Discussion of "Low Strain Integrity Testing of Piles: Three-Dimensional Effects" by Y. K. Chow, K. K. Phoon, W. F. Chow, and K. Y. Young

November 2003, Vol. 129, No. 11, pp. 1057–1062
DOI: 10.1061/(ASCE)1090-0241 (2003)129:11(1057)

Joram M. Amir[1]
[1]Chairman, Piletest.com Ltd., 85 Hanassi St., Herzlia 46399, Israel. E-mail: jmamir@piletest.com

The authors have pointed to a subject that has intrigued many newcomers to pile integrity testing. In their own words, "The initial velocity response close to the impact area… goes negative after the first peak." This phenomenon is well known to practitioners, and as Fig. 1 here shows, is evident in most test results presented in time domain (reflectograms). The authors attribute this feature to three-dimensional effects close to the pile head, and offer a revised testing procedure to overcome it.

In the discusser's opinion, there is an alternative explanation to this phenomenon. While the one-dimensional wave equation is used in both pile driving and integrity testing applications, there is a major quantitative difference between the two: The purpose of pile driving is to produce a permanent set, thus the ratio between the mass of the hammer and that of the driven pile is relatively large, on the order of 1:10 to 1:100. In pile integrity testing, on the other hand, the corresponding ratio is between $1:10^5$ and $1:10^6$. Clearly, a handheld hammer blow can create no set in the pile, and any displacement produced is recovered elastically. Since displacement is the integral of velocity with respect to time, the negative and positive areas between the reflectogram and the time axis have to balance. This means that the downward velocity must be followed by a corresponding upward velocity. This result has nothing to do with reflections, as can be shown by finite-element analysis of a blow on a semi-infinite space, where no reflections are present.

The discusser would also like to differ with the authors' recommendation to improve the test results by holding the sensor at a distance of not less than half a pile radius from the hammer. This suggestion, which is incompatible with ASTM (2000), is rather problematic for the following reasoning: It is a well-known fact that in addition to body (*P* and *S*) waves that travel in all directions, the hammer blow also produces Rayleigh waves that travel horizontally on the pile's top surface. The Rayleigh waves carry two-thirds of the wave energy (Woods 1968) and are slower than either *P* or *S* waves. In concrete with a Poisson's ratio of 0.25, for example, Rayleigh waves propagate at a velocity that is almost 50% slower than that of the *P* body waves (Graff 1975). If the sensor is placed at a distance from the point of impact, it will not be triggered by the relatively weak *P*- or *S*-waves, but rather by the much stronger (and slower) Rayleigh wave.

Fig. 2 here shows the result of finite-element analysis for a sensor placed at a distance of 0.5 m from the point of impact. Clearly, the exact timing of triggering is strongly operator-dependent: If the trigger level is set at a velocity 5×10^{-4} m/s, the system will start logging 0.25 msec after the start of the hammer blow. If, on the other hand, the trigger level is set at 1×10^{-3} m/s, the system will start logging with a delay of 1.22 msec. At this point of time, the longitudinal wave front will already be 5 m below the pile's head, and the measured pile length will be 2.5 m shorter than the as-made figure. A situation in which the reported length of a pile (and hence acceptability) depends on the location of the sensor and on the trigger level as set by the operator, is intolerable. It can turn into a source of aggravation on-site, where the piling contractor is too often suspect of producing underlength piles.

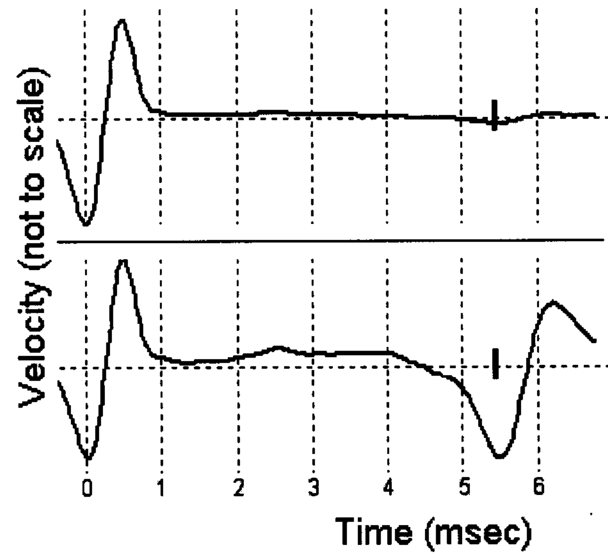

Fig. 1. Natural reflectogram (top) and amplified one (bottom)

Moreover, the fact that the sensor may be triggered by motion that can point either down or up adds another factor of uncertainty and may further complicate the analysis of the reflectogram.

To conclude, the discusser would like to offer two suggestions of general character: First, since force, displacement, and velocity are all vectors, plotting them in the right direction helps visualize wave-related phenomena. This means that downward motion

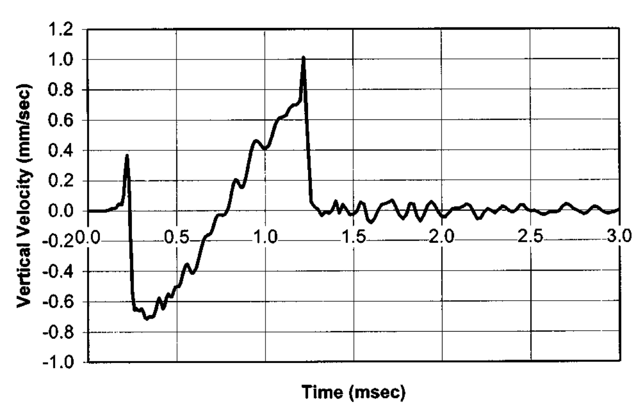

Fig. 2. Vertical velocity versus time at distance of 0.5 m from impact

J. Geotech. Geoenviron. Eng., 2005, 131(2): 269-270

must be plotted as negative ordinate, and vice versa. The presentation offered by the authors, in which downward velocity is plotted above the horizontal axis, is sometimes difficult to follow. Secondly, there are presently two distinct stress wave methods for pile integrity testing—sonic or low frequency (ASTM 2000) and ultrasonic or high frequency (ASTM 2002). To remove any ambiguity, it would be preferable to use the term "sonic testing" instead of "low strain integrity testing."

References

ASTM. (2000). "Standard test method for low strain integrity testing of piles." *Designation D 5882-00*, West Conshohocken, Pa.

ASTM. (2002). "Standard test method for integrity testing of concrete<> deep foundations by ultrasonic crosshole testing." *Designation D 6760-02*, West Conshohocken, Pa.

Graff, K. G. (1975). *Wave motion in elastic solids*, Dover, New York, p. 396.

Woods, R. D. (1968). "Screening of surface waves in soils." *J. Soil Mech. Found. Div.*, 94(4), 957–979.

Closure to "Low Strain Integrity Testing of Piles: Three-Dimensional Effects" by Y. K. Chow, K. K. Phoon, W. F. Chow, and K. Y. Wong

November 2003, Vol. 129, No. 11, pp. 1057–1062.
DOI: 10.1061/(ASCE)1090-0241 (2003)129:11(1057)

Y. K. Chow[1]; K. K. Phoon[2]; W. F. Chow[3]; and K. Y. Wong[4]

[1]Professor, Dept. of Civil Engineering, National Univ. of Singapore, Singapore 117576.

[2]Associate Professor, Dept. of Civil Engineering, National Univ. of Singapore, Singapore 117576. E-mail: cvepkk@nus.edu.sg

[3]Engineer, Maunsell Consultants (Singapore) Pte Ltd., 300 Beach Rd., #03-00, Singapore 199555.

[4]Technical Director, RPM Engineers Sdn Bhd, 34A Jalan SS21/39, Damansara Utama 47400, P.J., Malaysia.

The writers thank the discusser for his interest in the paper. The phenomenon in which the initial velocity response goes negative after the first peak observed in low-strain integrity tests on some piles is indeed well known to many practitioners, but the reason behind it is not known. The discusser has attempted to give an alternative explanation (using his convention of plotting the downward velocity as negative ordinate) and says that "…. the downward velocity must be followed by a corresponding upward velocity." It is well known that this phenomenon is not observed all the time in integrity tests and, as shown in the paper, is manifested in piles that exceed a certain size. The shape of the velocity response curve is dependent on the size of the pile and the distance of the sensor from the impact area; indeed it has nothing to do with reflections. In the case of a large-diameter pile, as the sensor distance is further away from the impact area, it approaches that given by the 1D stress wave theory.

The writers have recommended placing the sensor at a distance of about half a pile radius from the impact area so as to ascertain whether the observed phenomenon is indeed due to the 3D effect or if it is due to an anomaly in the pile. Ignoring it altogether, which is common practice, may not be prudent because an anomaly in the pile can also produce a similar effect, as explained in the paper. Having an impact at the center and placing a sensor at a distance of 0.5 m at half pile radius would imply a pile diameter of 2 m, which is not that common. Without the details of the finite-element analysis reported by the discusser, it is difficult to comment on his results. In any case, the recommended sensor distance is to help identify whether the observed velocity response is due to the 3D effect or the presence of an anomaly so that remedial work (if required) can be done. This recommendation is made in connection with the assessment of the integrity of the pile. If indeed the objective is to estimate the pile length, the location of the sensor may have an impact. This was discussed by Chow (1999), but the lack of space has prevented its inclusion in the paper. Having ascertained the integrity of the pile, the location of the sensor can always be brought closer to the impact area. Estimating the pile length from the velocity response in the integrity test is, however, not without problems. It is dependent on the assumed wave speed in the pile and, more importantly, on the ability to accurately identify the reflection from the pile toe, which is not always easy.

The discusser made two suggestions of a general character. The first is with respect to plotting the downward motion as negative ordinate. The commercial systems available for integrity testing present the results in both ways. As long as one understands what one is doing, it does not really matter. On the second, with respect to the term used to describe the test, it is a matter of preference.

Reference

Chow, W. F. (1999). "Three-dimensional effects in integrity testing of piles." MEng thesis, National Univ. of Singapore, Singapore.

Discussion of "Strain-Rate Effect on Soil Secant Shear Modulus at Small Cyclic Strains" by Leo Matesic and Mladen Vucetic

June 2003, Vol. 129, No. 6, pp. 536-549.
DOI: 10.1061/(ASCE)1090-0241(2003)129:6(536)

Charles M. O. Nwaiwu[1] and Kolawole J. Osinubi[2]

[1]Lect. I, Dept. of Civil and Water Resource Engineering, Univ. of Maiduguri, 600001, Nigeria.

[2]Associate Professor, Dept. of Civil Engineering, Ahmadu Bello Univ., Zaria 810001, Nigeria. E-mail: kosinubi@yahoo.com

Based on an extensive laboratory study involving six soils (three clays and three sands), the authors have shown that the secant shear modulus, G_s, of soils increase with shear strain rate, γ at small cyclic shear strain amplitudes γ_c under simple shear loading conditions. They have concluded that the G_s versus log γ data plot approximately along a straight line, the slope of which gives α_G, the strain-rate shear modulus parameter and the associated normalized value, $\alpha_G/G_{s(\text{low})}=N_{\gamma \cdot G}$, the shear strain-rate modulus factor.

J. Geotech. Geoenviron. Eng., 2005, 131(2): 269-270

The authors found no trend of either αG or $N\gamma \cdot G$ with the plasticity index (PI) or liquid limit (LL) of the clays despite clear differences between the effects of γ on sands and clays. This lack of trend should be expected because (1) the secant shear modulus Gs(low) from which α_G and $N\gamma \cdot G$ values were derived depends much on both void ratio (e) and vertical consolidation stress (σ_{vc}) as well as the overconsolidation ratio, OCR (Hardin and Black 1968, 1969; Lambe and Whitman 1979; Lanzo et al. 1997), and (2) the authors used soil samples that had different values of void ratio and vertical consolidation stress. Furthermore, disturbed and undisturbed clay samples with varying overconsolidation ratios were tested.

The authors conducted shear tests in a hand-controlled DSDSS device in order to avoid the effects of the vibrations caused by hydraulic or electrical motors, which cannot be tolerated in small strain testing. The discussers are of the opinion that hand operation of the DSDSS device will always yield fairly subjective results as accuracy will depend on the skill of the operator. The associated random testing errors will definitely cause scatter in data, thus leading to uncertainties in measured parameters and hence in geotechnical design.

References

Hardin, B. O., and Black, W. L. (1968). "Vibration modulus of normally consolidated clay." *J. Soil Mech. Found. Div.*, 94(2), 353–369.

Hardin, B. O., and Black, W. L. (1969). "Closure to 'Vibration modulus of normally consolidated clay.'" *J. Soil Mech. Found. Div.*, 95(6), 1531–1537.

Lambe, T. W., and Whitman, R. V. (1979). *Soil mechanics, SI version*, Wiley, New York.

Lanzo, G., Vucetic, M., and Doroudian, M. (1997). "Reduction of shear modulus at small strains in simple shear." *J. Geotech. Geoenviron. Eng.*, 123(11), 1035–1042.

Closure to "Strain-Rate Effects on Soil Secant Shear Modulus at Small Cyclic Strains" by Leo Matesic and Mladen Vucetic

June 2003, Vol. 129, No. 6, pp. 536–549.
DOI: 10.1061/(ASCE)1090-0241(2003)129:6(536)

Leo Matesic[1] and Mladen Vucetic[2]

[1]Geokon Zagreb, Ltd., Starotnjanska 16a, 10000 Zagreb, Croatia.

[2]Professor, Civil and Environmental Engineering Dept., Univ. of California, Los Angeles, CA 90095.

The interest of the discussers in our paper is greatly appreciated. The discussers expressed concerns about two issues that have been fully resolved by now. Below are the answers to the discussers' concerns.

1. The discussers concluded that we found no trend between the plasticity index of the soil PI and liquid limit LL, and the strain-rate shear modulus parameter α_G and shear strain-rate modulus factor $N_{\dot{\gamma}-G}$, because in our tests we used different void ratios, vertical consolidation stresses, and overconsolidation ratios, while the secant shear modulus, G_s, depends on these factors. A subsequent independent investigation showed that $N_{\dot{\gamma}-G}$ can be correlated to PI and LL. The results of that investigation are presented in Vucetic and Tabata (2003). It was possible to establish the correlations between $N_{\dot{\gamma}-G}$ and PI and LL because, to obtain $N_{\dot{\gamma}-G}$, the parameter α_G was normalized by G_s corresponding to the exactly same levels of the cyclic shear strain amplitudes, γ_c, and the same level of the shear strain rate, $\dot{\gamma}$. Such a precise normalization was not performed on the data presented in our paper, and thus the correlations between $N_{\dot{\gamma}-G}$ and PI and LL, which generally are not very strong, could not be clearly established. However, when the same procedure was applied to the data presented in our paper, the correlations between $N_{\dot{\gamma}-G}$ and PI and LL, though not very strong, have been established. These correlations are presented in Vucetic et al. (2003).

   Otherwise, the fact that G_s depends strongly on the void ratio, vertical consolidation stress, and overconsolidation ratio does not necessarily mean that $N_{\dot{\gamma}-G}$ also depends strongly on these factors. Although parameter α_G does depend on these factors, its dependence is similar to that of G_s. Consequently, when α_G is divided by G_s to obtain $N_{\dot{\gamma}-G}$, the effects of these factors on $N_{\dot{\gamma}-G}$ for the most part disappear. The analogous correlation is the one between $G_s/G_{\max}$ and PI (Vucetic and Dobry 1991). Both G_s and G_{max} are strongly affected by the void ratio, vertical consolidation stress, and overconsolidation ratio, but in a similar manner. As a consequence, G_s/G_{max} is relatively little affected by these factors.
2. The discussers are of the opinion that the hand operation of the dual-specimen direct simple shear (DSDSS) device will always yield fairly subjective results, since accuracy will depend on the skill of the operator. A relatively large number of papers containing the small-strain behavior data obtained in the DSDSS device has been published to date (Doroudian and Vucetic 1995, 1998; Lanzo et al. 1997; Vucetic et al. 1998b, 1988c; Vucetic and Tabata 2003). Among other data and charts, these papers also contain typical examples of the stress-time and strain-time histories and corresponding stress-strain curves, i.e., stress-strain loops. These time histories and curves reflect the true soil behavior regardless of the type of the load application, for they were obtained with the help of the load and displacement transducers that are attached practically directly to the soil specimen. If the measurements with these transducers are correct—and they certainly are—the above curves are also correct. Accordingly, all other cyclic soil properties and parameters that are derived from these curves, such as shear moduli and damping, are also correct. Furthermore, detailed results of more than 100 DSDSS tests are presented in a series of publicly available, comprehensive technical reports (e.g., Hsu and Vucetic 2002; Tabata and Vucetic 2002; Vucetic et al. 1998a, 1999). Such a large volume of comprehensive data accumulated over the last 10 years provides direct and compelling evidence about the high accuracy of the DSDSS testing and other attractive capabilities of the DSDSS device.

References

Doroudian, M., and Vucetic, M. (1995). "A direct simple shear device for measuring small-strain behvior." *Geotech. Test. J.*, 18(1), 69–85.

Doroudian, M., and Vucetic, M., (1998). "Small-strain testing in an NGI-type direct simple shear device." *Proc., 11th Danube-European Conf. on Soil Mechanics and Geotechnical Engineering*, Balkema, Rotter-

dam, The Netherlands, 687–693.
Hsu, C.-C., and Vucetic, M., (2002). "Dynamic and cyclic behavior of soils over a wide range of shear strains in NGI-type simple shear testing device." *UCLA Research Rep. No. ENG-02-228*, Civil and Environmental Engineering Dept., Univ. of California, Los Angeles, January.
Lanzo, G., Vucetic, M., and Doroudian, M. (1997). "Reduction of shear modulus at small strains in simple shear." *J. Geotech. Geoenviron. Eng.*, 123(11), 1035–1042.
Tabata, K., and Vucetic, M. (2002). "Results of cyclic simple shear tests on fifteen soils conducted for PEARL project and other research purposes." *UCLA Research Rep. No. ENG-02-233*, Civil and Environmental Engineering Dept., Univ. of California, Los Angeles, November.
Vucetic, M., and Dobry, R. (1991). "Effect of soil plasticity on cyclic response." *J. Geotech. Eng.* 117(1), 89–107.
Vucetic, M., and Tabata, K. (2003). "Influence of soil type on the effect of strain rate on small-strain cyclic shear modulus." *Soils Found.*, 43(5), 161–173.
Vucetic, M., Hsu, C.-C., and Doroudian, M. (1998a). "Results of cyclic and dynamic simple shear tests on soils from La Cienega site conducted for ROSRINE project and other research purposes." *UCLA Research Rep. No. ENG-98-200*, Civil and Environmental Engineering Dept., Univ. of California, Los Angeles, December.
Vucetic, M., Lanzo, G., and Doroudian M. (1998b). "Damping at small strains in cyclic simple shear test." *J. Geotech. Geoenviron. Eng.*, 124(7), 585–594.
Vucetic, M., Lanzo, G., and Doroudian, M. (1998c). "Effect of the shape of cyclic loading on damping ratio at small strains." *Soils Found.*, 38(1), 111–120.
Vucetic, M., Doroudian, M., and Matesic, L. (1999). "Results of geotechnical laboratory tests on soil samples from the UC San Diego Campus." *UCLA Research Rep. No. ENG-99-204*, Civil and Environmental Engineering Dept., Univ. of California, Los Angeles, May.
Vucetic, M., Tabata, K., and Matesic, L. (2003). "Effect of average straining rate on shear modulus at small cyclic strains." *Proc., Int. Conf. on Deformation Characteristics of Geomaterials*, Di Benedetto et al., eds., Balkema, Lisse, The Netherlands, pp. 321–328.

J. Geotech. Geoenviron. Eng., 2005, 131(2): 269-270