STATE OF THE ART REVIEW OF THE PREDICTION OF GROUND IMPROVEMENT USING IMPACT COMPACTION EQUIPMENT

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INTRODUCTION

Impact compaction offers the engineer and contractor many unique benefits in the compaction of in-situ materials. These include high productivity and the provision of a stronger, deeper foundation, as well as the ability to compact a wide range of problem materials. The capabilities of these locally developed machines have been documented over the years, yet a deeper understanding of the mechanisms of the improvement has not been forthcoming. Recent research into the effectiveness of impact compaction in ground improvement has however revealed patterns of improvement that appear to be predictable. With a view to the development of a predictive model for compaction by impact compactors, the pertinent literature has been reviewed.

The aim of this report is summarise and synergise the most promising models found. The report first briefly reviews the impact compaction literature, and then reviews other relevant models, particularly those undertaken in the field of dynamic compaction.

Some useful prediction models were found. Although much work has been done in the field of dynamic compaction, most models found are still semi-empirical in nature. It is concluded that this is probably due to the influence of the water table, compactor geometry and soil parameters being largely ignored. It is further concluded that the primary *compactor* parameters required for a prediction model are the compactor mass, drop height, contact area and total energy.

REVIEW OF IMPACT COMPACTION (IC) LITERATURE

A detailed review of the impact compaction literature has been undertaken by the CSIR (Paige-Green, 1998). The author makes the observation that *"Impact compaction…results in compaction at depth, with disturbance of the upper portion of the layer"*. This is the simplest form of prediction, and well known to most users of impact compaction. He also notes that "larger loads *and* larger contact areas are better for deep compaction". This is one of the main limiting factors of conventional cylindrical compactors in deep compaction: the contact width of the applied line load is difficult to enlarge. In considering the large force imparted by impact compactors Clifford (1978) notes that "principles that hold true for impact devices hold true for impact rollers, except that, in addition, an impact roller delivers generated momentum due to the rotational effect of the roller mass". In a report investigating this hypothesis, it was found that this was not the case (Heyns, 1998), and that the potential energy of the machines formed the bulk of the imparted energy. Typical decelerations were found to be in the order of $100 m/s^2$ to $200 m/s^2$ (10 to 20 g's). Clifford rightfully notes in his conclusion that "the paucity of mathematical studies on various aspects on compaction, from generated energy to the soil response limits, show how difficult evaluation is".

In a recent paper, Berry et al (1998) noted that the impact compaction trials undertaken at Kriel revealed a peak in the post compaction test pits that were dug, and that this appeared similar to the shape of the Schmertman strain influence diagram (Schmertman, 1970). The trial pit excavation results are shown in Figure 1.

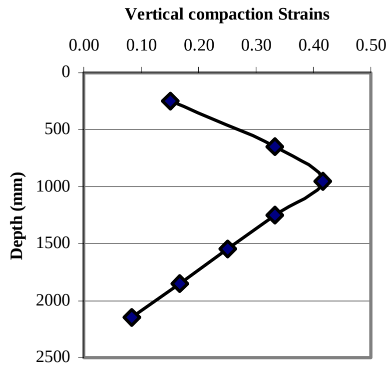

Figure 1: Settlement trial indicating degree of settlement with depth (Berry et al, 1998)

Apart from this observation, no mention was found of any prediction model in the IC literature, only descriptive trends.

OTHER COMPACTION SOIL IMPROVEMENT MODELS:

With the extensive use of the Menard dynamic compaction (DC) technique world-wide, much study has been undertaken in order to better understand the technology. The following parameters are typically predicted: patterns of improvement, depth of influence, surface settlement, settlement profile, surface stress, stress profile, residual horizontal stress profile and more recently, the void ratio reduction profile:

- Descriptive/observational – pattern of improvement

Initially, before the development of any mathematical prediction tools typical patterns of behaviour based on in-situ test results are all that is available to the engineer. Usually these offer little explanation. The most useful of these is given by Lukas (1986) and shown in Figure 2.

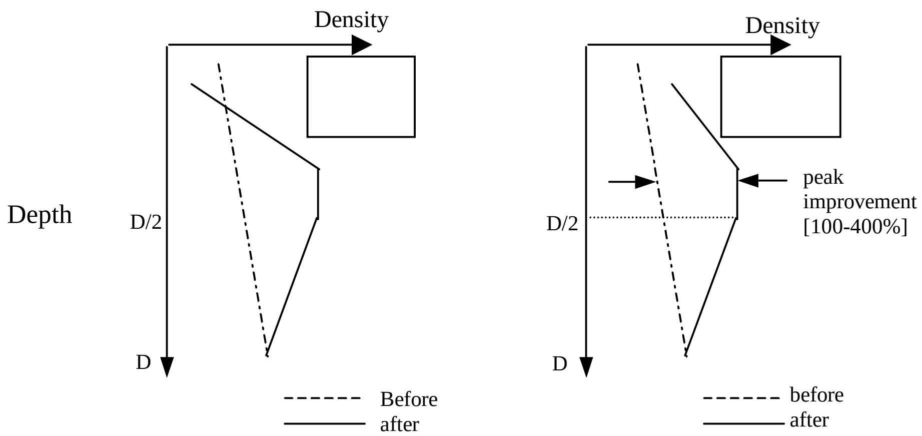

Figure 2: Descriptive pattern of DC soil improvement (Lukas, 1986)

This improvement pattern seems to tie in with the observation by Paige-Green above, that the surface is loosened, and compaction takes place deeper down.

- Predictions of depth of influence

One of the most important questions that needs answering is the depth to which improvement is achieved. To this end Menard & Broise (1976) suggested the well-known relation

$$d_{max} = \sqrt{W.H} \quad ,W=pounder\ mass\ (t),\ H=drop\ height\ (m) \qquad (1)$$

This was revised with experience and Lukas (1976) suggested

$$d_{max} = n\sqrt{W.H} \quad ,\ n=an\ empirical\ coefficient\ (0.3\text{-}0.8\ typically) \qquad (2)$$

The modified Menard equation (2) is still widely used in the industry, with the factor n=C.δ, where C=the velocity efficiency and δ= the stratigraphic coefficient (Varaksin, 1981). In the same publication Varaksin notes “In any type of unsaturated soil the shock causes a Proctor type compaction.” and that “the phenomenon becomes highly complex in saturated or impervious soil”. He then gives a formula to predict the increase in pore water pressure under saturated conditions. According to Varaksin, C=0.9 for cable drop and 1.2 for free fall. He also noted that 67% of the energy is dissipated in the Rayleigh surface wave, that this is represented by the δ coefficient. Once the point of liquefaction is reached, a rest period is required for the pore water pressures to dissipate. This rest period is of predictable duration. As impact rollers are generally used in non-saturated conditions, this aspect is not pursued any further, other than to note that the presence of a high water table is of great importance and needs to be considered. A typical energy-depth of influence chart from the use of the above equations is given in Figure 3.

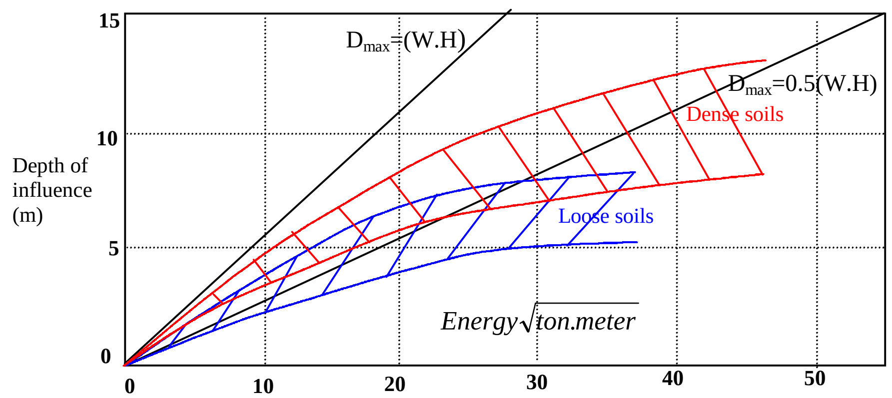

Figure 3: Typical energy-depth of influence chart for DC (Slocombe, 1993)

It is interesting to note that the depth of influence is thought to increase for denser materials.

- Predictions of impact displacements/settlement

In a massless soil of constant spring stiffness k (kPa/mm) the displacement is given by (Sears et al, 1982):

$$y = \frac{1}{2}\left[\frac{2mg}{k} + \sqrt{\left(\frac{2mg}{k}\right)^2 + \frac{8mgh}{k}}\right] \text{, where } \frac{mg}{k} = \text{static displacement} \qquad (3)$$

This is an elastic model however and would therefore rebound entirely if the theory was correct.

Kwang et al (1990) suggested that the ground improvement is related to the enforced (plastic) settlement curve and that this is uniquely related to the energy input and the pressuremeter limit pressure. The proposed curve is shown in Figure 4.

The energy intensity characteristic I_s is a function of only the energy imparted per unit area (E_B) and the pressuremeter limit pressure (P_L). The method indicates a "saturation energy intensity" after which there are limited returns. It fails to clearly describe the influence of moisture, however, and gives no guidance as to the distribution of the improvement with depth.

The enforced strain, η_{SE}, is defined below as

$$\eta_{SE} = \frac{S_E}{H_t} \quad , S_E = \text{enforced settlement} \quad , H_t = \text{thickness requiring treatment}$$

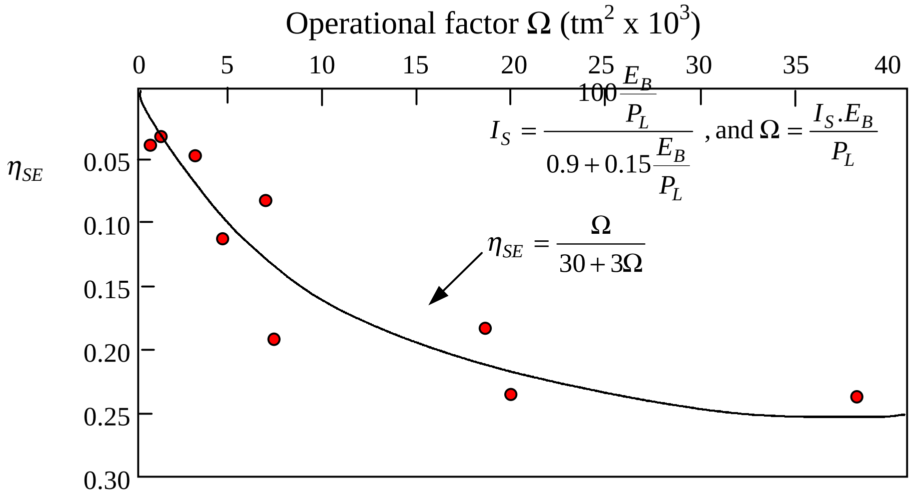

Figure 4: Unique enforced strain diagram (Kwang et al, 1990)

The selection of the depth requiring treatment, H_T, is left to the engineer, and leaves the method open to overestimation of this parameter. It is nevertheless a step forward as it demonstrates that there is a predictable level of energy input after which there is little gain in the ground improvement. It confirms that the ground improvement is a function of the enforced surface settlement. The critical parameters required by the method are the input energy (Σmgh) and the limit pressure of the soil.

- Prediction of settlement profile

Wallays (1983) suggested a method to predict the settlement at various depths below the compacted surface, i.e a settlement profile. The potential energy from the drop of the mass is equated to the work done by the vertical stress induced in the soil, plus the work done in moving the soil mass by the residual settlement. The derivation results in equations for a layered soil, predicting the surface stress, the surface settlement and the settlement profile:

$$\therefore \sigma_{\max} = \frac{1}{B^2}\sqrt{\frac{\eta.G.H}{A}}\left[\sqrt{1+\left(\frac{F}{2\sqrt{\eta.G.H.A}}\right)^2} - \frac{F}{2\sqrt{\eta.G.H.A}}\right] \quad (4a)$$

$$s_{\max} = \sigma_{\max}.B^2.A \quad (4b)$$

$$s_z = \sigma_{\max}.B^2.\left[\sum_{i=j+1}^{n}\frac{1}{E_i}\left(\frac{1}{B+z_{i-1}} - \frac{1}{B+z_i}\right) + \frac{1}{E_j}\left(\frac{1}{B+z} - \frac{1}{B+z_j}\right)\right] \quad (4c)$$

σ_{max} = max contact stress, B = load diameter, E_i = stiffness of layer i, z_i = depth to top of layer i
s_{max} = max settlement, A & F are influence factors, s_z = settlement at depth z
the efficiency factor $\eta = \eta_w \eta_i \eta_d$, *where* η_w = mass efficiency (typically = $2/3$)
η_i = impact efficiency (typically = $1/3$)
η_d = heave loss factor (typically = $2/3$) $\quad \therefore \eta \cong 4/27$

The method does not directly predict improvement, but may well be used or extended to obtain more measurable parameters such as density or void ratio. It does not clearly indicate the effect of the water table or the effect of Poisson's ratio (lateral strains). Material properties are dealt with indirectly through the stiffness used in the equations. Results are given in charts showing the measured settlement compared to the predicted settlement.

- Predictions of impact stresses

Intuitively, the contact stress ($\sigma_{o, dyn}$) has a large influence on the ground improvement. Estimates of this were of the first to be made, as this could then readily be input into stress distribution formula. Jessberger and Beine (1981) proposed laboratory testing with an accelerometer attached to a falling mass to determine the relationship between the decelerations and the impact velocity. The constant of proportionality, α, was then used in the equation:

$$\sigma_{0,dyn} = \alpha \frac{m}{A}\sqrt{2gh} \quad \text{,m=mass, A=base area of rammer, h=drop ht ,g=9.81m/s}^2 \qquad (5)$$

This means that the contact stress is proportional to the impact momentum, since the impact velocity, $v = \sqrt{2.g.h}$, for a constant base area. Mayne & Jones (1983) proposed a slightly different form of equation, based on the integral of the area under measurements of the impact deceleration-time graph:

$$\sigma_z = \frac{V_s\sqrt{WHB}}{4(B)^2} \quad \text{,Vs=shear wave velocity, H=drop ht, B=contact diameter, W=mass (t)} \qquad (6)$$

The formula Mayne & Jones give for the deceleration ratio (a/g), gives values close to those measured by Heyns (1998) on the tube axles of impact compaction plant:

$$\frac{a_{max}}{g} = V_s\sqrt{\frac{HB}{W}} \quad \text{,where } a_{max}\text{=maximum acceleration of pounder, g=9.81m/s}^2 \qquad (7)$$

Lewis (1957) proposed an equation that related the contact stress to the impact energy:

$$p = \sqrt{\frac{1}{2}.mv^2.\frac{k_s}{A}} \quad \text{,m=mass, v=impact velocity, g=9.81m/s}^2\text{, A=base area, } k_s\text{= spring constant} \qquad (8)$$

Therefore, to maintain a constant impact pressure the energy (½mv^2) must be proportional to the square root of the base area, or, for a square base, proportional to the side dimension B. i.e It is difficult to keep the contact stresses down as you raise the energy levels, as B can't be adjusted much.

The critical parameters for determining contact stress are therefore the mass, the pounder base area, the drop height and the soil stiffness. The deceleration, impact velocity, energy and momentum are related to these parameters.

- Predictions of dynamic stress profile

The proponents of the above contact stress predictions usually assumed some form of distribution of stress with depth to give a dynamic stress profile estimate. Jessberger and Beine (1981) proposed the following stress distribution base on Frolich's 1934 equation:

$$\frac{\sigma_{z,dyn}}{\sigma_{0,dyn}} = 1-\left(\frac{z}{\sqrt{z^2+r^2}}\right)^{v} \qquad \text{with } 7 < v < 15 \qquad (9)$$

$\sigma_{o,dyn}$=contact stress (from equation (5)), $\sigma_{z,dyn}$=stress at depth z, r=contact radius

Similarly, Mayne (1983) proposed the dynamic stress distribution:

$$\sigma_z = \frac{V_s\sqrt{WHB}}{4(B+z)^2} \qquad \textit{variables defined in equation (6) above} \qquad (10)$$

The authors assume that having this information allows the likely compaction to then be evaluated. No guidance was found on how to convert the applied dynamic stress into effective compaction. It seems that it is assumed that the higher the stress and the deeper the stress profile, the better the compaction.

- Prediction residual stress profile

A method commonly used to predict the increase in horizontal stresses against retaining structures by compaction plant (Norvais Ferriera, 1983) shows the residual horizontal stresses after compaction (Figure 5):

This method is usually used for the prediction of the increase in lateral stresses against retaining structures, but may also be used in compaction away from structures (Duncan et al, 1986). It is interesting to note that there is a peak in the residual lateral stress diagram and that this peak is a function of the assumed active and passive pressure lines and the applied dynamic stress profile. This means that the larger the applied stress and plate/pounder size, the deeper the peak residual horisontal strain. As the method was aimed mainly at the prediction of residual stresses, no attempt was made to use the method for prediction of the compaction profile.

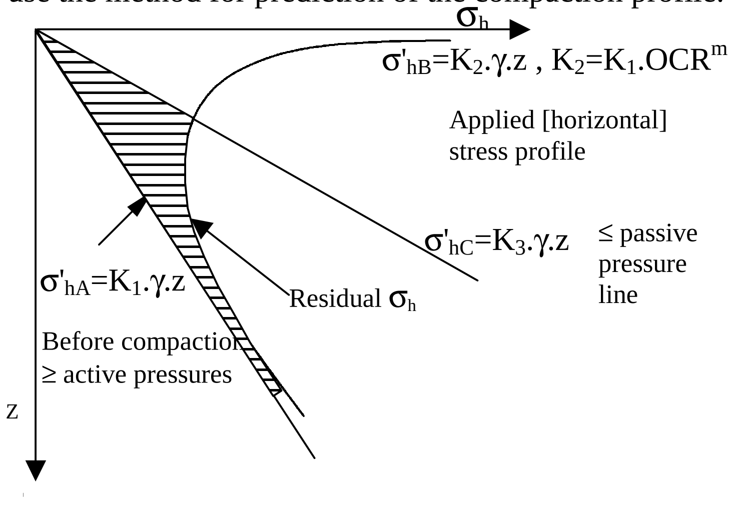

Figure 5: Predicted residual horizontal stresses after compaction (Norvais Ferriera, 1983)

It is notable that the predicted profile appears to correspond to that found by Berry et al (Figure 2).

- Prediction of void ratio reduction

Oshima & Takada (1997) proposed a model that predicts the degree of compaction achieved in terms of the relative density, D_r, based on model testing in sand. They showed that the improvement could be predicted in terms of the total momentum of the pounder:

$$\begin{aligned} Z &= a_z + b_z \log(mvN) \\ R &= a_R + b_R \log(mvN) \end{aligned} \qquad (11)$$

where Z=the vertical depth of improvement, R=radial improvement ,mvN=ram momentum, and a & b are empirical constants from laboratory testing.

The method was specifically aimed at dynamic compaction, and if used for the much lower energy/momentum levels of impact compactors, results in negative answers from below 15 passes of a 25kJ machine. With a different format of equation, the model may give better results. A notable omission from the model is the pounder base area. Empirical constants are available for changes in D_r of 40%, 20% and 10% respectively. This enables the bottom half of the profile of improvement to be drawn, including the depth of influence. The model does not predict the entire improvement profile, as the improvement immediately below the pounder is not evaluated.

A similar model was postulated by Poran & Rodriguez (1992). Their model used total energy rather than momentum. The model equations are:

$$\frac{b}{D} = j + k \log\left(\frac{N.W.H}{A.b}\right) \text{ , b = influence depth, D = tamper diameter, N = blows, W = mass,}$$
$$\text{H = drop ht, A = base area \& j, k are empirical constants} \qquad (12)$$

$$\frac{\mathrm{a}}{\mathrm{D}} = l + m \log\left(\frac{N.W.H}{A.b}\right) \text{ , l \& m are empirical constants from lab testing} \qquad (12a)$$

The equations must be solved itteratively. The model does not incorporate the effect of the water table (testing was on dry sand), and specific correlation coefficients must be obtained relevant to the conditions under consideration.

Figure 6 shows the limitations of the Menard type equations, where the depth of influence continues increasing indefinitely. Charles' solution (1978) for cohesive materials gives the lowest results. The behaviour is contrary to the other methods, as the depth of influence decreases with increasing pounder dimension (A_p=B^2):

$$D = 0.4\left(\frac{E_d B}{A_p c_u}\right)^{0.5} \text{ , where } \frac{E_d}{\mathrm{A_p}} = \text{energy/area, B = pounder width, } \mathrm{c_u} = \text{undrained shear strength} \qquad (13)$$

A wide scatter is found. This is probably due to the level of the water table and the pounder contact area not being addressed in some of the models.

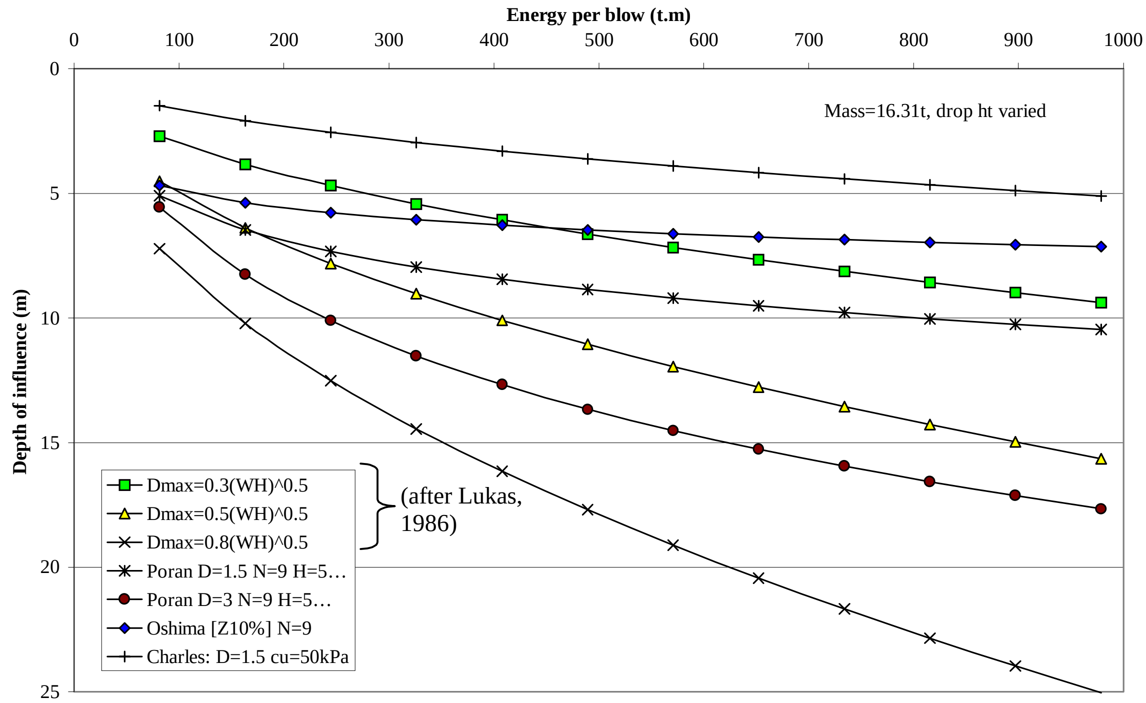

Figure 6 : Comparison of various prediction models - depth of influence

- Computer simulation based on the wave equation-profile of improvement prediction

In a paper presented to the American Society of Civil Engineers, Chow et al (1992), gave the most comprehensive (and complicated) predictive model found in the literature surveyed. This method predicts the reduction in the void ratio as measured by the relative density D_r. At the core of the method is a computer program that solves partial differential equations of a non-linear (spring and dashpot) soil model that takes plastic behaviour of the soil into account. Good correlation was found between predicted and measured parameters. An example of the measured versus computed prediction is given in Figure 7:

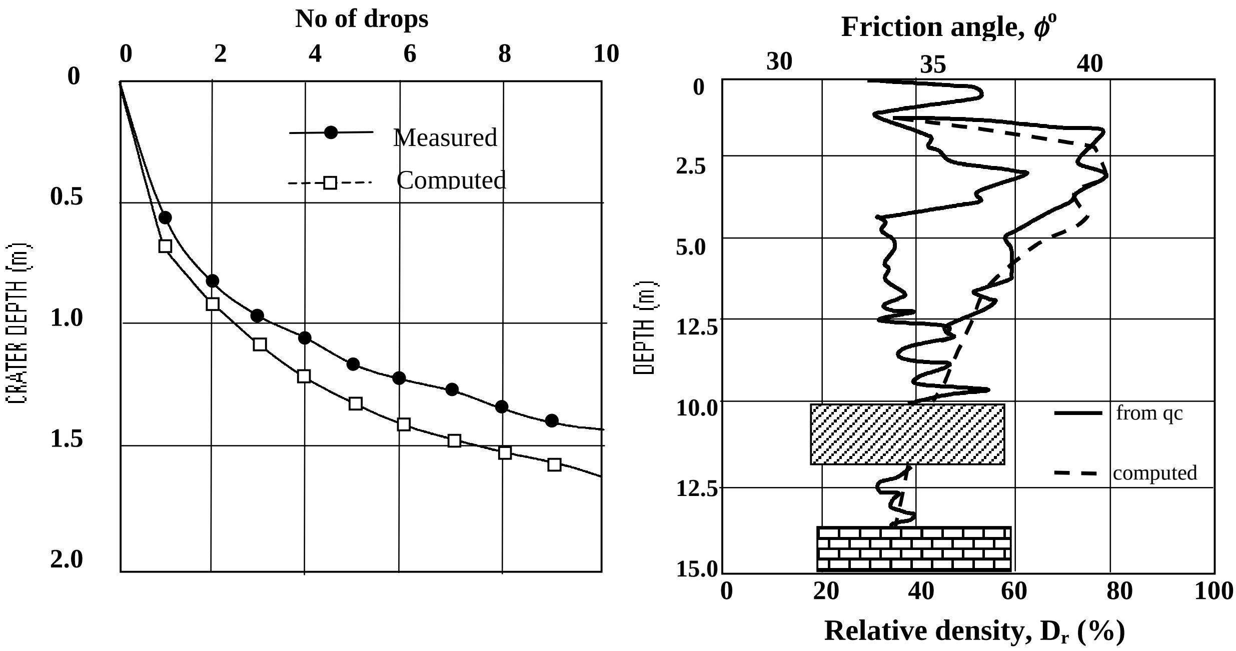

Figure 7 : Wave equation modelling of soil improvement (Chow et al, 1992)

It is again noteworthy that a peak in the improvement profile is also predicted by this model.

The model could well be used to predict the behaviour of the DMM and Impact compactors, but has the following drawbacks:

- The modelling is complex: it requires a computer to solve the wave equation model. This means that no understanding of the patterns of behaviour can be obtained without the use of the software. [i.e a black box solution].
- Lab testing is required to determine the "phenomenological" soil model.
- The spring and damping constants (k_s & c_s) have to be measured in the laboratory
- The soil springs behave in an elastic-perfectly plastic manner
- ϕ is estimated from empirical equations [ϕ'=28+15.D_r]
- The ratio of vertical to horizontal stresses is estimated from empirical equations (the analysis is sensitive to this)-i.e the model is sensitive to the value of Poisson's ratio used.

The method is able to predict both the settlement and the reduction in void ratio as measured by the relative density [D_r], and then, using Meyerhof's empirical equation, an estimate of the increase in friction angle is made. This is by far the most sophisticated and impressive method found to date.

CONCLUSIONS

Various useful prediction models were found, ranging from the estimation of settlement, stress, residual stress and void ratio. It is concluded that the *compactor* parameters that are critical to a comprehensive predictive model are the compactor mass, the drop height, the contact area and total energy (or total momentum). Most models did not address the soil parameters directly. The *soil* parameters deemed important by the authors are the initial void ratio, the initial moisture content and depth of water table, the grading and Atterberg limits, and the Poisson's ratio of the material. Most other parameters are a function of the above. Cementation and soil structure also play an important role.

For the prediction of ground improvement by impact compactors, several of the models may be used as an initial indication of the improvement. No single model simple was found that could confidently be used for impact compaction. It is likely that a combination or modification of these into one model may be required for reliable predictions.

Several of the models reviewed predicted a peak in the improvement profile from about B/2 to B below the surface of the ground, where B is the contact diameter of the impact load.

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Graduated from University of Natal in 1980. Worked for Protekon (Projects, KZN) for 3 years. Transferred to Protekon Bridge Office for 5 years, designing both Bridges and heavy duty pavements, where an interested in geotechnics was started. Completed BHons degree at UP in geotechnics during this period. Moved to Landpac, where the need to predicted the improvement in the ground after impact compaction, formed the basis of a Mastered degree, which is currently being finalised. Have just joining Franki Africa as Senior Design Engineer.