DISCUSSION

# Dynamic penetration resistance and the prediction of the compressibility of a fine-grained sand—a laboratory study

C. R. I. CLAYTON, M. B. HABABA and N. E. SIMONS (1985). *Géotechnique* **35**, No. 1, 19–31

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The Writers are in total agreement with the main theme of the Paper, namely that the factors affecting compressibility and penetration resistance of sands are not homologous and therefore, in general, it is not possible to assess compressibility from correlations with penetration resistance. The errors are particularly large if the sand has a stress or cyclic strain history. This was also the main thrust of the paper by Lambrechts & Leonards (1978), which has been fully confirmed by extensive tests conducted in recent years in Italy by Jamiolkowski and his co-workers (Jamiolkowski, Ladd, Germaine & Lancellota, 1985). In spite of this general agreement, there are some specific items in the Paper that warrant further discussion.

PENETRATION TESTING

The introduction to the Paper and the references used in the discussion of the factors affecting dynamic penetration resistance suggest that the results obtained with a cone are applicable to the standard penetration test. Although qualitative effects may be similar, the relative importance of some of the factors is not expected to be the same, either in drained and or in undrained conditions.

In undrained shear, prefailure compressibility has a strong effect on the porewater pressure that is generated; therefore saturation has a large influence on the dynamic penetration resistance. There is no indication given by the Authors on whether penetration was performed under drained conditions or not. If partly undrained conditions existed, then clarification of what considerations were given to excess porewater pressures in the analysis of the data would be welcomed.

FACTORS AFFECTING COMPRESSIBILITY

The Authors have summarized the factors affecting compressibility (Table 2 in the Paper) and estimated their maximum quantitative effects based on experimental evidence reported in the literature. While these values may reflect a 'possible maximum effect' for all granular materials, it is important to realize that for a specific sand the relative effect of each factor may differ considerably from these maxima.

The Authors have indicated a maximum effect of angularity of the order of 7–14 on the basis of a review of work by Holubec & D'Appolonia (1973). If, however, the data for glass beads are discarded, whose properties are clearly extreme with respect to naturally occurring sands, then a more reasonable effect of angularity of the order of 2–3 is observed for sand at a given relative density.

The differentiation between void ratio and relative density as presented in the Authors' Table 2 is unclear. Perhaps the mechanism described under void ratio should have been labelled relative density and the influence of what the Authors referred to as relative density could have been more clearly stated in terms of fabric, which includes both particle arrangement and particle orientation, as follows.

(*a*) *Particle arrangement.* In Fig. 1, the same number of identical (two-dimensional) particles, or portions thereof, are contained in the outlined area; thus the void ratio and the relative density are the same. However, they have very different fabric owing to particle arrangement. The matrix in Fig. 1(b) can be seen to be more compressible than that in Fig. 1(a). The 'possible maximum effect' of particle arrangement is far more important in loose sands than in dense sands; at a relative density of the order of 50% the compressibility is affected by at least a factor of 2–3 (Seed, 1976).

(*b*) *Particle orientation.* The particles shown in Fig. 2 have the same void ratio and relative density but different particle orientation; the matrix shown in Fig. 2(a) can be seen to be less compressible in the vertical direction than that in Fig. 2(b). The effect of particle orientation is more important in dense sands than in loose sands, with a 'possible maximum effect'

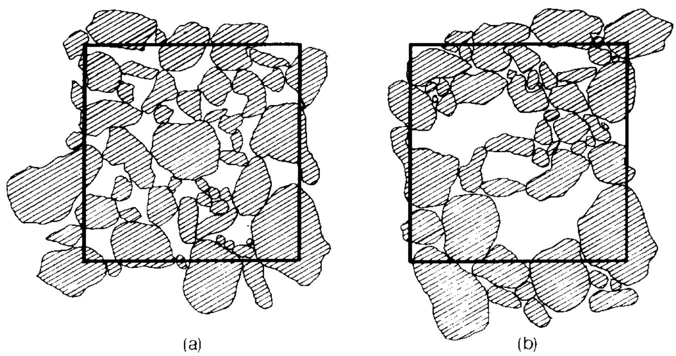


**Fig. 1. Illustration of potential ranges in the arrangement of identical particles at the same relative density**

of the order of 2–3 (Oda, 1972). It is also evident that preferred particle alignments result in properties such as strength and compressibility that are inherently anisotropic.

The first column in the Authors' Table 2 lists the factors affecting compressibility in two categories

(*a*) material dependent
(*b*) stress dependent.

On the basis of the additional factors noted above, a reorganization of the table to consider a third category, namely fabric-dependent factors, is suggested. It would include particle arrangement, particle orientation and relative density. Depositional anisotropy would be included under this category.

COMPARISON OF SOIL MODULI

The data presented by the Authors in their Figs 9 and 11(a) have been replotted by the Writers in Fig. 3. The shaded area shows the constrained modulus for Ticino sand (data from Bellotti, Crippa, Morabito, Pedroni, Baldi, Fretti, Ostricata, Ghionna, Jamiolkowski & Pasqualini, 1985). The figure clearly indicates the important effects of strain path during prestressing and mean stress level during testing on the intepreted value of the modulus.

In the Authors' tests, both $\sigma_v'$ and $\sigma_h'$ were incremented but only $\Delta\sigma$ was considered in calculating the modulus. In the paper by Lambrechts & Leonards (1978), Young's modulus, based on an incremental stress increase in $\sigma_v'$ alone, was measured. Owing to inherent and induced anisotropy, which are a function of material characteristics, specimen preparation techniques and relative density, the effects of changes in stress ratio along different loading paths cannot be evaluated from the available data. Thus, it is not possible to compare directly the Authors' moduli *vis-à-vis* those presented by Lambrechts and Leonards. However, a modular ratio MR (the ratio between the overconsolidated and the normally consolidated moduli) permits visualization of behavioural trends because the effects of the factors previously listed are compensated when the modular ratio is formed. A plot of MR versus mean normal stress is shown in Fig. 4. The modular ratio values include constrained moduli (used by Bellotti *et al.* (1985)), Young's moduli (used by Lambrechts & Leonards (1978)) and the

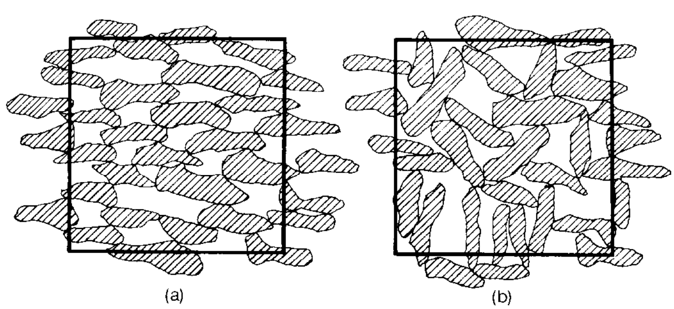


**Fig. 2. Illustration of potential ranges in the orientation of identical particles at the same relative density**

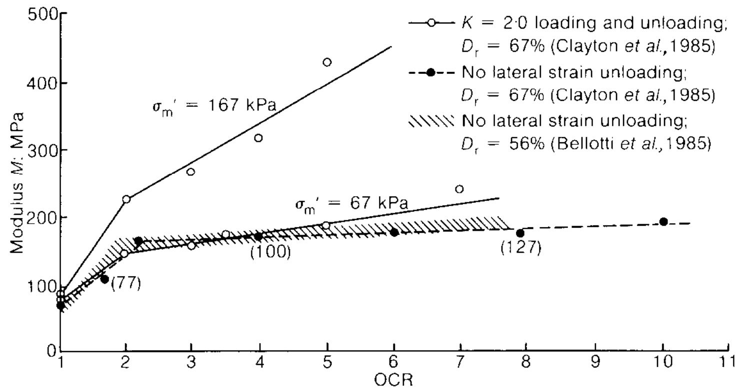


**Fig. 3. Effect of overconsolidation ratio OCR on the tangent deformation modulus (values in parentheses are estimates of $\sigma_m'$ in kilopascals)**

moduli defined by the Authors. (It should be noted that $\sigma_m'$ increases with the overconsolidation ratio when unloading is at no lateral strain. If the overconsolidated constrained moduli were corrected to the same $\sigma_m'$ value for the normally consolidated modulus, the lines shown in Fig. 4 would be shifted downwards without affecting the trend.) It can be inferred that MR is strongly dependent on the strain conditions during unloading, and that MR increases as the mean confining stress decreases or as the relative density decreases. (The range for OCR = 2–7 is much larger when the stress ratio $K$ is held constant during unloading, compared with the condition of no lateral strain. The reason for this is apparent from the plots in Fig 3.) The influence of material-dependent factors is also indicated. Accordingly, a particular value of MR should not be described as 'too high' or 'too low' without due consideration of, at least, the strain path during prestressing, the mean stress level and the relative density.

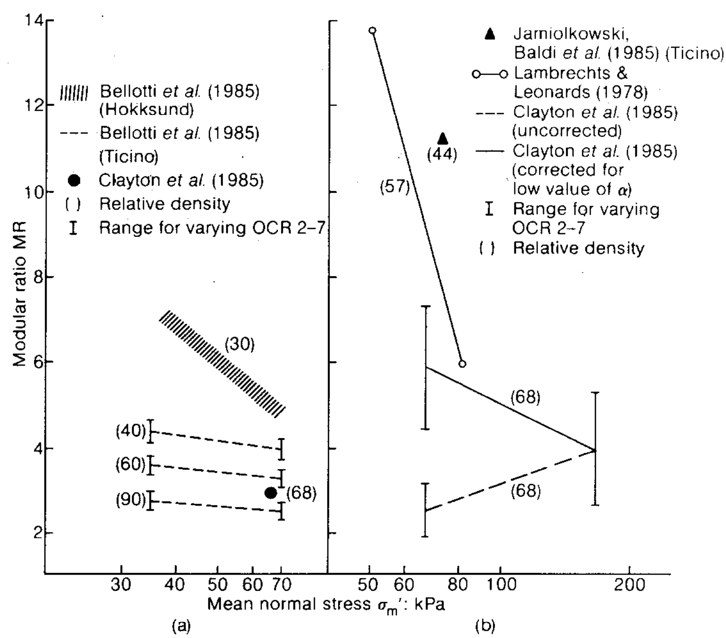


**Fig. 4. Effects of relative density, stress level and strain path on the modular ratio: (a) no lateral strain unloading; (b) constant $K$ loading and unloading (values in parentheses denote the relative density)**

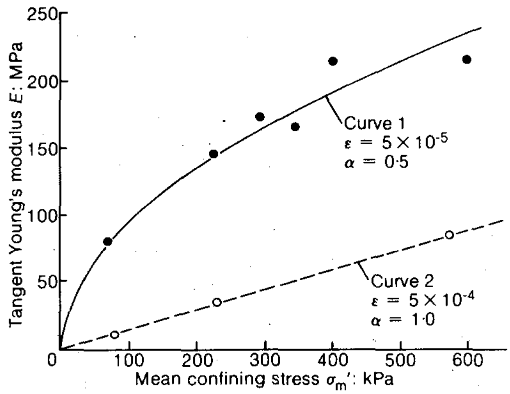


**Fig. 5. Effect of mean confining stress on tangent Young's modulus**

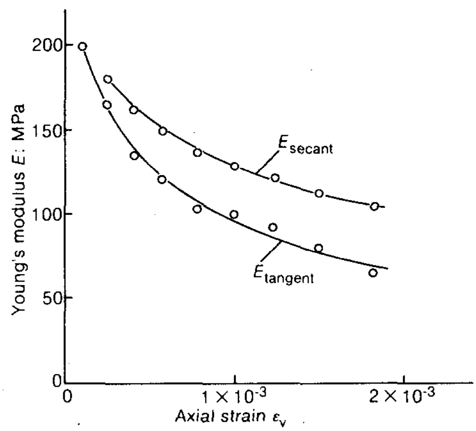


**Fig. 6. Effect of strain level on modulus value**

## EFFECTS OF STRESS AND STRAIN LEVEL

A triaxial test on Ottawa sand ($D_{50} = 0{\cdot}76$ mm and $C_u = 1{\cdot}1$) at a relative density of about 60% was performed by the Writers using sensitive new apparatus (described by Alarcon, Chameau & Leonards (1985)) that combines triaxial compression capabilities with resonant column and torsional shear features. A constant value of $K = 2{\cdot}0$ was used for loading and unloading.

Normally consolidated tangent Young's moduli at low strains ($\varepsilon_v \approx 5 \times 10^{-5}$) were determined at different consolidation stress levels (Fig. 5, curve 1). The curve corresponds to an exponent $\alpha = 0{\cdot}5$ in the relation

$$E = m\sigma_0 \left(\frac{\sigma_3'}{\sigma_0}\right)^{\alpha} \qquad (1)$$

The values obtained by Lambrechts & Leonards (1978) are also shown in the figure (curve 2). They correspond to a similar sand at about the same relative density but were determined at a higher strain level ($\varepsilon \approx 5 \times 10^{-4}$). This curve represents a value of $\alpha = 1{\cdot}0$. Normally consolidated Young's moduli were also determined at $\sigma_m' =$ 600 kPa for various strain levels. The degradation curves for the tangent and secant moduli are shown in Fig. 6 and indicate that important reductions occur, even at relatively small strain levels; therefore, for any comparison of moduli to be valid, tests should be performed at comparable strain levels. (The degradation curves are dependent on the dilatancy characteristics of the sand, which are influenced by the mean confining stress level.) If the modulus determined for curve 1 (Fig. 5) at $\sigma_m' = 600$ kPa is corrected for strain level using Fig. 6, a value close to the curve obtained by Lambrechts and Leonards at this stress level is determined.

The importance of Figs 5 and 6 is that they emphasize that the modulus is strongly influenced by the strain level at which it is evaluated, and that the value of $\alpha$ is neither unique nor approximately equal to 0·5 but depends on the strain level and varies from about 0·5 to about 1·0 for the range in strains noted in the above tests. (Alternatively, the effects of strain amplitude could be considered by adding a stress ratio term to equation (1) (Yu & Richart, 1984).) Other factors such as sand characteristics and relative density should also be considered; for example, Scheidig (1931) reported data on the initial tangent modulus for sand in which $\alpha$ ranges from 1·0 for loose conditions to 0·4 for dense conditions.

The compressibility of granular soils varies in a complex way with material, fabric and stress-dependent factors. Much further research is needed before the in situ value of this parameter can be evaluated reliably.

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## Authors' reply

The Authors wish to emphasize their support for the use of the standard penetration test. Despite the well-known problems of the test it continues to find favour for routine investigation over much of the world. In the Authors' opinion its usefulness is likely to increase in coming years, not only as a result of improved reference procedures, but also as a result of a better knowledge of its predictive capabilities and inadequacies.

The work described in the Paper was, at least partly, stimulated by the two comparisons between quasi-static penetration resistance in normally consolidated and prestressed sand presented in the last page of Lambrechts & Leonards' (1978) paper. However, there were doubts about the validity of the results *vis-à-vis* dynamic penetration testing because

(*a*) a quasi-static cone was used
(*b*) the conclusion that stress history does not influence cone penetration resistance was reached on the basis of only two reported comparisons
(*c*) the penetrometer-to-specimen diameter ratio was only 5·6, which is widely believed to be unacceptably low: with such a small specimen it would not be surprising if the stresses on the boundary were the dominant influence on penetration resistance.

It is encouraging that it has now been accepted (Jamiolkowski, Ladd, Germaine & Lancellota, 1985) that the main conclusion of the Paper, namely that there is only a tenuous link between compressibility and dynamic penetration resistance, is also true for quasi-static penetration tests.

### PENETRATION TESTING

The Authors agree with the discussion that while the qualitative influence of the factors in Table 3 of the Paper may be similar for both dynamic and quasi-static penetration tests the relative importance of each factor would not necessarily be expected to be the same.

With regard to drainage conditions during penetration resistance, it is of primary importance firstly to establish that boundary conditions do not influence test results. Given a sufficiently large chamber to prevent such an influence, it remains extremely doubtful whether any dynamic test could be considered to be 'drained' (i.e. not subject to excess pore pressures during driving). In these tests, although the specimen boundaries were 'drained' during penetration testing, excess pore pressures were measured at the penetrometer tip (Hababa, 1984). Such measurements have previously been reported by Clayton & Dikran (1982).

Dynamic penetration tests on a much coarser, less uniform sand ($D_{50} = 0{\cdot}8$ mm, $C_u = 7$) also gave rise to excess pore pressures, although their duration was shorter (Dikran, 1983). Excess pore pressures measured at the penetrometer tip were not used in the analysis of data presented in the Paper.

### FACTORS AFFECTING COMPRESSIBILITY

As stated in the Paper, the Authors believe that the early optimism that relative density would be an important unifying parameter to describe the behaviour of granular soils (for example Burmister, 1948) has not been borne out by experimental studies. Leonards and his co-workers find the differentiation between void ratio relative density unclear in Table 2 of the Paper; relative density was included (albeit bracketed) only because of its widespread use, and not because the Authors believe that two different sands at the same relative density should be expected to exhibit similar compressibility characteristics.

Particle arrangement and particle orientation are factors which affect compressibility. Indeed Table 2 includes depositional anisotropy, which results from particle orientation, as a material-dependent factor. It would be of value to have experimental data to allow a numerical assessment of the relative compressibilities of the two parts in Figs 1 and 2, but presumably such data are not available. Although the Authors agree that it would be desirable to add a third class to the two proposed by Daramola (1978), following Rowe (1972), the Authors suggest that this class should be termed 'structure-dependent factors'.

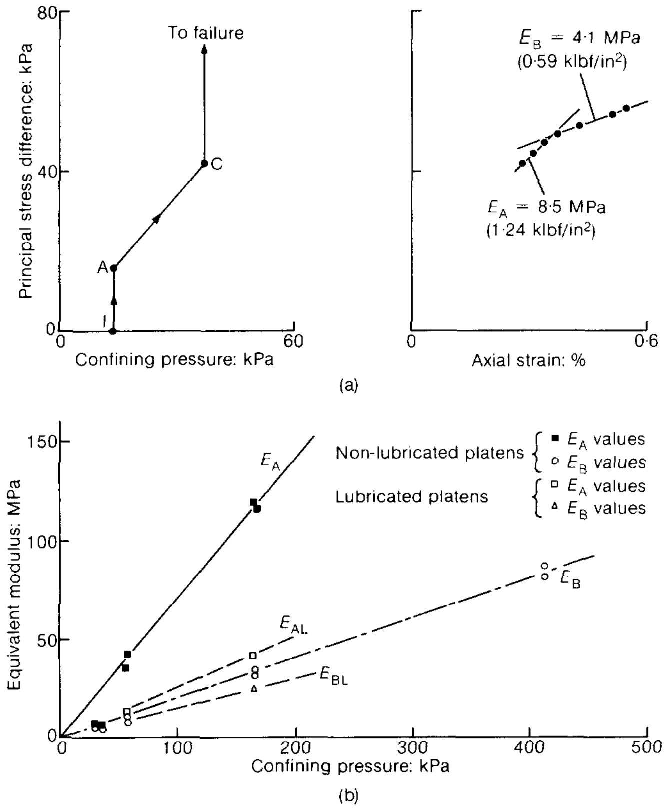


**Fig. 7. Results from Lambrechts & Leonards (1978): (a) stress path and strains in test AC-1; (b) equivalent modulus versus confining pressure for normally consolidated samples**

COMPARISON OF SOIL MODULI

The Authors agree with Leonards and his co-workers that soil modulus depends on many factors. As a generalization, however, most engineers would expect the compressibility of a sand to be less than that of a firm clay, which might well have a drained compressibility similar to the 8·3 MN/m$^2$ value given by Lambrechts & Leonards. Part of Figs 4 and 5 from Lambrechts & Leonards (1978), redrawn here as Fig. 7, shows the typical principal stress difference versus strain response observed by them and the modulus values plotted as a function of minor principal stress $\sigma_3'$. $E_A$ and $E_B$ represent the equivalent moduli of the first and second linear portions of their stress–strain curves, for the stress path from C towards failure.

The subscript L (as in $E_{AL}$) denotes the use of lubricated end platens. It can be seen (Fig. 7(b)) that the values of $E_A$ for the two specimens tested at $\sigma_3' = 37$ kN/m$^2$ (5·3 lbf/in$^2$) fall well below that straight line for $E_A$ suggested by the four specimens tested at 59 kN/m$^2$ and 165 kN/m$^2$. Not surprisingly, considering the date of publication, Lambrechts and Leonards were not concerned with this behaviour as they felt that the lubricated end platen results were correct and suggested that the higher $E_A$ values were incorrect, resulting from the 'stiffening and restraining effect of rough platens'. Following the work of Daramola (1978) and Sarsby, Kalteziotis & Haddad (1982), it is now understood that the effects of bedding at the platens more than compensate for any stiffening due to platen friction, and that lubricated end platens can reduce measured modulus values by many times. The only satisfactory technique is to measure strains on the middle third of the height of the specimen.

Figure 8 shows the results of a test on a specimen 102 mm in diameter by 204 mm high of the Leighton Buzzard sand used by the Authors, following the AC stress path used by Lambrechts and Leonards. The void ratio of the specimen was 0·73, with a relative density of 63%, so that the results might be expected to be comparable with, or to exhibit more compressibility than, those in Fig. 7(a). Two curves are shown. That with the greater strains results from conventional measure-

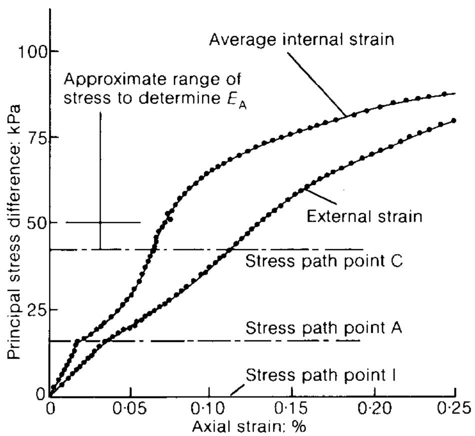


**Fig. 8. Test on Leighton Buzzard sand to Lambrechts and Leonards' stress path AC (Khatrush, 1986)**

ment of the displacement of the cell ram relative to the top of the cell. A rigid internal load cell was used, with a combined non-linearity and hysteresis of less than about 0·1% and a displacement at its maximum 5000 N load of only 0·1 mm. The contribution of load cell compressibility to the externally measured strains would be of the order of 0·004% at a principal stress difference of 50 $kN/m^2$ for a specimen of these dimensions.

The curve with the lower level of strains is the average of two measurements of strain made inside the triaxial cell at diametrically opposed positions on the middle third of the height of the specimen, using a displacement measuring device functioning on the Hall effect principle, developed at the University of Surrey (Khatrush, 1986).

The modulus determined externally for the same increment of principal stress difference as used by Lambrechts and Leonards for $E_A$ is 50 $MN/m^2$ (7·3 $klbf/in^2$). That determined from internal strain measurement is 115 $MN/m^2$ (16·7 $klbf/in^2$). As shown in Fig. 7(a), the strain increment over which $E_A$ was measured by Lambrechts and Leonards was about 0·07%. For this strain increment the results in Fig. 8 give a modulus value of 37 $MN/m^2$ for external strains and 44 $MN/m^2$ for internal strains. The influence of change in stress path direction on specimen stiffness is much more apparent when internal strain measurement is used.

The values of the modulus determined above take into account the factors of density, stress path, stress increment and strain increment discussed by Leonards and his co-workers and yet continue to be in line with the values reported by the Authors in their Paper. They do not take into account the differences in test apparatus and technique between the two sets of tests. Either very large differences in specimen stiffness may be generated by these differences, or the apparent similarity of the two sands is deceptive.

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