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DISCUSSION

Back analysis of offshore pile driving with an improved soil model

B. R. DANZIGER, A. M. COSTA, F. R. LOPES and M. P. PACHECO (1999).
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A. E. Holeyman, *Université Catholique de Louvain*

The discusser commends the authors for their re-analysis of field data: it demonstrates that fine-tuning efforts of pile–soil dynamic interaction models still provide for interesting developments. The discusser fully supports the authors' quest for a more physical modelling of the dynamic interaction between the driven pile and the surrounding soil. In that regard, the sophistication of models should be tolerated only inasmuch as they lead to a more truthful representation of the physical reality, accompanied by a deeper insight into the mechanisms at play, and in particular into 'damping'.

However, the discusser questions that rate-dependent effects be pegged into only two categories, namely radiation damping and viscous damping. Although Simons' (1985) suggestion to lump those two effects into two distinct components helps one to differentiate between near-field and far-field effects, that model cannot properly account for: (*a*) hysteretic damping (energy loss during *unloading*), (*b*) intrinsic viscous damping at *low strains*, and (*c*) non-linear rate dependence of the *ultimate* soil strength.

A more comprehensive model was developed earlier by the discusser for both friction and end bearing (Holeyman, 1984; also described in Holeyman, 1985, and Holeyman, 1988). The one-dimensional radial model to represent skin friction and its simplified lumped parameter system, along with their mobilisation curves, are shown in Fig. 24. The simplified system was suggested after checking its equivalence against numerous simulations of the more complete model under several kinetic input signatures, typical of pile driving (see e.g. Fig. 24(c) and 24(d)).

The model shown in Fig. 24(a) consists of a succession of concentric cylinders with a linearly increasing depth. The equations of movement are integrated for each cylinder based on their dynamic shear equilibrium in the vertical direction, in a manner similar to that used by Smith (1960) in the longitudinal direction. The model allows the shear strain/shear stress

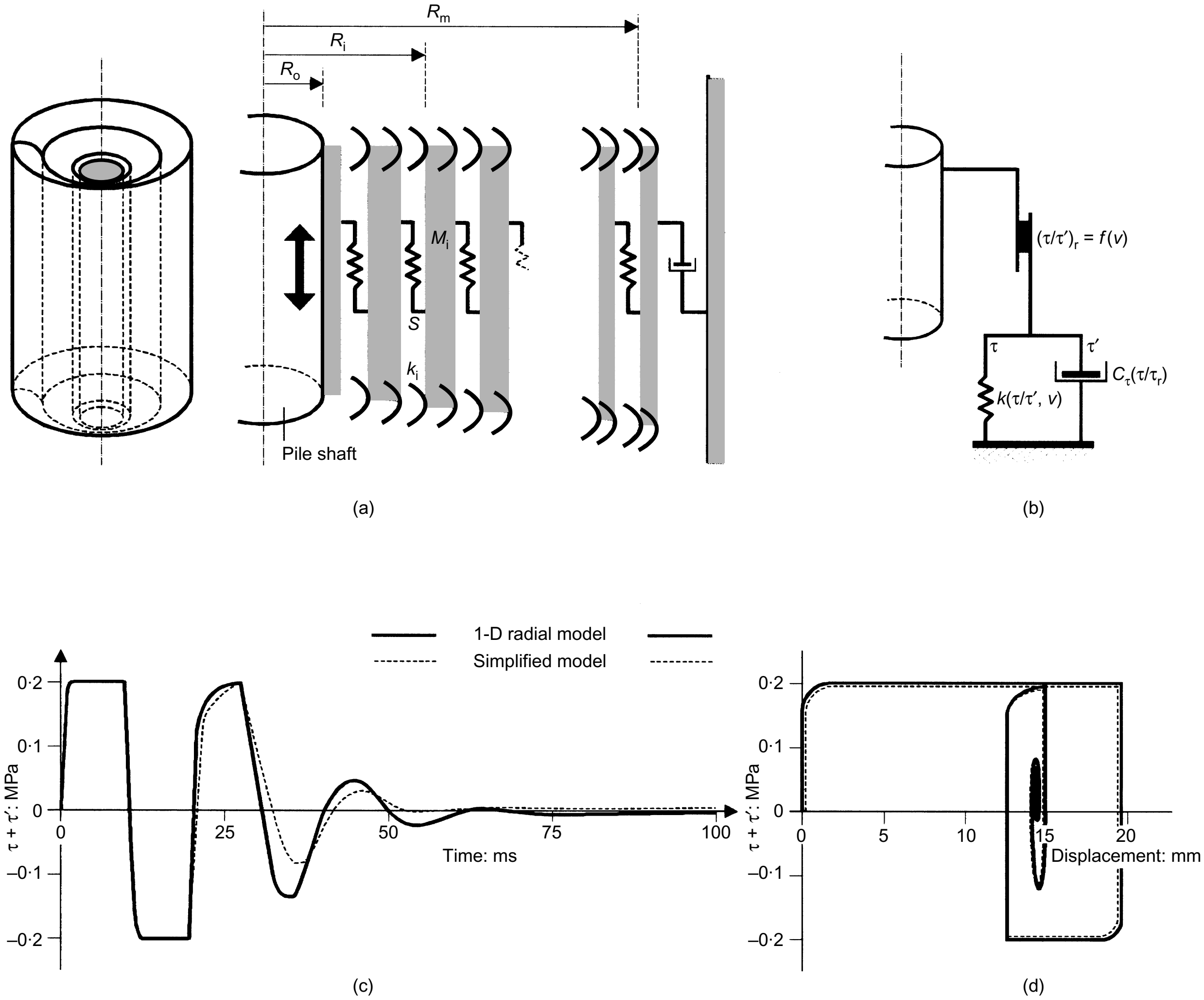

Fig. 24. Pile friction models: (a) one-dimensional radial model; (b) simplified model; (c) time response; (d) shear stress—displacement mobilisation

constitutive relationships already developed by soil mechanics researchers to be readily deployed. The major advantage of that shear wave propagation model is to follow the onset of soil softening as shear develops. It can also provide insight into vibration levels in the vicinity of the pile.

The more complete one-dimensional model allows the discusser also to identify two regions around the pile shaft: a very localised zone within which very high shear strains and slippage concentrate, and a more continuous medium zone within which lower strains propagate. That observation was used to cast the simplified system shown in Fig. 24(b). That model, in spite of adding no complication to the model later suggested by Simons (1985), then made it possible to take into account: (*a*) soil softening by updating the shear modulus, *G*, as a function of the shaft displacement, thereby *reducing* radiation damping as the transient displacement progresses; (*b*) energy losses through appropriate *unloading* paths, playing a role during pile rebound; and (*c*) localised velocity dependence of the *ultimate* shear strength.

Studies of the more realistic one-dimensional radial model showed that geometric damping consistently drives the soil to fail well before the shaft reaches displacements that would trigger its static failure. As noted by Holeyman (1992), shear failure takes place almost instantaneously along the shaft, overshadowing friction static stiffness. It is therefore not surprising that match quality of the inverse modelling becomes insensitive to skin friction static stiffness (k_S). Values of friction static stiffness derived from dynamic inverse modelling of events involving shear failure along the shaft have therefore to be evaluated with extreme caution.

Conversely, at the pile toe, geometric damping has a limiting role in mobilising resistance, relative to static stiffness. In addition, the maximum enforced displacement at the toe during a blow is generally quite small compared with 10–25% of the pile base diameter, a range commonly accepted to produce failure at the toe. A direct consequence of that limitation is that, in most cases, inverse modelling will provide at best an estimate of the toe static stiffness, and its associated radiation damping (consistent with the *G* value). Relying on some estimate of the ultimate end bearing is hazardous, especially at refusal (where the authors insist on the soil deformation remaining nearly elastic).

It is the discusser's opinion that if soil parameters have to be assessed based on pile driving inverse modelling, they can be ranked, in decreasing order of degree of reliability as follows: (*a*) ultimate skin friction, and (*b*) base stiffness. In the case of easy driving, one may add (*c*) ultimate end bearing. If low strain blows are modelled, it then becomes possible to estimate (*d*) skin friction radiation damping, but then losing hope of obtaining (*a*). *G* values derived from the estimated skin friction stiffness at large velocity are questionable, and in that regard the discusser agrees with the authors that it is preferable to adopt a consistent set of k_S, k_b, C_S based on a single choice of *G*.

The discusser suggests that it would be more appropriate to state that the improved model leads to increased toe stiffness, in lieu of increased ultimate end bearing. He would also like to raise the following questions:

Do the skin friction elements concentrated at the conical toe allow for uplift? (Comparison of Fig. 9(c) with Fig. 9(b) of the paper implies that allowance, as total resistance is incrementally less than the static resistance.) If so, how does their behaviour compare with that of standard toe resistance models that allow for at most cavitation?

Different distributions of the skin friction of the same pile (A2) at different depths appear assigned relative to the pile toe, rather than to depths. If one tries to derive soil parameters, would it not be more suitable that soil properties be assigned independently from the penetration of the pile?

G values derived at the conical point are vastly superior to those just above the point, leading to the consideration that a geometrical effect is at play more than a contrast in soil properties. What is the correlation between an equivalent *G* value reflecting the stiffness of a circular base ($k_b = 4GR/(1-\nu)$) and the average *G* value necessary to model a stepped stack of cylinders acting only in shear?

Back-figured soil parameters are useful inasmuch as they can be related to soil investigation data usually available before a drivability analysis can be conducted. Was any attempt made to relate τ_S and *G* from pertinent or statistical CPT or other in-situ test?

Can the profile of f_S (unit skin friction measured on local CPT sleeve) or the cumulated skin friction be provided? Why was the skin friction distribution modelled without following the CPT profile shown in Fig. 2 (real or idealised)?

Authors' reply

The discussion by Holeyman has been focused mainly on his model developed earlier for both friction and end bearing (Holeyman, 1984; also described in Holeyman, 1985, and Holeyman, 1988). The authors feel that Holeyman's model has contributed to a better understanding of pile behaviour during driving. However, it is not yet suited to back-analysis applications since numerous parameters are required to describe pile behaviour, and this can be a problem for real-time professional developments. It is important to point out that the model adopted by the authors aimed at a better interpretation of the instrumented piles regarding the unique conditions prevailing at the Campos Basin, such as high-capacity closed-end pipe piles driven offshore through dense to very dense calcareous sands. The model proved its practical usefulness under such peculiar conditions through the back-analysis of many offshore records.

The authors agree with the discusser that fine-tuning efforts to model soil dynamic interaction still provide for interesting developments. The discusser, however, pointed out that a more comprehensive model should not only include radiation and viscous damping, but should also incorporate hysteretic damping, intrinsic viscous damping at low strains, and non-linear rate dependence of the ultimate soil strength. The authors also agree that those aspects are likely to influence pile-driving behaviour to some extent. The authors point out, however, that radiation damping seems to prevail in the low-strain range, whereas viscous damping turns out to be an important contribution to pile driving resistance in the medium-strain range. In addition, non-linearity prior to failure, according to Randolph (2000), is probably of secondary importance to inertial and viscous effects for dynamic applications. Therefore, the authors feel that the inclusion of so many variables in a back-analysis will require a much longer interpretation, and will impair practical applications.

The discusser points out the ability of his model to cope with (*a*) soil softening by updating shear modulus *G* as a function of the shaft displacement, reducing radiation damping as the transient displacement progresses, (*b*) energy losses through appropriate unloading paths, playing a role during pile rebound, and (*c*) localised velocity dependence of the ultimate shear strength. The authors emphasised in the paper that hard driving conditions prevailed in the back-analyses, so they feel that aspects such as (*a*) and (*c*) above would not lead to a more truthful representation of the closed-end pipe piles. The energy loss during unloading, referred to above as aspect (*b*), was also taken into account by the authors. Fig. 9(a) of the paper shows the static and total resistance for pile A1 at 43 m penetration, indicating the energy loss during pile unloading and the ability of Simons' (1985) model to represent the reduction in radiation damping as the transient displacement progresses. Appropriate unloading paths during pile rebound could also be incorporated into Simons' model. However, this inclusion would not be consistent with the theory of dynamic elasticity. Instead, a much more dominant aspect in the interpretation of piles driven through very dense calcareous sands is the ability of Simons' model to cope with the residual stresses locked at the pile toe. The capability of the discusser's model in that respect is not clear, unfortunately. Also relevant in the analyses carried out by the authors was the proper modelling of the steel conical point.

The driving records were sensitive to a great extent to the sharp variation in pile impedance.

Another aspect pointed out by the discusser is the fact that studies of his one-dimensional radial model showed that geometric damping consistently drives the soil to fail well before the shaft reaches the displacements that would trigger its static failure. The authors point out, however, that radiation damping prevails only at the earlier stages of shaft loading, when the mobilised soil resistance is still lower than τ_S. When loading proceeds, the static soil resistance will increase up to τ_S as particle velocity and radiation damping decrease. Therefore radiation damping overshadows the friction static stiffness only at the earlier stages of loading. At the end of loading, just before unloading, static friction plays a major role as the static soil resistance reaches τ_S. The match quality is therefore sensitive to skin friction static stiffness, k_S.

The discusser mentions that the maximum enforced displacement at the pile toe during a blow is generally quite small compared with 10–25% of pile base diameter, a range commonly accepted to produce failure at the toe. The authors are entirely aware of this important point, and emphasise that the dynamic soil–pile interaction carried out at the Campos Basin showed stiffness coefficients much greater under dynamic conditions, which in turn have led to much lower quakes, in agreement with Randolph (1991) and Randolph & Deeks (1992). It was also stressed in the paper, especially in Table 6, that the soil resistances obtained in back-analyses were related to mobilised resistances during continuous driving, rather than values at failure. In the session related to bearing capacity the authors also pointed out that the end-bearing stress, q_p, actually mobilised during pile monitoring may be lower than the end-bearing capacity, mainly in hard driving conditions when small penetrations occur. In fact, redriving some piles with a heavier hammer indicated a higher toe resistance than that back-analysed at the end of driving (Danziger *et al.*, 1992).

For the hard driving conditions prevailing at the Campos Basin, the authors found it possible to estimate the mobilised resistance, the soil stiffness and radiation damping, as these parameters proved to be very helpful to pile drivability control in the layered calcareous sands. Therefore, upon careful interpretation of the driving records, it is possible to estimate

Table 7. Soil shear modulus as in Table 3 of the paper and equivalent shear modulus, $G_{equivalent}$

Record	$G_{equivalent}$: MN/m^2	G: MN/m^2	$G_{equivalent}/G$
A14300	511·5	603·5	0·85
A24325	760·5	896·6	0·85
A24425	936·4	1069·0	0·87
B24200	933·2	1103·5	0·85

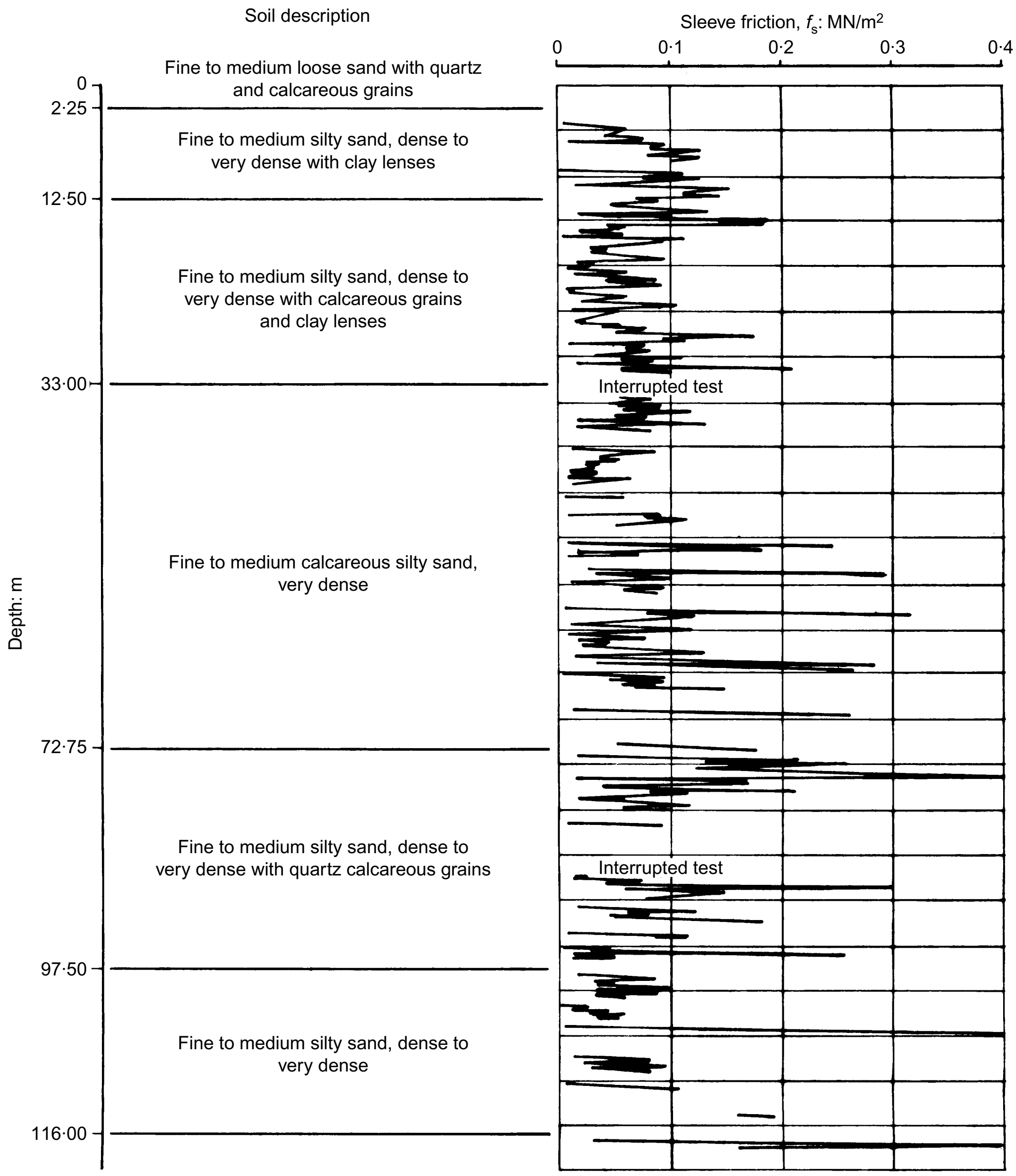

Fig. 25. Sleeve friction at Pargo 1A site (McClelland, 1985)

radiation damping at the pile shaft without losing track of reasonable estimates of ultimate skin friction, as each one plays its major role in distinct phases of the driving process.

The authors agree that G values derived from the estimated skin friction stiffness at large velocity are questionable, but they recall that large velocity conditions prevail mainly in easy driving operations, which was not the case described in the paper. Randolph & Deeks (1992) indicated the range 400–1000 as typical of shear modulus to limiting shaft friction ratios. However, this range is not applicable to calcareous sands, where a much smaller skin friction is developed, consistently with the back-analysed raio of 175 reported in the paper. Accordingly, Beringen *et al*. (1982) and Noorany (1985) observed that reduced lateral stresses in piles driven in calcareous soils are the main cause of reduced skin friction.

The discusser asks whether the skin friction elements concentrated at the conical toe allow for uplift. They really do, as illustrated in Fig. 9 of the paper, where residual stresses can be assessed upon complete unloading. A comparison with toe resistance models that allow for cavitation, in the authors' view, would not provide more realistic results in view of more relevant aspects such as residual stresses and sharp variation of pile impedance at the steel conical point.

The discusser observes that different distributions of skin friction relative to pile toe were assigned at different depths. The discusser ought to observe skin friction distribution related to depths, not to the pile toe. The authors would like to recall the nature of back-analysis, similar to CAPWAP, comprising an iterative procedure. In the present analyses the simulated velocity is calculated and compared with the measured velocity until a fine-tuned adjustment is attained, leading to a much more realistic distribution of skin friction related to the pile toe. In most programs the skin friction and toe resistance are represented as a percentage of the total bearing capacity. At the end of the matching procedure, a toe resistance and skin friction distribution related to a percentage of the total bearing capacity is obtained.

The discusser comments on the fact that G values obtained at the conical point are vastly superior to those just above the point, leading to the consideration that a geometrical effect is at play more than a contrast in soil properties. The discusser suggests that the authors obtain the correlation between an equivalent G value reflecting the stiffness of a circular base ($K_b = 4GR/(1-\nu)$) and the average G value as back-calculated by the authors. Accordingly, K_b was estimated as the ratio between the end-bearing resistance and the maximum soil displacement from Table 1 of the paper, whereas the equivalent soil shear modulus was calculated as

$$G_{\text{equivalent}} = \frac{K_b(1-\nu)}{4R}$$

The authors assumed $\nu = 0{\cdot}3$ and $R = 0{\cdot}835$ m. The comparison between $G_{\text{equivalent}}$ and soil shear modulus G obtained in Table 3 of the paper, and the ratio between both, are indicated in Table 7. It is seen that G values derived at the conical point are of the same order as that derived from a circular-shaped base with the same diameter as the shaft. The ratio $G_{\text{equivalent}}/G$, however, may reflect some geometrical effect.

The authors did not attempt to relate τ_S and G from in-situ tests as a database of G values is not yet available for calcareous sands at the Northeast Pole of the Campos Basin in Brazil.

Figure 25 illustrates the sleeve friction measured in the only CPT nearby. As expected, the skin friction back-analysed does not follow the same trend as the sleeve friction in the CPT. It was emphasised in the paper that the back-calculated values of shaft friction were very low, and different from CPT results. This was attributed to the reduced resistance offered by calcareous sands during continuous driving. Even in CPT there is a considerable decrease in the lateral resistance when the friction resistance is measured along the whole length of the rod. As mentioned in the paper, this was the reason why Begemann (1963) proposed measuring the lateral friction close to the cone with the so-called friction jacket cone.

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