기초기술의 최근 연구동향(II)

FOUNDATION ENGINEERING: SOME RECENT CONTRIBUTIONS

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This article summarizes some recent contributions by the author and his students at Texas A&M University in foundation engineering. The topics include the scale and embedment effects for shallow foundations, the load-settlement curve method for shallow foundations, a simple method for horizontally loaded piles, a method for calculating downdrag loads and for reducing them, a method for calculating the scour depth next to a bridge pier, a deflection based analysis for tieback retaining walls, and some background on the National Geotechnical Experimentation Sites in the USA. In each case, the objective is presented, the project is outlined, and the essence of the results is described.

References are given for further details.

INTRODUCTION

Einstein said: "Everything should be made as simple as possible but not one bit simpler than that" (Safir, Safire, 1982). This should be the goal of any researcher when proposing a solution to a problem. If the solution is too complicated, it is unlikely that it will be of much use to the engineer; yet if it is too simple, it is unlikely that the solution will be able to capture the complete process. Finding the threshold of optimum simplicity (Figure 1) is at the same time desirable and difficult. To find this threshold, it is much easier to simplify an over-complicated solution than to complicate an over-simplified solution.

Foundation engineering problems tend to be complicated geotechnical problems. Some 100 years ago the tendency for foundation engineering solutions was to resort to experimental correlations because theoretical soil mechanics was in its infancy. In the last 50 years, soil mechanics theory and the associated numerical simulations have made remarkable progress and have more than caught up with sampling and testing developments. There is a need to aim for solutions with a proper balance between

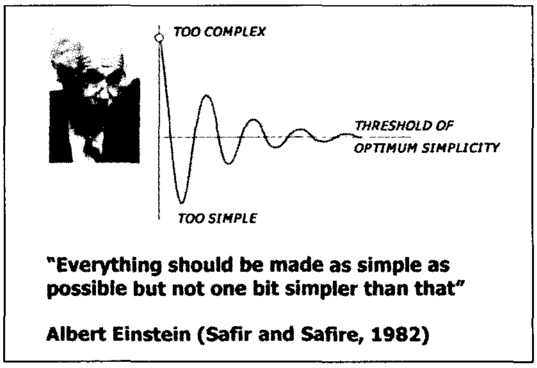

Figure 1. The Einstein Threshold of Optimum Simplicity

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theoretical and experimental considerations. In foundation engineering, this appears to give the best potential for reaching the threshold of optimum simplicity.

The methods presented here have been developed over the last 10 years by the author and his students. They include methods for shallow foundations, deep foundations, and retaining walls.

SHALLOW FOUNDATIONS: IS THERE SCALE AND EMBEDMENT EFFECT?

The students who worked on this project at various times are Philippe Jeanjean, Bob Gibbens, and Jayson Barfknecht. The sponsor was the Federal Highway Administration. The case considered is the one of a square footing in sand subjected to a vertical load applied at the center of the footing surrounded by a flat and horizontal ground surface. The average pressure under the footing is pf, the settlement is ρ, the footing width is B, the depth of embedment is D, and the average soil strength within the zone of influence of the footing is sa. The question raised is: is there a scale effect and a depth of embedment effect on the load-settlement curve in this case?

The answer is based on theoretical considerations and on the results of experiments. The theoretical considerations included the bearing capacity equation and the theory of elasticity. The experiments performed for the study included five large footing load tests and over 30 plate load tests at the National Geotechnical Experimentation Site at Texas A&M University. The footings were 3x3m, 3x3m, 2.5x2.5m, 1.5x1.5m, and 1x1m; they were all embedded 0.75m into a medium dense silty sand. The results are presented in Briaud and Gibbens(1994, 1997, 1999). Figure 2 gives an example. The plates were square,

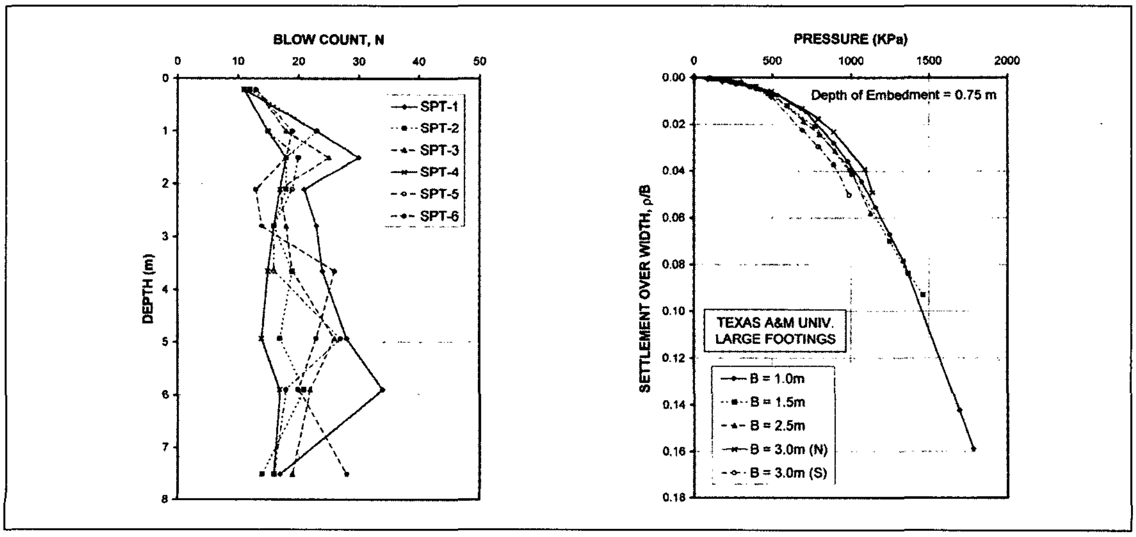


Figure 2. Results of the Large Footing Tets at Texas A&M University

the sizes were 0.1m, 0.2m, 0.3m, and 0.4m, and the embedment varied from 0m to 0.8m. The results are presented in Barfknecht and Briaud(1999). A review of the literature on this topic yielded four additional studies with footing load tests(Ismael, 1985, Pu and Ko, 1988, Khebib, Canepa, and Magnan, 1997, Lutenegger, 1995).

The answer to the question posed is limited by the evidence mentioned above and used to reach the following conclusions.

1. There is no scale effect and no embedment effect for the curve p_f/s_a vs. ρ/B regardless of the soil profile.
2. There is no scale effect and no embedment effect for the curve pf vs. ρ/B when the soil strength profile is constant with depth.
3. There is a scale effect and an embedment effect for the curve p vs. ρ/B when the soil strength profile is not constant with depth. The effect is an increase or a decrease depending on whether the strength increases or decreases with depth. This effect disappears if the curve is normalized as pf/sa vs. ρ/B.
4. The general bearing capacity equation for sands assumes a soil strength profile which increases linearly with depth because ϕ and γ are constant. In this case, N_γ and N_q are constant and the equation gives the right influence of B and D. For any other strength profile, this equation does not represent the true variation of the bearing capacity because the assumptions no longer correspond to the strength profile.

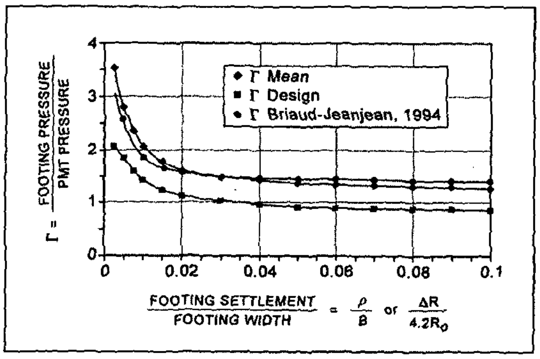
SHALLOW FOUNDATIONS: THE LOAD SETTLEMENT CURVE METHOD

The students who worked on this project at various times are Philippe Jeanjean, Kabir Hossain, Bob Gibbens, Jayson Barfknecht, Jong Hyub Lee. The sponsor was the Federal Highway Administration. The load-settlement curve method is used to generate the complete load settlement curve for a footing. The Load Settlement Curve method(LSCM) replaces the calculations of bearing capacity and settlement which were done separately in the past. The LSCM was proposed by Briaud and Jeanjean(1994) for square footings in sand resting on a flat ground surface and subjected to a centered vertical load. This method is based on the point-by-point transformation of the pressuremeter curve(Briaud, 1992) into the load-settlement curve for the footing through the use of two equations.

$$\rho/B = 0.24\,\Delta R/R_o \qquad (1)$$

$$p_f = \Gamma p_p \qquad (2)$$

where $\Delta R/R_o$ is the relative increase in pressuremeter radius, pp is the pressure on the cavity wall applied by the pressuremeter probe, and Γ is a transformation function obtained experimentally and theoretically (Jeanjean, 1995) (Figure 3). As discussed in the previous section, the scale and embedment effect are directly tied to the soil strength profile within the zone of influence of the footing, and the p/s_a vs. ρ/B curve is inde-



Figrue 3. The Γ Function for the Load-Settlement Curve Method

pendent of the foundation width B and the depth of embedment D. This indicates that the Γ function is independent of B and D.

This method was extended to the case of a rectangular footing(B wide, L long) subjected to an eccentric inclined load (eccentricity e and angle of inclination δ) at a distance d from the crest of a slope(Hossain, 1996). A number of correction factors are proposed based on numerical simulations calibrated against the large footing tests mentioned in the previous section. These factors are as follows:

Influence of the shape

$f_{L/B} = 0.8 + 0.2(B/L)$ (3)

Influence of eccentricity

$f_e = 1 - 0.33(e/B)$ center (4)

$f_e = 1 - (e/B)^{0.5}$ edge (5)

Influence of inclination

$f_\delta = 1 - (\delta/90)^2$ center (6)

$f_\delta = 1 - (\delta/360)^{0.5}$ edge (7)

Influence of a slope

$f_{\beta,d} = 0.8(1 + d/B)^{0.1}$ slope at 3 to 1 (8)

$f_{\beta,d} = 0.7(1 + d/B)^{0.15}$ slope at 2 to 1 (9)

For the time being, the superposition of cases is taken into account by multiplying the influence factors as is common practice. There is some evidence that this approach is conservative(Hossain, 1996). More research is needed in this area.

The load-settlement curve method therefore consists of the following steps:

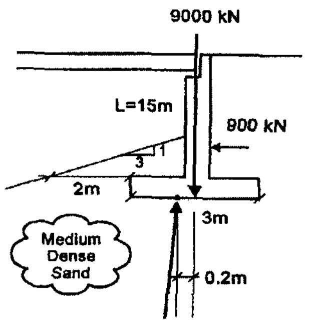
1. Perform preboring pressuremeter tests (PMT) within the depth of influence of the footing; usually at depths equal to 0.5B, 1B, and 2B.
2. Prepare the PMT curves and obtain the average curve (Briaud, Jeanjean, 1994).
3. Transform the average PMT curve point by point into the footing pressure vs. relative settlement curve:

$$\rho/B = 0.24\ \Delta R/R_o \quad (10)$$

$$p_f = f_{L/B}\, f_e\, f_\delta\, f_{\beta,d}\, \Gamma\, p_p \quad (11)$$

where ρ is the settlement of the footing, B is the footing width, ΔR et R_o the increase in radius and the initial radius of the cavity in the PMT test respectively, p_f the footing pressure corresponding to ρ/B, $f_{L/B}$, f_e, f_δ, et $f_{\beta,d}$ the influence factors for shape, eccentricity, inclination, and proximity of a slope given by equations (3) to (9) above, Γ the function given in Figure 3 and which already includes the scale

PROBLEM: A bridge abutment rests on a spread footing 3m wide and 15m long. The vertical load on the footing is 9000kN. The active pressure behind the abutment wall develops a horizontal load equal to 450kN. The resultant of the two loads is applied at an eccentricity of 0.2m. The soil is a medium dense sand characterized by the pressuremeter curve.



SOLUTION: Load Settlement Curve Method

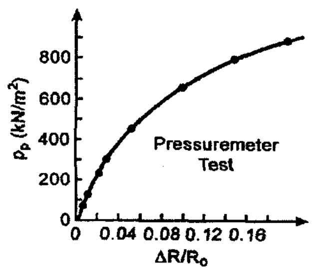
$f_{L/B} = 0.8 + 0.2 \times 3/15 = 0.840$

$f_e = 1 - 0.33 \times 0.2/3 = 0.978$

$f_\delta = 1 - \left(\frac{\text{Arc tan } 900/9000}{90}\right)^2 = 0.996$

$f_{\beta,d} = 0.8(1 + 2/3)^{0.1} = 0.842$

$f = f_{L/B}\, f_e\, f_\delta\, f_{\beta,d} = 0.689$



$\Delta R/R_0$	p_p (kN/m²)	ρ/B	ρ (mm)	[illegible]	f	p_f (kN/m²)	Q (MN)
0.005	60	0.0012	3.6	2.25	0.689	93.0	4.18
0.01	120	0.0024	7.2	2.02	0.689	167.0	7.51
0.02	220	0.0048	14.4	1.72	0.689	260.7	11.73
0.03	300	0.0071	21.3	1.54	0.689	318.3	14.32
0.05	450	0.0119	35.7	1.33	0.689	412.3	18.55
0.10	650	0.0238	71.4	1.15	0.689	515.0	23.17
0.20	800	0.0357	107.1	1.02	0.689	562.2	25.30
0.30	900	0.0476	142.8	0.97	0.689	601.5	27.07

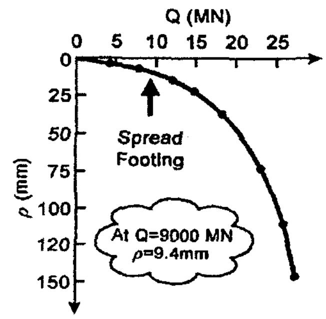


Figure 4. An Example of the Load-Settlement Curve Method

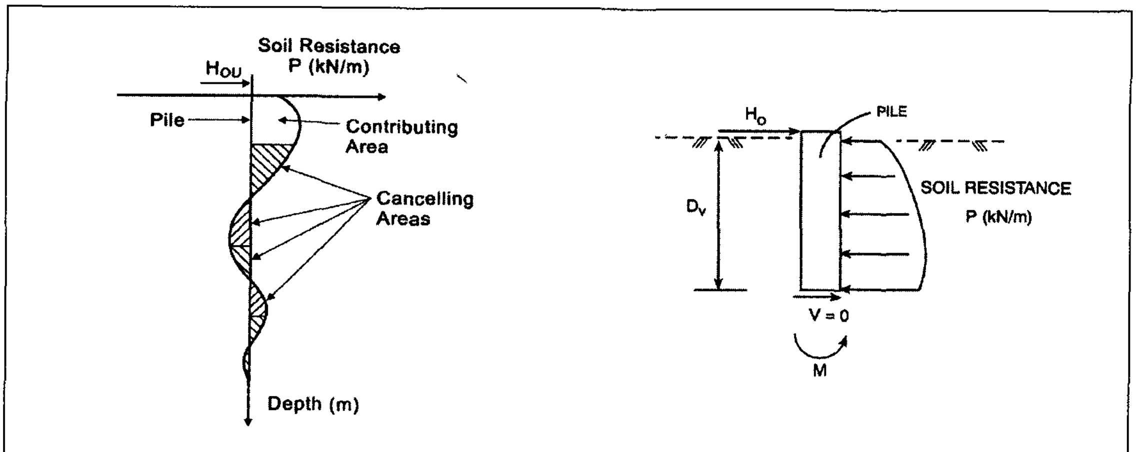
and depth of embedment effect, and pp the pressure in the PMT test corresponding to $\Delta R/R_o$.

4. Prepare the load settlement curve for the footing once the p_f vs. ρ/B curve is known.

Figure 4. shows an example of the load settlement curve method.

DEEP FOUNDATIONS UNDER HORIZONTAL LOADS: S.A.L.L.O.P.

The students and colleagues who worked on this project were Larry Tucker, Srini Donthireddy, and Marc Ballouz. The early part of the project was sponsored by the National Science Foundation. The problem is



(a) Soil Resistance versus Depth (b) Free Body Diagram of Shallow Segment

Figure 5.

the one of a single pile subjected to a horizontal static load. This problem is often solved by assuming that the pile is an elastic member and that the soil can be represented by a series of non-linear horizontal springs called P-y curves. A method was developed at Texas A&M University to obtain the P-y curve directly from the pressuremeter curve. This method was simplified and lead to the Simple Analysis for Lateral Load On Piles(S.A.L.L.O.P.) (Briaud, 1997).

The following observation is the basis for the simplification. A conceptual plot of the soil resistance P per unit length of pile as a function of depth z is shown in Figure 5 a. The sinusoidal nature of the P-z profile is such that the soil resistance alternates direction and essentially cancels itself out except for a shallow zone close to the ground surface which contributes most to the lateral resistance. More specifically there is a depth Dv where the shear force in the pile is zero(Figure 5 b). The horizontal equilibrium of this shallow segment of pile is the basis of the SALLOP method.

The method consists of obtaining the lateral capacity H_{ou}, the horizontal movement at 1/3 of that load, and the maximum bending moment under $H_{ou}/3$. The lateral capacity H_{ou} is defined as the horizontal load corresponding to a horizontal movement at the pile head equal to B/10 where B is the pile diameter. The method was developed on a theoretical basis but was adjusted after studying a database of 20 full-scale pile load tests(Figure 6).

The steps of the method are as follows:

1. Perform preboring PMT tests within a depth corresponding to $2D_v$.
2. Reduce the data and obtain the profile of limit pressures p_l and the profile of first load modulus E_o. Select a design p_l value and a design E_o value from the profiles within the depth D_v. Use 1.5m if D_v is not known.
3. Calculate the zero shear depth D_v by using:

$$D_v = (\pi/4)\ l_o \qquad \text{for} L > 3l_o \qquad (12)$$

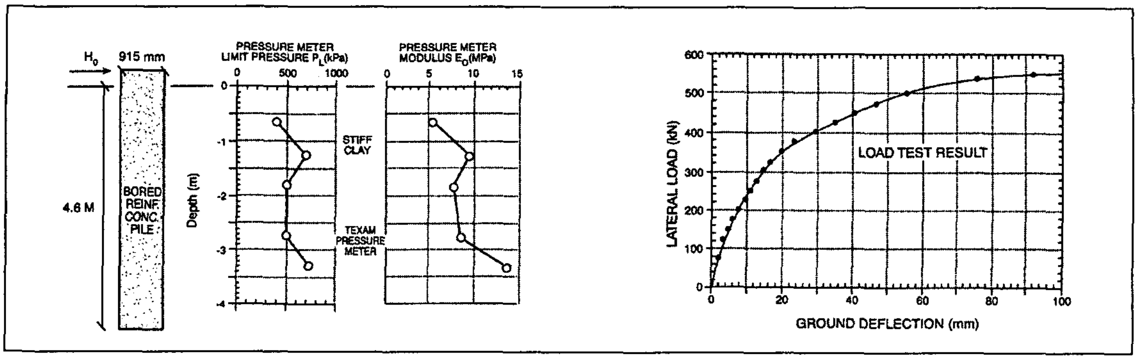


Figure 6. Example of Full Scale load Test in the Database

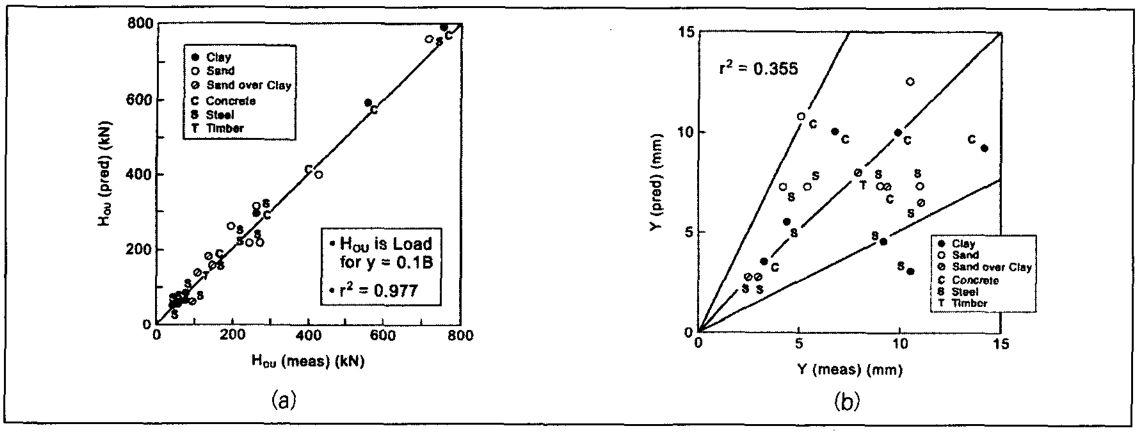


Figure 7. Predicted vs. Measured Results for Horizontal Capacity and Movement.

$D_v = L/3 \qquad \text{for } L < l_o \qquad (13)$

where

$l_o = (4EI/K)^{1/4} \qquad (14)$

and L is the pile length, E the pile material modulus, I the moment of inertia of the pile, and K the horizontal soil modulus taken as $2.3E_o$ after studying the pile database. If the pile length is between l_o and $3l_o$ use linear interpolation.

4. Calculate the lateral capacity Hou by using:

$H_{ou} = \frac{3}{4} p_l B D_v \qquad (15)$

5. Estimate the horizontal deflection y_o under H_o(a safe fraction of H_{ou}) by

$y_o = 2 H_o / l_o K \qquad \text{for } L > 3l_o \qquad (16)$

$y_o = 4 H_o / L K \text{ for} \qquad L > 3l_o \qquad (17)$

These equations are theoretically based and were used to find the best fitting value of K ($K = 2.3 E_o$).

The accuracy and precision of the SALLOP method can be evaluated on Figure 7. If a moment is also applied to the pile head, the reader is referred to Briaud (1997).

DEEP FOUNDATIONS: DOWNDRAG AND BITUMEN COATING

The students and colleagues who worked on this project were Larry Tucker, Randy Bush, Sangseom Jeong, Rajan Viswanathan, Mohamed Quraishi, and Zaid Al Gurgia. The sponsor was the National Cooperative Highway Research Program. The project was aimed at developing a methodology to select bitumen coatings to reduce downdrag. It lead to publications outlining the procedure for uncoated piles and for bitumen-coated piles (Briaud, Tucker, 1997, Briaud, 1997), to a computer program called PILNEG(http://ce-profs.tamu.edu/briaud/pileneg.htm), and to a videotape on how best to coat the piles with bitumen.

The method to calculate the downdrag load and the allowable top load for an uncoated pile is outlined by working through an example by hand in this article. The computer program, the case of a pile group, and the bitumen selection process can be found in the publications mentioned. The example is the one of a single pile(Figure 8) driven in a soil deposit that will experience a settlement profile shown on the figure. The maximum shear stress that the soil can exert on the pile is taken as a constant equal to $25kN/m^2$ to simplify the calculations; it is assumed that the movement will be large enough to mobilize the full friction load in all cases. The point resistance is given by a load transfer curve as shown with a maximum point load of 1000kN. The question is: how much load can be placed on top of the pile if the top settlement must be less than 14 mm; the problem is solved first for the uncoated pile and then for the coated pile.

Uncoated Pile. The first step is to calculate the ultimate capacity of the pile in positive friction.

$$Qu = (25kN/m^2 \times 1.2m \times 30m) + 1000kN = 1900\ kN$$

Lets try a top load Qt of 500kN. The neutral point is found at a depth where the movement of the pile is equal to the movement of the soil. The calculations advance as a trial and error process.

If the neutral point is at a depth of 20m, then according to the soil profile, the movement of the soil at that depth is $w_{NP(soil)} = 50mm$. The pile point carries a load Qp of:

$$Qp = 500kN + (20m \times 1.2m \times 25kN/m^2) - (10m \times 1.2m \times 25kN/m^2) = 800kN$$

For a point load of 800kN, the point movement is given by the point load transfer curve as 4mm. Now it is possible to calculate the pile movement at the neutral point by adding the pile compression between a depth of 30m and a depth of 20m(depth of NP) to the 4mm movement at the point.

$$w_{NP(pile)} = 4mm + (950kN \times 104mm / 0.09m^2 \times 2 \times 10^7 kN/m^2) = 9.3mm$$

Since $w_{NP(soil)} \neq w_{NP(pile)}$ the initial guess of 20m for the depth of the neutral point is in-

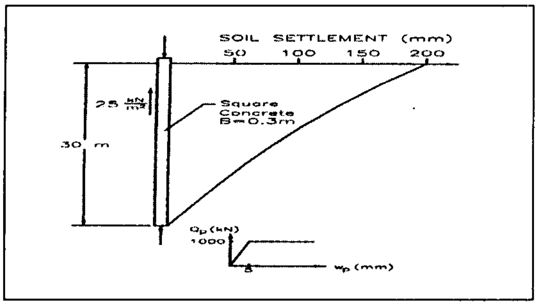


Figure 8. Example Problem for Downdrag Calculations

correct and a new guess is required.

If the neutral point is at a depth of 29m, then according to the soil profile, $w_{NP(soil)}$ = 5mm and Q_p = 500kN +870kN - 30kN = 1340kN. This is not possible since the maximum point load is 1000kN. The pile point will therefore reach the maximum load of 1000kN and vertical equilibrium of the pile gives

$$500kN + X = 1000kN + (900kN - X) \text{ or } X = 700kN$$

This downdrag value corresponds to 23.3m of friction.

If the neutral point is at a depth of 23.3m, then $w_{NP(soil)}$ = 35mm and the movement at the top of the pile is:

$$w_{top} = 35mm + (850kN \times 23300mm / 0.09m^2 \times 2 \times 10^7 kN/m^2) = 46mm$$

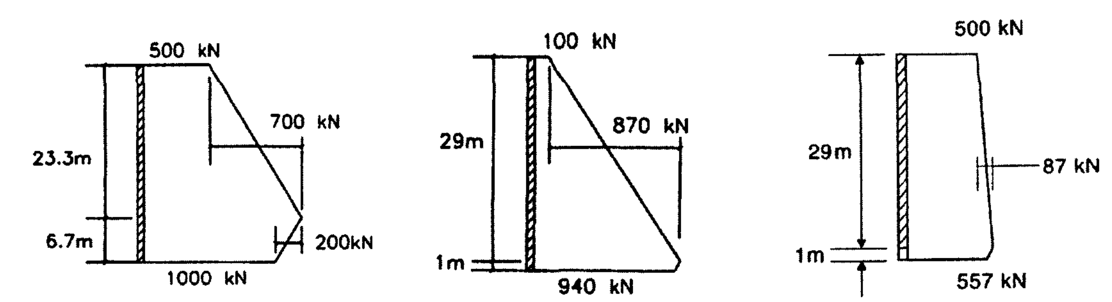
This is more than the allowable movement of 14mm. Therefore the top load must be reduced. Figure 9 (a) gives the load distribution in the pile for a top load of 500kN.

Lets try a top load Qt of 100kN, the same approach is taken. The final iteration is shown here.

If the neutral point is at a depth of 29m, then according to the soil profile, $w_{NP(soil)}$ = 5mm and Q_p = 100kN +870kN - 30kN = 940kN. The movement at the pile point is therefore w_p = 4.7mm

$$w_{NP(pile)} = 4.7mm + (955kN \times 103mm / 0.09m^2 \times 2 \times 10^7 kN/m^2) = 5.2mm$$

In this case, $w_{NP(soil)} \approx w_{NP(pile)}$ and the neutral point position is indeed at a depth of 29m. The



(a) the uncoated pile and a top load of 500kN

(b) the uncoated pile and a top load of 100kN

(c) the bitumen coated pile and a top load of 500kN

Figure 9. Load distribution in the pile for (a), (b), (c)

top movement can now be calculated:

$$w_{top} = 5mm + (535kN \times 29000mm / 0.09\ m^2 \times 2 \times 10^7 kN/m^2) = 13.6mm$$

This settlement is acceptable. The load distribution in the pile is shown on Figure 9 (b). The distribution indicates that this pile which has a capacity of 1900kN, can only be allowed to carry 100kN(Figure 9 (b)) because of the settlement criterion and of the downdrag. In addition this pile has a point resistance under working load which has a very low factor of safety against plunging. In this case, it becomes very advantageous to coat the pile with bitumen or other bond breakers as shown in the following.

Bitumen-Coated Pile. A bitumen coating which reduces the maximum shear stress that the soil can exert on the pile from $25kN/m^2$ to $2.5kN/m^2$ is selected. The coating however must only be applied to the part of the pile which will be subjected to downdrag. The neutral point is found in the same fashion as previously.

Lets try a top load Q_t of 500kN.

If the neutral point is at a depth of 29m, then according to the soil profile, $w_{NP(soil)} = 5mm$ and $Q_p = 500kN + 87kN - 30kN = 557kN$ (Figure 9 (c)). This corresponds to a point movement of 2.8mm and a pile movement at the neutral point w NP(pile) close to $w_{NP(soil)}$

Therefore the neutral point is at a depth of 29m. The settlement of the pile top is:

$$w_{top} = 5mm + (543.5kN \times 29000mm / 0.09m^2 \times 2 \times 10^7\ kN/m^2) = 13.8\ mm$$

This is acceptable. The capacity of the pile is:

$$Q_u = (2.5kN/m^2 \times 1.2m \times 29m) + (25\ kN/m^2 \times 1.2m \times 1m) + 1000kN = 1117kN$$

The factor of safety against plunging failure is therefore 1117/500 = 2.23. Figure 9 (c) shows the load distribution in the pile at working loads. By coating the pile with bitumen, the allowable load has been increased from 100 to 500kN: coating the pile is estimated to increase the pile cost by 20 to 30 %.

SCOUR OF FOUNDATIONS: THE SRICOS-EFA METHOD

The students and colleagues who worked on this project were H.C. Chen, Francis Ting, Kiseok Kwak, Ya Li, Jun Wang, Prahoro Nurtjahyo, Seung Woon Han, Rao Gudavalli, Gengsheng Wei, Yiwen Cao, Suresh Perugu. The sponsor was the Texas Department of Transportation followed by the National Cooperative Highway Research Program.

Scour of foundations is the removal of soil by water flowing around the foundation. Bridges, dams, and other structures built near flowing water can be subjected to scour. The depth of the scour hole must be predicted since it impacts the design of the foundation. The depth of scour in soils depends on three factors: water flow, soil resistance, and geometry of the obstacle. For a constant water velocity v, there is a maximum depth of scour z_{max} which can be reached. The water flow in a river is not constant however, it varies significantly over time but in coarse-grained soils one flood usually will last long enough to create

z_{max}. Therefore, in coarse-grained soils scour calculations to not include time as a factor. The situation is different in fine-grained soils because they are typically more erosion resistant and one flood is usually not long enough to create z_{max} and a time-based accumulation procedure must be used. Indeed using the coarse-grained soils procedure for fine-grained soils can lead to excessive conservatism. The purpose of the project was to develop a method to predict the depth of scour holes as a function of time next to piers founded in fine-grained soils.

A new apparatus was developed to measure on a site-specific basis the erodibility of he soil(Briaud et al., 2001a). This apparatus called the EFA(Erosion Function Apparatus; Figure 10:http://www.humboldtmfg.com/pdf2/hm4000ds.pdf) consists of taking a 76mm diameter Shelby tube sample at the site, bringing it back to the lab without extruding the sample, placing the tube through the bottom of a conduit, pushing the sample to protrude out slightly into the conduit, flowing water on top of the sample at a chosen velocity v, and recording the rate at which the sample is eroded $\dot{z}$. The shear stress τ applied by the water on the soil surface is calculated using Moody's chart(Moody, 1944). By testing the soil at various velocities, the $\dot{z}$ vs. τ curve is obtained and represents a measure of the erodibility of the soil(Figure 10). Typically the erosion rate $\dot{z}$ is zero until the critical shear stress τ_c is reached and then $\dot{z}$ increases as τ increases.

Once the $\dot{z}$ vs. τ curve is obtained, the method to predict the pier scour depth as a function of time proceeds as follows. First, the maximum shear stress tmax around the bridge pier is calculated (Briaud et al, 1999)(a)):

$$\tau_{max} = 0.094\,\rho\,\nu^2\left(\frac{1}{\log Re} - \frac{1}{10}\right) \qquad (18)$$

where ρ is the density of water, ν the mean approach velocity, and Re the pier Reynolds number (νD/v, where D is the pier diameter and v the viscosity of water, ~ $10^{-6}m^2/s$). Second, the initial scour rate $\dot{z}$ corresponding to τ_{max} is read on the $\dot{z}$ vs. τ curve. Third, the maximum depth of scour z_{max} is calculated(Briaud et al., 1999)(a)):

$$z_{max}(mm) = 0.18Re^{0.635} \qquad (19)$$

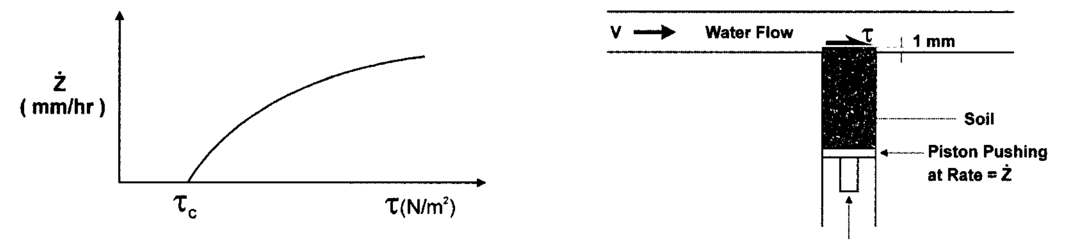
where Re is the pier Reynolds number. Fourth, the equivalent time teq is calculated. The time t_{eq} is defined as the time over which the design velocity ν_{des} would have to be applied for the depth of scour z to be equal to the depth of scour reached after the hydrograph spanning the design life of the bridge tlife has been applied. The time t_{eq} is calculated as (Briaud et al., 2001b):

$$t_{eq}(hrs) = 73(t_{life}t(years))^{0.126}(\nu_{des}(m/s))^{1.706}(\dot{z}mm/hr))^{-0.20} \qquad (20)$$

Fifth, the scour depth $\dot{z}$ versus time t curve is given by:

$$z = \frac{t}{\frac{1}{\dot{z}} + \frac{t}{z_{max}}} \qquad (21)$$

and the depth of scour at the end of the design life of the bridge is calculated by using equation



(a) Diagram of the Apparatus (b) Result of an EFA

Figure 10. The EFA

(21) with the t_{eq}, z_{max}, and z_i values obtained from equation (20) and (19) and from the erodibility curve respectively.

Figure 11 shows an example of calculations. In this case the scour depth after 75years (1.76m) is approximately 50% of the scour depth(3.62m). Note that the equation to obtain the equivalent time teq was developed by regression on Texas Rivers and should not be used outside of that database without proper evaluation of its applicability to other cases. It is also possible to make predictions by applying a detailed velocity history over the design life of the bridge if one is available. This requires the use of the SRICOS computer program (Kwak et al., 2001) which can also consider the case of a layered soil system(Briaud et al., 2001b). An example of results for a run using the SRICOS program is shown in Figure 12.

TIEBACK WALLS: PRESSURES BASED ON DEFLECTIONS

The students and colleagues who worked on this project were David Weatherby, Moonkyung Chung, Yujin Lim, Nak-Kyung Kim. The sponsor was the Federal Highway Adminsitration and Schnabel Foundations. The goal of the project was to advance the state of knowledge on the deflection based design of tieback walls as permanent retaining structures.

The project consisted of reviewing the current state of knowledge, gathering data on full-scale cases of instrumented tieback walls, performing numerical simulations, and building and monitoring for several years a full-scale instrumented tieback wall at the National Geotechnical Experimentation Site at Texas A&M University (Figure 13).

The result was an improved P-y curve approach for such walls with specific recommendations(Briaud, Kim, 1998). The P-y curve analysis can be performed by simulating the construction sequence (several successive P-y analyses) or by wishing the wall in place in a single P-y curve analysis. The construction sequence analysis was developed and compared to the single step analysis. The comparison shows that there is little difference between the two analyses for bending moment predictions but that the deflections are better predicted with the construction sequence ap-

Problem: *Maximum flood velocity = 3 m/s*
Bridge design life = 75 years
Pier diameter = 2 m
Water depth = 5 m
What is the depth of scour after 75 years?

Solution: *S-SRICOS Method*

1. *Results of EFA tests gave the $\dot{z}$ vs τ curve shown.*
2. *Maximum hydraulic shear stress around the pier is:*
   $$\tau_{max} = 0.094\ \rho v^2\left(\frac{1}{\log Re} - \frac{1}{10}\right) = 40\ N/m^2$$
3. *The initial rate of scour $\dot{z}_i$ is read on the EFA curve at $\tau = \tau_{max}$. $\dot{z}_i = 6$ mm/hr*
4. *The maximum depth of scour z_{max} is*
   $$z_{max} = 0.18\ Re^{0.635} = 3626\ mm$$
5. *Equivalent time*
   $$t_e = 73\left(t_{hydro}\right)^{0.126}\left(v_{max}\right)^{1.706}\left(\dot{z}_i\right)^{-0.2} = 573\ hrs.$$
6. *The equation for the $z(t)$ curve is*
   $$z = \frac{t}{\frac{1}{\dot{z}_i} + \frac{t}{z_{max}}} = 1765\ mm \text{ after 75 years}$$
   $z_{75\ years}$ *is 49% of* z_{max}

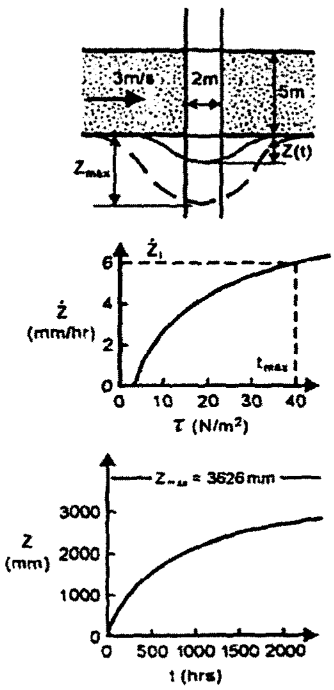


Figure 11. Example of Scour calculations by the S-SRICOS method.

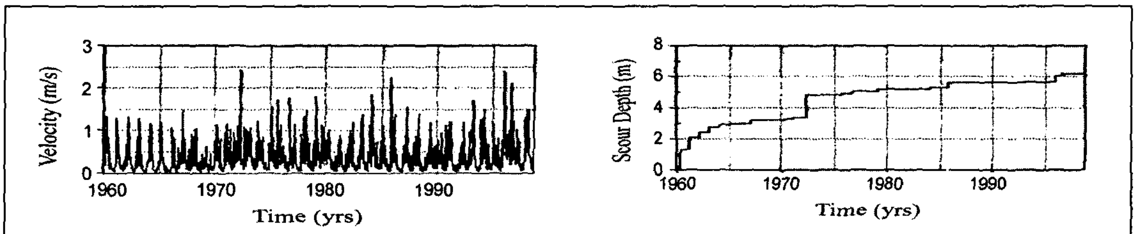


Figure 12. Velocity Hydrograph and Predicted Scour Depth vs. Time Curve for Pier 1 of the Existing Woodrow Wilson Bridge in Washington D.C.

proach. It is also shown that the P-y curve approach, even with the construction sequence, does not give good predictions of movements when the deflections are large; this is attributed to the fact that the P-y curve approach does not include the mass movement of the soil behind the wall. Predicting such movement requires the use of the finite element method(Lim, 1996, Briaud, Lim, 1999). The P-y curve approach can give, through the use of vertical load transfer curves, an idea of the vertical load distribution. This distribution does not include the downdrag which loads the wall due to the sagging of the retained mass. This vertical load component and the associated vertical move-

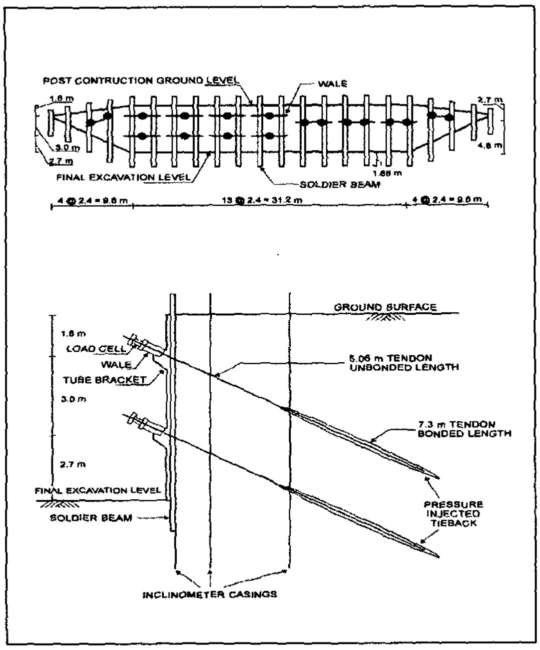


Figure 13. Full Scale Instrumented Tieback Wall at Texas A&M University

ment can create significant horizontal movement by rotation of the wall facing around the anchor bulbs. This leads to the odd conclusion that in order to minimize horizontal movements, one must minimize vertical movements by ensuring that the tieback wall has sufficient vertical capacity, as well as a sufficient tieback capacity of course. A parametric analysis indicated that it is best to place the first anchor as high as possible when the excavation starts and that anchor loads are the most effective way to limit deflections.

One of the major advantages of tieback walls is that the engineer can control movements by choosing the anchor loads. The 0.65 $K_a \gamma H$ pressure that is recommended by Terzaghi and Peck is only one possible value. The engineer can elect to design for different pressures in order to have deflections either larger or smaller than the ones associated with the 0.65 $K_a \gamma H$ pressure diagram. Through the use of case histories and numerical simulations calibrated against the large-scale wall at Texas A&M University, two graphs of an earth pressure factor K as a function of wall deflections were generated. The mean earth pressure for each case history was obtained by adding the anchor loads and dividing by the wall facing area. The earth pressure coefficient K was then simply taken as the pressure divided by γH. The deflection of the top of the wall u_{top} and the wall deflection averaged over its height u_{mean} were normalized with respect to the wall height H. The plots of K vs. u_{top}/H and K vs. u_{mean}/H are shown on Figures 14 and 15 respectively. These graphs enable the engineer to design tieback walls on the basis of deflections. Note that the Terzaghi and Peck 0.65 $K_a \gamma H$ pressure corresponds to a K value of about 0.2 and to a normalized deflection at the top of the wall u_{top}/H averaging about 0.003.

NATIONAL GEOTECHNICAL EXPERIMENTATION SITES IN THE USA

There are 5 National Geotechnical Experimentation Sites(NGES) in the USA. They are located at Texas A&M University, on Treasure Island near the University of California at Berkeley, at the University of Massachussetts, at Northwestern University, and near Auburn University. The purpose of

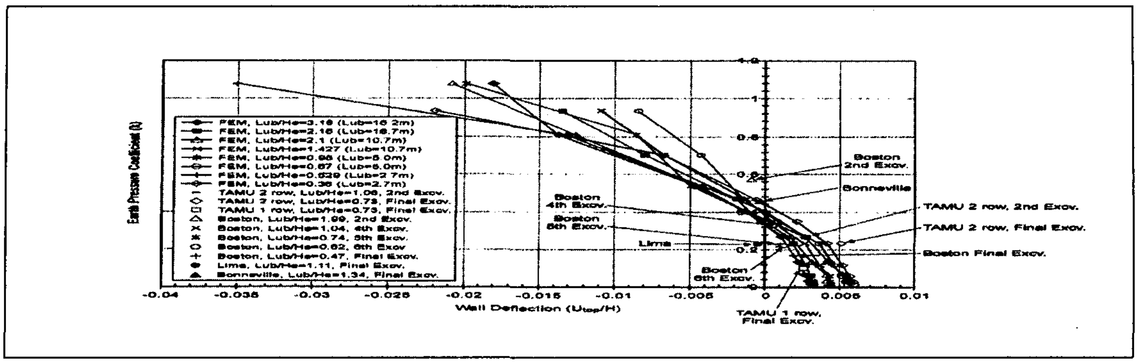


Figure 14. Mean Earth Pressure Coefficient vs. Normalized Top Wall Deflection

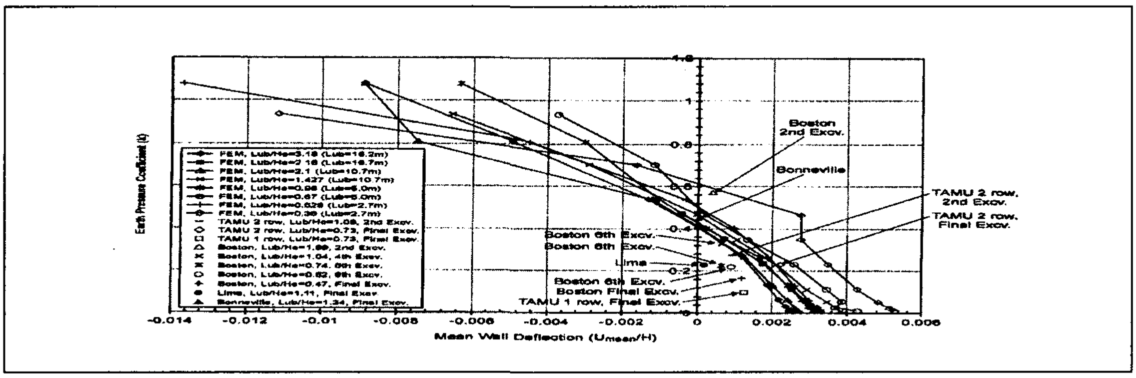


Figure 15. Mean Earth Pressure Coefficient vs. Normalized Mean Wall Deflection

these sites is to investigate the behavior of geotechnical structures at full scale in a research environment; it was initiated in the early 1990s under the sponsorship of the Federal Highway Administration and the National Science Foundation. The soil at these sites is very well characterized and researchers throughout the USA and the world can come and conduct controlled research experiments. The results are organized in a database maintained by the Federal Highway Administration. The projects include shallow foundations, deep foundations, retaining walls, embankments, culverts as well as innovative in-situ testing and non-destructive detection methods. Information on these sites can be found at http://www.unh.edu/nges/