Pipe Jacking Forces in Soft Ground Construction During Utility Installation Related to Central Artery/Tunnel Project Construction

Based on an analysis of certain case studies and the application of theoretical models, it may be possible to accurately predict jacking forces in different soils.

JOHN M. PECORA, III &
THOMAS C. SHEAHAN

Utility construction in urban areas and along highways has for many years included the use of trenchless technology methods for installation. Pipe jacking, as applied to large-diameter pipes, is the method by which a pipe is pushed through the ground by hydraulic rams or jacks, and as the pipe advances, soil is removed by augers, hydraulic excavators or hand-digging using pneumatic tools and shovels. Pipe jacking is clearly most advantageous over cut-and-cover installation methods in those situations where surface-level operations cannot be interrupted. It has gained tremendous popularity in the past twenty-five years due to the significant amount of utility installation and relocation required for urban and suburban construction projects.

The jacked pipe is installed from what is called a *jacking pit* that is excavated and shored, and has a thrust block constructed on its floor, which bears against the back wall of the pit, opposite the wall that will be penetrated with the jacked pipe. Because of the critical roles played by the thrust block and jacking pit in the installation, it is imperative that these elements be appropriately designed for the

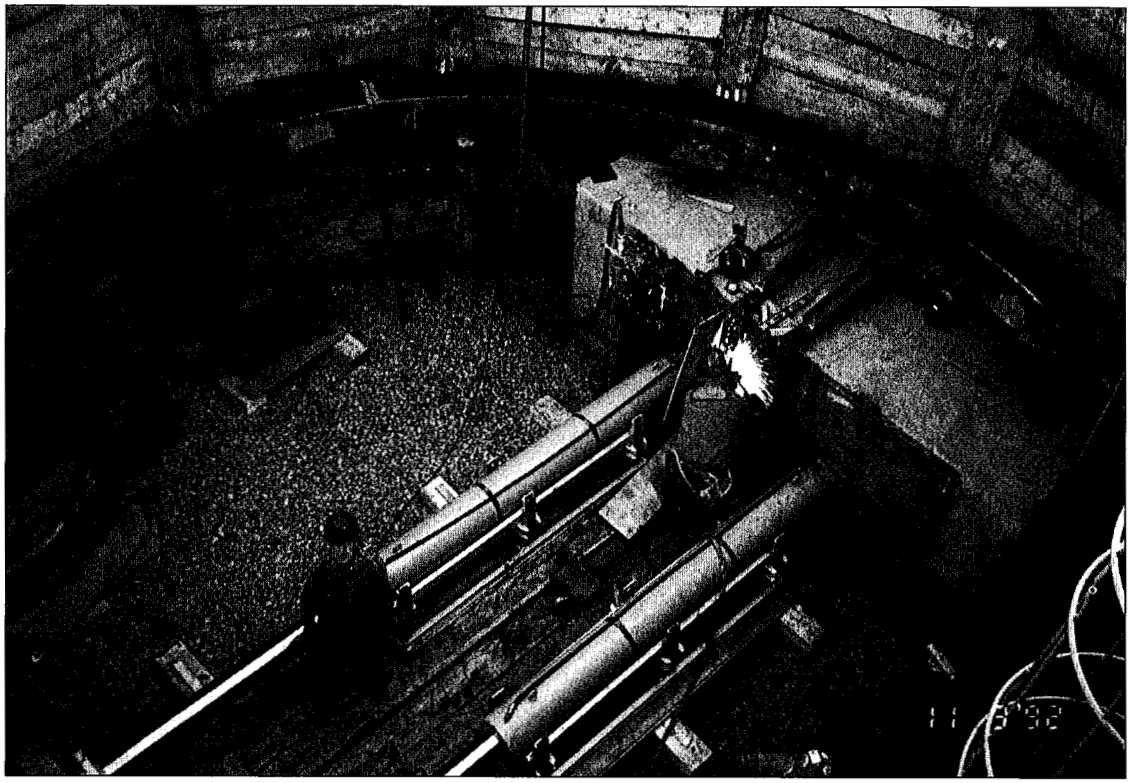

FIGURE 1. Circular jacking pit showing the hydraulic jacks and the concrete reaction block.

anticipated jacking forces. Jacking forces are a combination of the normal force acting on the front cutting edge of the pipe and the frictional force acting along the soil-pipe interface surface. However, while the mechanics of pipe jacking are reasonably simple, and there are methods for computing the forces associated with each component, there are only limited data and case studies available that illustrate how jacking forces behave in particular soils.

Review of Pipe Jacking Procedures

The installation of a jacked utility pipe with a large diameter follows a fairly standard set of procedures.[1-3] Prior to any jacking operation, jacking and receiving pits are excavated and associated pit construction is completed. A jacking or receiving pit is typically a shored excavation pit, with the shoring consisting of a variety of earth support systems — *i.e.*, soldier piles and lagging, steel sheeting or a modular box system (see Figure 1). The jacking pit is typically located at the lower elevation end of the pipe so that any water encountered at the jacking face drains back into the jacking pit where accommodations for drainage can be made.

The thrust block, typically consisting of reinforced concrete or steel sections with a cover plate, is then installed in the jacking pit. In most applications, the thrust block is a vertical plane on the jacking pit's back wall against which the hydraulic rams push. The block, in turn, distributes the jacking force across the back wall of the jacking pit (see Figure 1). The jacking pit also contains the jacking equipment, including the jacking shield or boring head, thrust ring, hydraulic rams and spoils cart. Once the surveyed alignment is provided, the jacking shield is set opposite the thrust block on line (axial location) and on grade (elevation location). The hydraulic rams and thrust ring are set between the thrust block and the shield or boring head. The thrust ring distributes the point loads of the hydraulic rams around the jacking shield, boring head or casing pipe.

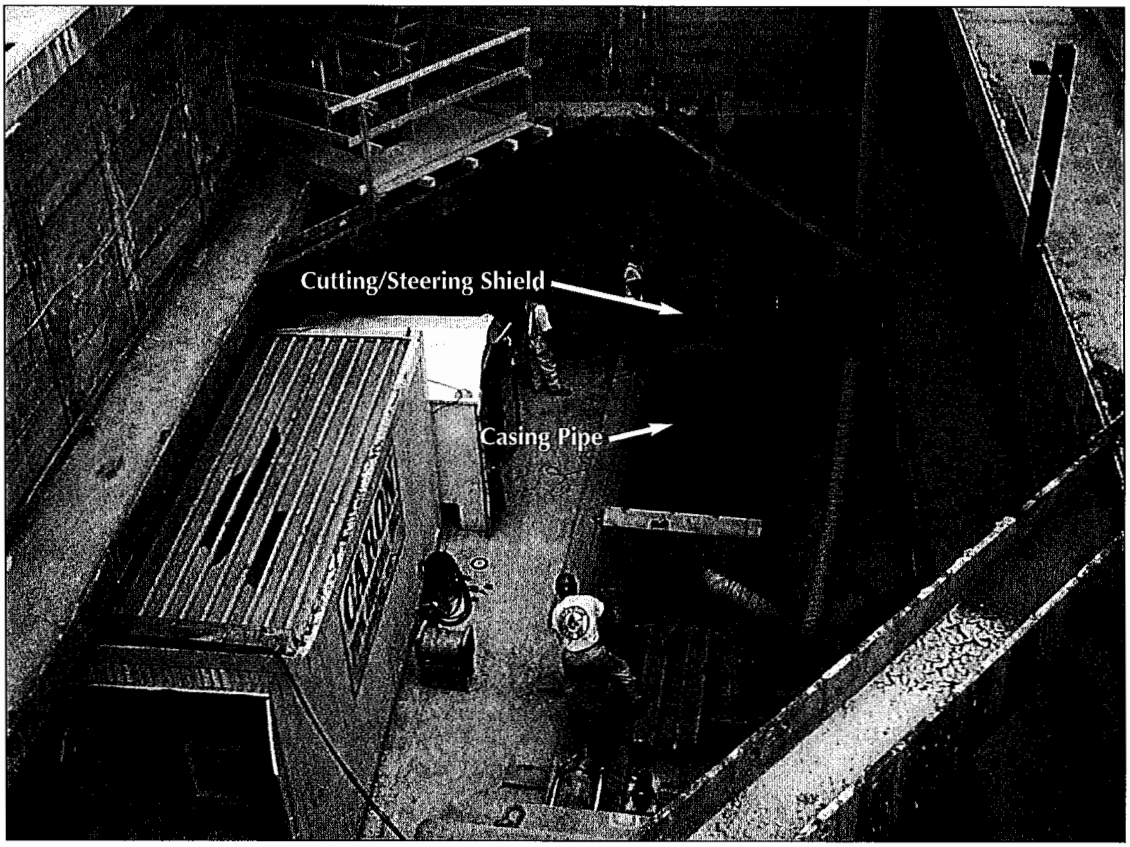

FIGURE 2. Jacking operations in the jacking pit showing the cutting shield and one pipe section being jacked.

After the equipment is in position in the jacking pit, jacking is set to begin. The shield, or boring head, is advanced up to the earth support wall where the earth support system is cut away around the circumference of the shield or boring head. The hydraulic rams extend to advance the lead element (shield or boring head), and spoil is either excavated from the shield or transported back by the boring head where it is removed (see Figure 2). With each extension of the hydraulic rams, the space between the thrust ring and the rams increases, and this space is taken up by steel spacer blocks. Spacer blocks are installed until the gap created by the advanced shield or boring head and the retracted position of the rams is large enough to accommodate another section of pipe. The process repeats itself with the thrust ring now bearing against the inserted pipe section instead of the shield or boring head.

Once the jacked tunnel reaches the receiving pit, a hole is cut in the earth support wall of the pit and the shield is advanced into the pit and removed. With jacking complete from the jacking pit to the receiving pit, a utility tunnel now exists as a final service line, or as a conduit for other utilities.

The Central Artery/Tunnel (CA/T) Project pipe jacking case studies presented here involved the use of conventional open face shields, which are those that permit direct access to the jacking face for observing conditions at the jacking face during excavation as well as observing the removal of any obstructions. The majority of the obstructions that have been encountered during the jacking operations for the CA/T Project include old granite foundations, timber pier pilings and existing or abandoned utilities. For example, Figure 3 shows personnel inside a jacked tunnel lining that has an open face, inspecting an

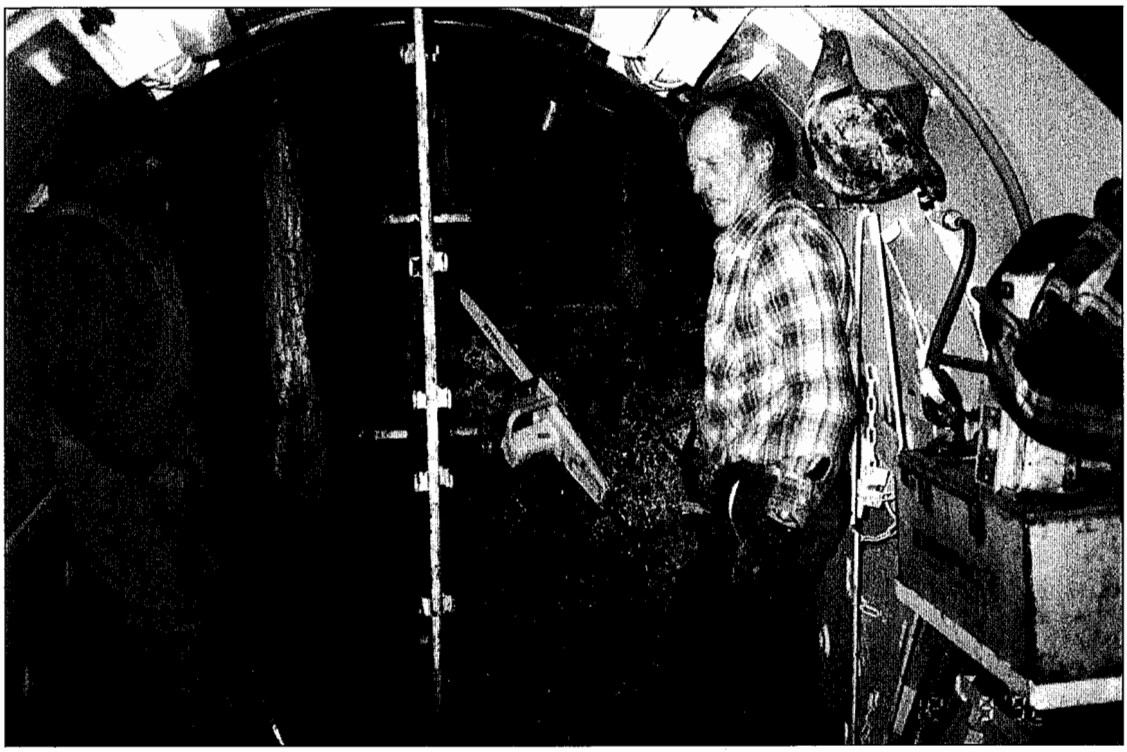

FIGURE 3. Construction personnel inside a pipe inspecting an abandoned timber pile seawall.

abandoned timber pile seawall that was encountered during excavation.

To improve safety in open face shields, a number of modifications can be made to enhance face stability and to temporarily isolate the jacking face from the inside of the pipe. On the CA/T Project, these modifications included:

- *Breast plates,* which are radial, steel "fins" extending from the inside surface of the jacking shield toward its center. They provide stability to soft and sandy soils, and as the jacking shield is advanced, they segment the penetrated soil for sectional removal.
- *Poling plates and spiles* are reinforcing posts or plates that are installed around the circumference of the shield's leading edge that point in the direction of the jacking. These plates help support the overburden soil and reduce soil lateral pressures on the jacking face.
- *Shield face doors* are manually or hydraulically actuated covers on the front face of the jacking shield.[2] They close off the entire jacking face or a portion thereof, and are then opened or removed during excavation operations. Figure 4 shows an open face shielded with eight section doors, each independently actuated using a hydraulic cylinder. With such an arrangement, laborers can selectively excavate parts of the face.

Components of the Pipe Jacking Force

The jacking forces that are required for open face jacking depend on the penetration resistance of the jacking shield (commonly known as the cutting edge resistance), and the interface frictional resistance between the outer pipe surface and the surrounding soil. Additional factors must be considered when using compressed air or fluid pressure balance methods, but these factors are not relevant to the CA/T Project case histories presented here since these methods were not applied in these cases.

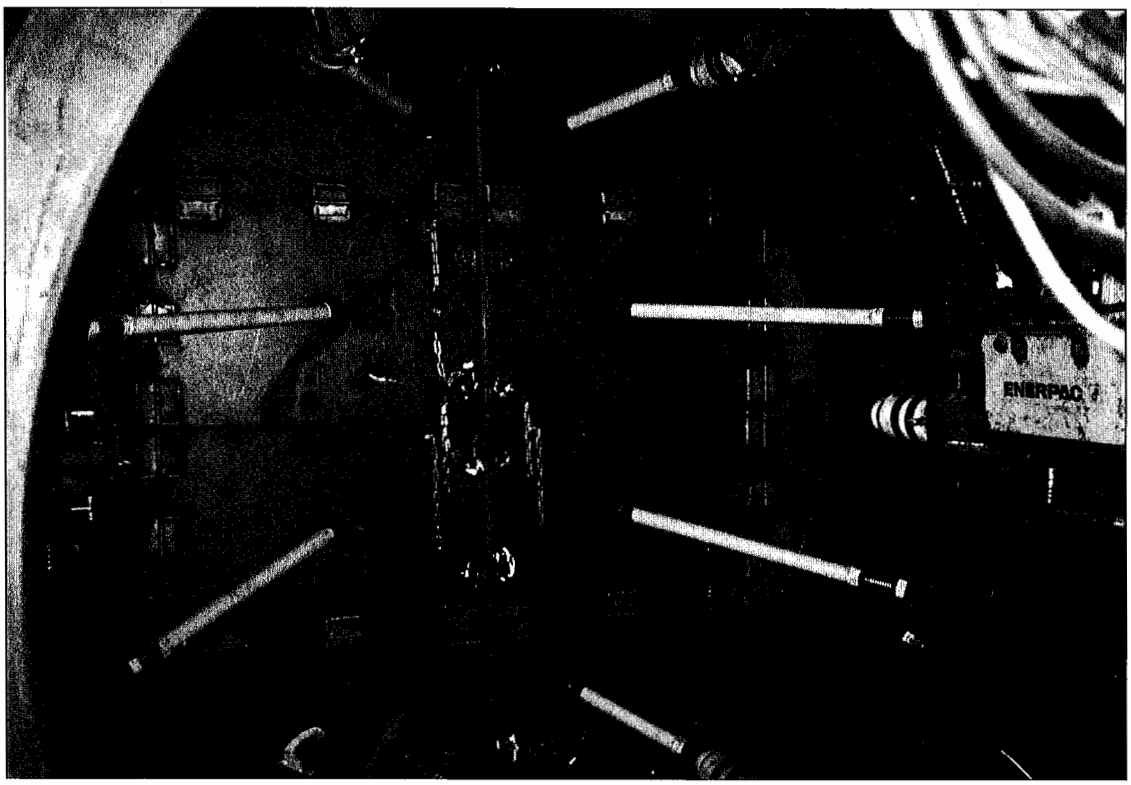

FIGURE 4. A view of a shield face with independently actuated sectional doors.

Cutting Edge Resistance. There are three common methods used for determining the cutting edge resistance. Each method employs at least some degree of empirical data, and tends to be a variation on determining the penetration resistance as a product of the cutting edge area and the soil resistance at the tip (based on soil type). As outlined by Stein *et al.*, the three most common methods used in the pipe jacking industry are the Herzog, Scherle and Weber methods.[1]

Herzog suggested the following formula can be used to determine the cutting edge resistance, P_s:[1]

$$P_s = \pi \cdot D_s \cdot t_s \cdot p_s \quad \textbf{(1)}$$

where:

D_s = external diameter of the shield
t_s = thickness of the shield cutting edge
p_s = tip resistance for specified soil type

Table 1 (on the next page) shows the values of p_s that Herzog developed based on the load-bearing capacity of cast-in-place piles. When these tip resistance values were applied in the geometric cutting edge resistance formula (see Equation 1) and the results were compared to data from previous pipe jacking projects, there was reasonably good agreement. Thomson provides further details on predicting Herzog resistance values in rock.[4]

Scherle's method for determining cutting edge resistance uses a resistance factor, f_1, of between 300 to 600 kN/m^2.[1] The overall cutting edge resistance is taken as the product of the cross-sectional area of the excavation face and a cutting edge factor, f_1:

$$P_s = ((\pi \cdot D_s^2)/4) \cdot f_1 \quad \textbf{(2)}$$

where:

f_1 = cutting edge resistance factor, ranging from 6,265 psf (300 kPa) for soils with low penetration resistance (*i.e.*, loose silts, clays and sands) to 12,531 psf (600 kPa) for soils with high pen-

TABLE 1.
Tip Resistance for Calculating Pipe Jacking Cutting Edge Resistance

Soil Type	p_s, psf (kN/m^2)
Soft Rock, Cemented Soil	250,625 (12,000)
Gravel	146,198 (7,000)
Dense Sand	125,313 (6,000)
Medium Sand	83,542 (4,000)
Loose Sand	41,771 (2,000)
Stiff to Hard Clay	62,656 (3,000)
Stiff to Firm Clay	20,885 (1,000)
Silt, Alluvium	8,354 (400)

Note: Data presented are derived using Herzog's method as given in Ref. 1

etration resistance (*i.e.*, dense sand, gravel and soft rock)

Clearly, Scherle's method is better suited to closed faced pipe jacking operations rather than open face since it treats the entire jacking face as the penetrating area rather than just the pipe's circumferential area as in Herzog's method (see Equation 1). Scherle's method also requires considerable experience to estimate f_1 values that will be reasonably accurate.

The third method for determining cutting edge resistance is known as Weber's method.[1] It is intended for use with pipe jacking operations in which a boring or cutting head is used, and was developed based on statistical data from construction site records as well as laboratory tests:

$$P_s = (\gamma \cdot z \cdot tan\phi + c) \cdot \lambda_c \cdot D \cdot d \qquad (3)$$

where:

γ = soil unit weight
z = height of soil cover over jacked pipe
ϕ = soil's angle of internal friction
c = soil cohesion
D = conveyor screw diameter
d = cutting edge thickness
λ_c = coefficient of load bearing capacity (unitless), obtained from Figure 5

Weber attempted to further refine his correlation by differentiating between pipe jacking using augers that are at the cutting edge, and those situations when the auger is retracted inside the cutting head. However, Equation 3 did not correlate well to laboratory data when the auger was placed at the cutting edge. Figure 6 (on page 36) shows a comparison among all three methods for computing P_s versus the cutting edge diameter, including the results from Weber's method for auger positions both at the cutting edge and retracted. These results are for a dense sand with γ equal to 22 kN/m^3 (ϕ equal to 42 degrees), a cutting edge thickness of 2 centimeters and the height of soil cover at 3 meters. These results show that Herzog's method is clearly the most conservative, and agrees quite well with Weber's method for the retracted auger. Scherle's method, when the lower limit of f_1 equal to 300 kN/m^2 is used, is least conservative. When the upper limit of f_1 equal to 600 kN/m^2 is used in Scherle's method, the resistance approaches that obtained using Herzog's method for larger diameter cutting edges.

Frictional Resistance at the Soil-Pipe Interface. The frictional resistance acting on the pipe surface at the soil-pipe interface acts along the entire pipe and cutting shield length. As in a typical friction law, the frictional resisting force, R, is the product of the skin friction

stress, M, between the tunnel shield/ jacking pipe with the surrounding soil, the pipe circumference and the jacking distance, or:

$$R = M \cdot D_s \cdot \pi \cdot L \qquad \textbf{(4)}$$

where:

M = skin friction ($\mu \cdot N$)
μ = soil-pipe interface coefficient of friction (unitless)
N = normal stress on pipe
D_s = outside diameter of tunnel shield or jacking pipe
L = jacking distance

Clearly, the only uncertain parameter in this equation is the friction coefficient between the soil and the tunnel wall. The Pipe Jacking Association suggests that, as a guide, the skin friction stress, M, be in the range of 111 to 556 psf (5.3 to 26.6 kN/m^2) of external circumferential pipe surface area around its circumference.[5] This range is based largely on historical data from pipe jacking contractors. Thomson has published a more detailed table of these M values, as given in Table 2 (on page 37).[4]

The wide range in values for particular soils as noted in Table 2 limits the value of such a listing. A more analytical approach to the determination of M is by using established values of μ and an appropriate normal stress on the pipe. Table 3 (on page 38) presents typical values for μ for static and sliding friction with different soil and pipe material interfaces, as well as for pipe jacking using bentonite slurry as a lubricant.

There have been numerous methods developed to determine the mean normal force acting on the pipe surface. This determination is complicated by the variable direction of the normal force from predominantly vertical on the pipe's crown and invert to horizontal on the sides of the pipe. Terzaghi *et al.* gave the following expression for the vertical stress, p_{Ev}, based on Terzaghi's so-called "trap door" experiment:[6]

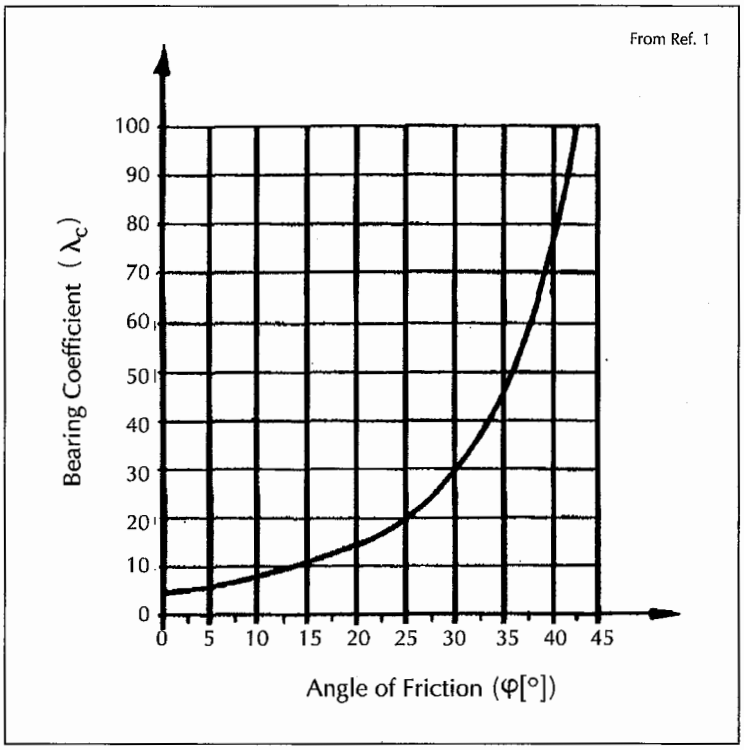

FIGURE 5. Weber's coefficient of loading bearing capacity for edge resistance.

$$p_{Ev} = \gamma \cdot h \cdot \kappa \qquad \textbf{(5)}$$

where:

γ = soil unit weight
h = height of soil cover above pipe crown
κ = coefficient of soil load, given by $[1 - exp(-2K tan\delta \cdot h/b)]/(2K tan\delta \cdot h/b)$ with K equal to the coefficient of lateral earth pressure above the pipe (unitless), δ equal to the angle of interface friction (this varies from $\phi/2$ to ϕ for the soil [unitless]) and b equal to the influence width of soil above pipe, given by $D_s \cdot [0.5 + tan(45 - \phi/2)]$ — however, the value of b is more conservatively and more easily computed as equal to $\sqrt{3} \cdot D_s$.

In addition, Stein *et al.* have suggested a more complex method for determining the lateral stress on the pipe.[1] A reasonable estimate for

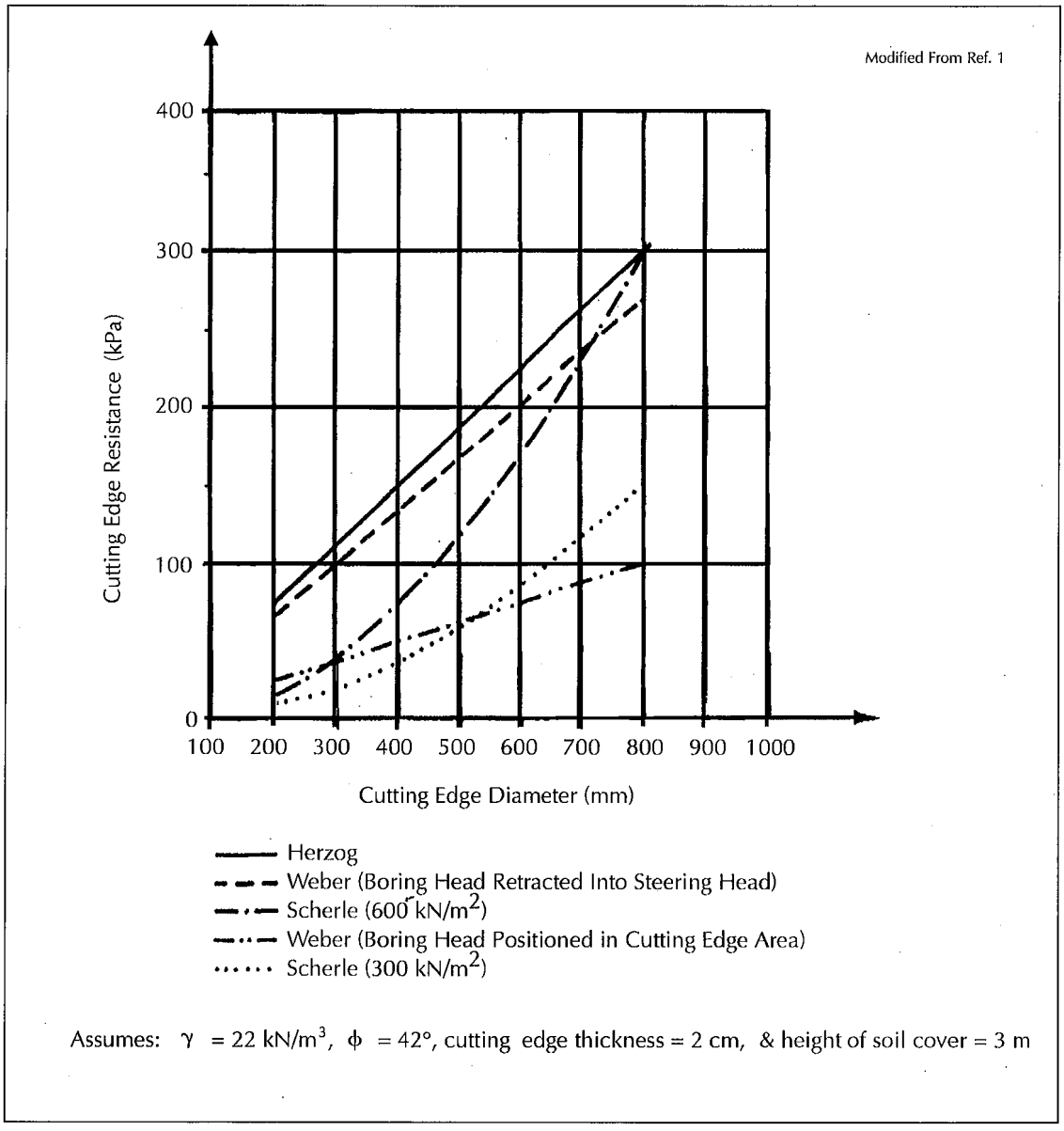

FIGURE 6. A comparison of the three different edge resistance computation methods.

the average stress around the pipe circumference can be determined by assuming a value of K equal to 0.5 for the coefficient of lateral earth pressure, then taking the average of p_{Ev} as computed in Equation 5 and the associated lateral pressure for this assumed K value — one-half of p_{Ev}. The resulting average stress around the pipe is then given is simply as three-quarters of p_{Ev}.

Because of the uncertainties associated with so many input parameters, it is apparent that this more fundamental approach may introduce more errors than it resolves. Hence, empirical data tables (such as Table 2) may be more useful to engineers and contractors if large variations can be avoided for a particular soil.

Pipe Jacking Data Details From Selected Cases in the CA/T Project

Pipe jacking cases are presented from four CA/T Project contract sections — C14A1,

TABLE 2.
Soil-Pipe Interface Skin Friction Values

Soil Type	Frictional Resistance Stress, M (kN/m^2)			
	France	U.K.	Australia	Germany
Rock	—	2 to 3	1	—
Boulder Clay	—	5 to 18	—	2.8 to 18.4
Firm Clay	8 to 10	5 to 20	5 to 7.5	5.3 to 9.3
Wet Sand	—	10 to 15	13	2.2 to 16.1
Silt	17	5 to 20	—	4.9 to 8.5
Dry Dense Sand	—	—	—	1.1 to 6.7
Dry Loose Sand	20 to 30	25 to 45	—	—
Fill	—	≤45	—	—
Dense Gravel	50	—	—	2.3 to 6.4

Note: Data From Ref. 4

C14A2, C15A1 and C09A4. Table 4 (on page 39) summarizes the details and soil conditions for these particular contracts. Detailed data on these sections are provided in a previous study.[7]

The C14A1 and C14A2 contracts were for the New East Side Interceptor (NESI) Sewer Project, located in the South Station/Financial District, Chinatown and South Bay sections of Boston. These contracts involved the installation of four 78-inch (1.98-meter) diameter sewer pipes, each with an outside diameter of 94.5 inches (2.4 meters). Through the course of construction, jacking operations were occasionally interrupted by obstructions in the path of the line, including timber piles and abandoned granite seawalls (for example, see Figure 3). The soil type was a mixed face of historic fill, organic silt and clay. Jacking for these lines took place approximately 25 feet (7.6 meters) below existing grade.

The C15A1 contract consisted of jacking six utility pipes near Boston's North End. The specific locations for pipe jacking were at the entrance to the Callahan Tunnel, the egress from the Sumner Tunnel and beneath North Washington Street. At the Callahan Tunnel, two 60-inch (1.52-meter) diameter steel casings, 161 feet (49 meters) long and parallel to each other, were jacked approximately 35 feet (10.7 meters) beneath the tunnel entrance. Beneath the Sumner Tunnel egress, two 125-foot (38-meter) long steel casings were also jacked 35 feet (10.7 meters) below the surface — one casing that was 60 inches (1.52 meters) in diameter, and the other was 48 inches (1.22 meters) in diameter. Finally, on this contract, pipe jacking was performed beneath North Washington Street to install a reinforced concrete sewer pipe with an outside diameter of 80.5 inches (2.05 meters) and an inside diameter of 66 inches (1.68 meters), as well as a 48-inch (1.22-meter) diameter steel pipe. Both of these pipes were jacked approximately 22 feet (7 meters) below existing grade.

The soil encountered under the Callahan Tunnel was a stiff silty clay, and the soil encountered at the Sumner Tunnel was a medium to stiff mixture of silty clay and sand and gravel (classified as "glaciomarine deposits"). For the North Washington Street section, the soil encountered when jacking the 80.5-inch (2.05-meter) diameter sewer pipe was a mixed face of loose to medium stiff organic silt and clay on the bottom half, and historic fill on the upper half. The entire length of the 48-inch (1.22-meter) steel sleeve was in

TABLE 3.
Standard Values for Coefficient of Friction of Pipes in Different Soils

Interface Type	μ
For Static Friction	
Concrete on Gravel or Sand	0.5 to 0.6
Concrete on Clay	0.3 to 0.4
Asbestos Cement on Gravel or Sand	0.3 to 0.4
Asbestos Cement on Clay	0.2 to 0.3
For Sliding Friction	
Concrete on Gravel or Sand	0.3 to 0.4
Concrete on Clay	0.2 to 0.3
Asbestos Cement on Gravel or Sand	0.2 to 0.3
Asbestos Cement on Clay	0.1 to 0.2
For Fluid Friction GBentonite)	0.1 to 0.3*

Note: Data are from Ref. 1
* When using bentonite suspension as supporting and lubricating fluid, μ depends on the liquid limit of the suspension.

the organic silt and clay layer. While its invert elevation was the same as that of the 80.5-inch (2.05-meter) pipe, it did not cross into overlying historic fill layer.

The primary scope of the C09A4 contract was tunnel jacking for Interstate 90, but it also included pipe jacking for utility relocation. A 72-inch (1.8-meter) steel sleeve was jacked for two sewer force main lines at 35 feet (10.7 meters) below the existing ground surface. The soil type encountered was a stiff to hard clay.

Pipe Jacking Data & Analysis

During each of the jacking operations, measurements were made of the jacking force required to advance the pipe. This force was computed based on the maximum jack hydraulic pressures during a particular drive (*i.e.*, the incremental advance of the pipe, typically 0 to 4 feet, or 0 to 1.2 meters). Figure 7 (on page 40)shows a typical jacking force versus jacking distance record, in this case for the Callahan Tunnel East 60-inch-diameter steel sleeve. It should be noted that the jacking force was composed of an edge resistance (which was assumed to remain approximately constant during jacking), and the frictional resistance that increases as the pipe was advanced. Therefore, in order to assess the validity of previously described methods for predicting jacking edge and frictional resistance, each of the pipe jacking cases shown in Table 4 was analyzed using Herzog's edge resistance method (see Equation 1 and Table 1), and the friction law (see Equation 4 and Table 3) adapted to each case, as follows:

- To compute the edge resistance, a tip resistance value, p_s, from Table 1 was assumed based on soil type. This value was multiplied by the edge area of the pipe to get the resulting edge force, P_s.
- The vertical stress on the pipe, p_{Ev}, was computed using Equation 5, with the soil unit weight, γ, in all cases assumed to be 120 lb/ft^3 (18.9 kN/m^3), and κ computed from $[1 - exp(-2K\tan\delta \cdot h/b)]/(2K\tan\delta \cdot h/b)$, assuming the influence zone, b, above the pipe to be $\sqrt{3} \cdot D_s$, where D_s is the outside diameter of the pipe.
- The average normal stress on the pipe, N, was computed as 0.75 times the p_{Ev}. This assumption was based on a coefficient of

TABLE 4.
Summary of CA/T Project Pipe Jacking Details & Analyses

Contract	Pipe Properties: Outside Dia., in. (m)	Pipe Properties: Type**	Location	Jacking Length, ft (m)	Depth, ft (m)	Soil Type(s)	Friction Analysis Input & Results*: Herzog's p_S, psf (kPa)***	Herzog's Resultant P_S, lb (kN)	Avg. Normal Stress on Pipe, psf (kPa)	Friction Coeff., μ	Avg. Meas. Friction, psf (kPa)	Computed Friction, psf (kPa)
C14A1	94.5 (2.4)	PCCP Sewer	Various	129 (39.3)	25 (7.6)	Organic Silts, Peat & Clay	8,354 (400)	17,216 (76.6)	1,548 (74.1)	0.15	252 (12.1)	232 (11.1)
C14A2	94.5 (2.4)	PCCP Sewer	S. Pit to CSMH#14	123 (37.5)	25 (7.6)	Organic Silts, Peat & Clay	8,354 (400)	17,216 (76.6)	1,548 (74.1)	0.15	254 (12.2)§	232 (11.1)
	94.5 (2.4)		CSMH#13 to #14	213 (64.9)							127 (6.1)	
	94.5 (2.4)		CSMH#13 to #12	285 (86.9)							159 (7.6)	
C15A1	60 (1.52)	Steel Sleeve	Callahan Tunnel E.	160 (48.8)	35 (10.7)	Stiff BBC	40,000 (1,915)	52,339 (232.8)	1,479 (70.8)	0.20	290§§ (13.9)	296 (14.2)
	60 (1.52)		Callahan Tunnel W.	160 (48.8)							317 (15.2)	
	60 (1.52)		Sumner Tunnel E.	125 (38.1)		Stiff BBC to Sand/ Gravel	40,000 (1,915)	52,339 (232.8)	1,479 (70.8)	0.25	430 (20.6)	370 (17.7)
	48 (1.22)		Sumner Tunnel W.	122 (37.2)				41,871 (186.3)	1,270 (60.8)		264 (12.6)	318 (15.2)
	80.5 (2.05)	PCCP Sewer	N. Wash. St.	282 (86.0)	22 (6.7)	Top Half Historic Fill, Bottom Organic Silt	8,354 (400)	14,666 (65.2)	1,752 (83.9)	0.10	164 (7.9)	175 (8.4)
	48 (1.22)	Steel Sleeve	N. Wash. St.	263 (80.2)				8,745 (38.9)	1,270 (60.8)		91 (4.4)	127 (6.1)
C09A4	72 (1.83)	Steel Sleeve	Pit D to B/C, I-90 Jacked Tunnels	241.5 (73.6)	35 (10.7)	Stiff to Hard Clay	62,656 (3,000)	98,380 (437.6)	1,649 (79.0)	0.25	419 (20.1)	412 (19.7)

Notes: * All analyses assumed the following: unit weight, γ = 120 pcf (18.9 kN/m^3); interface friction angle, δ = 25°; width of influence area, $b = \sqrt{3}\ D_S$; and coefficient of lateral earth pressure, K = 0.5.

** PCCP = Portland cement concrete pipe.

*** From Table 1.

§ Excludes net frictional stress from first 11 ft (3.4 m) and from readings during obstructions.

§§ Excludes net frictional stress from first 20 ft (6.1 m).

lateral earth pressure of 0.5, and derived by simply taking the average between vertical and horizontal stress on the pipe from the surrounding soil.

- The net jacking force at any given point due to friction alone (*i.e.*, the total jacking force minus the edge resistance computed using Herzog's edge resistance) was divided by the cumulative surface area of the jacked pipe to obtain the frictional stress per pipe surface area.
- The frictional stress obtained in the previous step was compared to the computed frictional stress from Equation 4, with an

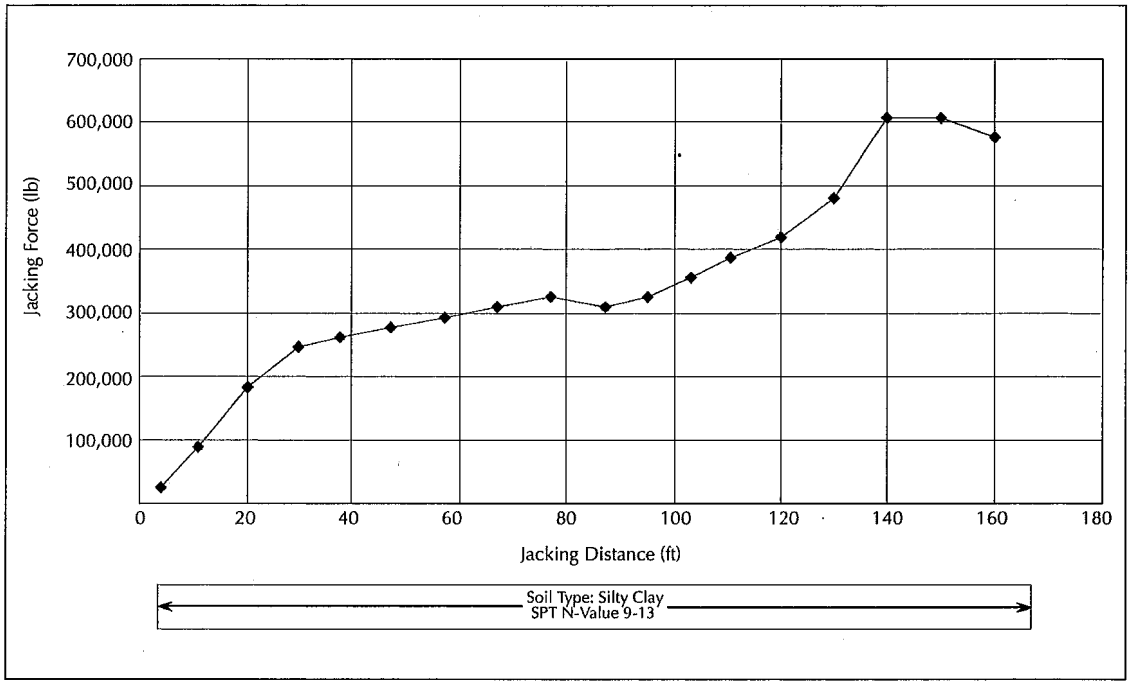

FIGURE 7. Typical total jacking force versus length jacking distance from the Callahan Tunnel East steel pipe installation.

assumed friction coefficient based on values in Table 3.

Table 4 summarizes assumed input parameters and the results of this analysis, which produces the average measured frictional resistance for each case. The most significant assumptions are the values of the edge resistance stress, p_s, from Table 1, and the friction coefficient from Table 3. The last two columns of Table 4 compare the average net frictional stress (*i.e.*, the frictional jacking force per pipe surface area jacked) for each pipe jacking stage undertaken, as determined from field data, to the computed values from Equation 4. This comparison is further illustrated in Figures 8 through 10, which show the summary plots of the net frictional stress for the three groups of jacked pipes versus pipe jacking length. The jacking stages were grouped as follows: Figure 8, the NESI sewer; Figure 9, Callahan Tunnel-Sumner Tunnel (CT-ST); and Figure 10, the North Washington Street-C09A4 (NWS) pipe jacking stages.

The set of NESI pipe jacking stages (Contracts C14A1 and C14A2 in Table 4 and Figure 8) were all 94.5-inch (2.4-meter) outside diameter pipes jacked in organic silts, peats and clays. Based on these soils, a Herzog edge resistance, p_s, equal to 8,354 psf (400 kPa), was selected from Table 1, and a friction coefficient, μ, equal to 0.15, was used to compute the theoretical frictional resistance using Equation 4. As Figure 8 shows, the theoretical frictional stress of 232 psf (11 kPa) was generally conservative except in the first 25 to 30 feet (7.6 to 9.1 meters), where higher jacking stresses were measured, and in the "South Pit to CSMH#14" stage, where timber pile obstructions were encountered. Piles were also encountered after about 80 feet (24.3 meters) of jacking length in "CSMH #13 to #12," but they did not cause a significant increase in frictional stress.

The CT-ST set of pipe jacking data is shown in Figure 9, including the theoretical values of net frictional stress. The Herzog edge resistance used in Equation 1 was 40,000 psf (1,915 kPa) based on the descriptions in Table 1; this value lies between the values for "stiff to firm" and "stiff to hard" clay. The theoretical frictional values were calculated using two differ-

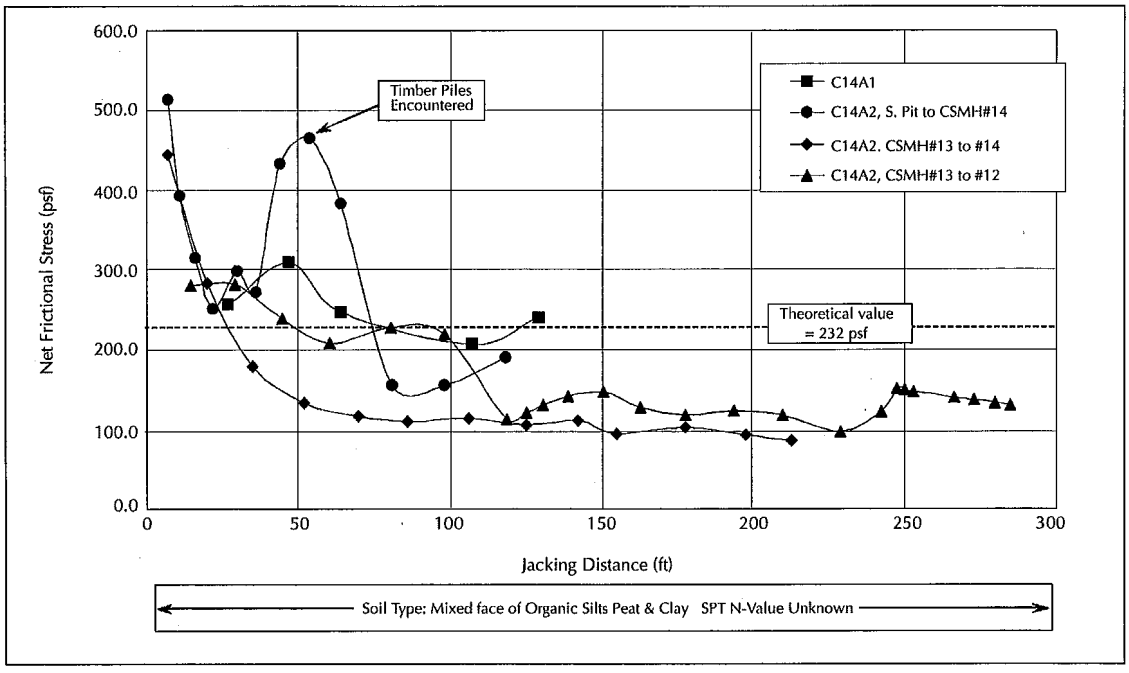

FIGURE 8. Net friction stress versus jacking distance length compared to theoretical friction on the NESI contract.

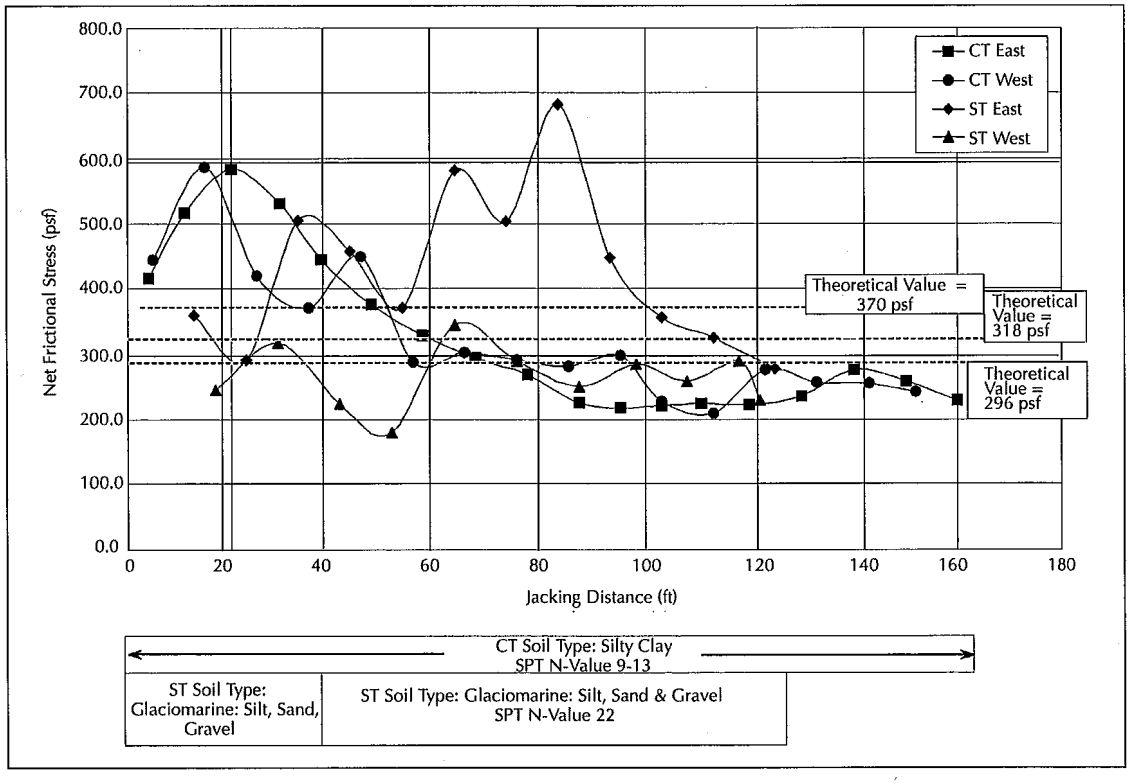

FIGURE 9. Net friction stress versus jacking distance length compared to theoretical friction on the Callahan Tunnel-Sumner Tunnel contract.

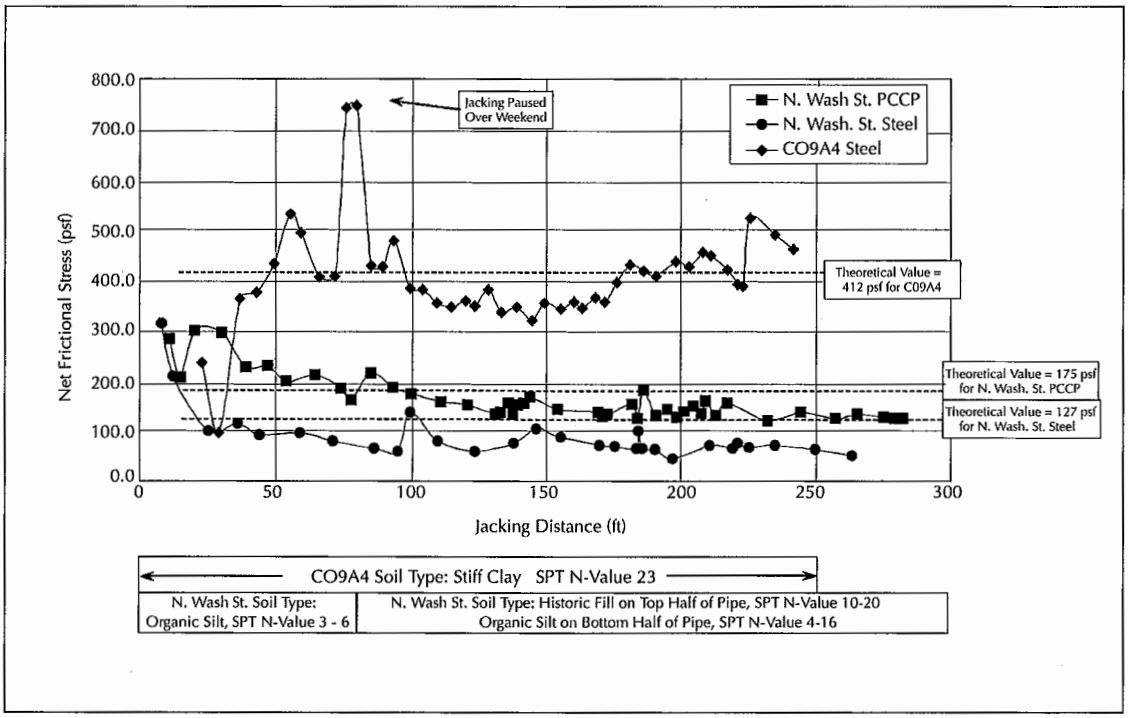

FIGURE 10. Net friction stress versus jacking distance length compared to theoretical friction for the North Washington Street and C09A4 contracts.

ent μ values: 0.20 for the CT pipes in stiff Boston blue clay (BBC), and 0.25 for the ST pipes in stiff BBC to sand and gravel. In Figure 9, the CT theoretical value of 296 psf (14.2 kN/m^2) is compared to both CT East and West field data (square and circle symbols, respectively), the ST East value of 370 psf (17.7 kN/m^2) to ST East field data (diamond symbols), and the ST West value of 318 psf (15.2 kN/m^2) to the ST West field data (triangle symbols). The theoretical values match reasonably well with the measured frictional stresses except in the first 40 to 50 feet (12.2 to 15.2 meters), and at a jacking length of about 80 feet (24.4 meters) in the Sumner Tunnel-East jacking, when frictional stresses were computed to be over twice the expected value (see Figure 9). There are no field notes to indicate what may have caused this discrepancy, although it can be speculated that either obstructions or a pause in jacking operations may have occurred.

The last set of jacking data, for the NWS and C09A4 pipes, is shown in Figure 10, and Table 4 gives the data summary for these jacking phases. For the NWS pipes, which were jacked through historic fill and organic silt, a μ-value of 0.10 and the minimum Herzog edge resistance of 8,354 psf (400 kN/m^2) were assumed to compute the theoretical frictional stress. For the C09A4 pipe in stiff to hard clay, a Herzog edge resistance of 62,656 (3,000 kN/m^2) and a μ value of 0.25 were assumed. Figure 10 reveals that there was excellent agreement between the measured and theoretical values for all three pipes except for the portion of C09A4 when jacking was paused over the weekend. Table 4 also shows close agreement between the average frictional stress based on field data and the theoretical frictional stress for these cases. Unlike the other two sets of data shown in Figures 8 and 9, for these data there is no appreciable increase in measured frictional stress at the start of jacking.

Discussion of the Data Analysis

The analysis performed to predict jacking forces for these CA/T Project pipe jacking cases consisted of computing an assumed

Herzog edge resistance and subtracting it from the total measured jacking resistance to obtain the pipe's measured frictional resistance, and comparing these measured values to theoretical frictional resistance values obtained using computed normal stresses (*i.e.*, Terzaghi's stress calculations and an assumed lateral stress coefficient) and assumed values for the interface friction coefficient. In general, the theoretical friction provided a reasonable to conservative estimate of the actual measured frictional resistance — *i.e.*, it predicted a frictional resistance greater than or equal to that actually measured after the Herzog edge resistance was accounted for. However, this agreement was not as good at the beginning of pipe jacking in two out of the three data sets (in the NESI and CT-ST pipe jacking cases — see Figures 8 and 9, respectively, and Table 4), where the jacking stresses were much higher. This divergence may be due to Herzog's resistance factors being much higher at the start of jacking than in the steady state. Measured and computed friction values were also not in agreement when obstructions were encountered or jacking operations were suspended for an appropriate amount of time.

The average values of frictional resistance from field data, as presented in Table 4, can also be compared to values from the literature provided in Table 2 and by the Pipe Jacking Association.[5] However, these previously published values have limited usefulness due to the large range in the values given. Thus, it can be concluded that, with careful selection of input parameters, the methodology detailed here may provide a more reliable means for *a priori* prediction of pipe jacking frictional resistance values. Clearly, case studies of pipe jacking in other soil types need to be analyzed to confirm the method's widespread applicability.

Conclusions

A set of eleven pipe jacking cases from contracts associated with Boston's CA/T Project were presented (including soil conditions, pipe properties and construction problems encountered) to describe a methodology used to isolate the measured frictional resistance and compare that to a theoretical frictional resistance. The frictional resistance was obtained from the total measured jacking resistance by accounting for the edge resistance (based on Herzog's edge resistance values) and computing the remaining frictional stress on the pipe's outside surface area. Terzaghi's method for tunnel stresses and an assumed lateral earth pressure coefficient were used to calculate an average normal stress on the pipe, from which the theoretical frictional resistance was computed. Comparisons of the measured and theoretical friction revealed that, in general, the theory provided reasonable to conservative estimates of frictional resistance except at the start of pipe jacking (in the first 25 to 50 feet of jacking length) where much higher frictional resistance values were measured. However, this difference may be due to higher, non-steady state edge resistance values at the start of pipe jacking than those predicted by Herzog's method. While pipe jacking case studies in other soil types need to be examined, the method used here for the given data offers the potential for the accurate prediction of jacking forces.

ACKNOWLEDGMENTS — *John M. Pecora would like to thank the J.F. White Contracting Company, which supported his graduate studies and provided access to the pipe jacking data described herein.*

JOHN M. PECORA, III, *is a Project Engineer with J.F. White Contracting Company in Framingham, Massachusetts. He has twelve years of experience in the heavy and highway construction industry. He received his B.S. in Civil Engineering at the University of Massachusetts at Amherst in 1993. He received his M.S. in Civil Engineering with a concentration in Geotechnical Engineering from Northeastern University in 2002.*

THOMAS C. SHEAHAN *received his B.S. from Union College, and his M.S. and Sc.D. from the Massachusetts Institute of Technology. He has been on the faculty at Northeastern University since 1991, where he teaches*

and conducts research in the fields of geotechnical engineering and engineering education. He has been the undergraduate program director for the Department of Civil and Environmental Engineering since 1997. He serves as the co-editor of the ASTM Geotechnical Testing Journal, *and is active on the Transportation Research Board.*

REFERENCES

1. Stein, D., Möllers, K., & Bielecki, R., *Microtunneling: Installation and Renewal of Nonman-Sized Supply and Sewage Lines by the Trenchless Construction Method*, Berlin: Ernst and Sohn, 1989.

2. Kramer, S.R., McDonald, W.J., & Thomson, J.C., *An Introduction to Trenchless Technology*, New York: Van Nostrand Reinhold, 1992.

3. Iseley, T., & Gokhale, S.B., *Trenchless Installation of Conduits Beneath Roadways*, Nat. Coop. Hwy. Research Progr., Synthesis of Practice No. 242, Washington: National Academy Press, 1997.

4. Thomson, J.C., *Pipejacking and Microtunneling*, Glasgow: Blackie Academic & Professional, 1993.

5. Pipe Jacking Association, *A Guide to Pipe Jacking and Microtunneling Design*, London: Pipe Jacking Association, 2003.

6. Terzaghi, K., Peck, R.B., & Mesri, G., *Soil Mechanics in Engineering Practice*, third edition, New York: John Wiley & Sons, 1996.

7. Pecora, J.M., *Geotechnical Engineering Influences on the Design and Construction of Pipe Jacking Operations.* M.S. report, Northeastern University, Dept. of Civil and Environmental Engineering, 2002.