

DISCUSSION

Design of piled bridge abutments on soft clay for loading from lateral soil movements

D. P. STEWART, R. J. JEWELL, M. F. RANDOLPH (1994). *Géotechnique* 44, No. 2, 277–296

S. M. Springman, *University of Cambridge*

The paper describes approaches for designing bridge abutments subjected to lateral thrust due to horizontal soil movements in a soft layer underlying the approach embankment and presents empirical design charts. This discussion is concerned principally with the importance of modelling the situation which pertains to conditions in the field, to allow valid use to be made of these charts. In particular, reference is made to

- (a) the geometry of the pile cap relative to the surcharge or the embankment and abutment loading
- (b) the influence of lateral loading on the abutment wall
- (c) shear transfer between the embankment and the soft clay layer
- (d) time dependency
- (e) non-linear effects

and refers to some of the data published in the paper, namely tests carried out by Springman (1989), and additional data published by Bransby (1993) and Ellis (1993) (see Fig. 19).

Figure 19 shows a full-height bridge abutment, supported on a group of piles. The pile row furthest from the embankment (called the rear row)

is raked. The approach embankment is shown to apply lateral pressure to the abutment wall (defined here as p_w) combined with lateral loading arising from differential soil–pile movement.

However, in Stewart’s (1992) centrifuge model tests, the pile cap was 2 m (prototype scale) above the ground surface. Springman (1989) used this option (Fig. 19) because of the difficulty of dealing with the fixity of the pile into the cap and the instrumentation of the pile using strain gauges to determine bending moment profiles. However, this modification would have reduced the lateral thrust on the piles, and particularly on the rear row of piles, because the soil had an extra degree of freedom allowing movement of soil vertically upwards under the pile cap. This was observed in the tests. Consequently the pile bending moments and displacements would be less than for the equivalent prototype. The authors’ test data in Fig. 2(b) show that the bending moment profile in the rear pile is almost linear in the soft clay layer, implying minimal differential lateral soil–pile movement, with the lateral loading supplied mainly by the component of shear force at pile cap level due to p_w .

Subsequently, Bransby (1993) resolved this modelling problem by repeating the tests, using

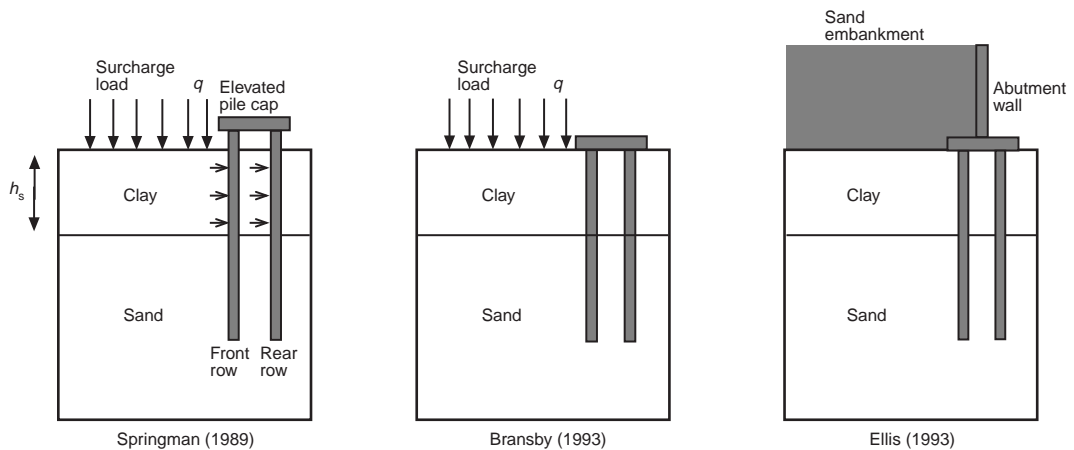


Fig. 19. Comparison of geometry for different tests

Table 4. Data for a surcharge load of 140 kPa, $h_s = 6$ m, immediately after completion of loading*

| Year | Loading category | K_R | Maximum bending moment at pile cap: MNm | Maximum bending moment in stiff layer: MNm | Pile cap displacement: mm | Deduced p_m for full clay depth: kPa | Vane shear strength: kPa |
|------------------|---|-------|--|--|---------------------------|---|--------------------------|
| Springman (1989) | Surcharge load, pile cap above ground level | 1 | -0.9 F, C -1.2 R, C -1.1 F, S -1.3 R, S | 0.8 F, C 1.2 R, C 1.0 F, S 0.8 F, S | 20 | 130 F, C 45 R, C over top 5 m 64 F, S 67 R, S over top 4 m | 16 |
| Bransby (1993) | Surcharge load, pile cap at ground level | 0.33 | -2.6 F, C -3.7 R, S | 2.0 F, C 3.4 R, S | 78 | 90 F, C 100 R, S | 40 |
| Ellis (1993) | Embankment built in flight behind a full height abutment wall | 0.53 | -9.5 R, S -8.9 F, S -8.8 R, C | 3.5 R, S 3.2 F, S 3.8 R, C | 95 | 53 R, S 72 F, S 77 R, C | 24 |

* C centre pile of three in each front and rear row and spaced at 5.25 diameters in each row; F front pile (nearest embankment); R rear pile (farthest from embankment); S side pile.

the same below-ground pile group geometry, but with the pile cap located on the surface of the soil (Fig. 19). Both bending moments and pile cap displacements were found to be 100–200% larger than those for the same model geometry for an equivalent case from earlier tests (Table 4). The average vane shear strength of the clay in Bransby's test was 2.5 times greater, which would imply lesser differential movement between the soil displacement and the pile displacement for equivalent surcharge loads, but greater soil stiffness (and also lower K_R), to which equation (8) for determining lateral pressure refers.

The lateral pressures acting on the pile in the soft layer p_m have been deduced from double differentiation of the bending moment profile, and it can be seen from Table 4 that

- in Springman's tests the pressure on the front central pile was greater than that on the rear pile where the pressure extended over a shallower depth
- in Bransby's tests the pressures were more evenly spread on each row of piles, showing the difference in the soil displacement mechanism when the pile cap was nominally in contact with the soil.

A further comparison is made with tests in which the pile cap was in contact with the ground, and the load was provided by building an embankment in flight behind the abutment wall (Fig. 19; Ellis, 1993; Springman, Ng & Ellis, 1994). This increased the bending moments and displacements to a more significant degree (Table 4) and provided two additional interaction effects (Fig. 20), the first of which was lateral earth pressure on the abutment wall p_w —which is easy to predict if it is assumed that this reduces to active pressure with significant lateral soil movement, although Department of Transport (1987) guidelines propose that the abutment structure should be designed to carry earth pressures of $1.5K_0$. This earth pressure may be assumed to provide a lateral force at pile cap level, and this option is offered in the computer programs SIMPLE and SLAP (Springman & Symons, 1992; Randolph & Springman, 1991) which solve for the design method proposed (Springman, 1989) and on which the author's method is based. The second additional interaction effect were shear stresses at the interface between the embankment and soft clay layer τ_t (which was referred to but not quantified and under the pile cap τ_c (Springman, 1989; Bransby, 1993; Ellis, 1993). τ_c was also present in Bransby's tests. These shear stresses are significant and raise the net lateral load on the abutment and pile cap. Further analysis is given in Springman *et al.* (1994). Furthermore, the lateral pressures on the piles p_m in Ellis's tests were less than in

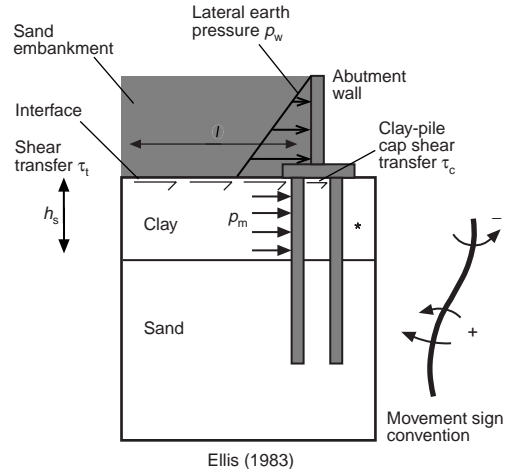


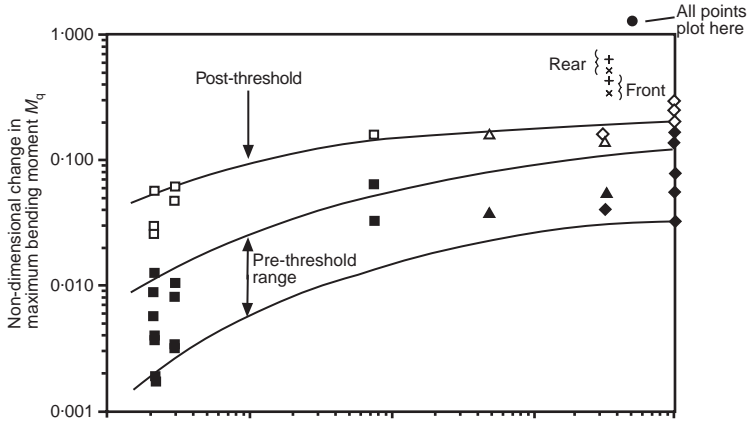
Fig. 20.

Bransby's tests because of the additional lateral forces applied at and above pile cap level by these two effects, which reduced the differential lateral displacement between the soil and the pile.

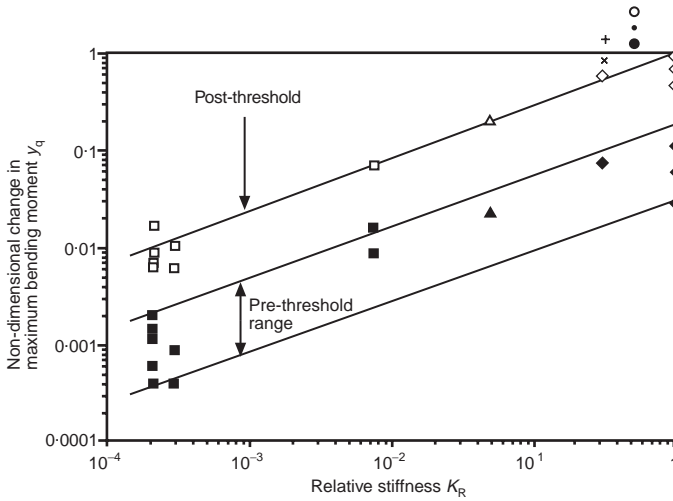
It can be seen that the values of bending moment increase in each case as the loading system becomes closer to the field prototype as shown in Fig. 1. The bending moment values which include the loadings due to an embankment behind a full height abutment are up to ten times larger than those on which Fig. 6 has been based, and the displacements are up to five times larger. Fig. 21 shows the data from Table 4 superimposed on Fig. 6.

In interpreting the field test data referred to in Fig. 21, it should be noted that the data of Bigot, Bourges, Frank & Guegan (1977) are for a sloped embankment, with a single free-headed pile at the toe of the embankment, and Heyman's (1965) data are similarly for a sloped embankment with a single pile, propped at the head. The soil is therefore permitted to move vertically upwards around the pile. Furthermore, the use of logarithmic scales will tend to reduce the scatter at the larger values of K_R .

It is therefore suggested that these charts in Fig. 21 and the exponents derived from them as shown in Figs 9 and 10 are relevant for sloped embankments and piled bridge piers in which the pile cap is elevated above ground level, but not for full height bridge abutments. However, for a full height abutment, if an embankment–pile cap–pile system (e.g. a highly compressible synthetic layer under the pile cap) can be designed to allow vertical movement of soil under the pile cap, this will reduce the pile cross-section required and the charts may be applicable.



(a)



- Centrifuge data pre-threshold (Stewart, 1992)
- ◆ Centrifuge data pre-threshold (Springman, 1989)
- ▲ Field data pre-threshold
- × Threshold (Bransby, 1993), $q \sim 120$ kPa
- + Maximum (Bransby, 1993), $q \sim 209$ kPa
- 31% embankment (Ellis, 1994)
- 63% embankment (Ellis, 1994)
- 100% embankment (Ellis, 1994)
- Centrifuge data post-threshold (Stewart, 1992)
- ◇ Centrifuge data post-threshold (Springman, 1989)
- △ Field data post-threshold
- Threshold, $3s_u \sim 50\%$ embankment

(b)

Fig. 21. (a) Non-dimensional change in maximum bending moment (after Stewart *et al.*, 1994); (b) non-dimensional change in pile head deflection (after Stewart *et al.*, 1994)

Both Bransby (1993) and Springman (1989) observed only marginal increases in bending moment and displacement with time after the vertical surcharge load had been applied, whereas Ellis (1993) found that bending moments increased

on average by 33% in the rear piles during the two years (at prototype scale) following embankment construction, whereas the displacements increased by 50%. It is therefore important to allow for the shear transfer mechanism, which were approxi-

mated by between 150 kN/m (short term) and 225 kN/m (long term) acting laterally on the pile cap (Springman *et al.*, 1994): equivalent to an average interfacial shear stress τ_i of 15–20 kPa over a length l of 10–11 m when including time-dependent effects.

In Fig. 22 there appears to be a clear threshold for Bransby's tests if bending moments are plotted against surcharge load q ; this occurs for $2s_u < q < 3s_u$. However, in Ellis's tests, a breakpoint was not evident for the data shown, which were obtained immediately following embankment construction. Subsequent shear transfer occurred with time, whereupon significant increases in

bending moments and displacements were observed. This would have created a breakpoint, although it would not be clear whether this was at $q = 3s_u$ or elsewhere, unless a slow drained loading had been pursued.

The authors discuss three-dimensional effects, given that the analysis is based on plane strain formulation. In three dimensions, soil will displace in a transverse sense, so that the resultant lateral soil movement away from the embankment along the centre-line of the overpass road will be less. Therefore the lateral thrust on the piles will be reduced and so the plane strain model should provide the worst case for design.

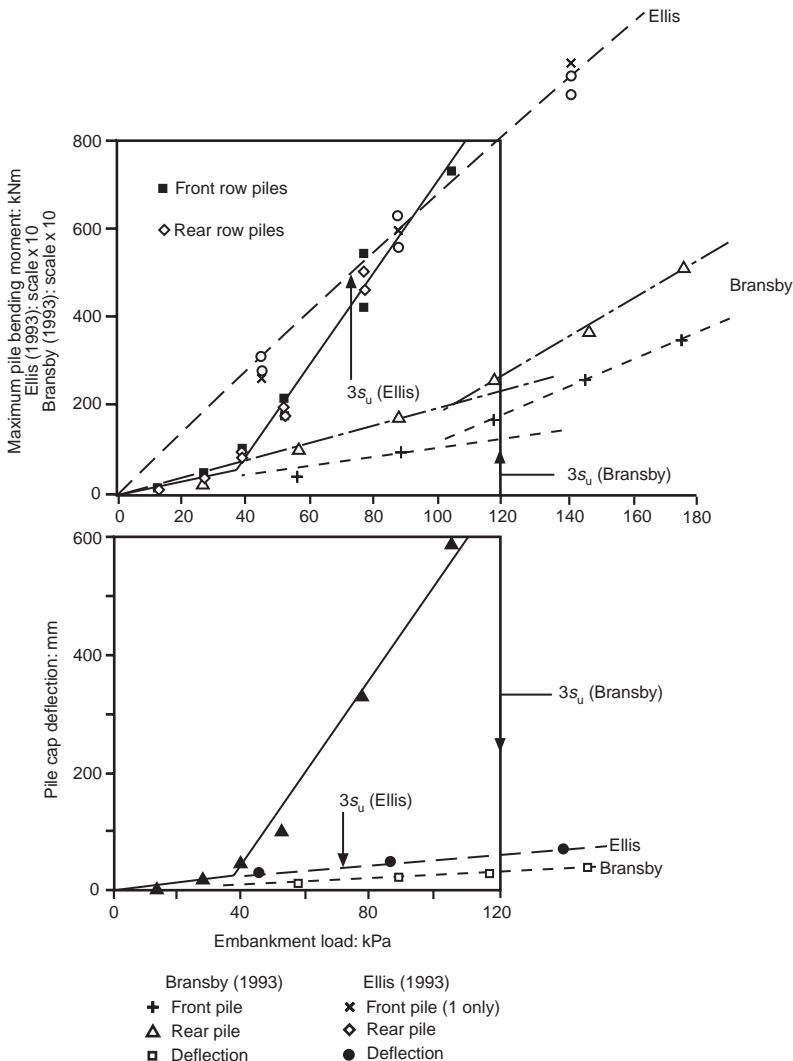


Fig. 22. Pile group response with embankment loading level for centrifuge test 9 (after Stewart, 1992) with additional data superimposed

It is also possible to model non-linear effects using the same soil displacement mechanism on which the authors based their extended non-linear analysis, by using a power law of the form

$$\tau = a\gamma^b$$

where a and b are constants, into which substitutions may be made for $\gamma = 2y_{so}/h_s$ and $q = 4\tau_{mob}$, so that

$$q = 4a(2y_{so}/h_s)^b$$

From the authors' data, a hyperbolic equation has been deduced and is shown in Fig. 23 for the authors' centrifuge tests. It can be seen that a power law curve with $a=25$ and $b=0.3$ fits the hyperbola at low displacements, with values of $a=22.5$ and $b=0.2$ approaching the hyperbola in the mid range and a final power law curve with $a=20$ and $b=0.1$ providing the best approximation at large displacements.

The triangular constant strain mechanism is more suited to a shallower layer of clay, and so, in comparison with Stewart's test data for surcharge against deflection for the 8 m deep clay layer, it can be seen (Figs 24 and 25) that the same trends are observed, with the power law curve fitting best for high values of the exponent b at low displacements and lower values of b as displacements increase. The fit with $a=22.5$ and $b=0.2$ is better than for a linear prediction (Fig. 24), but not as close as the authors' combination of a non-linear hyperbolic stress-strain relationship and an embankment geometry correction (Fig. 25).

Unless designers can assure themselves that the pile cap is not in contact with the ground, then allowance needs to be made for the significant additional lateral loads applied by the embankment lateral pressure on the abutment wall, and the shear transfer at the base of the embankment and under the pile cap.

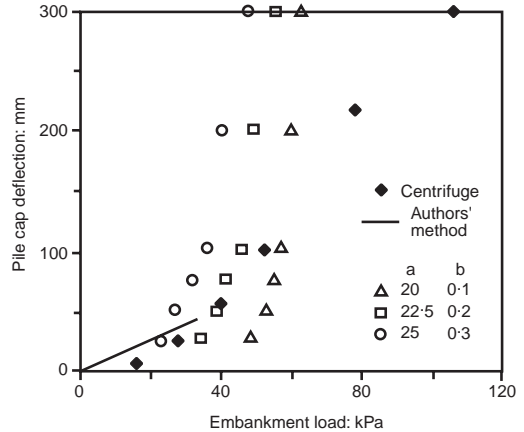


Fig. 24. Comparison of results from the proposed method with data from centrifuge test II (8 m thick soft layer) (after Stewart et al., 1994) with power law data superimposed

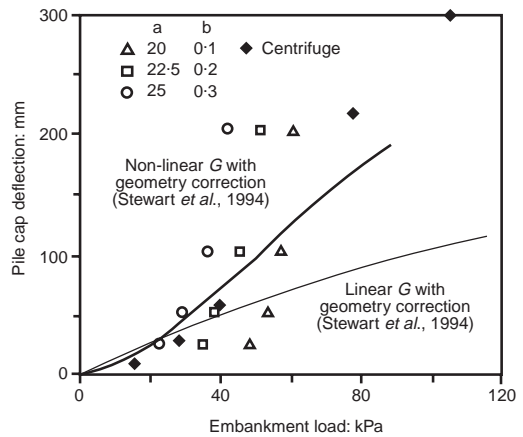


Fig. 25. Comparison of results from the proposed method (incorporating non-linear effects) with data from centrifuge test II (8 m thick soft clay layer) (after Stewart et al., 1994) with power law data superimposed

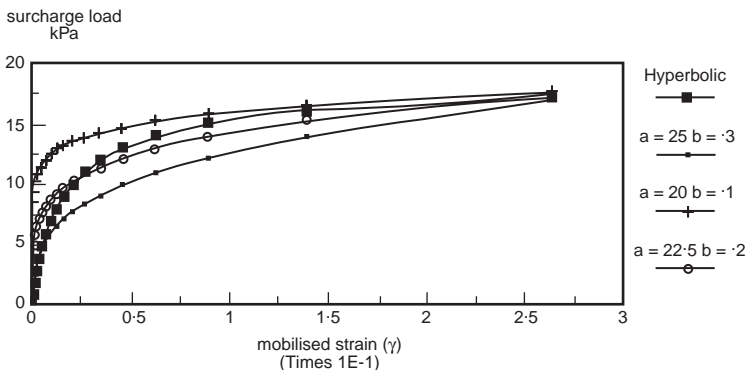


Fig. 23. Design of piled bridge abutments: stress-strain law comparison

Authors' reply

Dr Springman raises several issues which were not emphasized in the paper, or are now more evident after the examination of recently obtained data.

Dr Springman shows that significantly larger bending moments and deflections will occur when the pile cap is in contact with the ground. In the new data she presents an increase in moments and deflections by a factor of about 3 was observed, as shear stresses were able to develop along the underneath of the pile cap and vertical soil movement was constrained so that higher lateral pressures were imparted to the rear row of piles. The suppression of upward soil movement by the pile cap will lead to an increase in the cap-soil contact pressure. It might be expected that higher

contact pressure will develop when the soft clay is relatively shallow, as the zone of deformation is more localized, and these would then result in increased shear stress under the pile cap. Similarly, the effect of vertical soil movement constraint on increasing lateral pressures on the rear row of piles would also be expected to be higher where the soft clay is relatively shallow and/or the piles are relatively stiff. It is therefore our opinion that both of these effects will become more prominent as K_R increases.

This view is reinforced by the model test data of Watabe, reported by Kimura, Takemura, Watabe, Suemasa & Hiro-oka (1994) which were for full height piled abutments with the pile cap in contact with the ground ($K_R = 10^{-2}$ to 10^{-1}), and were shown by us (Stewart, Randolph & Jewell (1994))

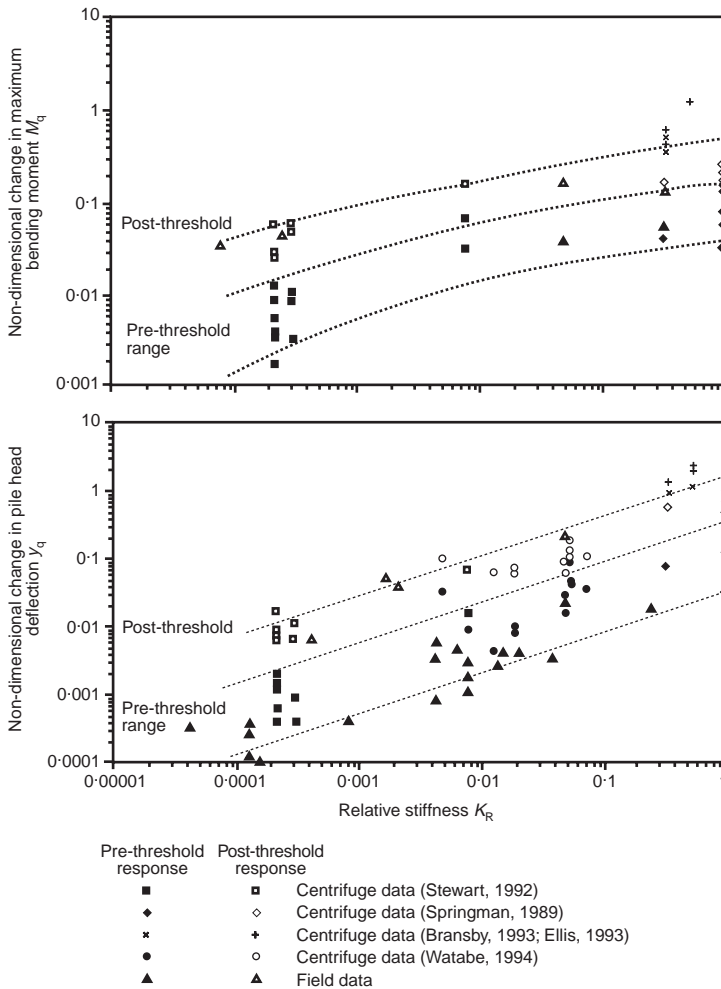


Fig. 26. Non-dimensional change in maximum bending moment and pile head deflection, updated with additional data; approximate envelopes have been altered from Fig. 6

to compare well with the envelopes proposed in Fig. 6. An updated version of Fig. 6 is shown in Fig. 26 which incorporates a great deal of additional field and centrifuge data (Stewart *et al.*, 1994). It also includes the new data presented by Dr Springman. Further work is in progress to examine these aspects more fully and to confirm the trends suggested by the test data.

Dr Springman also shows that a sand fill behind a full height abutment gives rise to significantly higher bending moments and deflections than a surface pressure loading, and several factors contributing to this effect are outlined. Data presented in Fig. 6 included a test with a sand embankment constructed behind a retaining wall. This test was with $K_R = 2 \times 10^{-4}$, and bending moments and deflections 2–3 times those in an identical test with a sloping embankment were recorded. This increase correlates with the observed differences between Ellis's and Bransby's data in Table 4, although $K_R = 0.53$ and 0.33 respectively. Further details of the influence of the loading conditions are given by Stewart *et al.* (1994) where identical tests were performed to quantify the individual contributions from embankments of differing geometry and loading imparted by earth pressure on a retaining wall.

Dr Springman appears to infer that the envelopes proposed in Fig. 6 may require adjustment. However, on the basis of our comments it is likely that the curves need only to be moved upwards in the region of relatively high K_R , relevant to data presented by Dr Springman. The curves in Fig. 26 have been adjusted on the basis of the additional data now available. The issues raised by Dr Springman are covered adequately in the range of low to medium K_R by the original data in Fig. 6 and the additional data in Fig. 26.

Another important point discussed in the paper is the influence of the time of pile installation relative to embankment placement. Pile deflections and bending moments can be reduced considerably by placement of the embankment prior to pile installation. However, all the available centrifuge test data represent the extreme loading case, where piles are installed before embankment construction. The design charts are intended as an aid to initial design; detailed design should incorporate consideration of all individual factors contributing to loading, such as the timing of pile installation.

The threshold loading of $3s_u$ observed in most of the data is likely to depend on a number of factors, including the loading configuration and the degree of overconsolidation. Stewart (1992) noted that the threshold was less discernible in tests where the soft clay was relatively shallow, which specifically relates to the data presented by Dr Springman.

Non-linear stress–strain response of the soft clay is an important feature of this problem and Dr Springman has presented a useful alternative expression that can be readily incorporated in the solution. However, the expression we used contains an additional parameter and can be used to represent a wider range of stress–strain curves.

REFERENCES

- Bigot, G., Bourges, F., Frank, R. & Guegan, Y. (1977). Action de déplacement latéral du sol sur un pieu. *Proc. 9th Int. Conf. Soil Mech., Tokyo* **1**, 407–410.
- Bransby, M. F. (1993). *Centrifuge test report. Investigation of the buttonhole foundation technique. Tests EC3 & 4*. University of Cambridge.
- Department of Transport (1987) *Backfilled retaining walls and bridge abutments*. Highways and Traffic Departmental Standard BD30/87. London: DoT.
- Ellis, E. A. (1993). *Centrifuge modelling of full height abutments on soft clay: Tests 3 & 4*. ANS&A data report to Transport Research Laboratory.
- Heyman, L. (1965). Measurement of the influence of lateral earth pressure on pile foundations. *Proc. 6th Int. Conf. Soil Mech., Montreal* **2**, 257–260.
- Kimura, T., Takemura, J., Watabe, Y., Suemasa, N. & Hiro-oka, A. (1994). Stability of piled bridge abutments on soft clay deposits, *Proc. 13th Int. Conf. Soil Mech., New Delhi* **2**, 721–724.
- Randolph, M. F. & Springman, S. M. (1991). Analysis of pile response due to external loads and soil movement. *Proc. 10th Eur. Conf. Soil Mech., Florence* **2**, 525–528.
- Springman, S. M. (1989). *Lateral loading on piles due to simulated embankment construction*. PhD thesis, University of Cambridge.
- Springman, S. M., Ng, C. W. W. & Ellis, E. A. (1995) *Centrifuge and analytical studies of full height bridge abutment on piled foundation subjected to lateral loading*. Project report 98, DoT (E450A/BG). Crowthorne: Transport Research Laboratory.
- Springman, S. M. & Symons, I. F. (1992). The design of piled full-height bridge piers and abutments. *Geotechnique et Informatique Colloque International, Paris*, pp. 343–350.
- Stewart, D. P., Randolph, M. F. & Jewell, R. J. (1994). Recent developments in the design of piles loaded by lateral soil movements. *Proceedings of international conference on design and construction of deep foundations*, vol. 2, pp. 992–1006. Orlando: Federal Highway Administration.
- Watabe (1994). Private communication.

ERRATA TO THE PAPER

On page 286, in equation (2b) L_{eq}^2 should be replaced by L_{eq}^4 .

On page 287, the scales on the vertical axes for constants a_1 and a_2 are in reverse order: they should both read from 1 at the bottom.

On page 289, in expression F_1 for equation (6)

$2m[(L - h_f)^3 - h_u^3]/L$ should be replaced by $2m^2[(L - h_f)^3 - h_u^3]/L$ and in expression F_2 for equation (7) [$h_u^2(L - h_f)^3/L$] should be replaced by [$h_u^2(L - h_f)^3/L$].

Also on page 289, in equation (9) $(h_s - h_1)/2$ should be replaced by $(h_s - h_1/2)$ and in equation (10) $(h_s - h_1)/2$ should be replaced by $(h_s - h_1/2)$.