Interpretation of static pile loading test results and application for design of pile groups

David Tara, M.Sc.A., P.Eng.
President, Thurber Engineering Ltd., Vancouver, B.C.

Steven Coulter, M.Sc., P.Eng.
Geotechnical Engineer, Thurber Engineering Ltd., Vancouver, B.C.

ABSTRACT Recent major bridge projects in the Greater Vancouver Area showcase the experience of local geotechnical engineers. Large static pile loading tests have been carried out on a number of these projects including the Pitt River Bridge, Golden Ears Bridge and Port Mann Bridge projects. This paper briefly discusses different analytical approaches to load test interpretation and reviews the approach as applied to some classic and recent local case histories. Where applicable, the paper considers the influence of reaction piles on the initial stiffness of the test pile and explores the application of different methods of interpretation to published test data including some local projects. The paper summarizes the results of the analyses and provides recommendations for design.

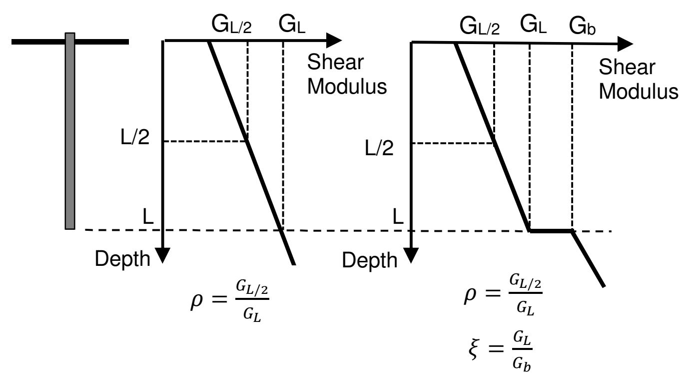
Background

There currently are two distinctly different design philosophies based on elastic methods that are promoted for interpretation of static loading test (SLT) results and their application to design of pile groups. These methods are the equivalent linear elastic approach, which is based on the secant modulus (Poulos and Davis 1980, Fleming et al. 1992) and the linear elastic approach which is based on the initial stiffness augmented by the measured movement (Randolph 1994, Fleming et al. 2008, Viggiani et al. 2012). Although both methods have a similar basis, they differ significantly in application. This paper will demonstrate the use of these methods for six published case histories including some local ones. To accurately assess the local case histories, the influence of the reaction piles will also be taken into consideration in the analysis.

Methods of analysis

Elastic methods of analysis have been available to the geotechnical community for many years (eg. Poulos and Davis 1980 and Fleming et al. 1992). The software program PIGLET (Randolph 2003a) is one example of this method of analysis. PIGLET was developed to model the elastic response of single piles and pile groups subject to compression, tension and lateral loading. A detailed example of its application to the prestigious My Thuan bridge project in Vietnam is provided in a recent Rankine Lecture (Randolph 2003b). Program inputs include the soil's Poisson's ratio and shear modulus along the pile shaft and at the pile toe, and the pile's dimensions and elastic modulus. The soil model and program inputs are defined as shown in Figure 1.

Fig. 1. PIGLET soil model showing variation of shear modulus with depth (Fleming et al. 2008)



(a) Floating Pile (b) End-bearing Pile

[1] $$\frac{P_t}{w_t d G_L} = \frac{\frac{2\eta}{(1-\nu)\xi} + \frac{2\pi\rho}{\zeta}\frac{\tanh(\mu L)}{\mu L}\frac{L}{d}}{1 + \frac{8\eta}{\pi\lambda(1-\nu)\xi}\frac{\tanh(\mu L)}{\mu L}\frac{L}{d}}$$

Where

[2] $$\eta = \frac{d_b}{d}$$

[3] $$\xi = \frac{G_L}{G_b}$$

[4] $$\rho = \frac{\bar{G}}{G_L}$$

[5] $$\lambda = \frac{E_p}{G_L}$$

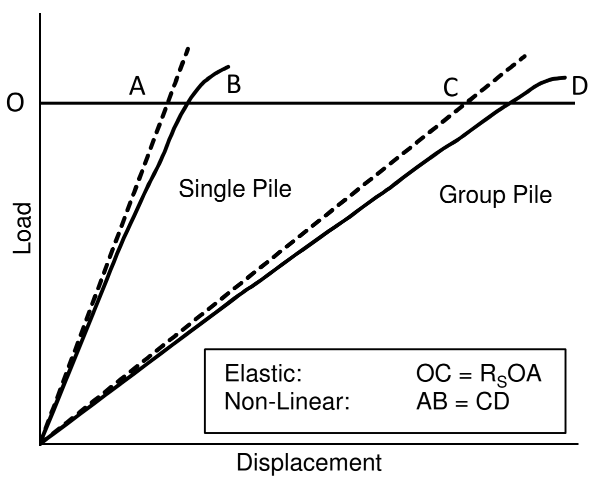
[6] $$\zeta = \ln\left(\frac{2r_m}{d}\right)$$

[7] $$\mu L = 2\sqrt{\frac{2}{\zeta\lambda}}\left(\frac{L}{d}\right)$$

We have considered two methods of elastic analyses; the linear elastic (LE) and elastic secant (ES) methods described below. For an elastic analysis to accurately represent reality, an appropriate level of strain must be assigned. The equivalent linear elastic approach for pile group analysis uses the secant modulus derived from a single pile static loading test. This method will be referred to as elastic secant (ES) and is described in detail in Poulos and Davis (1980) and Fleming et al. (1992). Poulos (2000) suggests that, for typical working loads, a modulus equivalent to about 25% to 30% of the small-strain value (i.e. initial stiffness derived for a single pile, k) is generally suitable for pile group analysis. Mayne (2007) and others suggest that the elastic secant method can be extended from small strain to large strain, even failure, using a modulus degradation approach to capture the overall movement of a single pile. While the elastic secant method has been in use for a number of years and has been shown to work reasonably well for single piles, it provides conservative results and tends to overpredict pile group movement, especially at larger strains (or lower factors of safety).

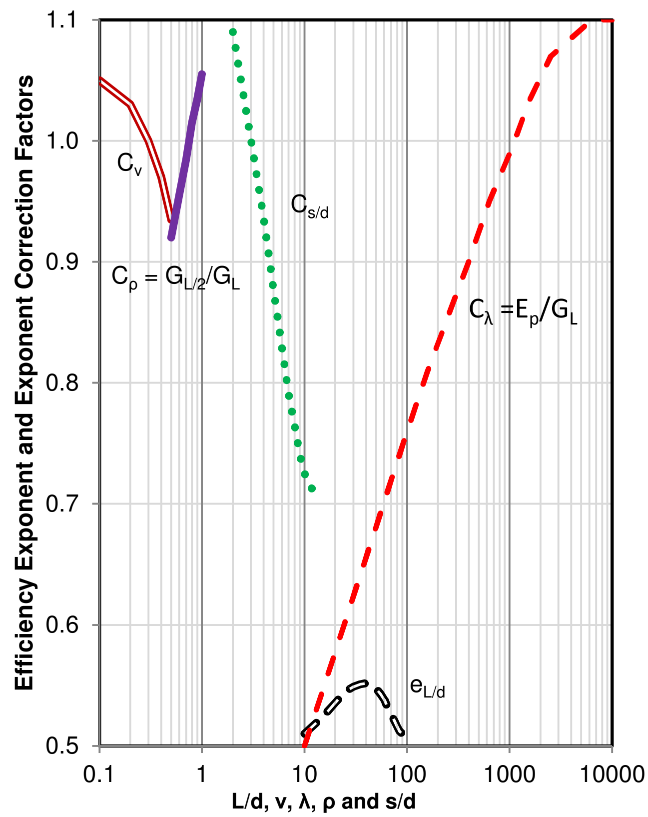
Shortly after the publication of the second edition of "Piling Engineering" (Fleming et al. 1992), Randolph (1994) suggested that to analyse the load-movement response of a pile group, calculations should be based on elastic calculations of interaction using the initial stiffness (i.e. tangent) of the single pile response augmented by the non-linear component of load-movement response for a given load level. Mandolini and Viggiani (1997) and Viggiani, Mandolini and Russo (2012) presented the results of back-analyses of measured vertical response of large pile groups and concluded that the load-movement response could be predicted reasonably well by assuming elastic soil response, and adopting a shear modulus close to the small-strain modulus (G_0). This method will be referred to as linear elastic (LE) and is shown conceptually in Figure 2.

Fig. 2. Calculation of interaction with non-linear load transfer response (data from Fleming et al. 2008)



To understand the relative importance of the various parameters used in elastic pile group analyses, Fleming et al. (1992, 2008) describe a means of estimating the stiffness of group of n piles as $K = \eta_w \cdot n \cdot k$ where η_w is a factor describing group efficiency equal to n^{-e} and k is the stiffness of a single pile. The exponent e is an efficiency correction factor and typically in the range of 0.4 to 0.6, depending on pile slenderness ratio (L/d), pile stiffness ratio ($\lambda = E_p/G_L$), pile spacing ratio (s/d), variation of soil modulus with depth ($\rho = G_{L/2}/G_L$) and Poisson's ratio (v). Using PIGLET, Fleming et al. (1992) developed the design curves replicated in Figure 3 for square pile groups. The base $e_{L/d}$ value is selected for the slenderness ratio (L/d) and then adjusted with the other factors (C_λ = pile stiffness correction factor, C_v = Poisson's ratio correction factor, C_ρ = soil homogeneity correction factor and $C_{s/d}$ = spacing correction factor). By inspection, the most important parameters are the pile stiffness (C_λ) and spacing ratios ($C_{s/d}$). Slenderness ratio ($e_{L/d}$), soil homogeneity (C_ρ) and Poisson's ratio (C_v) contribute less than 10% each to the pile group efficiency exponent.

Fig. 3. Efficiency exponent and exponent correct factors (data from Fleming et al. 1992)

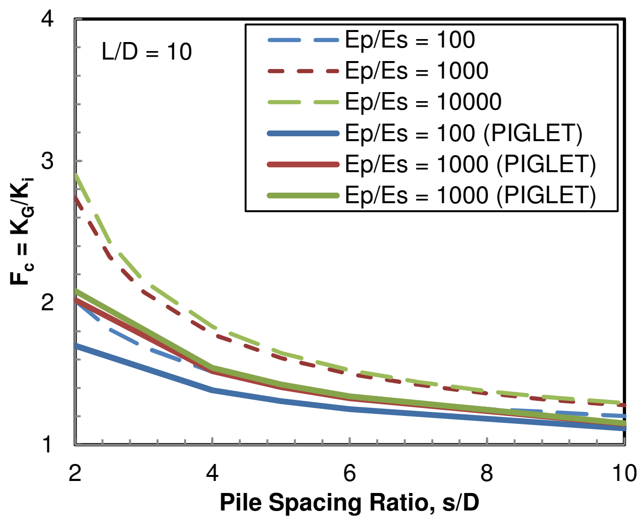


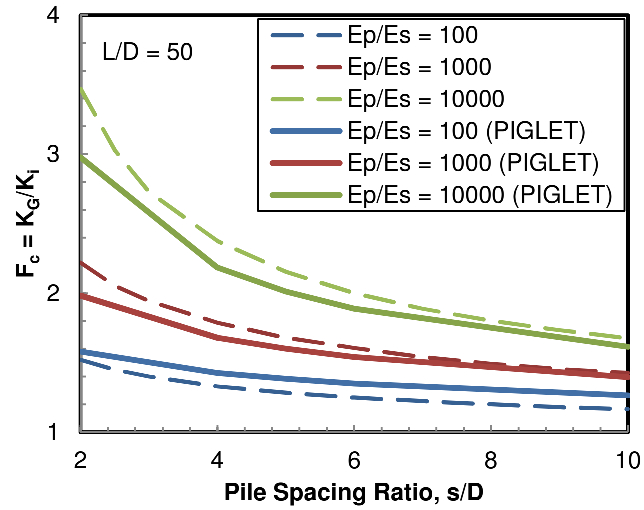
Influence of reaction piles

When analysing the results of static load tests, Poulos and Davis (1980), Poulos (2012) and Kityodom, Matsumoto and Kanefusa (2004) suggested that the measured stiffness of a test pile may be influenced by the presence of reaction piles and thus could be artificially high. Poulos and Davis (1980) provided a simplified means of correcting the inferred pile stiffness for the case of two reaction piles whereas Kityodom et al. (2004), using the software program PRAB, developed a series of design curves for correcting initial pile head stiffness for cases with four reaction piles (as is more typical in Japan). While ASTM D1143 indicates that the separation between a test pile and reaction piles should typically be at least 6 test pile diameters centre-to-centre to be able to ignore interation effects, Kityodom et al. (2004) showed that at some spacings, depending on the relative stiffness of the pile and soil (E_p/E_s) and slenderness (L/D) ratio, the measured test pile initial stiffness could overestimate the true stiffness by 30% to 100% for a floating pile and up to 40% for a pile bearing on a rigid stratum.

The Kityodom et al. (2004) initial stiffness correction curves were used to test the appropriateness of using PIGLET to assess the influence of reaction piles on a test pile for L/D = 10 and 50 and ν = 0.3. The results of parametric analyses for floating piles are shown in Figure 4. By inspection, PIGLET is able to replicate the Kityodom et al. (2004) correction curves reasonably well at higher slenderness ratios and shows the same trend at lower slenderness ratios.

Fig. 4. Correction factor for floating piles embedded in semi-infinite soils for L/D = 10 and 50 (data from Kityodom et al. 2004).





Case histories

The six case histories examined here are described in detail in Briaud et al. (1989), O'Niell et al. (1982), Cooke, Bryden-Smith, Gooch and Sillett (1981), Tara (2012), Amini et al. (2008) and Naesgaard et al. (2012), and Naesgard et al. (2006, 2012). Table 1 summarizes the author, soil type, small-strain shear modulus profile and pile type, toe condition and dimensions. Table 2 provides relevant information regarding slenderness ratio, average pile spacing and number of piles. For discussion purposes, the case histories will be referred to by number. Cases 1 to 3 have been reviewed in the literature by a number of authors.

Table 1. Summary of Relevant Case History Information

Case	Author	Soil Type	Small Strain Shear Modulus Profile [MPa]	Pile Type	Toe Condition	Embedded Length, L [m]	Total Length [m]	Diameter, d [m]	Wall Thickness, t [m]
1	Briaud et al. (1989)	Sand	38.3	Steel pipe	Floating	9.15	9.15	0.273	0.0093
2	O'Neill et al. (1982)	Clay	47.9 + 7.8z	Steel pipe	Floating	13.1	13.1	0.273	0.0093
3	Cooke et al. (1981)	Clay	45 + 3.2z	Precast concrete	Floating	13	13	0.45	0.225
4	Tara (2012)	Varies	32.5 + 1.1z	Open end steel pipe	End bearing	100	101.5	1.83	0.0244
5	Amini et al. (2008)	Varies	45 + 0.9z	Cast in situ concrete	Floating	32	32	2.5	1.25
6	Naesgaard et al. (2006)	Varies	0.125 + 1.25z	Closed end steel pipe	Floating	45	52.9	0.61	0.0127

Table 2. Summary of Relevant Case History Information (continued)

Case	Location	Slenderness Ratio, L/d	No. of Piles	Spacing Ratio, s/d	Pile Stiffness Ratio, E_{pile}/E_{soil}	Pile Stiffness Ratio, λ = E_{pile}/G_L	Soil Homogeneity, ρ = G_{avg}/G_L	Group Efficiency Exponent, e	Group Efficiency, η_w	Single Pile Initial Stiffness, k [kN/mm]
1	Briaud et al.	34	5	3.3	308	708	1.00	0.574	0.397	263
2	O'Neill et al.	48	9	3	91	181	0.66	0.412	0.404	437
3	Cooke et al.	29	351	3.56	178	297	0.76	0.489	0.057	590
4	Tara	55	9	5	41	76	0.61	0.309	0.508	2183
5	Amini et al.	13	3	3	144	349	0.80	0.442	0.615	3875
6	Naesgaard	74	5	5.25	198	298	0.50	0.341	0.577	280

Cases 1 and 2 presented data for relatively small groups of driven steel pipe piles. Cases 1 and 2 are a single pile and pile group in sand at Hunter's Point and in stiff clay at the University of Houston, respectively. The load-movement plots for single piles and the average response of the piles in the pile groups for Cases 1 and 2 are shown on Figures 5 and 6, respectively. The initial pile head stiffness was estimated for the single pile with the software program PIGLET and using the information in Tables 1 and 2. To estimate the response of the pile group using PIGLET for the elastic secant method, the modulus was degraded at an equal rate along the shaft and at the toe to fit the single pile load-movement curve.

The results of the PIGLET analyses are also shown on Figures 1 and 2. The figures also show the estimated pile group movement using the linear elastic method with the initial stiffness of the single pile plus the non-linear component of the movement (measured movement minus linear component from initial pile head stiffness). By inspection of the two figures, the elastic secant method works well for limited modulus degradation (i.e. small-strain) or high factors of safety. However, as soon as the piles start to exhibit strong non-linear behaviour, the elastic secant method overpredicts group movement. Conversely, the linear method combined with the non-linear component of the movement from the single pile loading test tends to predict the average group response reasonably well.

Case 3 is an example of a facility (Stonebridge Park) founded on driven pre-cast concrete piles. In this example, the applied load remains entirely within the elastic range and, as such, the elastic secant and linear elastic approaches give identical results in terms of overall, average foundation response. Figure 7 presents the load-movement curve of a constant rate of penetration test. For the applied building load of 156 MN, the average load per pile is about 444 kN which is clearly in the elastic range of the constant rate of penetration static load test. The ratio of small-strain shear modulus to undrained soil strength (G_0/S_u) is reported to be in the range of 240 to 465 for London Clay according to Vardanega and Bolton (2013). Using G_0/S_u = 450 and a Poisson's ratio of 0.5 was found to fit the initial portion of the static loading test curve reasonably well as also shown on Figure 7 .

Fig. 5. Single pile and average pile group response in sand (data from Briaud et al. 1989)

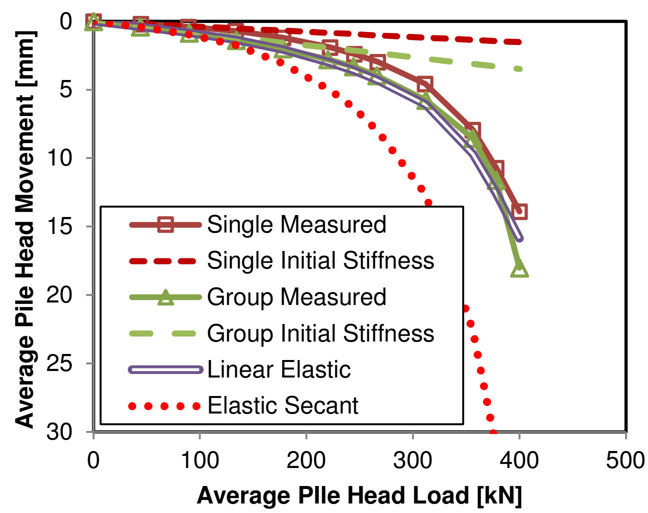


Fig. 6. Single pile and average group pile response in clay (data from O'Neill et al. 1982).

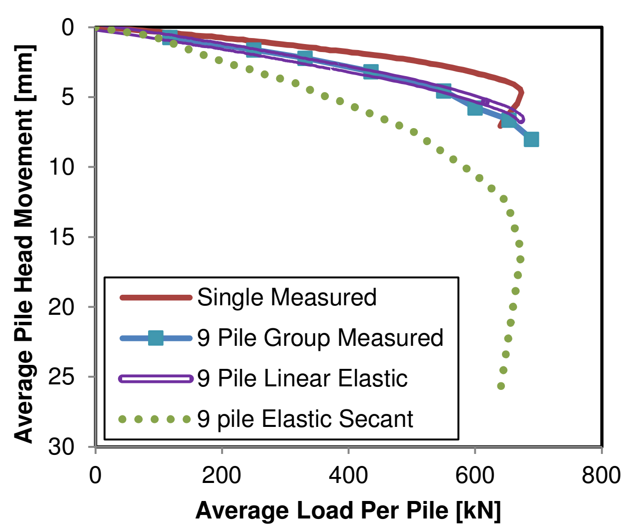
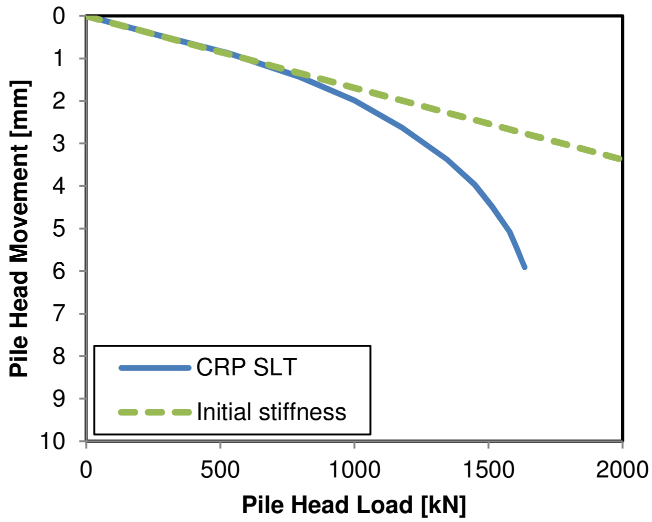
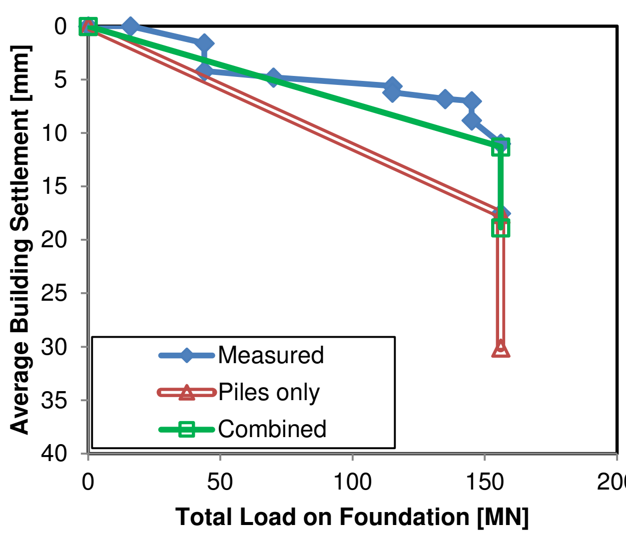


Fig. 7. Single pile response in London clay (data from Cooke et al. 1981).



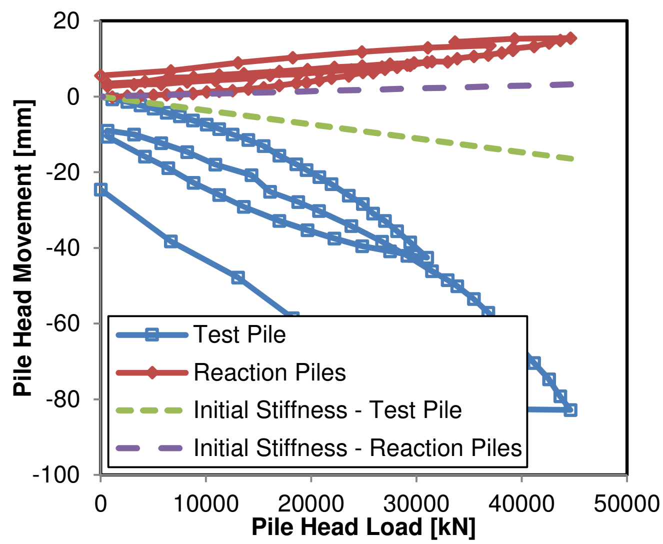
PIGLET was then used to model the pile group response shown on Figure 8 using the parameters provided in Tables 1 and 2. By inspection, PIGLET somewhat overestimates the average measured pile group response. However, if the contribution of the resistance of the raft area between the piles is considered using the method outlined in Fleming et al. (1992, 2008), the average undrained response of the foundation falls approximately in-line with the measured settlement shown. Further, the transition from undrained to drained response, where the Poisson's ratio is decreased from 0.5 to 0.1, for both the pile group and raft can also be captured reasonably well in the simple model as shown on the figure.

Fig. 8. Average pile group and piled-raft response at Stonebridge Park (data from Cooke et al. 1981)



Case 4 (Pitt River Bridge) is a local project described in Tara (2012). The main cable-stayed bridge and backstays are supported on 100 m long, driven steel pipe piles terminated a few metres in very dense glacial deposits. The 1830 x 24.4 mm piles were installed open-ended and cleaned out and filled with reinforced concrete to 30 m depth to enhance moment resistance and facilitate the structural connection with the pile cap. Although not included in the 2012 paper, reaction pile movements were also monitored during the quick load test. Figure 9 shows the load-movement response of the test and reaction piles. For discussion purposes and ease of visualization, the combined load on all four reaction piles is shown on the figure. PIGLET was used to estimate the initial stiffness of the pile group (4 piles in tension and one in compression) for the linear elastic method and also to estimate the secant modulus that would be required for a group analysis. As PIGLET does not allow the use of variable pile properties with length, average pile properties were applied over the length of the piles. In addition to the load-movement response of the test and reaction piles, Figure 9 also shows the estimated initial stiffness determined using the shear modulus profile provided in Tara (2012). Figure 10 shows the corrected load-movement response of a single pile for both the linear elastic and elastic secant methods. By inspection, ignoring the reaction piles overestimates the initial pile head stiffness by about 25%. For comparison, the Kityodom et al. (2004) findings suggest that an increase of about 20% to 30% would be expected for a pile-soil stiffness ratio similar to that at Pitt River. In the load range of interest (up to about 24 MN), the difference in predicted movement is in the range of 5 to 10 mm for a single pile.

Fig. 9. Test pile and reaction piles at Pitt River Bridge (data from Tara 2012).



Pier E1 of the Pitt River Bridge is supported on the test pile, four reaction piles and four additional piles (Tara 2012). Taking the linear elastic and elastic secant data and simply applying it to the overall group response (assuming a flexible pile cap) is shown on Figure 11. By inspection, the elastic secant response is significantly softer than the linear elastic except at very high factors of safety. For example, at a load of about 24 MN, the linear elastic and elastic secant methods predict about 40 and 75 mm of movement respectively. At 40 MN, the two methods predict movements of about 90 and 175 mm.

Fig. 10. Corrected load-movement of test pile at Pitt River Bridge using elastic secant and linear elastic methods (data from Tara 2012).

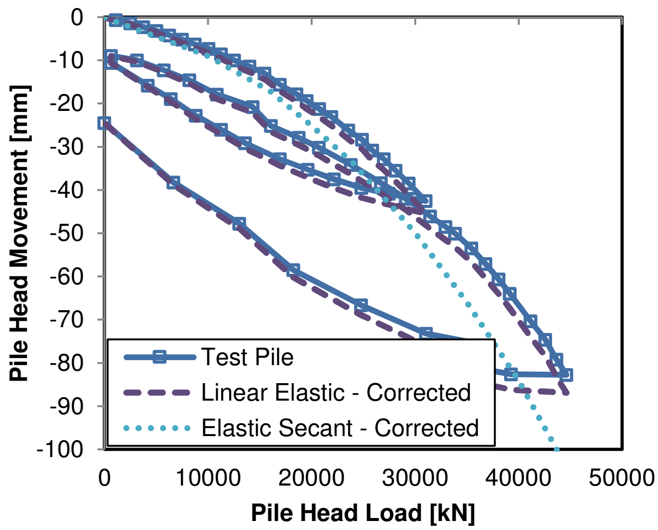
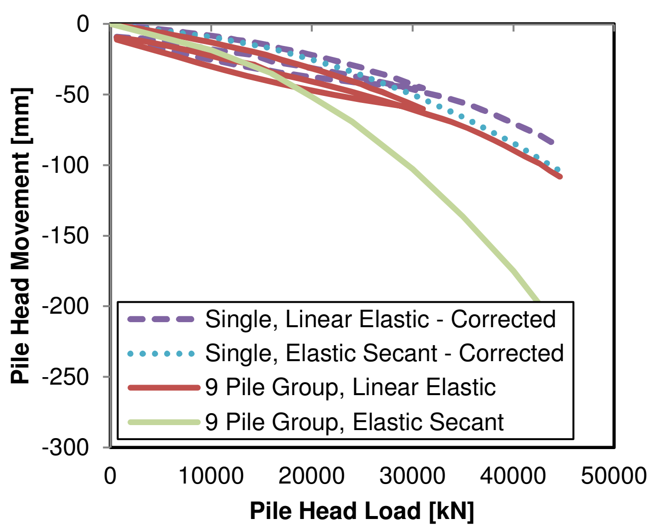
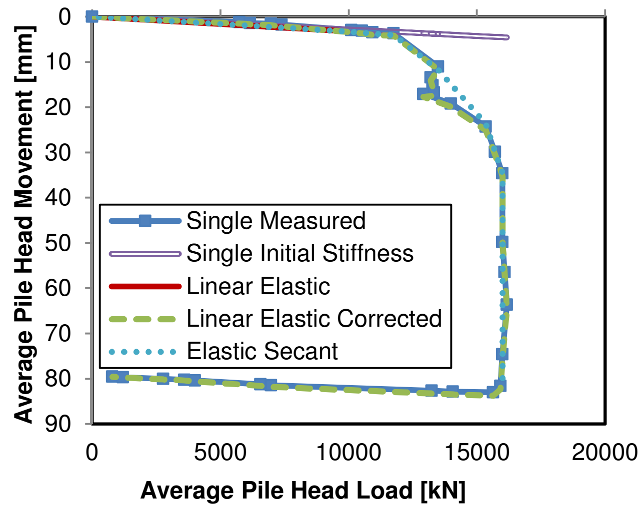


Fig. 11. Estimated average linear elastic and elastic secant pile group response at Pitt River Bridge.



Case 5 is also local (Golden Ears Bridge). The test involved three large diameter bored piles. The slenderness ratio (L/d) of the test pile and reaction piles was only 13 and 20 respectively. No movement monitoring data for the reaction piles or shear wave velocity data were provided. However, data in Amini et al. (2008) indicate that the soil conditions at the site are predominantly clay with an undrained shear strength that can be approximated as linear over the length of the pile by $S_u \approx 50 + 1 \cdot z$ (in kPa, where z is the depth below ground surface). Experience in these local deposits suggests that the ratio of small-strain shear modulus to undrained soil strength (G_0/S_u) is about 900. Accordingly, the inferred shear modulus would be about $45000 + 900 \cdot z$ kPa. Using this data and taking into consideration the weight of the reaction frame and piles, the test pile elastic secant and linear elastic load-movement response in the presence of the reaction piles was modelled using PIGLET as shown in Figure 12.

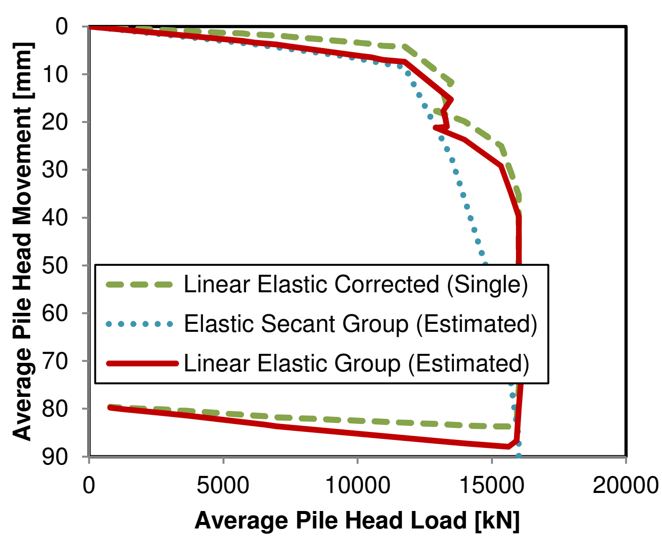
Fig. 12. Test pile and estimated reaction pile load-movement responses (data from Amini et al. 2008).



Our analysis indicates that the initial pile head stiffness was increased by about 11% due to the close proximity to the reaction piles. This magnitude of increase is relatively small but, given the small slenderness ratio, is consistent with findings of Kityodom et al. (2004) and Poulos and Davis (1980).

Figure 13 provides the estimated elastic secant and linear elastic load-movement response of a group of three piles having the same length as the test pile. This figure suggests that, while both methods provide essentially the same estimated response at small strain and up to about 12 MN, the response generated using elastic secant method shows group plunging occurring much earlier and more dramatically than would be expected.

Fig. 13. Estimated elastic secant and linear elastic pile group response.



Case 6 is the W.R. Bennett Bridge in Kelowna reported in Naesgaard et al. (2006, 2012). The test pile program included five, 610 mm diameter by 12.7 mm steel pipe piles installed by vibratory and impact driving to nominally 45 m embedment in Okanagan Lake bottom sediments. The test pile and two reaction piles were installed closed-ended and the remaining two reaction piles open-ended. No monitoring of the reaction piles was reported and, given that the piles were installed over water, no shear wave velocities values were measured. For this case, the initial pile head stiffness is estimated to be about 275 kN/mm at the pile head as shown on Figure 14. According to PIGLET, this stiffness corresponds to G ≈ 125 + 1250·z kPa. Using the curves shown in Figure 4 with ν = 0.49 and E_p/E_s ≈ 200, Kityodom et al. (2004) would suggest a correction factor (F_c) of about 1.4. In single pile mode, PIGLET gives an initial pile head stiffness of about 235 kN/mm. Transferring these stiffnesses to the mudline (recall that the load is applied 7.9 m above) would suggest F_c ≈ 1.3 which seems reasonable. The corrected single pile head movement is also shown on the figure.

PIGLET was then used to estimate the corrected response for the same group of five piles loaded in compression. The results are also shown on Figure 15. For factored resistances in the range of 2000 to 2800 kN (as would be typical), the predicted pile head movement is almost double the corrected movement of the single pile.

Fig. 14. Measured and corrected load-movement response of single pile and five pile group (data from Naesgard et al. 2006, 2012).

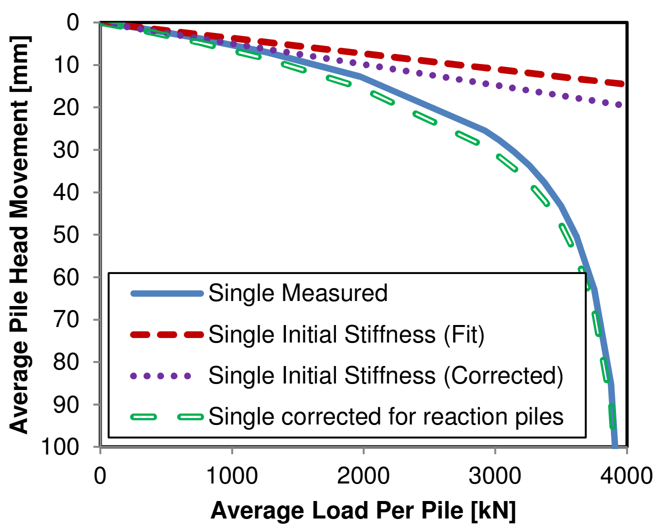
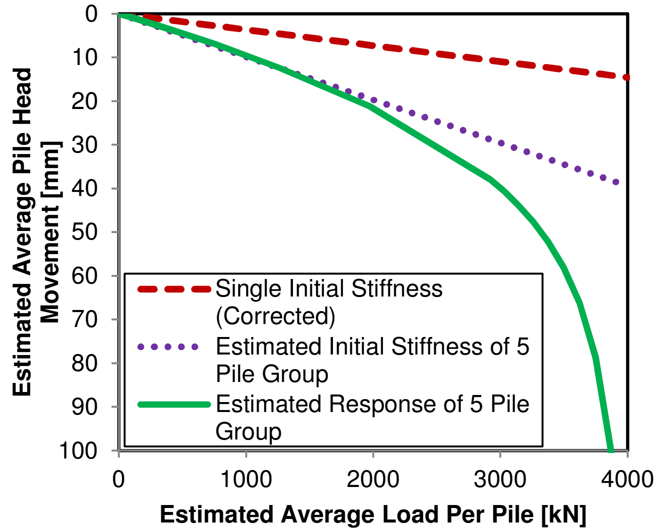


Fig. 15. Estimated load-movement response of five pile group.



Conclusions

This paper reviews reaction pile influence and the application of elastic secant (ES) and linear elastic (LE) methods of analysis to the interpretation and use of static loading test data. Our findings show that the methods and concepts reported in Randolph (1994) and Kityodom et al. (2004) can be applied to real-world problems using simple elastic solutions with programs such as PIGLET in combination with small-strain shear moduli. The findings reported herein support the importance of obtaining site-specific shear wave velocity data where pile foundations are contemplated as this data greatly assists in the interpretation of static loading tests and pile group modelling. Accurate monitoring of reaction pile head movements during static load testing should also be conducted to further enhance our ability to analyse test data. Finally, group elastic effects can be significant, especially for large pile groups.

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List of symbols

C_λ = pile stiffness correction factor

C_ν = Poisson's ratio correction factor

C_ρ = soil homogeneity correction factor

$C_{s/d}$ = spacing correction factor

d = diameter of pile

E_p = Young's modulus of an equivalent pile

E_s = Young's modulus of soil

F_c = correction factor for a vertical load test

$e_{L/d}$ = slenderness ratio base efficiency correction factor

G = shear modulus

G_0 = small-strain shear modulus

$\bar{G}$ = average shear modulus of soil over embedded depth

G_b = shear modulus of soil at base of pile

G_L = shear modulus of soil at length L

k = stiffness of single pile

K = stiffness of a group of n piles

K_G = measured pile head stiffness

K_I = actual pile head stiffness

L = embedded depth

n = number of piles in a group

P_t = total load

s = pile spacing

S_u = Undrained shear strength

w_t = settlement of pile head

ζ = measure of radius of influence of pile

η = ratio of underream for underream piles

η_w = factor describing group efficiency

λ = pile-soil stiffness ratio

μ = coefficient of compressibility

ν = Poisson's ratio

ξ = ratio of end bearing for end bearing piles

ρ = variation of soil modulus with depth