during this research programme, a common basis for comparison is proposed.

A normalisation process is proposed which allows, to some degree, direct comparison of the data. Since, the process comprises a discrete number of events, with each event (impact) providing data with respect to particle velocity and pressure, it is proposed to present the soil response as a function of an increasing number of impacts. Whilst this may be a convenient method to examine the behaviour of a model being subjected to a predetermined and controlled series of blows in a centrifuge, the results derived from this analysis may be used only to predict the trend in behaviour and should not be extrapolated explicitly to predict soil response to some field situation.

The three variables, which characterise the response of the instrumentation are:

- Initial relative density of the soil (D_r)
- Distance of the instrument from the impact point (r)
- Orientation of the instrumentation in the soil mass (active face horizontal or vertical)

The objective is to normalise the signals to the extent that the overall soil response to an increasing number of blows may be evaluated. Examination of the data suggests that the magnitude of the peak acceleration signal and peak pressure signal is inversely proportional to the relative density of the soil (Fig. 7, 8). This is confirmed by the increase in magnitude of the peak signals with each succeeding impact as the relative density is increased with each successive blow on the target.

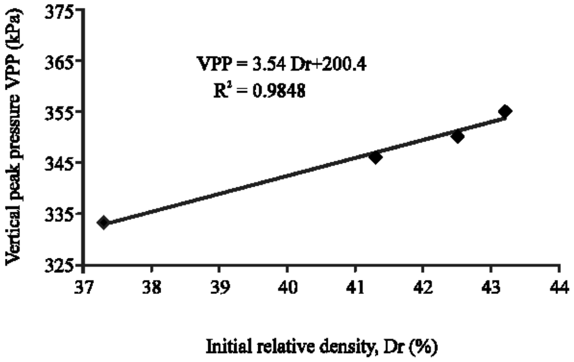

Fig. 7: Vertical peak pressure against initial relative density

The attenuation of the signal away from the source is assumed to approximate to r^{-2}. This is a commonly held assumption in the consideration of dynamic loading in soil (Wiss, 1981).

The earth pressure cells and accelerometers are oriented in the vertical and horizontal axis at different locations within the soil mass. The peak values of both the earth pressure response and the particle velocity for each blow in each test may be normalised against the initial relative density of the soil and the distance of the active face from centre of the target (r), (Fig. 9, 10). Since,

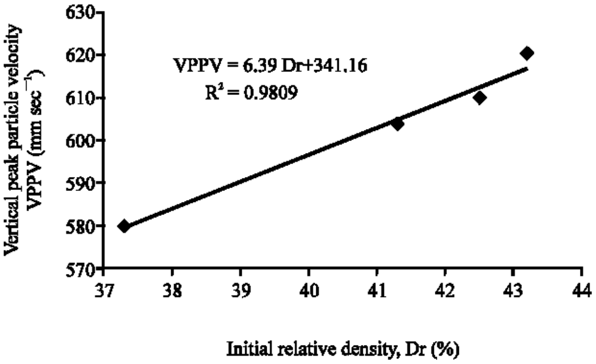

Fig. 8: Vertical peak particle velocity against initial relative density

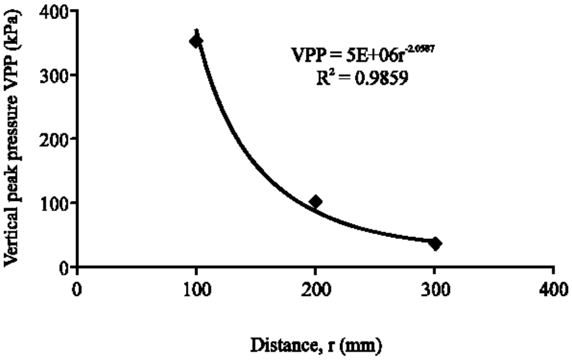

Fig. 9: Vertical peak pressure against distance

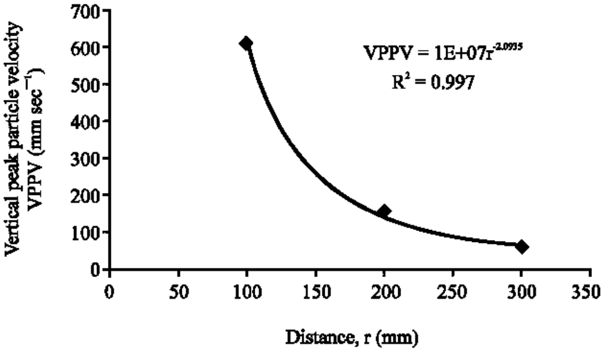

Fig. 10: Vertical peak particle velocity against distance

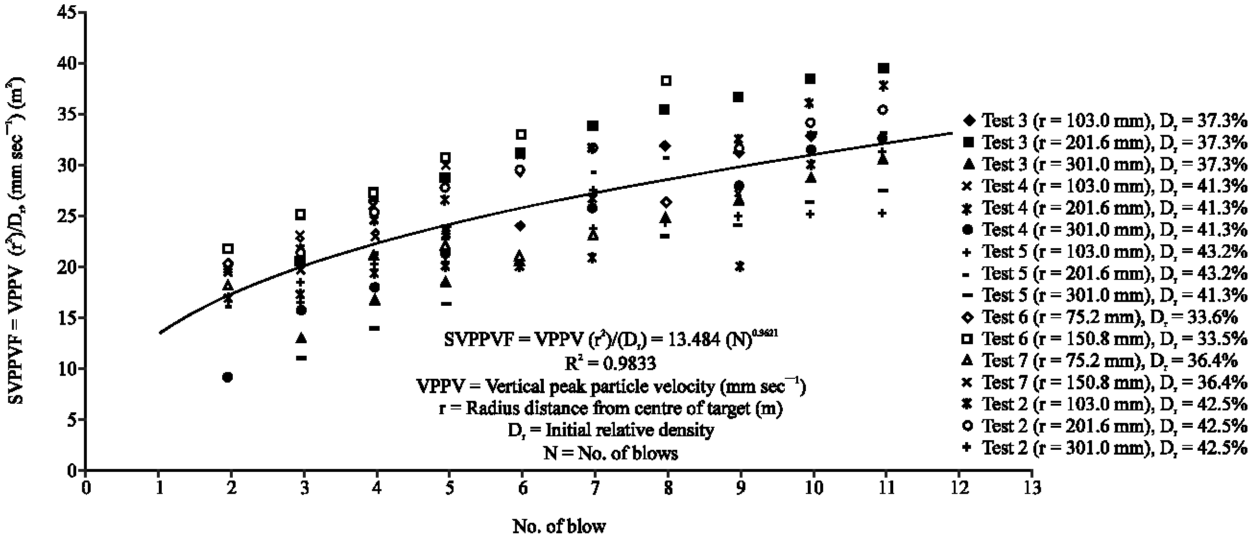

Fig. 11: Normalised peak particle velocity in the vertical axis (SVPPVF) for all instrumented locations against blow number

the active faces of the instruments were oriented either vertically or horizontally, the component of the peak magnitude of the signal in either the vertical or the horizontal direction has been adopted.

Vertical Peak Particle Velocity (VPPV) model: Tests 2, 3, 4, 5, 6 and 7 have been chosen for this analysis since the initial relative density of the soil bed in each case was approximately the same (39.8±3.4%) and they were all conducted at 20 g. The normalised vertical peak particle velocity may then be expressed as:

$$\text{Scaled vertical peak particle velocity factor (SVPPVF)} = \frac{\text{VPPV}(r^2)}{D_r} \tag{1}$$

The value of (SVPPVF) for all tests and the mean value relationship is shown in Fig. 11. It can be seen that whilst there is a variation in data for each blow the vast majority of points lie within two standard deviation of the mean. The following relationship was found to fit the data.

$$\text{SVPPVF} = \frac{\text{VPPV}\left(r^2\right)}{D_r} = 13.484(N)^{0.3621} \tag{2}$$

($R^2 = 0.9833$)

Horizontal Peak Particle Velocity (HPPV) model: Only results from tests 1, 6 and 7 have been used for this analysis, since it was only from these models that data from horizontally oriented accelerometers was available. The initial relative densities were 62.8, 33.6 and 36.4%, respectively. The horizontal peak particle velocity was normalised in the same way as the vertical peak particle velocity i.e.,

$$\text{Scaled horizontal peak particle velocity Factor (SHPPVF)} = \frac{\text{HPPV}(r^2)}{D_r} \tag{3}$$

SHPPVF is plotted against No. of blows in Fig. 12. From a regression analysis the following equation best expresses the relationship:

$$\text{SHPPVF} = \frac{\text{HPPV}\left(r^2\right)}{D_r} = 3.4225(N)^{0..195} \tag{4}$$

($R^2 = 0.826$)

Vertical Peak Pressure (VPP) model: The peak earth pressure response with increasing number of blows was normalised as described above. Data from the following tests, i.e., tests 1, 2, 3 and 5 have been used. The initial soil relative densities were 62.8, 42.5, 37.3 and 43.2%, respectively. The following equation describes this relationship:

$$\text{Scaled vertical peak pressure factor (SVPPF)} = \frac{\text{VPP}(r^2)}{D_r} \tag{5}$$

Again, a best-fit line has been derived from a regression analysis of the data and is described as:

$$\text{SVPPF} = \frac{\text{VPP}(r^2)}{D_r} = 7.217(N)^{0.3719} \tag{6}$$

($R^2 = 0.9249$)

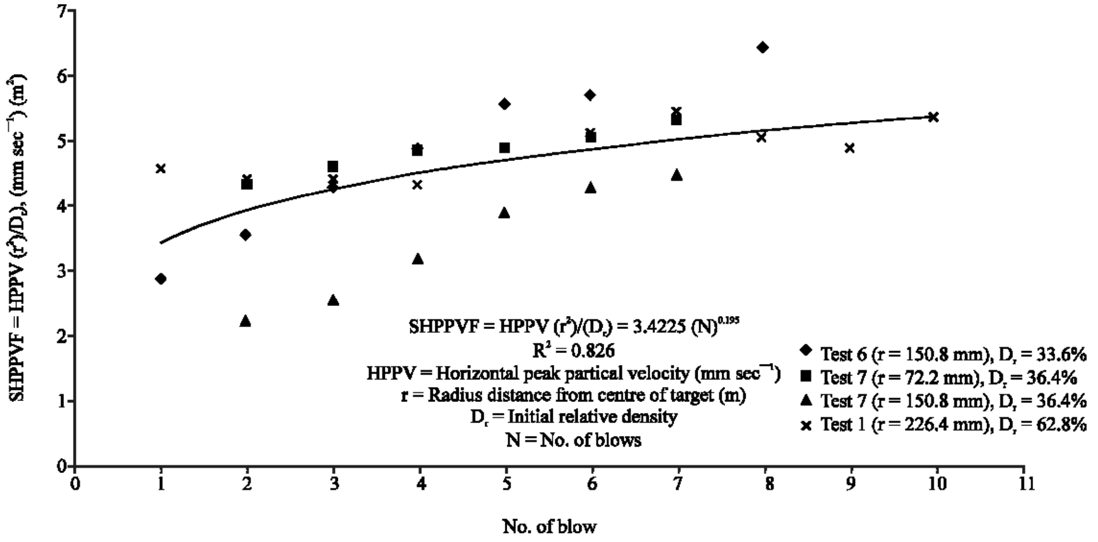

Fig. 12: Normalised peak particle velocity in the horizontal axis (SHPPVF) for all instrumented locations against blow number

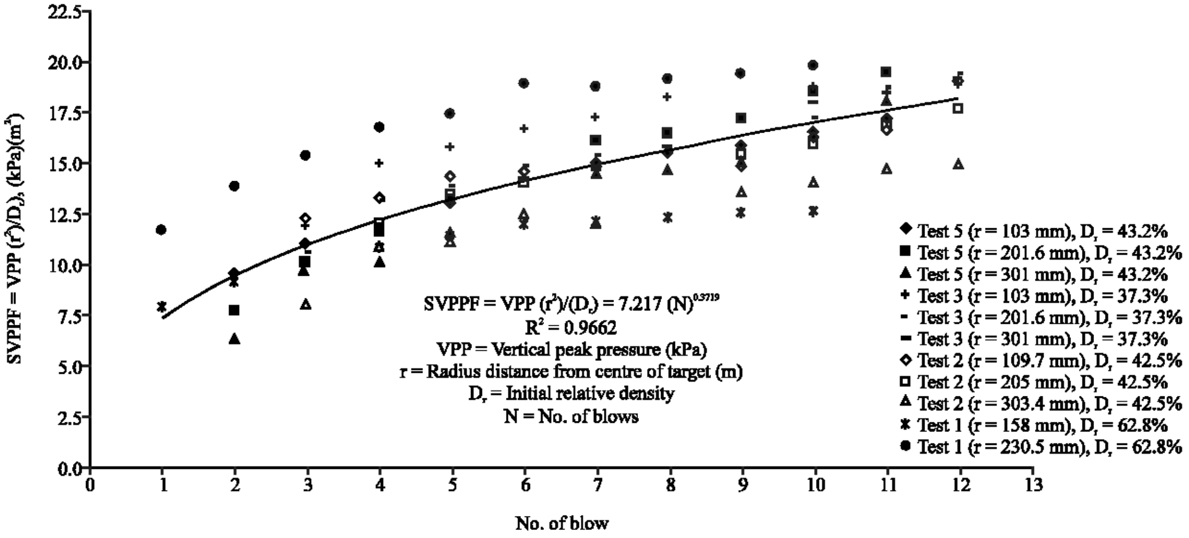

Fig. 13: Normalised peak pressure in the vertical axis (SVPPF) for all instrumented locations against blow number

The vertical peak pressure is plotted against increasing number of blows in Fig. 13.

Horizontal Peak Pressure (HPP) model: Only data from two tests of the test series were available for the analysis. These were from tests 1 and 2 where the initial relative densities of the soil beds were 62.8 and 42.5%, respectively. A total of five EPCs were used in these two tests.

The normalised horizontal peak pressures derived from all the EPCs are plotted against blow number in Fig. 14. The following relationships were derived from regression analysis:

$$\text{Scaled horizontal peak pressure factor (SHPPVF)} = \frac{HPP(r^2)}{D_r} \quad (7)$$

$$SHPPF = \frac{HPP(r^2)}{D_r} = 4.0258(N)^{0.2167} \quad (8)$$

$(R^2 = 0.9128)$

Discussion: In all cases the relationship between the normalised peak value of both particle velocity and normal earth pressure induced by the impact on the target were found to be best described by a power function. The exponents of Eq. 2 and 6 (0.362 and 0.3719) and

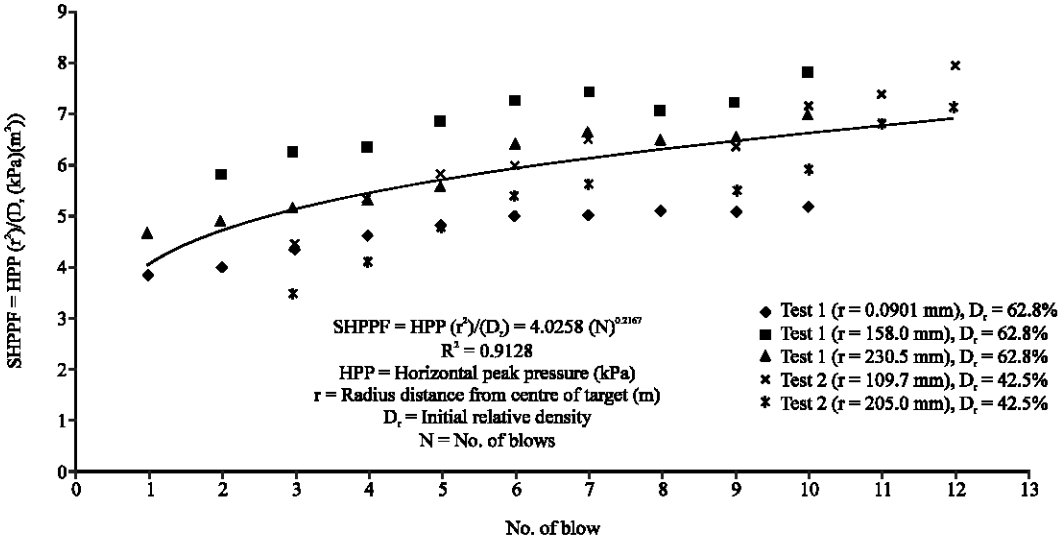

Fig. 14: Normalised peak pressure in the horizontal axis (SHPPF) for all instrumented locations against blow number

Eq. 4 and 8 (0.195 and 0.216) suggest that response in the vertical and horizontal directions for both particle velocity and transient earth pressure are consistent.

Whilst there is no explicit evidence to suggest that it is the case, there is a suggestion that the exponent should be zero (i.e., independent of the number of blows received) if the relative density of the vibrated mass of soil immediately preceding the blow from which the data are derived, was adopted in the normalisation. This would reinforce the commonly held understanding that dynamic loading attenuates by r^2 where, r is the distance measured from the point of impact to the location being considered.

Furthermore, the ratio of the exponent of N (number of blows) in Eq. 2 to the exponent of N in Eq. 4 is 1.86. Likewise the ratio of the exponent in Eq. 6 and 8 is 1.72. This is further evidence suggesting that if the behaviour of the soil in the vertical and horizontal direction for two measured parameters is the same, the behaviour is governed by the soil condition, or, in this case, relative density.

This relationship, which is similar to that derived for the stiffness measured by the WAK test plotted against blow number (Parvizi, 1999; Parvizi and Merrifield, 2000) further show the effect of dynamic compaction with increasing number of blows. The densifying effect, furthermore, is most dominant during the first seven to eight blows. Thereafter the effect diminishes steadily. This behaviour has particular relevance to the design of a soil densification process.

Table 2: Centrifuge scaling factors for dynamic compaction

Parameters	Model scale	Field scale
Length (r: m)	1	N
Initial relative density (D_r: %)	1	1
Velocity (v: mm sec^{-1})	1	1
Pressure (p: kPa)	1	1

Table 3: Field scaled coefficients

Eq. No.	Centrifuge model scale coefficient	Field scale coefficient
2	13.484	5392
4	3.423	1369
5	7.127	2851
6	4.026	1610

NORMALISATION OF THE RESULTS TO FIELD SCALE

The relationships for both peak particle velocity and peak pressure against number of blows may be expressed in terms of field values by up scaling the relationships using conventional centrifuge scaling laws (Taylor, 1995; Parvizi and Merrifield, 2004). The relevant scaling factors for soil dynamic response are given in Table 2.

To derive normalised relationships at field scale for Eq. 2, 4, 6 and 8 the constant coefficient in these equations should be multiplied by n^2 (20^2). These coefficients are then altered as in Table 3.

CONCLUSIONS

By monitoring the soil response in terms of magnitude and arrival times of both peak particle velocity and peak pressure at various locations within the soil bed subjected to controlled surface impacts and presenting this normalised against relative density and distance to the point of impact, evidence shows that the change in the soil response is due to a change in the soil properties, in this case the relative density. This is also corroborated by analysis of the transfer function associated with the input force and the output accelerations at the point of impact (Allen *et al.*, 1994; Taylor, 1995; Parvizi and Merrifield, 2004).

These normalised relationships may also be used to predict the attenuation of the dynamic pressure wave in congested areas surrounded by sensitive structures and assist in the choice of an appropriate number of blows required providing a modest but efficient and cost-effective improvement to the foundation soils.

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