Preliminary Design and Engineering of Pipe Ramming Installations

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Abstract: Installation of new buried pipes and culverts, and replacement of existing ones utilizing trenchless technologies, is increasing in popularity because these methods mitigate many of the surface disturbances associated with conventional open-cut placement. Pipe ramming is an efficient technique that allows installation of casings in soils that can present difficulties for other trenchless technologies. Despite increasing usage, little technical guidance is available to owners and engineers who plan installations with pipe ramming. This paper provides an overview of the pipe ramming technique, possible design procedures, and governing mechanics associated with pipe ramming, with the goal of providing a baseline for engineered installations and identifying areas for further research. Methods to estimate soil resistance to ramming, analysis of ground deformations, and ground vibrations are discussed and compared with measurements observed in field installations. Soil resistance predictions based on conventional jacking methods are shown to underpredict measured resistances inferred from dynamic load testing. Empirical Gaussian settlement models commonly employed in tunnel engineering were shown to result in somewhat inaccurate predictions for an observed pipe ramming installation in cohesionless soils. Field measurements of the ground vibrations resulting from ramming are presented and compared with commonly used safe vibration standards developed for residential structures; the frequencies of vibration generally range from 20–100 Hz, are considerably high for small source-to-site distances, and attenuate rapidly with radial distance. In general, the study lays a basis for planning pipe installation projects with the intent of providing technical advancement in pipe ramming. **DOI:** 10.1061/(ASCE)PS.1949-1204.0000107. © *2012 American Society of Civil Engineers.*

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Introduction

Traditional methods for pipeline and culvert installation, such as conventional open cut techniques, are proven but have become increasingly unpopular because of long duration of excavation, installation and back-filling, road or rail closures, traffic delays, detours, and loss of access to homes and businesses (Ariaratnam et al. 2006). In response, engineers and contractors are gradually abandoning open trench cutting and adopting trenchless methods of pipe installation, which can mitigate many of the logistical issues associated with traditional approaches. Pipe ramming is a simple non-steerable trenchless construction technique, used primarily in horizontal applications in which a ramming tool (i.e., an encased piston) is used to hammer a pipe casing into the ground with high-frequency percussive blows. Pipe ramming allows casing installation in soils with large cobbles and boulders, which are ground conditions that may pose greater difficulty to other trenchless techniques without specific modifications. Among the various trenchless technologies available, such as micro-tunneling and horizontal directional drilling (HDD), pipe ramming is a preferred method for shallow pipe or culvert installation under roads and railways, for which other trenchless methods could cause unacceptable ground movement.

Despite the growing popularity of and experience with pipe ramming, there is surprisingly little technical guidance available for engineers to appropriately plan pipe ramming installations. Engineers must be confident in specifying all aspects of a pipeline or culvert installation to inform owners of the factors that govern construction layout, rate of progress, and impacts to the urban built environment. Simicevic and Sterling (2001) and Najafi (2008) provide the most helpful information to date for planning pipe ramming projects. However, detailed technical guidance, such as methods to predict ground movement, ground vibrations, soil resistance to ramming, compressive and tensile installation stresses, and required wall thickness is not available in the literature. Likewise, few measurements of the performance of pipe ramming installations have been taken or reported, such that empirical experience has been largely confined to the contracting industry.

This paper is intended to provide a baseline for the technical advancement of pipe ramming installations. Because pipe ramming is an emerging trenchless technology, an overview of the advantages and disadvantages, procedures, and considerations for planning an installation is provided. Methods to estimate soil resistance to ramming, based on traditional pipe jacking and driven pile design procedures, are presented and compared with measured loads. Procedures to estimate ground movement developed from tunnel engineering are discussed and compared with field measurements from a pipe ramming installation. Finally, methods to estimate ground vibrations are compared, and placed in the context of ground vibrations observed during ramming. This document intends to provide owners, consultants, and contractors a common basis for the planning of pipe ramming projects, and encourage

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consideration of and confidence in the use of the pipe ramming for new culvert installations.

Advantages and Limitations of Pipe Ramming

Evaluation of pipeline installation feasibility requires an assessment of the opportunities and risk associated with available installation methods and contractor experience. Similar to other trenchless technologies, there are advantages and disadvantages associated with the use of pipe ramming. Perhaps the most significant benefit is the cost-effectiveness of the operation (Najafi 2008). Relative to other trenchless technologies, ramming equipment requires the simplest design and equipment, and features significantly less moving parts relative to the rotating cutting heads and jacking actuators associated with micro-tunneling and HDD. Therefore, maintenance costs throughout the project are lowered. Smaller work crews and less heavy equipment are needed in pipe ramming operations, reducing operational costs (Piehl 2005). Social, economic, and environmental costs related to traffic delays or detours are reduced or eliminated entirely, as is the case with most trenchless technologies.

Ramming is possible in a wide variety of soils, including cobbles and boulders, and stable (nonflowing) in addition to unstable (flowing) ground conditions (Schrank et al. 2009). When using open-faced ramming, the spoil moves steadily into the cavity of the pipe, reducing damage, deviations in alignment, creation of voids, and surface disruptions (Simicevic and Sterling 2001). Subgrades characterized with boulders or other obstacles of considerable size can be traversed, as long as their diameter is smaller than that of the pipe; diameters of up to 3.66 m have been installed in soils with boulders (Zubko 2003). One disadvantage of pipe ramming is that the materials which are used tend to be limited to steel. However, an advantage in the use of steel is that damage to the leading edge of the driven pipe may be minimized relative to other common pipe materials. Conversely, the frictional characteristics of steel tend to be similar to that of concrete and vitrified clay, which are larger than those of centrifugally cast fiberglass-reinforced polymer mortar (CCFRPM) and polymer concrete pipe surfaces. For example, Iscimen (2004) found that the residual coefficient of friction, μ, for Ottawa sand sheared against painted steel surfaces ranged from 0.44–0.49 for normal stresses ranging from 40–120 kPa, whereas the CCFRPM and polymer concrete pipe alternatives produced $\mu = 0.42$–0.44 over the same range of normal stresses. The practical result is that the magnitude of soil resistance to driving steel materials may tend to be larger than that corresponding to a driven polymeric material in the absence of lubrication. Because of resistance to ramming, typical pipe length is limited to approximately 75 m (Najafi 2008); however, lengths of up to 108 m (TT Group 2011) have been reported. Another disadvantage of pipe ramming stems from the inability to actively steer the pipe following initial setup and driving. Following installation of the first 1.2 m of pipe, there are very few options to correct the alignment and grade of a rammed culvert (Najafi 2008).

Pipe Ramming Procedure

Installation of pipes or culverts with pipe ramming begins with excavation, as necessary, of insertion and receiving pits, locations in which the pipe is launched and breaks out, respectively. The length and width of the working pits depend on the length of the pipe segments to be installed, required working space to set up and facilitate the ramming operation, and available right-of-way. The insertion pit is stabilized, if necessary, by placing and compacting a leveled gravel bed and/or placing steel sheets. For areas in which the ground water table is high, dewatering may be required to avoid flooding of the working area and facilitate welding of pipe segments. Steel tracks typically used for auger boring are placed on the working surface to guide the pipe at the required grade and alignment (Fig. 1), and are subsequently used when removing spoils with auger boring methods. The pipe and ram are then assembled. Setting the grade and alignment of the initial pipe segment is the most critical aspect of construction preparation, given that correcting alignment after approximately 1.2 m of insertion is extremely difficult (Najafi 2008). When misalignment requires pullback of the pipe, loose soil at the insertion face may prevent the installation from proceeding at the proper line and grade. Thus, the operations must progress cautiously such that grades can be monitored at every foot of penetration (Najafi 2008). A general pipe ramming operation is depicted in Fig. 1.

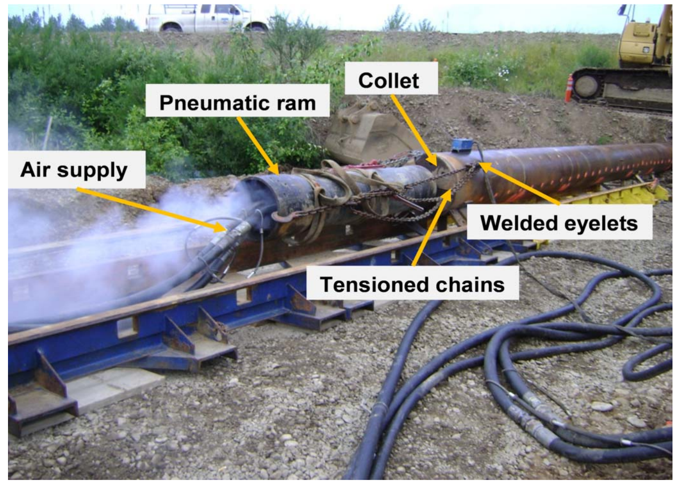

Fig. 1. Typical ramming setup with 400 mm pneumatically-driven hammer, delivering 231 blows per min

Pipes with diameters of 200 mm or less are installed with a closed face, whereas larger pipes are generally installed with an open face. In the former (closed pipe), a cone-shaped head is often welded to the front of the leading pipe to ease penetration (Najafi 2008). In the latter (open pipe), a cutting shoe is welded to the leading edge to help reduce frictional soil resistance, and ease movement through the soil by helping break up cobbles and small boulders. The ramming hammer can be fitted directly to the rear end of the casing pipe if the diameter of the pipe is less than or equal to the largest hammer diameter. However, if the diameter of the casing is larger than the hammer, a series of adjustment collets or ram cones and cotter segments are used to downsize the diameter and allow connection to the hammer. These heavy steel connections help evenly distribute the percussive force of the hammer to the pipe. The entire arrangement is held in place by tensioned chains (Fig. 1), which must be tightened throughout the ramming operation, given that tension within the chains tends to reduce, which is attributable to vibrations associated with hammer impact. Loss of seating between the hammer and pipe results in an undesirable loss of energy transfer efficiency.

As ramming proceeds, the pneumatic hammer provides rapid impacts, or hammer blows, to advance the pipe into the ground while simultaneously pulling itself along with the pipe. Ramming continues until all but 1.2–1.5 m of the pipe has penetrated the subsurface; the exposed length of pipe is required to allow adequate space for welding operations and/or inspection of the connection. Pipe segments are connected by a full structural weld or by using

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proprietary joint systems, which must feature adequate strength to facilitate transfer of impact energy during installation and the bending associated with changes in alignment which are attributable to impact with obstructions.

For open-faced pipe ramming, removal of the soil plug can be carried out continuously using a soil removal cone, or at some intermittent interval that depends on installation length. Soil plug removal helps reduce the weight carried by the pipe; maintain grade in soft cohesive soils and very loose, saturated cohesionless soils; and reduce internal friction within the casing. Lubrication can also lessen the frictional soil resistance to driving and maintain the integrity of the overcut created by a cutting shoe. Spoil removal and lubrication is discussed in detail next.

Considerations for Planning Pipe Ramming Projects

Ramming Tools (Hammers)

Impact hammers consist of a pneumatically- or hydraulically-powered reciprocating piston within a specially-designed steel shell. The piston strikes the inside of the shell which is connected to the culvert, and imparts energy in the form of a stress wave that advances the hammer/pipe system into the ground. The piston in the hammer is designed to propel rapidly on the forward stroke and is regulated on the backstroke such that it does not reverse the hammer/pipe system out of the bore. The most commonly-used type of ramming tools are pneumatic hammers [Fig. 2(a)], which employ compressed air to drive the piston (Schrank et al. 2009; Currey et al. 2009; Yin et al. 2003). These hammers require a large external air compressor to provide the air supply. Some hammers may be operated in reverse to assist in decoupling from the pipe following completion of driving a particular pipe segment. Variation of the impact energy magnitude and frequency is possible during the operation by varying the air pressure supplied to the hammer. Hydraulic hammers [Fig. 2(b)] operate through the use of pressurized fluid that powers the ram's piston and gas within an accumulator system. A hydraulic hammer requires a hydraulic power unit, adaptors, and a support device for the hammer to rest and in which to glide. A hydraulic system allows for an overall smaller hammer and a hydraulic power unit that weighs less than a pneumatic system. Blow energy and frequency can be varied by adjustments to the charge pressure of the gas within the accumulator or flow rate of hydraulic fluid, depending on soil conditions, pipe size, and other considerations (Yin et al. 2003). Yin et al. (2003) reported that hydraulic systems utilize 25–50% of the energy required to operate pneumatic systems; however, these hammers are less common than pneumatic hammers. Pipe ramming hammers have been used successfully by several contractors to assist in salvaging product pipes, assist HDD operations in difficult ground, and other applications. The authors refer interested readers to Najafi (2008) and Orton (2008).

Pipe Section Design

The nature and magnitude of ramming forces delivered to a pipe necessitates that the pipe be made of steel. Simicevic and Sterling (2001) recommend that pipes installed by ramming follow the ASTM A139 standard (ASTM 2010) and exhibit a minimum yield strength, f_y, of 241 MPa. High thrust forces and poor steering control can cause damage if the compressive impact stresses exceed the yield strength of the pipe, and should therefore be limited to $0.9\ f_y$, similar to pile driving operations (Hannigan et al. 2006) The anticipated compressive stresses attributable to ramming can be determined from wave mechanics. When the hammer strikes the pipe, a small zone of the pipe compresses initially and causes a strain; thereafter, the strain propagates down the length of the pipe by compressing neighboring elements of the pipe. The compressive stress, σ_0, resulting from imposed strain at the head of the pipe is a function of the pipe's velocity, v_p, and can be derived from Hooke's law, expressed in the following:

$$\sigma_c = \frac{E}{c} \cdot v_p \tag{1}$$

where E = elastic modulus of the pipe and c = wave speed of the steel pipe (approximately $5,960\ \mathrm{m/s}$). The maximum compressive stress will almost always occur at the rear of the pipe (i.e., in proximity to the hammer), because there is no opportunity for soil to resist pipe particle motion at this location. Eq. (1) will usually overestimate the actual maximum compressive stress for perfectly-seated hammers, because of inefficiencies of hammer-pipe energy transfer; however, when the pipe is not seated properly, or eccentricities occur, the hammer energy may be concentrated over a smaller cross-sectional area, potentially leading to locally higher stresses and possible yielding. Thus, maintaining hammer alignment and seating is critical in pipe ramming operations.

The potential for buckling needs to be investigated to ascertain the integrity of the pipe under installation loads, which include earth pressure and service loads. External loads generally cause the cross section of the pipe to deflect such that the vertical dimension decreases while the horizontal dimension increases; the resulting

Fig. 2. (a) 610 mm pneumatic hammer driving 1,070 mm diameter pipe; (b) hydraulic hammer driving 3,050 mm diameter pipe (adapted from Piehl 2005)

compressive ring stress in the pipe wall should therefore be maintained below the pipe wall buckling or crushing stress, given by Watkins and Anderson (2000) in the following:

$$\sigma_c = \frac{\sigma_v \cdot D}{2 \cdot t} \quad (2)$$

where σ_c = compressive ring stress; σ_v = vertical soil pressure; D = external diameter; and t = thickness of the pipe. Additional guidance on structural load combinations may be found in the German standard ATV-A 161E (ATV 1990).

Cutting Shoes

Cutting shoes are commonly welded to the tips of open-ended pipes to reinforce the leading edge of the pipe and minimize damage from boulders and cobbles. Cutting shoes are generally constructed using T1 steel [Fig. 3(a)], also known as high-speed steel (ASTM A 514 T1), preferred over mild steel because of its durability and strength. Alternatively, a pre-fabricated steel band [Fig. 3(b)] may be welded around the internal or external edge of the pipe (Simicevic and Sterling 2001). Both alternatives should be of a diameter slightly greater than the pipe edge to create an overcut. Cutting shoes are typically beveled to improve directional control

Fig. 3. Photos of typical cutting shoe options: (a) T1 steel, external cutting shoe with interior bevel (Oregon State Univ. Pipe Ramming Project); (b) steel band cutting shoe with lubrication conduit and port shown in the foreground (Najafi et al. 2005, with permission from the Missouri Dept. of Transportation)

during pipe advancement, direct spoil into the pipe, compact soil around the pipe to stabilize the overcut, and fracture boulders and cobbles. Historically, either internal or external bevels have been used at the edge of the cutting shoe; however, an internal bevel is recommended. Cutting shoes may be strengthened by addition of a hardened steel bead welded to the edge of the bevel.

Spoil Removal

Open-ended pipe ramming results in movement of soil into the casing, which increases the total weight of the pipe, and reduces penetration rate. Limited spoil removal is necessary on all installations to facilitate welding of subsequent pipe sections. For ramming in loose, saturated granular and soft, cohesive soils, the additional weight in the pipe may result in downward movement of the pipe relative to the intended grade. Spoil removal is accomplished using water pressure, air pressure, augers, or scrapers, depending on the diameter of the pipe, soil type, and accessibility. Auger boring is commonly used for large-diameter installations. Relatively short installations may allow soil removal following installation of the entire casing.

Lubrication

Similar to other trenchless technologies, lubrication may be readily applied to pipes installed by ramming, to reduce the frictional soil resistance to ramming and stabilize the annulus created by an overcut. Pressurized lubricant is delivered through one or more small-diameter steel conduits to the rear of the cutting shoe, where it is pumped into the overcut. Bentonite-based slurries, often improved with synthetic polymers and additives, are commonly used in pipe ramming applications. Although bentonite-based lubricants are good candidates for soils with low plasticity and large voids that can be filled, they are less favorable for clay soils with moderate to high plasticity, which tend to swell and become sticky with introduction of water (Piehl 2005; Marshall 1998). Polymer slurries do not need to be mixed with water for activation and application, and are better suited for highly plastic cohesive soils and shales.

Lubricants are characterized by rheological parameters such as viscosity, gel strength, (i.e., the measure of the lubricant to hold particles in suspension), density, and filtration. Although a thorough lubricant design will consider all of these parameters, practical considerations have led to the popularity of the Marsh funnel for determining the appropriate qualitative viscosity in the field. A Marsh funnel is a simple device made of plastic or steel (ASTM 2004) used to time the flow of a given volume of lubricant. The target viscosity is dependent on the soil type, and generally ranges from 55–65 and 40–45 s for sandy and clay soils, respectively. For planning purposes, the theoretical minimum flow rate of lubrication required may be computed by multiplying the cross-sectional area of the overcut with the anticipated rate of penetration. However, this quantity will typically be less than the actual demand, attributable to seepage into the surrounding soils, and must be increased depending on the hydraulic conductivity and saturation of the soils being penetrated.

Soil Resistance to Ramming

Accurate prediction of the soil resistance to ramming can guide selection of equipment and operation by providing an estimate of the thrust or ramming forces required to complete the installation without damage. Despite the advantages of being able to predict the soil resistance to ramming, calculations are currently not performed on a routine basis, likely stemming from the lack of methods available to the design engineer. Until further research and progress in this area is made, engineers may resort to traditional quasi-static pipe jacking computations to estimate the soil resistance to ramming. Resistance developed during pipe jacking results from a combination of face resistance (i.e., resistance to driving developed at the leading edge of the pipe) and casing resistance (i.e., resistance to driving generated along the cylindrical surface of the pipe) (Auld 1982; Norris 1992; Bennett 1998; Marshall 1998; Chapman and Ichioka 1999; Staheli 2006).

Estimation of Soil Resistance

The face resistance can be computed as the product of projected cross-sectional projected area of the leading edge (e.g., the cutting shoe), A_f, and the unit face resistance, q_f, given by the following:

$$Q_f = q_f \cdot A_f \tag{3}$$

As determined from traditional pile capacity calculations. The unit face resistance can be determined using an approach proposed by Weber and Hurtz (1981), and summarized by Stein (2005), for jacking load predictions of micro-tunneling machines. Based on statistical evaluation of instrumented case histories and laboratory tests, Weber and Hurtz (1981) proposed the unit face resistance equal to the following:

$$q_f = \lambda\left[c + \gamma\left(h_c + \frac{D}{2}\right)\tan\phi'\right] \tag{4}$$

where c = soil cohesion; h_c = height of soil cover above the pipe crown; D = bore diameter; ϕ' = effective soil friction angle; and λ = an empirical coefficient of load-bearing capacity. The relationship for λ generated by Weber and Hurtz (1981), and plotted in Stein (2005), may be approximated for $\phi' \leq 45°$ by the following relationship:

$$\lambda = \frac{3\pi}{2} e^{\pi \cdot \tan\phi'} \tag{5}$$

Although facing resistance can be considerable in closed-ended applications, the soil-pipe interface friction along the casing generally accounts for most of the resistance to installation in typical pipe ramming applications. One method to compute the casing resistance considers the ground in contact with the entire surface area of the pipe. The friction force attributable to the normal stress of ground pressure is given by the following (Pellet-Beaucour and Kastner 2002):

$$Q_s = \mu \cdot F_N = \mu \cdot 2 \cdot \int_{-\pi/2}^{\pi/2} \sigma_n L \frac{D}{2} d\theta \tag{6}$$

where σ_n = normal stress acting on the pipe and L = penetration length of the pipe. The coefficient of friction, μ, varies as a function of the interface friction angle, $\delta = \tan^{-1}\mu$, which ranges from 0.25–0.75ϕ' depending on the roughness of the soil-pipe interface and use of lubricant (Pellet-Beaucour and Kastner 2002).

The distribution of normal stress on a pipe may be estimated by considering the vertical and horizontal stresses in the immediate vicinity of the pipe. The most widely-used model to estimate the vertical stress acting on a pipe is given by Terzaghi's (1943) trapdoor theory (Staheli 2006). Based on Terzaghi (1943), soil above a trapdoor of width $b = 2B$ is assumed to move downward between two vertical planes [Fig. 4(a and b)]. As the soil column moves downward, friction mobilizes on the vertical shear surfaces at a distance B from the central alignment, resulting in an induced

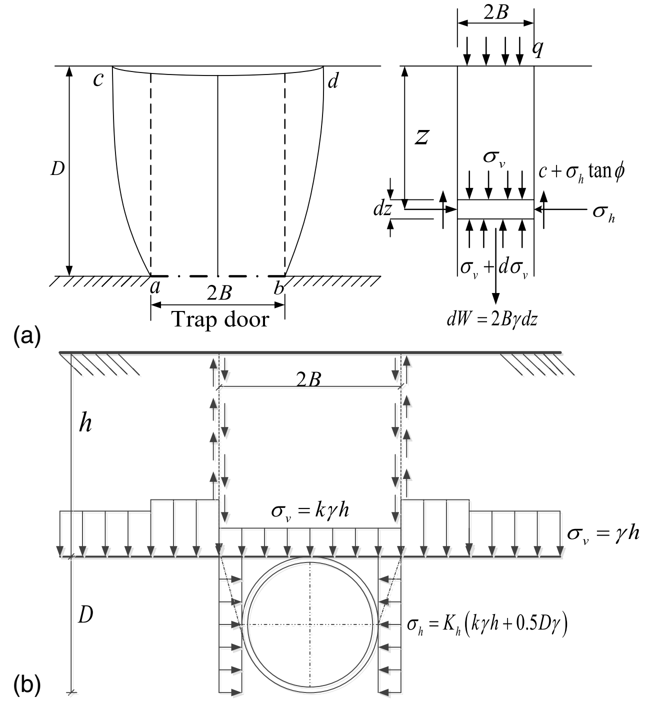

Fig. 4. Schematic illustrating Terzaghi (1943) soil arching theory: (a) trap door model (adopted from Terzaghi 1943); (b) assumed vertical and horizontal stresses on the pipe (adopted from Stein 2005)

compressive stress along the borders of the parallelepiped soil column. This behavior is termed soil arching, which is presumed to occur during installation of pipes with pipe jacking and ramming methods, particularly if an overcut is specified. At equilibrium, the sum of vertical forces on an infinitesimal element of a soil equals zero. The differential form of the vertical force equilibrium is given by the following (Stein 2005):

$$\frac{\partial \sigma_v}{\partial z} = \gamma - \frac{2c}{b} - 2K\sigma_v \frac{\tan\delta}{b} \tag{7}$$

where c = cohesion; $\delta = \phi'$ for soil-to-soil shearing surfaces; γ = unit weight of the soil; K = coefficient of earth pressure above the pipe; and σ_v = vertical overburden soil pressure. Integration of the differential equation for the scenario in which depth, z, equals the depth of cover, h, results in the following expression for the vertical stress at the top of the pipe:

$$\sigma_v = \frac{b(\gamma - \frac{2c}{b})}{2K\tan\delta}[1 - e^{-2K\frac{h}{b}\tan\delta}] \tag{8}$$

The vertical stress formulation at the crown of the pipe can be rewritten in a simplified form as the following:

$$\sigma_v = k \cdot \gamma \cdot h \tag{9}$$

where k = the stress reduction factor contributed by loosening of the ground around the pipe, and is given by the following:

$$k = \frac{1 - e^{-2K\frac{h}{b}\tan\delta}}{2K\frac{h}{b}\tan\delta} \tag{10}$$

Table 1 summarizes the various expressions for parameters K, b, and δ proposed by different researchers and design codes. The stress reduction factor, k, is plotted as a function of friction angle

Table 1. Soil Parameters Used to Compute the Normal Stress (Adapted from Pellet-Beaucour and Kastner 2002)

Soil parameter	Terzaghi (1943)	ATV	PJA	Staheli (2006)
b	$D + 2D\tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$	$D\sqrt{3}$	$D\tan\left(\frac{3\pi}{8} - \frac{\phi}{4}\right)$	$D\cos\left(45 + \frac{\phi}{2}\right)$
δ	ϕ	$\phi/2$	ϕ	ϕ
K	1	0.5	$\frac{1-\sin\phi}{1+\sin\phi}$	1

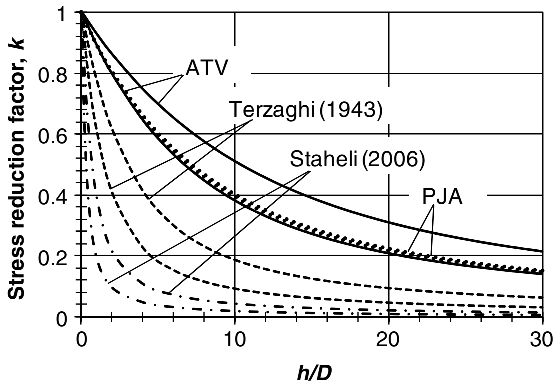

Fig. 5. Variation of stress reduction factor with normalized depth of embedment (h/D); upper bound curves given by $\phi = 30°$, lower bound curves given by $\phi = 45°$ (note: $c = 0$)

and the ratio of depth of cover to diameter (h/D) in Fig. 5. As shown in the figure, k can vary dramatically from method to method for a given frictional angle.

The horizontal stress acting at the central axis of the pipe [Fig. 4(b)] can be determined as the following:

$$\sigma_h = K_h\left(k\gamma h + \frac{D}{2}\gamma\right) \tag{11}$$

where K_h = coefficient of earth pressure at rest (i.e., $1 - \sin\phi$). The normal stress acting radially on the pipe can be obtained using the Mohr's circle of stress (and assumed level ground conditions) as the following:

$$\sigma_n = \frac{(\sigma_v + \sigma_h)}{2} + \frac{(\sigma_v - \sigma_h)}{2}\cos 2\theta \tag{12}$$

where θ = orientation of the elemental soil surface with the horizontal axis. Substitution of Eq. (12) into Eq. (6) followed by simplification yields the following expression for casing resistance:

$$Q_s = \mu \cdot \pi \cdot L \cdot \frac{D}{2}(\sigma_v + \sigma_h) \tag{13}$$

Separately, the pipe ramming force required to install a pipe has been estimated using the following expression (TT Technologies, personal communication, 2011):

$$Q \geq Q_s + Q_f = A \cdot q_f + (C_i + C_o) \cdot L \cdot q_s + V_s \cdot \gamma + W_p \tag{14}$$

where Q = total required force; C_i and C_o = internal and external circumferential length, respectively; q = unit casing resistance as recommended in Table 2; V_s = volume of soil plug in the pipe; and W_p = weight of the steel pipe.

Table 2. Unit Face and Frictional Resistances Recommended by TT Technologies (Adapted from TT Technologies, Personal Communication, 2011)

Soil type	Unit face resistance (kPa)	Unit frictional resistance (kPa)
Cohesive low density	5,000	30
Cohesive medium density	6,000	40
Cohesive high density	8,500	50
Noncohesive low density	4,000	40
Noncohesive medium density	5,000	50
Noncohesive high density	6,500	60

Comparison to Observed Soil Resistance

The performance of existing analytical models for soil resistance to pipe ramming can be evaluated by comparing the predicted resistance with that observed in the field. Meskele and Stuedlein (2011) present the observed performance of a pipe, 0.61 and 32 m in diameter and length, respectively, installed through a granular embankment. Force and velocity measurements were taken at the rear end of the pipe, in accordance with dynamic pile load test procedures (ASTM 2008), over the embedded length of the pipe corresponding to a penetration of 11.7–20.4 m, following welding of the second pipe segment. Each hammer impact blow was converted to total soil resistance in the field using the Case Method (Rausche et al. 1985; Goble et al. 1975, 1980), which assumes that all resistance is concentrated at the pipe toe. Meskele and Stuedlein (2011) compared the static soil resistance component with the jacking methods. The comparison made in this paper is with respect to total soil resistance (Fig. 6), including both static and dynamic resistance, because both components act to resist pipe motion during installation.

Initially, the soil resistance at 11.7 m was relatively high, likely attributable to a loss of soil arching over the time required to complete welding of the new pipe segment (Fig. 6). As ramming continued, soil resistance decreased as the soil arch above the pipe was re-established; thereafter, soil resistance to ramming increased as additional surface area of the pipe was installed within the embankment. Based on the embankment characterization (Meskele and Stuedlein 2011), soil resistance to ramming was modeled using an interface friction coefficient, $\mu = 0.35$, $\gamma = 20.5$ kN/m^3, and $\phi' = 42°$. The ATV and PJA methods produced similar estimates to one another, underestimating total soil resistance to ramming. The Terzaghi (1943) and Staheli (2006) methods produced lower estimates than the code-based methods, and under-predicted total

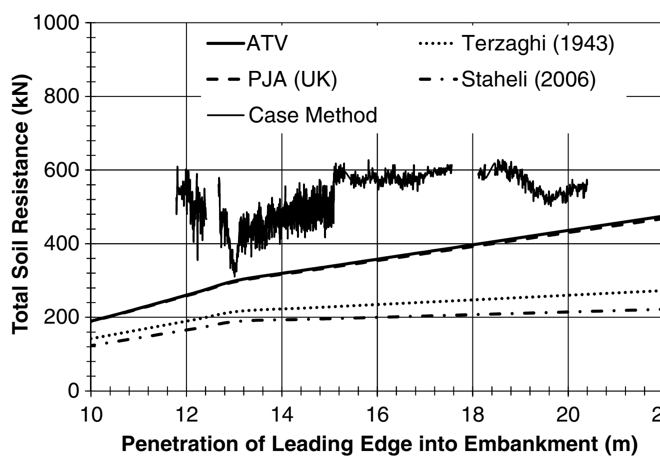

Fig. 6. Measured and predicted soil resistance along the length of the pipe

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soil resistance to driving by a factor ranging from 2–3. However, Eq. (14) provided an estimate of the required pipe ramming force, varying linearly from 2064–4401 kN for penetration lengths of 10–22 m (not shown in Fig. 6), which overestimated the observed soil resistance approximately five-fold. Based on the comparison of observed and predicted soil resistance, it appears that there is a strong need to develop new or improved ramming resistance methods.

Ground Deformation Associated with Pipe Ramming

Analysis of ground deformations associated with installation of pipes and other underground conduits, particularly in urban areas or in proximity to adjacent structures or buried utilities, is critical for designers planning new construction. The nature and magnitude of soil displacement attributable to a new installation depends on soil characteristics, groundwater conditions, depth and size of the culvert, and the construction technique (Leca and New 2007). Soil deformation generally occurs because of a change in the position of soil particles resulting from shearing and volumetric strains imposed by the pipe; thus, ground deformation during pipe ramming is a complex three-dimensional problem [Fig. 7(a)]. A simplified empirical method proposed by Peck (1969) for estimating surface settlements attributable to tunneling has been employed over a wide range of soil conditions and trenchless technologies. The method is popular because of the similarity of the resulting settlement prediction to frequently-observed settlement profiles and its simplicity (Loganathan et al. 1998). The technique assumes that the shape of the transverse settlement profile immediately behind the advancing pipe can be approximated by a Gaussian distribution, in which the ground displacement is given by the following:

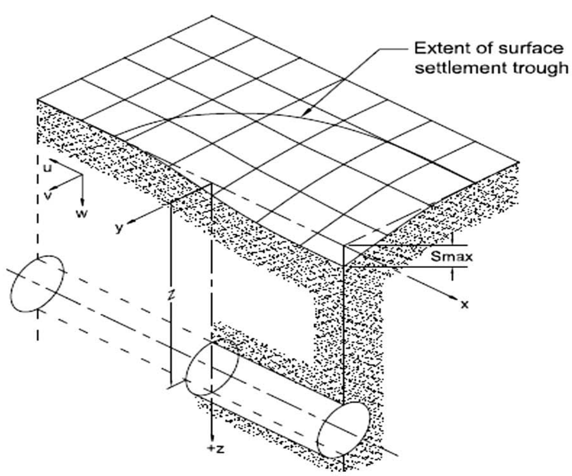
$$S_z(y) = S_{\max} e^{-\frac{y^2}{2i^2}} \tag{15}$$

where $S_z(y)$ = settlement at a distance y from the center line; $S_{\max}$ = maximum (centerline) settlement; and i = standard deviation of the settlement curve. The maximum settlement $S_{\max}$ is defined as the following:

$$S_{\max} = \frac{U_s}{i\sqrt{2\pi}} \tag{16}$$

where U_s = soil loss per unit of length (m^3/m) attributable to an overcut, and $i = 0.28z_0 - 0.1$ or $0.43z_0 + 1.1$ (non-cohesive and cohesive soils, respectively), where z_0 = depth to the center of the pipe (Stein 2005). The soil loss per unit of length equals the area of the annulus between the casing and the soil that is initially generated behind the leading edge by the cutting shoe.

Fig. 7(b) presents the total ground settlement measured and predicted using Eq. (15) along the transverse direction behind an advancing 0.61 m diameter pipe (Meskele and Stuedlein 2011). The Gaussian model appears to model the observed settlement fairly well close to the center of the alignment; however, the observations at radial distances greater than 1 m indicate a greater extent of settlement than that predicted. In general, the observed settlements do not agree with the shape predicted by the model, which represents ground movements anticipated for tunneling and pipe jacking applications. This finding has been observed by other researchers (e.g., O'Reilly and New 1982), which suggests that the Gaussian function may not be appropriate for cohesionless soils. However, the settlement measurements of several pipe ramming installations performed in saturated sands by Jensen et al. (2007) appear to follow the predicted Gaussian profile. Additional research is currently underway to further evaluate the applicability of Eq. (15) to cohesionless soils.



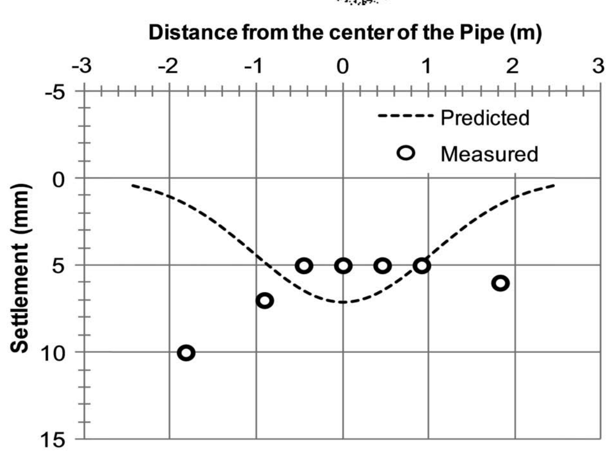


Fig. 7. Gaussian distribution settlement model: (a) conceptual sketch of 3D settlement during pipe jacking (adopted from Attewell 1988); (b) comparison of settlement measurements to that predicted using the Gaussian model

Ground Vibrations Generated from Pipe Ramming

The impact force of the hammer causes a stress wave to travel along the pipe, resulting in vibrational transfer to the ground. Ground vibration can cause aesthetic and structural damage to buildings, disturbance to nearby people, and densification and settlement in sandy soils. The degree of vibration depends on the hammer type and size, dynamic properties of the soil, and distance of the leading edge to a location of interest. During ramming, the hammer imparts a compressive stress wave to the rear end of the pipe, which travels to the leading edge of the pipe at the wave speed of steel. Energy dissipates as the wave travels down the pipe, attributable to material damping and soil resistance to the relative motion of the pipe. A portion of the energy transmitted to the leading edge of the pipe is reflected back to the source of the impact, whereas the remaining energy propagates outward into the soil in the form of spherical P-waves (Massarch and Fellenius 2008). Along the casing, the relative motion between the pipe and the surrounding soil creates horizontally-polarized shear waves (S-waves) that propagate outward in a conical form (Massarch and Fellenius 2008). The velocities of the compressive primary waves (i.e., P-waves, C_p) and shear secondary waves (i.e., S-waves, C_s) are controlled by the small strain elastic properties of the soil. The wave velocities are expressed as the following:

$$C_p = \sqrt{\frac{M}{\rho}} = \sqrt{\frac{\lambda + 2\eta}{\rho}} \tag{17}$$

$$C_s = \sqrt{\frac{\eta}{\rho}} \tag{18}$$

where λ and η = Lamé's constants; ρ = mass density; M = constrained modulus; and ν = Poisson's ratio for soil. Although typical values for P- and S-waves can be estimated from typical values of the constrained and shear modulus, respectively, and mass density of the soil, site-specific in-situ tests, such as downhole or multi-channel analysis of surface wave (MASW) tests, are recommended for projects in which critical infrastructure may be impacted by ground vibration.

When P- and S-waves encounter the ground surface, part of their energy is reflected back into the ground, whereas the remainder convert to surface waves that consist of Rayleigh waves (R-waves), and in some limited cases, Love waves (L-waves) (Kramer 1996). These pipe-ramming-induced vibrations attenuate with distance at a rate that depends on the energy delivered to the pipe, pipe geometry, and ground conditions and geometry. Decay of the vibration amplitude can be attributed to radiation (geometric) damping and material damping, expressed as the following (Kim and Lee 2000):

$$v_2 = v_1 \left(\frac{r_1}{r_2}\right)^{\beta} e^{-\alpha(r_2 - r_1)} \tag{19}$$

where v_1 and v_2 = peak particle velocities at distances of r_1 and r_2, respectively; and β and α = attenuation coefficients attributable to geometric and material damping, respectively (Table 3). The attenuation coefficient, α, can be given as the following (Massarsch 1992):

$$\alpha = \frac{2\pi f \zeta}{C} \tag{20}$$

where ζ = damping ratio of the soil; C = ground surface wave velocity; and f = vibrational frequency.

Wiss (1981) developed a simplified vibration attenuation model based on field observations of vibrations attributable to pile driving. The power law model relates the pile driving hammer energy, W_E, and radial distance, d, between the source of the energy and a location of interest to vibration, given by the following:

$$v = K_d \left[\frac{d}{\sqrt{W_E}}\right]^{-n} \tag{21}$$

where K_d = intercept value of vibration amplitude at "scaled-distance;" $d/\sqrt{W_E} = 1$; and n = attenuation rate. Both Eqs. (19) and (21) show that ground vibrations decay as a function of radial distance.

Table 3. Coefficient of Radiation Damping (adapted from Kim and Lee 2000)

Source location	Source type	Wave induces	β
Surface	Point	Body wave	2.0
		Surface wave	0.5
	Infinite line	Body wave	1.0
		Surface wave	0.0
At-depth	Point	Body wave	1.0
	Infinite line	Surface wave	0.5

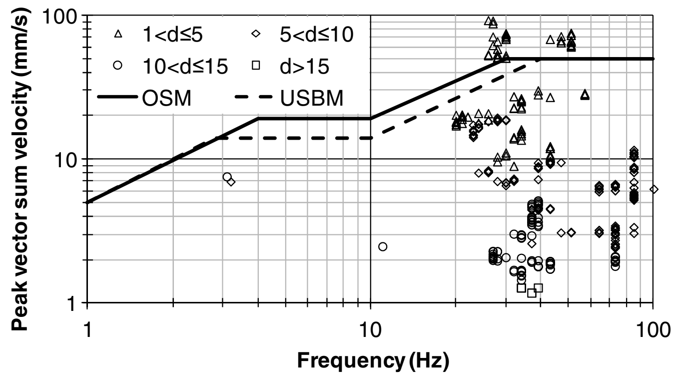


Fig. 8. Distribution of peak vector sum velocity with dominant frequency of ground vibration; safe vibration criteria from the Office of Surface Mining and U.S. Bureau of Mines shown for comparison

The U.S. Bureau of Mines (USBM) and the Office of Surface Mining (OSM) developed "safe" vibration criteria for residential structures based on frequency content and peak particle velocity. The safe vibration criteria use the peak vector sum (PVS) velocity, which is a vector sum of the peak particle velocities in the longitudinal, vertical, and transverse directions and dominant frequency at maximum particle velocity. The criteria, shown in Fig. 8, indicate that the potential for damage that is attributable to low-frequency vibrations (<10 Hz) is considerably higher than those of high-frequency vibrations (>30 Hz). Fig. 8 also shows the PVS and dominant frequencies observed during installation of a pipe 1.07 m in diameter and 36.5 m long with a 406 mm pneumatic hammer. The vibration data plotted in Fig. 8 shows the effect of distance from the source (defined as the leading edge of the pipe) on vibrational attenuation. The vibration levels are significantly high for some observations at small source-to-site distances (e.g., 1 m < d ≤ 5 m), whereas other vibrations in range and all of those observed beyond 5 m fall below the established OSM and USBM threshold levels. Thus, vibrations induced by pipe ramming may impact existing structures or buried utilities when the proposed alignment passes nearby, and efforts to mitigate vibrational amplitudes may need to be developed.

Summary, Conclusions, and Future Research Needs

This paper reviews the state of practice of pipe ramming technology by providing an overview of the advantages and disadvantages, procedures, and considerations for planning a pipe ramming installation. Methods to estimate total soil resistance, ground deformations and vibrations associated with pipe ramming were addressed and compared with measurements observed during actual, full-scale ramming operations. The total soil resistance comprises the combination of the static and dynamic face and casing resistance. Methods to estimate total soil resistance to ramming based on traditional pipe jacking methods were evaluated, as well as a method used by a ramming manufacturer. These methods were compared with measurements of total soil resistance to ramming obtained from dynamic load testing commonly used in pile driving applications. The comparisons showed that the considered conventional pipe-jacking methods appeared to under-predict the observed soil resistance to ramming. Further research is required to understand the mechanisms of pipe resistance to ramming, and develop new or

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improved estimates of total soil resistance for driveability studies and hammer selection.

Prediction of ground deformation and vibration must be performed for planning pipe ramming installations. The simplified Gaussian curve commonly used in tunneling and pipe jacking operations was compared with ground surface settlement measurements for a ramming operation in a granular embankment. The observations presented in this paper and by other researchers indicated that the Gaussian model may not be appropriate for cohesionless soils; research is currently underway to further evaluate settlement prediction. Methods to evaluate vibrational transmission from a driven pipe to the ground surface were presented, as well as measurements observed during pipe ramming. The measurements confirmed theoretical expectations of vibration attenuation with radial distance, and that care must be observed when ramming nearby existing structures or buried utilities. Additional research should be performed to determine vibration model parameters for different hammer types, energies, and soil conditions.

Pipe ramming holds excellent promise for providing a cost-effective trenchless solution for installation of new culverts and pipes. Despite its growing use, very little research has been performed on the fundamental mechanics of pipe ramming operations within the geologic environment. Continued research must be performed to develop reliable methods for prediction of total face and casing resistance, dynamic installation stresses, ground deformations, and ground vibrations that may develop during installation. This will help owners, consultants, and contractors optimize efficient use of pipe ramming technology.

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