Response of buried steel pipelines subjected to relative axial soil movement

Dharma Wijewickreme, Hamid Karimian, and Douglas Honegger

Abstract: The performance of buried steel pipelines subjected to relative soil movements in the axial direction was investigated using full-scale pullout testing in a soil chamber. Measured axial soil loads from pullout testing of pipes buried in loose dry sand were comparable to those predicted using guidelines commonly used in practice. The peak values of axial pullout resistance observed on pipes buried in dense dry sand were several-fold (in excess of 2 times) higher than the predictions from guidelines; the observed high axial pullout resistance is primarily due to a significant increase in normal soil stresses on the pipelines, resulting from constrained dilation of dense sand during interface shear deformations. This reasoning was confirmed by direct measurement of soil stresses on pipes during full-scale testing and numerical modeling. The research findings herein suggest that the use of the coefficient of lateral earth pressure at-rest (K_0) to compute axial soil loads, employing equations recommended in common guidelines, should be undertaken with caution for pipes buried in soils that are likely to experience significant shear-induced dilation.

Key words: buried pipelines, pipe–soil interaction, dilation, soil loading on pipes, interface friction.

Résumé : La performance de tuyaux en acier enfouis soumis à des mouvements axiaux de sol a été investiguée à l'aide d'essais d'arrachement des tuyaux à l'échelle réelle dans une boîte de sol. Les charges axiales de sol mesurées dans les essais à l'échelle réelle effectués sur des tuyaux enfouis dans un sable sec non compacté étaient comparables à celles prédites en utilisant les directives usuelles. Dans le cas d'un sable dense, le pic de résistance axiale observé sur les tuyaux enfouis était de plusieurs fois plus élevé (plus de deux fois) que les prédictions provenant des directives. Ceci est principalement dû à une augmentation significative des contraintes normales globales du sol sur les tuyaux en raison de la dilatation en milieu restreint du sable dense durant les déformations en cisaillement des interfaces. Cette explication a été confirmée par des mesures directes des contraintes du sol sur les tuyaux durant les essais à l'échelle réelle, en plus d'être supportée par des modélisations numériques. L'utilisation du coefficient de pression latérale des terres au repos (K_0) pour représenter les contraintes du sol sur les tuyaux lors des essais d'arrachement, tel que recommandé par les directives usuelles, devrait être faite avec précautions dans le cas de tuyaux enfouis dans des sols qui peuvent se dilater lors du cisaillement des interfaces.

Mots-clés : tuyaux enfouis, interaction tuyau–sol, dilatation, chargement du sol sur les tuyaux, friction à l'interface.

[Traduit par la Rédaction]

Introduction

Buried pipeline systems form a key part of global lifeline infrastructure and any significant disruption to the performance of these systems often translates into undesirable impacts on regional businesses, economies or the living conditions of citizens. One of the major risks to buried pipelines arises from permanent ground movements where high soil loads can lead to potentially unacceptable strains in the pipelines. Common causes of permanent ground displacement are related to slope instability and ground subsidence, including earthquake-induced ground displacements.

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Based on the direction of relative soil displacement with respect to the pipe alignment, buried pipeline restraints against soil movement can be categorized into four different modes: vertical uplift, vertical bearing, transverse (or horizontal–lateral), and axial (or longitudinal). Effects arising due to a combination of these modes are generally considered in the evaluation of the response of buried pipelines subjected to differential ground movements. This paper focuses on the estimation of loads on the pipe due to relative axial soil movements.

Current understanding of the mobilized soil loads on pipes due to axial pipe restraint (or differential axial displacement between soil and pipe) is mainly based on soil–pipe interface parameters and an assumed failure mechanism of the soil. The commonly used approach for determination of axial loads on pipes buried in cohesionless soils is through the use of a simple formula

$$[1] \qquad F_A = \gamma' H \left(\frac{1 + K_0}{2}\right) \tan(\delta)(\pi D L)$$

where F_A is the axial soil load on the pipe (soil resistance), γ' is the average effective unit weight of the soil, H is the depth from the ground surface to the pipe centreline (spring line), K_0 is the coefficient of lateral earth pressure at rest for the soil, δ is the interface angle of friction between the soil and pipeline, D is the external pipeline diameter, and L is the buried pipeline length. This formula has been in wide use in the design of pipeline systems, and Newmark and Hall (1975) used this equation to calculate axial soil loads on pipes subjected to strike-slip fault movement. Moreover, the equation has been recommended for the computation of axial soil loads in ASCE (1984) and Honegger and Nyman (2004).

In using eq. [1], the values of the geometric parameters (H, D, and L) are known and the material parameters (γ' and δ) can be assessed reasonably well using laboratory experimentation (Karimian et al. 2006); arguably, the most uncertainty would arise in determining the term $[(1+K_0)/2]$ used to represent the normal stress on the pipeline. This $[(1+K_0)/2]$ term is based on the implied assumption that the average normal soil stress on the perimeter of pipe is equal to the arithmetic mean of the effective vertical overburden stress (σ'_{v0}) and lateral effective stress at-rest ($\sigma'_{v0} = K_0\sigma'_{v0}$) computed at the centreline level of the pipe (i.e., spring line). Considering that the stress conditions σ'_{v0} and $K_0\sigma'_{v0}$ would ideally correspond to those in a semi-infinite soil mass under "at rest" (or zero lateral strain) condition, it is of importance to review the suitability of this assumption to represent average stresses around a pipe. It is also important to note that, during axial displacement, there is a tendency for shear-induced volumetric strains to occur in an annular shear soil zone adjacent to the pipe. The response of the soil mass surrounding the shear zone to these volume changes will very likely affect the normal soil stresses on the pipe (i.e., dilation in the shear zone could increase the normal stress on the pipe).

The approach in eq. [1] does not account for the above-noted potential changes to the normal stress and there is a need to understand the normal soil stress distribution around the pipe surface under "at-rest" condition as well as its variation during relative axial soil movement. Data from controlled experimental work on pipelines subjected to axial movement, particularly conducted at a full-scale level, is needed to advance this knowledge. Reported experimental research on this subject is very limited (e.g., Trautmann and O'Rourke 1983; Paulin et al. 1998; Anderson et al. 2004). Most of the research testing to investigate axial soil loads during ground movement have been performed (by private entities) for specific uses and the results are either not published and documented or they cannot be generalized to other conditions.

For these reasons, full-scale laboratory testing was undertaken to study the response of buried steel pipes subjected to relative axial movements, as a part of an overall research program to study the response of buried pipelines subjected to ground movement. Full-scale laboratory testing was conducted on relatively large diameter pipes buried in a soil chamber. The pipes were subjected to axial pullout with direct measurement of axial soil loads and soil pressures developed on the pipe surface with respect to the pipe displacement. This paper presents the results from these experiments and corresponding interpretations with support from numerical modeling that was conducted to mimic the effects of soil dilation in the annular shear zone during relative axial soil movement.

Materials, experimental system, and test program

Testing system and methodology

A testing apparatus that was designed and constructed at The University of British Columbia (UBC), Vancouver, B.C., to conduct full-scale physical modeling research on pipe–soil interaction problems (Anderson et al. 2004) was modified for the present testing work. The published experience of the work performed at Cornell University (Trautmann and O'Rourke 1983) and C-CORE (Paulin et al. 1997) was considered in the design process. The apparatus allows investigation of the force–displacement behaviour of buried pipeline configurations subjected to axial and horizontal–lateral loadings. The details of the testing chamber, loading mechanisms, and data acquisition system are described in detail in Anderson (2004), Anderson et al. (2004), and Karimian (2006), and only the main features are presented herein for brevity.

The test setup was mainly comprised of a 3.8 to 5.0 m (length) × 2.5 m (width) × 2.5 m (height) chamber, hydraulic actuator, and a data acquisition system. A perspective view of the soil chamber arranged for axial pullout is shown in Fig. 1. The pipe was buried in a given soil backfill and, for the tests considered in this study, the pipe axis was aligned parallel to the longer direction of the chamber. The length of the pipeline test specimen was kept longer than the length of the chamber so that the pipe extended through both ends of the soil chamber. This ensured a constant soil–pipe test length and also avoided soil disturbance at the back of the chamber during pullout. Relatively soft "foam gaskets" were used at the soil chamber entry and exit locations of the pipe to allow for free movement of the sand around the pipe and to minimize frictional forces on the pipe at these locations.

The buried pipe was then pulled from one end (referred to herein as the leading end) by a double-acting hydraulic actuator while recording the resistance to axial pullout and the axial displacement. A load cell, with a maximum load capacity of 225 kN, was attached to the actuator and the pipe through a shackle and cable to measure the pulling force. The actuator was mounted on a loading pedestal that was bolted to the concrete strong-floor of the laboratory at a location separate from the soil chamber. Axial pipe displacements were measured relative to the strong-floor using string potentiometers passed through the inside of the pipe. All tests were performed at an ambient temperature of 20 ± 1 °C.

In all test configurations, the pipe was loaded in a displacement-controlled manner. The displacement rate chosen for the testing varied between 2 to 50 mm/s. Test results indicated that the loading rates selected had no noticeable effect on the results. Other considerations that controlled the loading rates included the capacity of the hydraulic pumps and costs related to the actuators. All measurements were recorded at 10 samples per second, which was considered suitable with respect to the rates used for axial pullout displacements.

Fig. 1. Test configuration showing the soil chamber and hydraulic actuator set up for an axial pullout test.

Measurement of soil stresses at the soil–pipe interface

In some of the tests, the normal soil stress on the pipe was measured using five total-pressure transducers mounted at selected circumferential locations as shown in Figs. 2*a* and 2*b*. The intent was to measure changes in the normal soil stress acting on a typical pipe cross section (*i*) while filling the chamber, (*ii*) during compaction of the backfill soil, and (*iii*) when subjecting the pipe to axial loading. As may be noted from these figures, the transducers were located along half of the pipe circumference at five equidistant locations (45° radial spacing) between the crown and invert of the pipe. Covering only half around the vertical plane of the pipe was judged acceptable as the soil pressures around the pipe during soil placement could be considered symmetric about the vertical axis.

Type AB/HP pressure transducers (capacity of 345 kPa with ±0.5 kPa resolution), manufactured by Honeywell, Freeport, Ill., were used for these tests (see left side of Fig. 2*c*). The transducers had a circular flat surface with a diameter of about 19 mm and they were initially secured in a custom-designed threaded tubular adaptor that had an outer diameter of about 29 mm (see right side of Fig. 2*c*). The steel pipe used for the pullout test was prepared by drilling threaded holes at the selected circumferential locations so that the holes would be compatible with the thread size and outer diameter of the adaptors; this approach allowed easy "screw-in" type installation of the pressure transducers to the pipe at the selected locations. The design was such that the pressure transducers (with the adaptors), when mounted on the pipes, would be flush with the pipe surface, thus ensuring minimum opportunity for localized soil arching or disturbance at the pipe–soil interface in the immediate vicinity of the transducer. The electrical cable connections to the installed transducers were passed through the inside of the pipe and were then connected to the data acquisition system. To simulate the roughness of the pipe surface, graded 0.4 mm uniform sand, which was comparable to the size of the asperities on the pipe surface resulting from sand blasting, was glued on top of transducers. Each transducer was then calibrated separately using an air-pressure calibrator. Each pressure transducer occupied an arc length of only about 5° on the 457 mm diameter pipe; as such, the localized deviation from the cylindrical pipe geometry due to the presence of the flat transducer zones could be considered negligible.

Another consideration is the effect of the shear stresses during axial pullout on the transducer on the measured normal stresses. Considering the relatively high stiffness combined with the small diameter of the pressure transducer used in this study, it is believed that the membrane distortion of the transducer leading to the shear stress effect is not significant. Detailed specifications of the transducer and the installation procedures provided in the previous paragraph should allow replication of this method in future studies along with the assessment of any shear effects as appropriate.

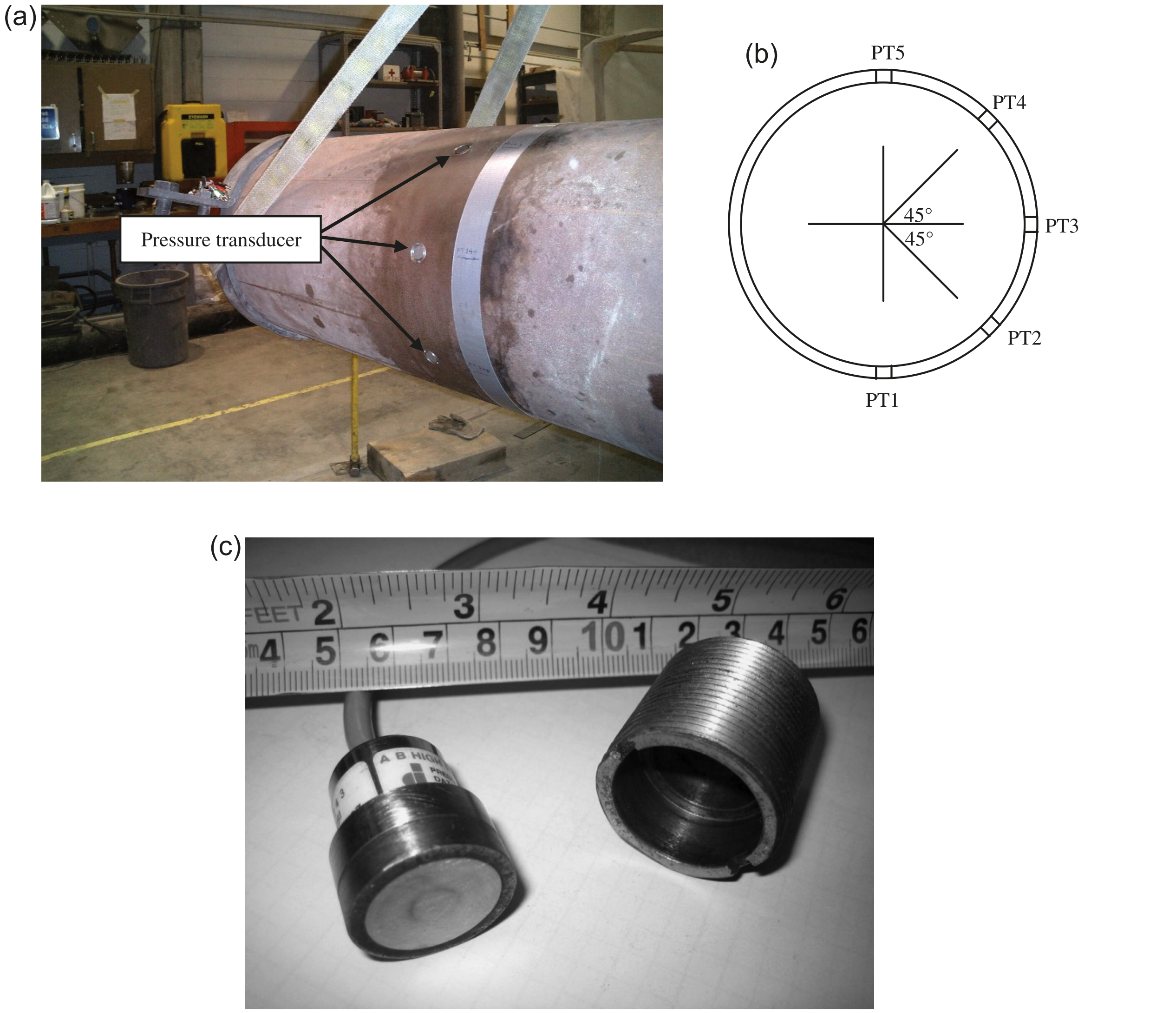
Effect of boundaries of the test chamber

Ideally, physical modeling of the real-life problem would be best represented if there were no artificial boundaries in the vicinity of the pipe (front, rear, sides or below the pipe) during pullout. As such boundaries are needed to keep the scale of testing to manageable levels, it is important to determine and minimize the impact of the boundaries on the examined pipe–soil interaction problem. In recognition of this, the possible effects of the boundaries of the soil chamber on the pipe–soil interaction were investigated in detail as a part of this research program. As detailed in the doctoral dissertation by Karimian (2006), the following specific activities were undertaken to study the boundary effects during axial pullout tests:

(1) Measurement of horizontal normal soil stresses on the front, rear, and side walls at selected locations using a total earth pressure transducer (a 150 mm diameter, 13 mm thick transducer with 200 kPa capacity and ±0.5 kPa resolution, Model LPTPC-06S fabricated by RST Instruments, Coquitlam, B.C., was used for this purpose).
(2) Performance of axial pullout tests using two different lengths of the soil chamber.
(3) Numerical modeling of a cross section of the soil chamber in a direction normal to the pipe axis while including the presence of the side walls.

As further discussed in the latter parts of this paper, the findings from these investigations indicate that the observed axial pullout response is not significantly affected by the effects of chamber boundaries.

Fig. 2. (*a*) Photograph of test pipe with mounted pressure transducers; (*b*) sketch showing circumferential locations of the pressure transducers; (*c*) photograph of type AB/HP bonded, 19 mm diameter, semiconductor strain gauge pressure transducers, manufactured by Honeywell, Freeport, Ill., (left side) and threaded tubular adaptor (right side). (Transducer locations: PT1, invert; PT2, haunch; PT3, spring line; PT4, shoulder; PT5, crown.)



Test materials

Locally available, uniformly graded Fraser River sand was selected as the soil medium for the tests because of the wide use of this sand in the region as an engineering material and for geotechnical research at UBC (Wijewickreme et al. 2005). The mineral composition of Fraser River sand has been reported as 40% quartz, 11% feldspar, 45% unaltered rock fragments, and 4% other minerals (Garrison et al. 1969). The minimum and average grain sizes of the sand are 0.074 and 0.23 mm, respectively, with a coefficient of uniformity of 1.5. The minimum and maximum void ratios (e_{min} and e_{max}) are 0.62 and 0.94, respectively (Wijewickreme et al. 2005).

The mechanical characteristics of Fraser River sand are available from past research at UBC as well as from laboratory element testing undertaken by Karimian (2006) as a part of current research work at UBC. Karimian (2006) conducted triaxial tests on this sand to determine the stress–strain response and strength at low confining stress (σ'_{3c}) levels ranging from 15 to 50 kPa representing the typical soil stress conditions around the buried pipe. A peak internal friction angle between 43.5° to 45.5° was obtained from the tests on Fraser River sand conducted at a relative density of ~75% under these stress levels. Based on previous research work, Fraser River sand has a constant volume friction angle (ϕ_{cv}) in the range of about 31° to 33° (Uthayakumar 1996). Sand used for all the tests was in an air-dried condition with moisture content less than 1%.

Steel pipe (Grade A524) with an outside diameter of 457 mm (18 in.) and a 12.7 mm (1/2 in.) wall thickness was used in the axial pullout test program presented herein. The surface of the pipe was prepared by sand-blasting prior to testing. The interface friction angle (δ) between the sand-blasted steel and Fraser River sand was directly available from specifically conducted direct shear tests (Karimian 2006). Peak δ values of 33° and 36° were obtained for loose and dense sand, respectively, and a large strain value of 31° was obtained for both loose and dense sand.

Testing program

Four axial pipe pullout tests were performed as summarized in Table 1. In three of the tests (AB-3, AB-4, and AB-6), dry sand fill was compacted to an average density of 1600 kg/m^3, corresponding to a relative density (D_r) of about 75%. The sand was placed in the chamber in approximately 200 mm lifts and mechanically compacted to achieve the target soil density. Density measurements made using manual and nuclear densitometer methods demonstrated that the compaction method provides very good control of density in the preparation of these relatively large specimens (Karimian et al. 2006). Test No. AB-5 was performed using loose, "as-placed," uncompacted sand with an average density of about 1450 kg/m^3 (D_r of approximately 20%). Tests were performed with a buried pipe length of 3.8 m, except for test No. AB-3 that had a buried pipe length of 5.0 m.

The depth of pipe burial in a given test is defined with respect to an H/D ratio, in which H is the height of the sand overburden above the centreline of the pipe and D is the outside diameter of the pipe. The H/D ratio was kept constant at 2.5 for the tests performed with dense sand. Test No. AB-5 (loose sand) was performed with an H/D ratio of 2.7 to provide a vertical effective stress at the pipe level nearly the same as that for tests with dense sand. Except for test No. AB-6, all the axial loading tests were conducted within 24 h after filling the soil chamber. Test No. AB-6 was performed 45 days after preparation to evaluate any effects of aging or relaxation of the soil on pipe response. In test No. AB-3, only the axial load and axial pipe displacement were monitored; whereas, in the other three tests, soil stress on the pipe was measured using total pressure transducers mounted on the pipe in addition to the measurement of load–displacement.

In test Nos. AB-4 and AB-5, some geometrically predefined zones in the immediate vicinity of the pipe were backfilled with colored sand. Following the axial pullout loading, the backfill was carefully removed and changes in the location of colored sand were used to identify active shear zones and particle displacements in the proximity of the pipe during pullout (see presentation later in the section entitled "Thickness of shear zone and degree of dilation at interface"). The objective was to make an assessment of the active shear zones during axial pipe pullout.

Experimental results

To facilitate comparison of different tests, the axial soil resistance is presented in the form of a normalized axial soil resistance (F'_A) defined as

$$[2] \qquad (F'_A) = F_A/(\gamma' H \pi D L)$$

The value of F'_A represents the average shear stress around the pipe normalized with respect to the vertical effective stress from the soil overburden at the centerline of the pipe. Such a normalization approach allows description of the observed response from different full-scale axial pullout tests in a directly comparable format.

Axial load versus displacement response

The normalized axial soil resistance (F'_A) versus displacement response observed for test Nos. AB-3, AB-4, and AB-6 (conducted on dense sand) are presented in Fig. 3. Test No. AB-4 was essentially identical to test No. AB-3, demonstrating very good repeatability in the test preparation and pipeline response.

Table 1. Summary of axial pullout tests.

Test No.	H/D ratio	Soil density	Time of axial loading after soil placement	Soil chamber length (m)
AB-3	2.5	Dense	Immediate loading	5.0
AB-4	2.5	Dense	Immediate loading	3.8
AB-5	2.7	Loose	Immediate loading	3.8
AB-6	2.5	Dense	Loading after 45 days	3.8

It is also of interest to note that test No. AB-4 was conducted with a pipe length of 3.8 m, whereas test No. AB-3 was conducted with a pipe length of 5.0 m. The essentially identical F'_A characteristics obtained from these two pipe lengths suggests that any effects arising from the boundary conditions at the entry and exit points of the soil chamber are negligible.

As indicated previously, the horizontal earth pressures experienced by the front and rear walls (i.e., pipe-exit and pipe-entry walls, respectively) and side walls of the soil chamber were measured using 150 mm diameter total earth pressure transducers. The earth pressure measurements were made with transducers located on the front and rear wall faces about 1 to 1.5 pipe diameters radially away from the centerline of the pipe and on side walls approximately at the pipe springline level. A review of the earth pressure measurements did not indicate any detectable change in pressures on the front and rear walls during axial pullout testing (when compared with the measured initial static pressure). This observation further supports the contention that the boundary walls on the entry and exit sides of the soil chamber do not seem to affect the pullout process in a significant manner.

A slightly higher (about 10%) peak F'_A obtained in test No. AB-6, performed after 45 days from the date of specimen preparation, likely reflects some effects due to aging of backfill. The value of peak F'_A observed for test Nos. AB-3 and AB-4 is about 1.02, and for test No. AB-6 is about 1.13. In all three cases, the peak load was achieved at an axial displacement of about 7 to 10 mm. The post-peak F'_A values for all tests approached a constant value between 0.75 and 0.8 after reaching axial displacements in the order of 200 mm.

The load versus displacement response for test No. AB-5, which was performed with the loose sand backfill, is presented in Fig. 4. The peak normalized axial soil resistance (F'_A) is about 0.42 and this value drops to about 0.37 when sheared to larger strain levels. As may be noted, the observed peak F'_A is less than half of the peak load observed for dense-sand test Nos. AB-3 and AB-4 conducted with similar vertical effective stress at the pipe level.

Soil pressures on pipe during specimen preparation and axial pullout

Soil pressures during specimen preparation

The normal stresses on the pipe recorded at transducers PT5 and PT3 during backfilling of the chamber (for the tests

Fig. 3. Load–displacement response during axial pullout in dense sand (test Nos. AB-3, AB-4, and AB-6).

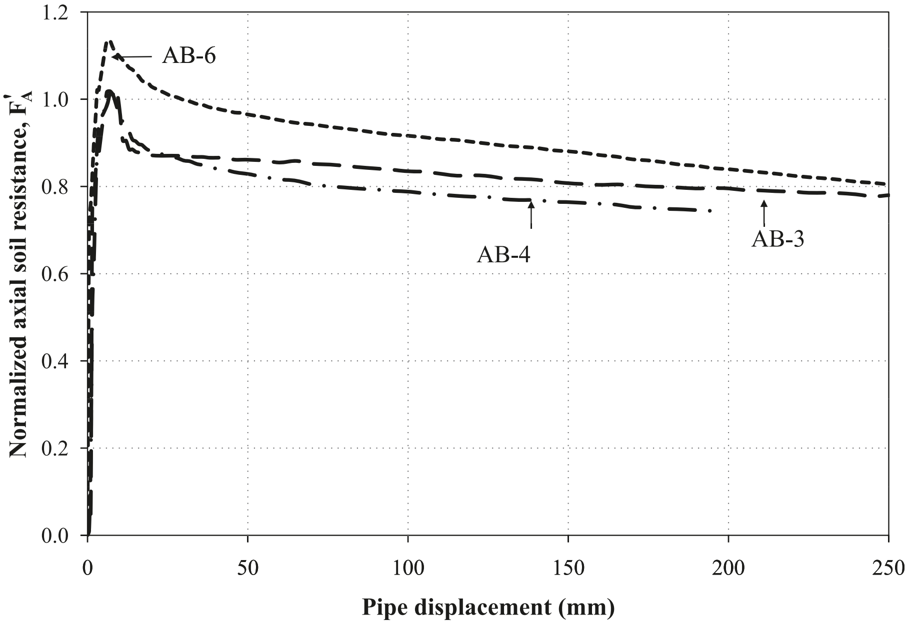
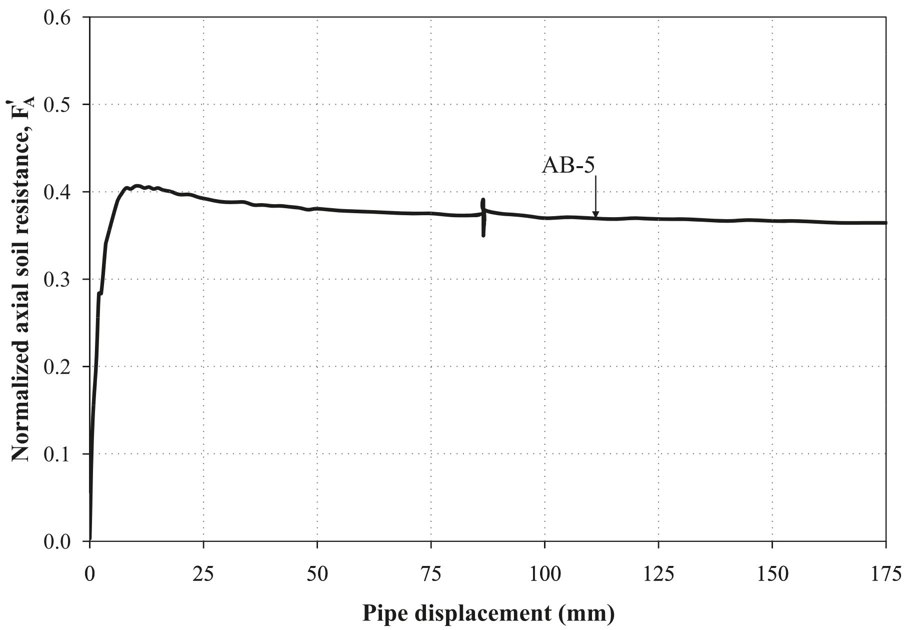


Fig. 4. Load–displacement response during axial pullout in loose sand (test No. AB-5).



Nos. AB-4 through AB-6) are plotted with respect to the estimated vertical effective overburden stress (σ'_{v0}) at the same locations, in Figs. 5 and 6, respectively (see Fig. 2 for the radial position of different pressure transducers on the pipe). The vertical effective stresses at the two transducer locations were estimated using the average density of the backfill soil. As may be noted, the measured vertical stress on the pipe at PT5 (crown of the pipe) is slightly higher than the estimated overburden effective stress. This observation is in accord with the vertical stresses computed using: (*i*) the concept of vertical arching factor (VAF) recommended for estimating vertical soil stresses on concrete pipes as per *Standard specifications for highway bridges* (AASHTO 1996) and (*ii*) Fast Lagrangian analysis of continua (FLAC) 2D (Itasca Consulting Group Inc. 2005) analyses conducted as a part of this study (see "Numerical modeling" section). As detailed in AASHTO (1996), in the presence of a relatively rigid pipe, the vertical stress at the pipe location can be higher than the "free field" overburden stress because of the attraction of more load to the relatively more stiff area in the vicinity of the pipe.

The measured horizontal stress from PT3 for the case of dense sand specimens (test Nos. AB-4 and AB-6, Fig. 6), was noted to be clearly higher than that obtained for the loosely prepared specimen (test No. AB-5), especially under low stress levels. It was also observed that the ratio between lateral and vertical stress decreases with increasing vertical stress for both dense and loose soil conditions. However,

Fig. 5. Measured normal soil stress on the pipe at transducer PT5 versus computed vertical overburden soil stress.

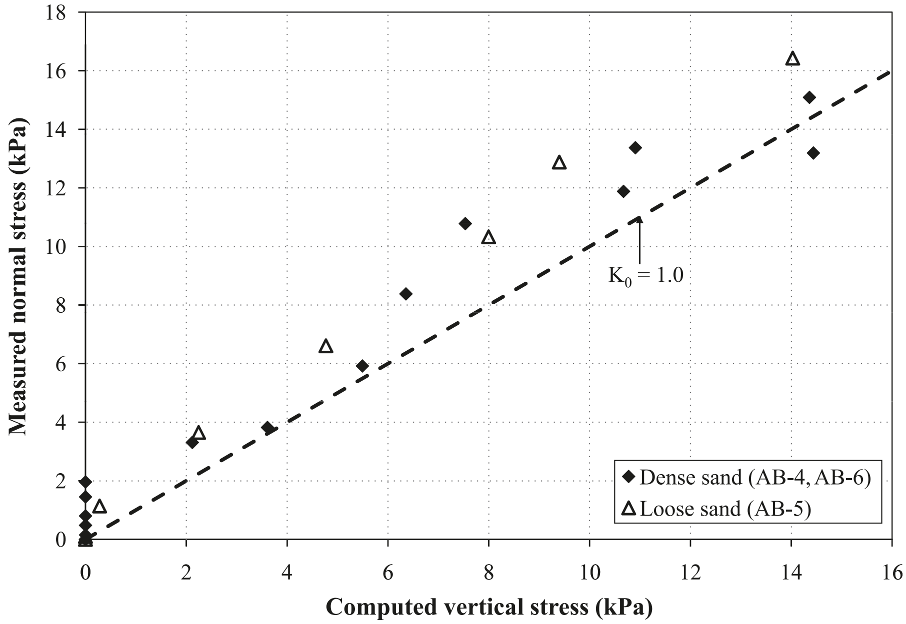
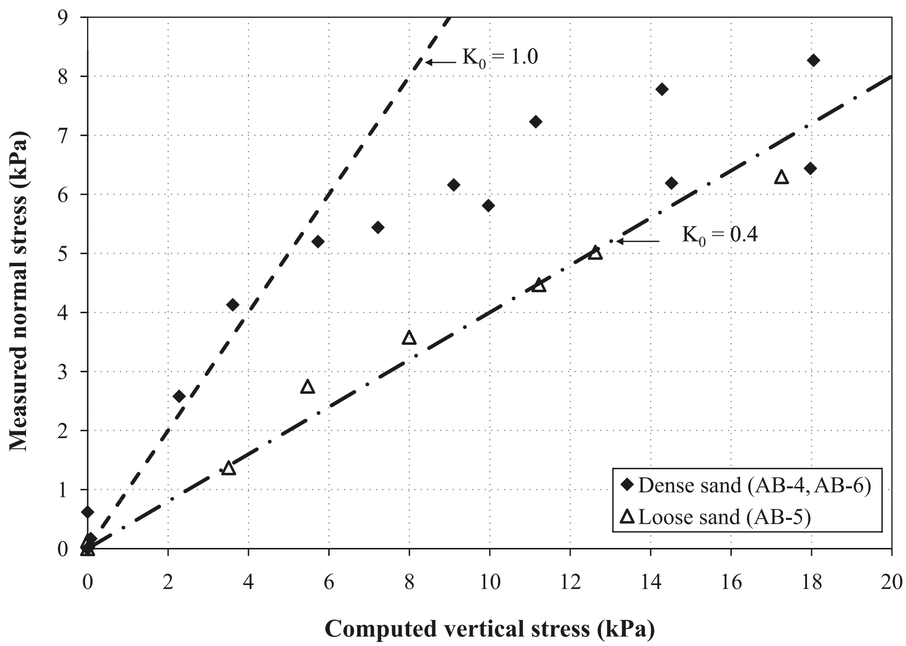


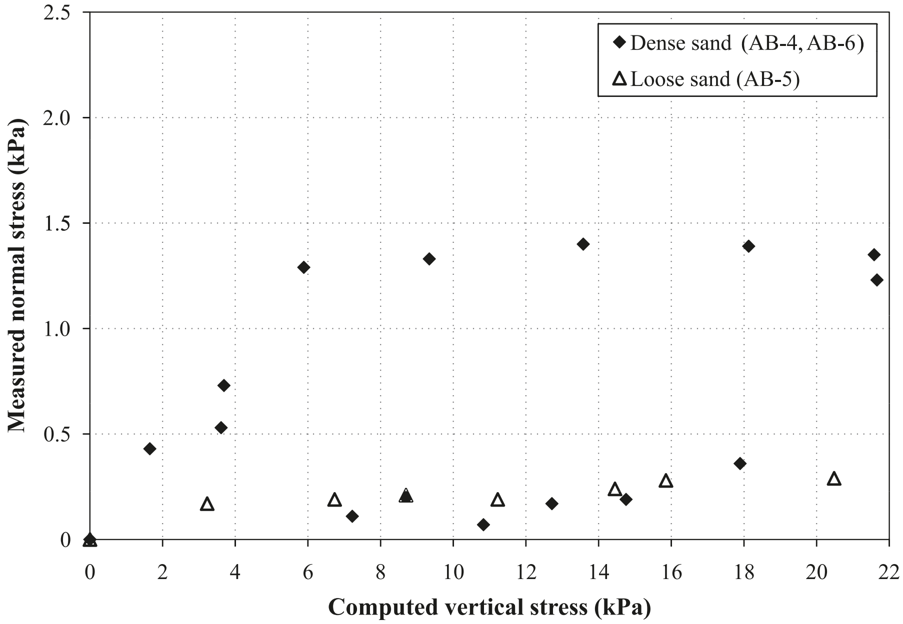
Fig. 6. Measured normal soil stress on the pipe at transducer PT3 versus computed vertical overburden soil stress.



the rate of decrease in the ratio between lateral and vertical stress is greater for the dense soil conditions, resulting in a decrease in the difference in measured lateral stresses between the loose and dense specimens with increasing overburden stress. These observations are generally in accord with the findings reported by Carder et al. (1977) and Duncan and Seed (1986) from their tests on retaining walls with regard to lateral earth pressures.

The measured pressure at transducer location PT1 (i.e., pipe invert level) is presented in Fig. 7. The transducer readings show significantly lower stresses than the estimated vertical effective overburden stress at the PT1 position, along with a considerable scatter. This low stress level is likely due to the unavoidable surface irregularities (i.e., minor surface unevenness) of the prepared invert-level soil surface prior to pipe placement, and consequently this led to a nonuniform stress distribution along the pipe length after pipe placement. These observations are also in agreement with the findings reported by Brachman et al. (2008) based on their extensive studies on strains within thermoplastic pipes; they have shown that the variation in density of soil below the haunches of the pipe (due to nonuniform compaction) could lead to potentially nonuniform support along the length of pipe.

Fig. 7. Measured normal soil stress on the pipe at transducer PT1 versus computed vertical overburden soil stress.



Soil pressures during axial pullout testing

As indicated earlier, soil pressure transducers were also monitored during axial pullout test Nos. AB-4, AB-5, and AB-6. To facilitate comparison, the measured pressures at a given transducer location were normalized with respect to the computed vertical overburden effective stress at the depth of the pipe springline to obtain the dimensionless normal stress (σ'_N) as per eq. [3]

$$[3] \qquad \sigma'_N = \frac{\sigma'_n}{\sigma'_v}$$

where σ'_n is the averaged pressures at a given transducer location and σ'_v is the computed effective overburden stress at the pipe spring line.

The measured soil stress on the pressure transducers exhibited relatively high frequency "noise" during pullout, possibly due to the localized particle movements and "abrasive" action at the soil–pipe interface. As such, to obtain a relatively smooth variation, the data were processed to obtain a moving average of the measurements. Dimensionless normal stress variations versus axial pipe displacements (i.e., computed moving average of σ'_N at different pipe circumferential locations with respect to axial pipe displacement) obtained for test No. AB-4 are shown in Fig. 8. It is noted that the pipe-displacement "window" (i.e., "*x*-axis window") used for computing the moving average had to be varied manually (by visual inspection of the original trace of the measurements) so that the key variations in the measured stresses with displacements were not unfairly obscured. Thus, computed moving average soil pressure readings were judged sufficient for cross-comparisons with axial load measurements and for identifying relative trends in the variation of soil pressures during axial pullout tests.

Discussion of experimental results

The normalized (dimensionless) axial pullout loads (F'_A) derived from the axial pullout tests for dense and loose sand are shown in Figs. 9 and 10, respectively. For the same test cases, the F'_A values for peak load and large-displacement conditions could also be estimated using eqs. [1] and [2] (as per ASCE (1984)); these values are also shown in the same two figures. The values of K_0 for use in eq. [1] were assessed as 0.37 and 0.42 for loose sand and dense sand, respectively, based on the final pressure transducer readings taken prior to pipe pullout for the two density levels. Based on the available laboratory data from interface direct shear tests (Karimian 2006), δ values of 31° and 36° were used in eq. [1] for the two density levels, respectively.

It is clear from Fig. 9 that the dense-sand tests (test Nos. AB-3, AB-4, and AB-6) exhibit much higher axial resistance than that predicted using eq. [1]. This is in contrast with the test results for loose sand (test No. AB-5) that are in good agreement with the predictions made using the same approach (see Fig. 10).

In lieu of K_0, an alternate K value was determined that, when substituted in eq. [1], would lead to F'_A values matching those from the measured axial soil load. (Note that as K_0 is the lateral earth pressure coefficient at rest, use of the symbol "K," indicating a lateral stress coefficient, is proposed herein to represent the back-calculated values of "K_0" using eq. [1].) As noted in Fig. 11, with a value of δ = 36° (as per interface direct shear tests), a value of K between 1.8 and 2.2 is necessary to obtain correspondence with peak axial resistance from tests Nos. AB-3, AB-4, and AB-6 in dense sand. Similarly, using a value of δ = 31° corresponding to large-strain friction (as per interface direct shear tests), a value of K between 1.5 and 1.8 is necessary to obtain correspondence with axial resistance at about 180 mm pipe displacement in the same tests.

The significant difference between back-calculated values of K and those based upon K_0 measurements just prior to the tests suggests that there is a substantial increase in the value of the normal soil stresses on the pipe during pullout. Inde-

Fig. 8. Variation of dimensionless normal stress (σ_N') on the pipe with axial pipe displacement during test No. AB-4 (with dense sand).

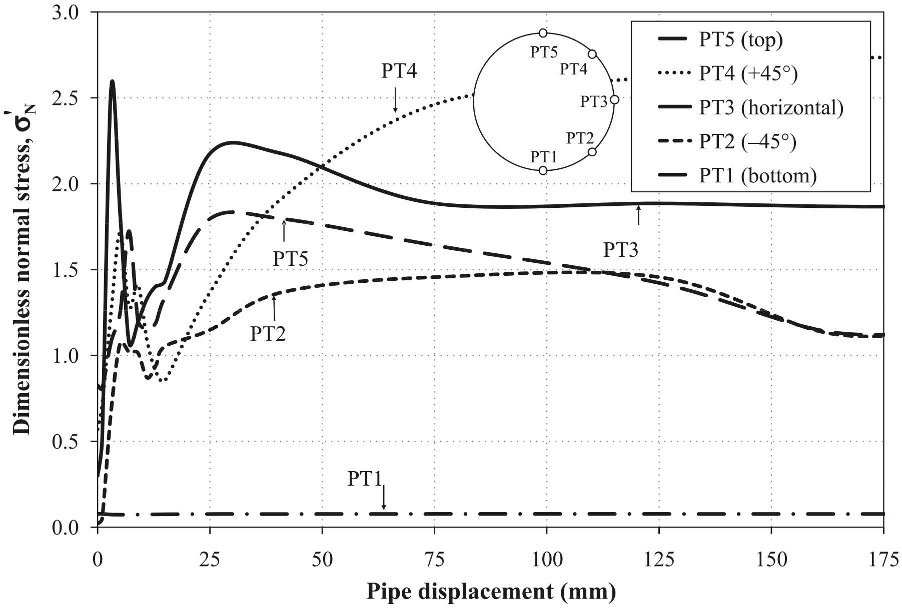
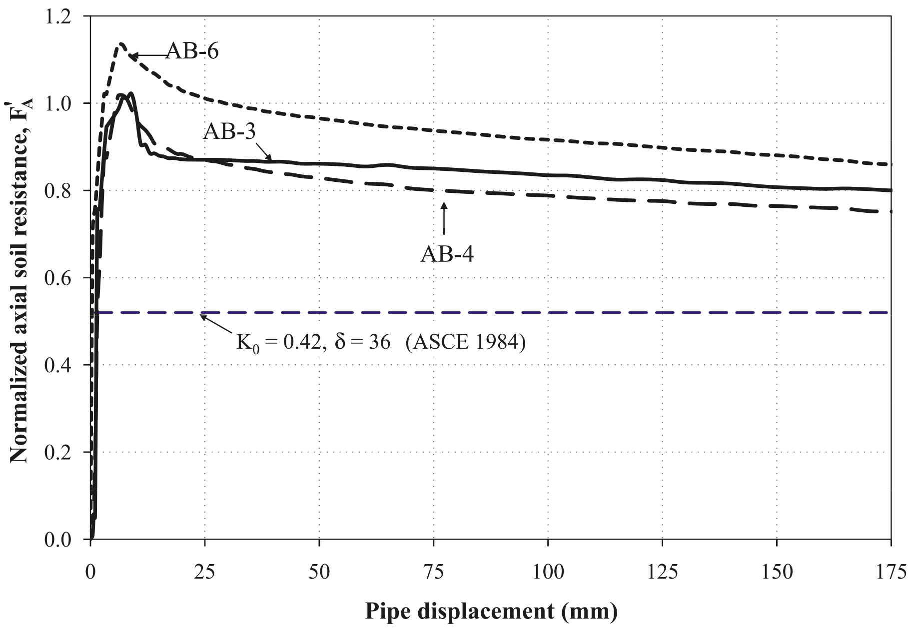


Fig. 9. Normalized axial soil resistance (F_A') versus axial pipe displacement from pullout tests on dense sand; predicted F_A' using ASCE (1984) plotted for comparison.



pendent soil pressure measurements on the pipe undertaken during some of the pullout tests (e.g., Fig. 8) provided an opportunity to directly investigate this further. The dimensionless normal stress (σ_N') at each pressure transducer location versus axial displacement during test Nos. AB-4 and AB-6 (with dense sand backfill) were averaged and plotted in Fig. 12 for the first 100 mm of the pipe displacement. For each transducer location, the σ_N' value corresponding to displacement at which the peak F_A' value occurred (about 10 mm of displacement) were extracted and presented as a radial plot on the right side of Fig. 13. The σ_N' values under "at-rest" conditions computed from the transducer measurements prior to pipe pullout are also plotted for comparison on the left side of Fig. 13. In a similar manner, the measured normalized radial pressure (σ_N') distribution corresponding to the occurrence of the peak F_A' at each transducer location during test No. AB-5 with loose sand is compared with the corresponding "at rest" normal pressure distribution in Fig. 14.

It is noted that, as presented in Fig. 7, the pressure measurements at the transducer location PT1 were found to be irregular and not reliable; this discrepancy is likely due to the unavoidable surface irregularities (i.e., minor surface unevenness) of the prepared invert-level soil surface prior to pipe placement, and consequently this led to a nonuniform stress distribution along the pipe length after pipe place-

Fig. 10. Normalized axial soil resistance (F_A') versus axial pipe displacement from pullout test on loose sand; predicted F_A' using ASCE (1984) plotted for comparison.

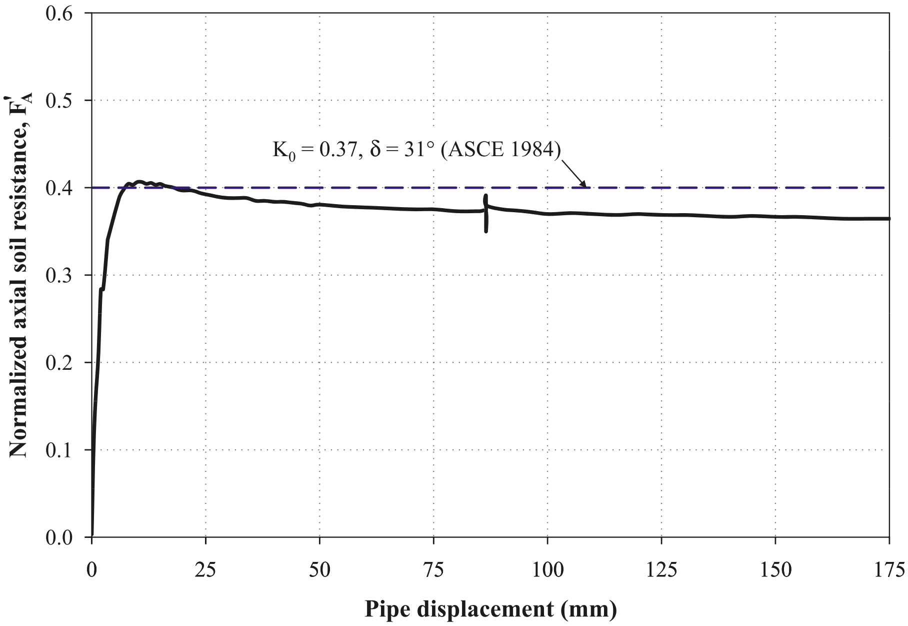
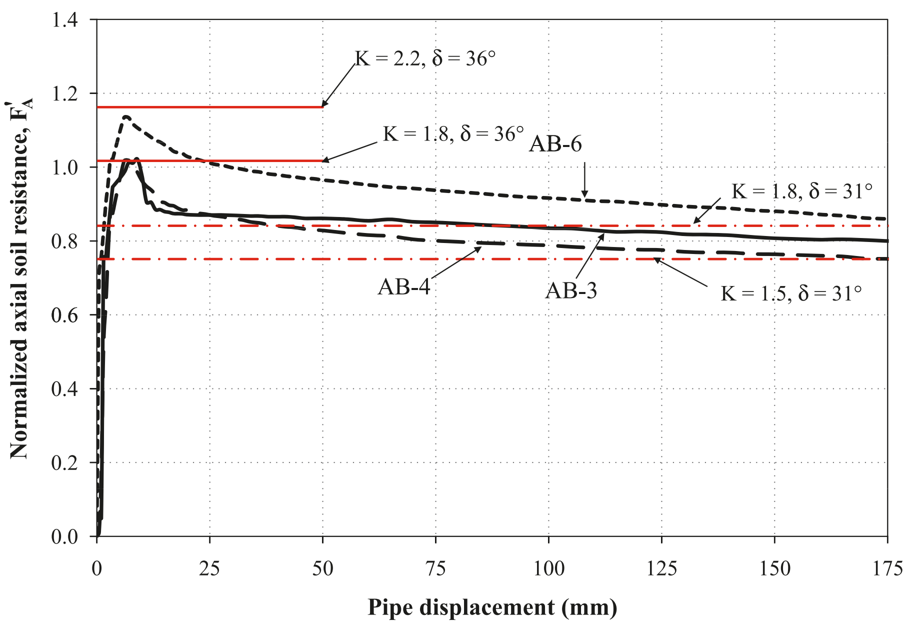


Fig. 11. Normalized axial soil resistance (F_A') versus axial pipe displacement from pullout tests on dense sand; predicted F_A' using ASCE (1984) formula for different K values plotted for comparison.



ment. Therefore, for the purpose of completing the σ_N' distributions presented in Figs. 13 and 14 over the full perimeter of pipe, it was judged reasonable to use the σ_N' value for the location PT1 as the value of ~1.0 estimated using the results of numerical analyses of this study (see next section "Numerical modeling").

It is clear from Fig. 13 that the average normal stresses on the pipe during pullout in tests with dense sand are significantly higher in comparison to the "at-rest" values. As may be noted, the largest stress increase appears to have occurred at the spring line, whereas the σ_N' distribution on the pipe during pullout in test No. AB-5 conducted in loose sand (Fig. 14) did not change significantly from those observed under "at-rest" conditions.

The term (1+K)/2 in eq. [1] represents an average σ_N' value over the perimeter of the pipe; therefore, it should also be possible to back-calculate the value of K, by averaging σ_N' distributions obtained from soil stress measurements on the pipe for tests in dense and loose sand presented in Figs. 13 and 14, respectively. Alternatively, the values of K can also be assessed from eq. [1] using the measured axial loads for the tests in dense and loose sands as described previously. The values of K determined using these two approaches are compared in the second and third columns,

Fig. 12. Variation of dimensionless normal stress (σ_N') on the pipe with axial pipe displacement: average of test Nos. AB-4 and AB-6 (with dense sand).

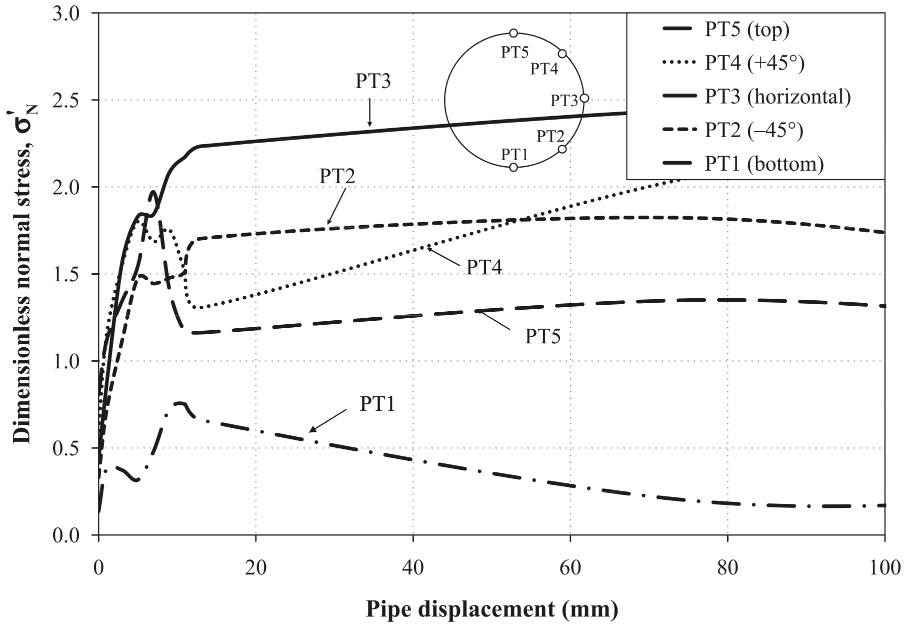
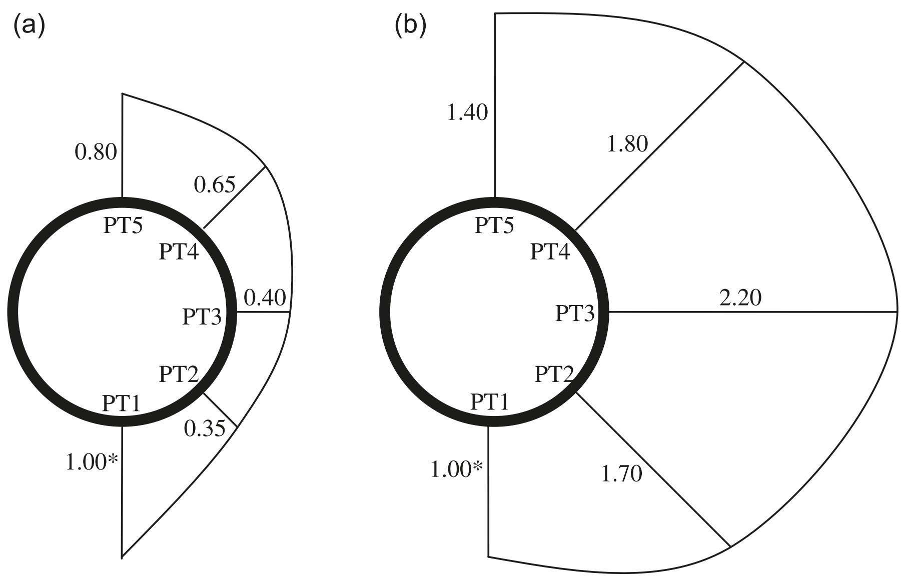


Fig. 13. Dimensionless normal stress (σ_N') on the pipe (*a*) just prior to axial pullout ("at rest conditions") and (*b*) during axial pullout (corresponding stresses at peak pullout load) based on pipe soil stress measurements averaged from test Nos. AB-4 and AB-6 (with dense sand). (Transducer locations: PT1 = invert; PT2 = haunch; PT3 = springline; PT4 = shoulder; PT5 = crown.) *, assumed value based on numerical modeling.



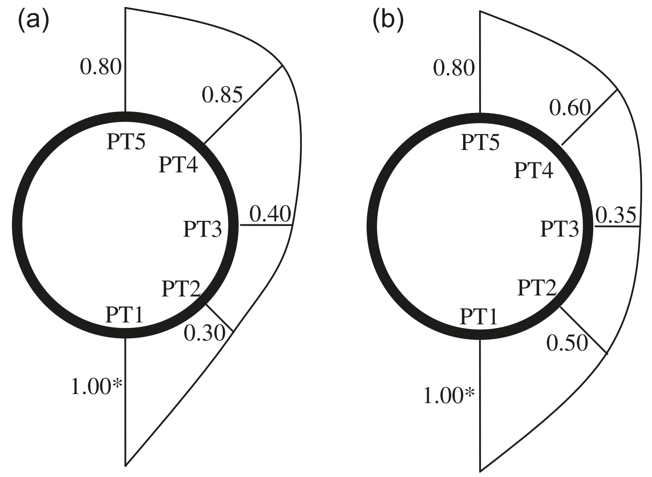
respectively, of Table 2. The value of lateral earth pressure just prior to axial pullout, estimated from the pipe stress measurements, is also shown in the same table. There is a general agreement between the *K* values back-calculated from axial load measurements and those obtained from soil pressure measurements, further confirming the validity of the observations.

The change in normal stress during pipe pullout can possibly be explained in terms of dilation (volumetric expansion) of the annular soil zone around the pipe. Dense sand would exhibit a tendency to dilate as it undergoes shear deformations at the interface during pullout. However, the tendency to dilate in the horizontal direction is significantly constrained by the surrounding soil mass (i.e., soil mass outside the shear zone), leading to the observed increase in lateral soil stress (up to 5 times the "at-rest" values) at the spring line of the pipe. Similar trends have been noted in pile research (Kraft 1991; Lehane et al. 1993; Randolph et al. 1994; Jardine and Overy 1996; Foray et al. 1998).

Numerical modeling

The increase in average normal soil stress on the pipe during pipe pullout observed in above full-scale laboratory

Fig. 14. Dimensionless normal stress (σ_N') on the pipe (*a*) just prior to axial pullout ("at-rest conditions") and (*b*) during axial pullout (corresponding stresses at peak pullout load) based on pipe soil stress measurements from Test No. AB-5 (with loose sand). (Transducer locations: PT1 = invert; PT2 = haunch; PT3 = springline; PT4 = shoulder; PT5 = crown.) *, assumed value based on numerical modeling.



testing suggests that the use of "at-rest" lateral earth pressure coefficient (K_0) in eq. [1] as per ASCE (1984) and Honegger and Nyman (2004) would lead to a significant underestimation of the peak load when the pipe is buried in dense (or dilative) material. Numerical modeling was undertaken to determine if an analytical determination of the effects of dilation in the sheared annular soil zone could match the experimentally observed increase in normal soil stress.

Considerations for development of numerical model

The fundamentals of soil dilation in an annular soil zone around the pipe during axial pullout are generally similar to the classical cavity expansion problem. The cavity expansion problem is typically analysed considering an axisymmetric stress state (Chadwick 1952; Vesic 1972). The presence of gravity in one direction and the nearby ground surface for the buried pipeline problem makes axisymmetric solutions unsuitable except for deeply buried pipes. Although the most realistic numerical modeling of the pipe axial pullout condition would require a three-dimensional (3-D) simulation, purely from the point of view of simplicity and cost effectiveness, it was decided to simulate the problem in a pseudo manner using plane strain two-dimensional (2-D) analysis, considering a vertical plane perpendicular to the pipe axis as shown in Fig. 15.

In the analytical model, it was assumed that the effect of volume change that takes place in the shear zone (see Fig. 15*a*) can be simulated by radially expanding the pipe "numerically" by the same amount as the estimated dilation effect (see Fig. 15*b*). Because of the relatively small thickness of the shear zone (as discussed later in this text), it was considered acceptable to assume that the perimeter of the pipe and the outside perimeter of the shear zone are coincident (see Fig. 15*b*). As this is only a 2-D model, the axial pullout that takes place normal to the plane of the paper is not modelled. The underlying premise is that if the dilation of the shear zone is mimicked through a radial expansion, a 2-D model would reasonably capture the changes in radial (normal) stress distribution, without the need for a 3-D model that would include simulation of the movements in the longitudinal direction.

The analysis was conducted using FLAC 2D (Itasca Consulting Group Inc. 2005), a commercially available computer program based on the two-dimensional explicit finite difference method. Soil material is represented by elements within an adjustable grid to fit the shape of the modeled object. The elements are quadrilateral with no intermediate node. This software is based on a "Lagrangian" calculation that is well suited to large deformations and material collapse.

The numerical model configuration developed on the above basis is shown in Fig. 16. The size of the model, position of the pipe, and boundary conditions were set identical to the physical model. The model consisted of 1073 soil elements and 38 beam elements that formed the pipe. The wall thickness and rigidity of the pipe were chosen to simulate the pipe that was used in the full-scale testing. The locations of pressure transducers mounted on the pipe during the physical model test are also shown in the same figure for completeness.

Constitutive relations and soil parameters

The constitutive model used in the numerical analyses to represent the soil stress–strain response is only briefly described below as detailed information related to the model is readily obtainable from the FLAC 2D 5.0 manual (Itasca Consulting Group Inc. 2005).

Soil elements were analytically modeled assuming a hyperbolic stress–strain response with Mohr–Coulomb failure criterion, as described by Duncan and Chang (1970) and Byrne et al. (1987). The key parameters required for the modeling are initial deformation parameters (e.g., initial Young's modulus, E_i, and Poisson's ratio, ν); failure ratio, R_f; and peak friction angle, ϕ_{peak}. R_f is a factor that changes the stress–strain curve for a better match with behaviour. In the current study, a R_f of 0.9 was assumed as per suggestions by Duncan and Chang (1970). The constitutive model in FLAC 2D 5.0 accounts for both elastic and plastic deformation of soil elements. Principal stresses and directions are evaluated from the stress tensor and corresponding strain increments are decomposed in the form of plastic and elastic strains. Dilation was modeled based upon the relationship proposed by Bolton (1986) as shown in eq. [4].

$$[4] \qquad \psi = \frac{\phi_{peak} - \phi_{cv}}{0.8}$$

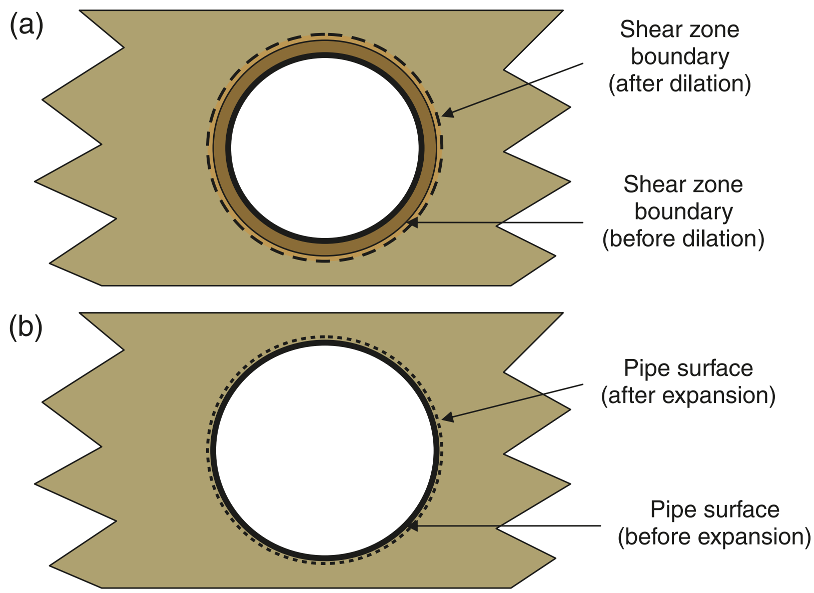
where ψ is the dilation angle.

Soil parameters for numerical analysis were obtained using data from laboratory triaxial element testing conducted specifically for this research (Karimian 2006). A peak friction angle of dense soil (ϕ_{peak}) of 45° was selected considering data from triaxial testing. The initial Young's modulus (E_i) of 36 MPa (for target dry soil density of $\sim$1600 kg/m^3 and for a confining stress level of 20 kPa (which is almost equal to the overburden stress at the pipe spring line) was also estimated from triaxial test results. Based on Bolton's

Table 2. Comparison of K values back-calculated from soil pressure measurements, axial load measurements, and numerical modeling.

Test backfill density	K derived from σ_N' estimated from soil pressure measurements on pipe	K back-calculated from axial load measurements on pipe (using eq. [1])	K based on numerical analysis using FLAC 2D	Coefficient of lateral soil pressure on pipe springline prior to pullout
Dense sand	2.4 (Fig. 13)	1.8 to 2.2 (Fig. 11)	1.9 to 2.2	0.42
Loose sand	0.23 (Fig. 14)	0.37 (Fig. 10)	N/A	0.37

Fig. 15. Configurations considered in modeling the effect of dilation on soil loads on pipe during axial pullout tests. (*a*) Size of annular shear zone before and after dilation; (*b*) expansion of pipe to mimic dilation effects in a simplified manner during numerical modeling.



relationship and considering a ϕ_{cv} of 33°, the dilation angle of Fraser River dense sand is calculated to be about 15°. It was assumed that the Poisson's ratio (ν) is constant and has a value of 0.3.

Unbonded interface elements with Coulomb shear-strength criterion as per eq. [5] were used to model the interface between the pipe and soil during radial expansion of the pipe.

[5] $$F_{smax} = cl + F_n \tan(\delta)$$

where F_{smax} is the shear force at the pipe–soil interface, c is the cohesion (in stress units) along the interface, l is the effective contact length along pipe, and F_n is the normal force on the interface. In the current model, because cohesionless material is considered, F_{smax} is only a function of interface friction and normal stress. The overall objective was to select a model that represents the soil–pipe system within acceptable error, yet is simple to use with parameters that are easily obtainable from simple laboratory tests. As indicated earlier, the peak and large-strain friction angle at the pipe–sand interface was estimated to be 36° and 31°, respectively. A constant interface friction angle value of 31° was selected for numerical analysis in this study. Sensitivity analyses indicated that variations in the interface friction angle from 31° to 36° have no significant effect.

Thickness of shear zone and degree of dilation at interface

One of the critical parameters for the proposed numerical modeling is the anticipated expansion of the annular shear zone around the pipe during pullout (see Fig. 15) in dense sand. Roscoe (1970) and Bridgewater (1980) suggested that the thickness of the actively sheared zone in a direct-shear mode of straining is about 10 times the mean particle size (d_{50}); this was supported later by observations via the radiography method by Scarpelli and Wood (1982). Based on micro-scale particle image velocimetry (PIV) observations, DeJong et al. (2006) observed that shear deformation and volume change is concentrated within a shear band with a thickness of 5–7 particle diameters adjacent to the interface. Accordingly, the thickness of the shear zone for Fraser River sand that has a d_{50} of 0.23 mm can be estimated to be in the order of 1 to 2.3 mm. Based on careful measurements of the movement of colored sand zones placed in the vicinity of the pipe during axial loading for sand-blasted steel pipes as well as polyethylene (PE) pipes, which was conducted as a part of the study presented herein, Karimian (2006) observed that only a small annular zone of about 1.2 to 2.8 mm in thickness is being sheared actively during pipe pullout (see Fig. 17 for an example); this compares well with the estimated shear zone thickness as per Scarpelli and Wood (1982) and DeJong et al. (2006). Observations during

Fig. 16. Geometry and mesh size for numerical modeling of the effect of dilation on soil loads on pipe during axial pullout tests.

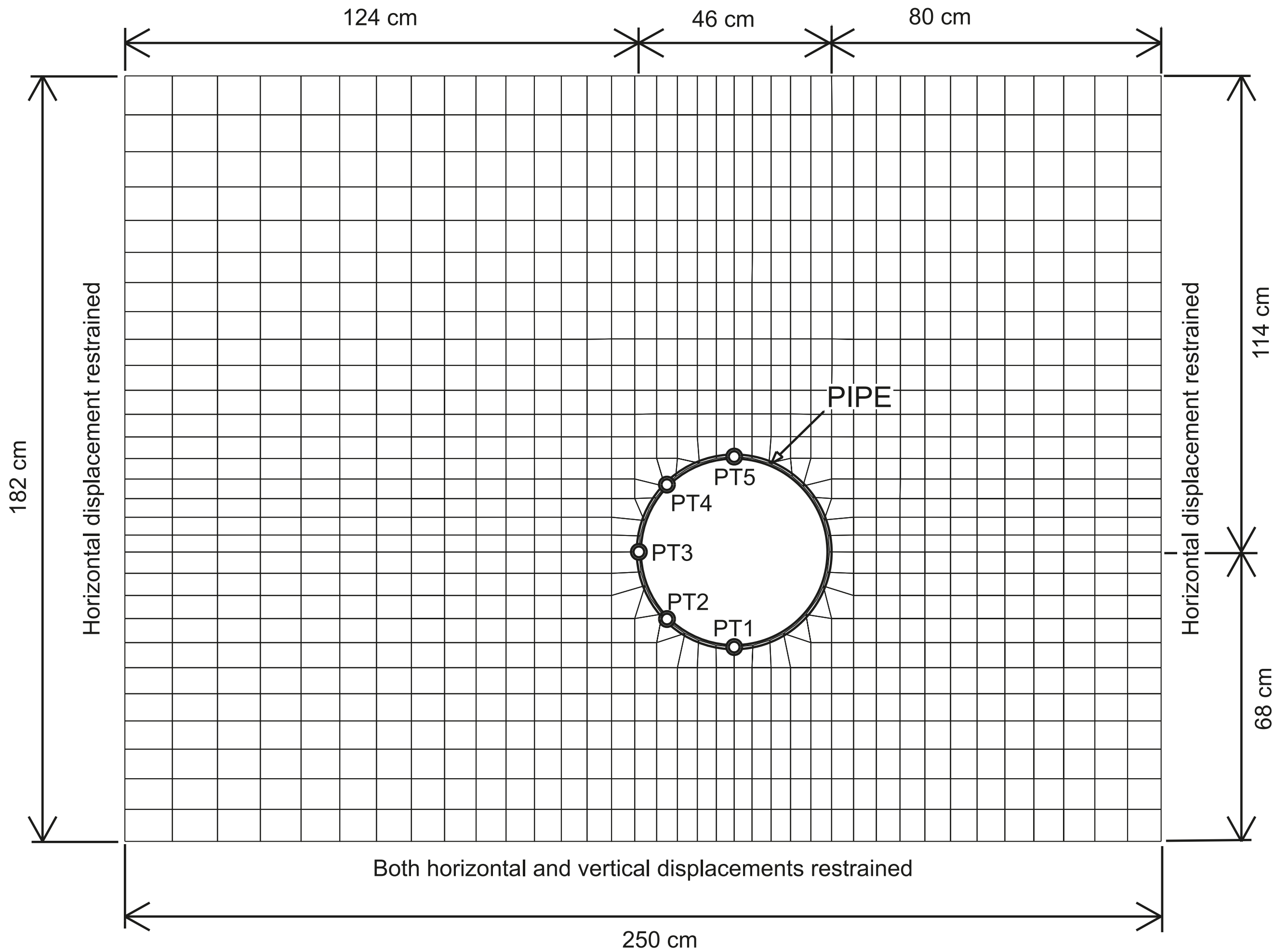
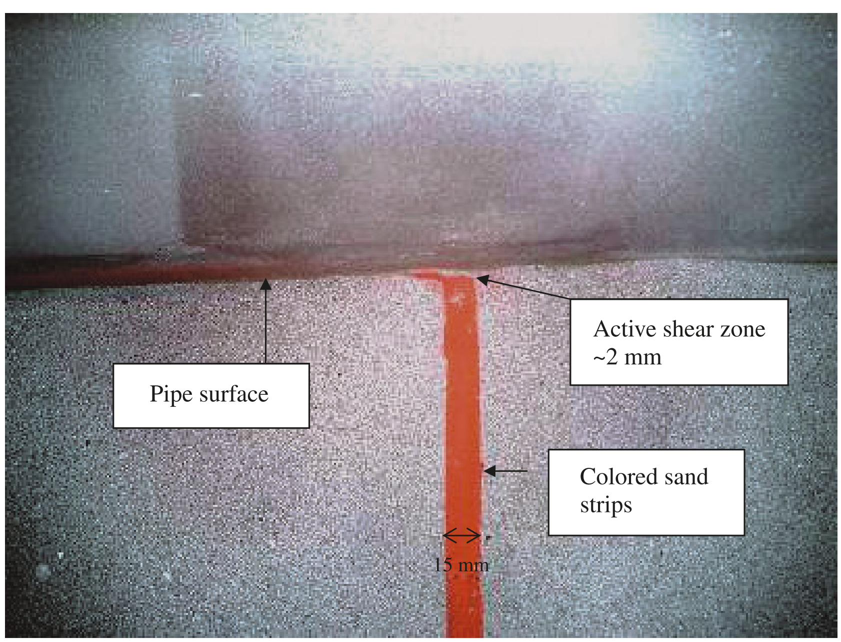


Fig. 17. Measurement of movement of sand particles in the shear zone during test No. AB-4 using colored sand zones.



direct shear tests of Fraser River sand were also available to assess the level of dilation when sheared up to the constant-volume phase. In this regard, the direct shear tests on compacted sand with an initial average relative density of 75% indicated that the average vertical deformation is about 0.72 mm, which is almost 30% of the above-estimated thickness of the active shear zone (2.3 mm). With this background, it was considered reasonable to assume a radial dilation of 0.7 to 1 mm at the interface for the purpose of numerical modeling.

Model validation

The measured normal stresses on the pipe at the locations of the pressure transducers during filling of the soil chamber and pipe pullout were used for comparison with those computed from numerical modeling. Independent of the above, the calculated average normal stress on the pipe from the numerical model could also be compared with average stress back-calculated from pullout loads in the tests.

Stresses under static conditions prior to pullout

To simulate the static condition around the pipe prior to pullout, the soil weight was applied in two steps during numerical modeling. At first, the soil up to the level of the pipe invert, along with the weight of the pipe, were simulated by activating gravity for the respective elements in the FLAC 2D model. Once equilibrium was reached under this condition, gravity was activated for the remainder of the model. Normal stress on the top (PT5) and bottom (PT1) of the pipe was directly available from the computed vertical stresses at those levels, and at spring line level (PT3) it was equal to the computed horizontal stress. Normal stress at positions PT2 and PT4 (on planes having –45° and +45° to the horizontal, respectively) were computed assuming the validity of Mohr's circle. The computed dimensionless normal stresses (σ'_N) at locations PT1 through PT5 are plotted in Fig. 18*a*. The values of σ'_N plotted for a given PT location in this figure were derived by considering stresses computed in elements in the immediate vicinity of the corresponding locations in the numerical model, and then averaging those values as appropriate. The σ'_N values derived from experimental measurements are depicted in Fig. 18*b*.

In an overall sense, the computed σ'_N at and above the spring line (at positions PT3, PT4, and PT5) are in general agreement with those derived from physical measurements. The agreement is poor at PT2; this, again, may be due to the difficulties in uniformly compacting sand backfill in the hard-to-access areas below the spring line in the preparation of physical models. As the experimentally measured pressure at PT1 was irregular, the σ'_N value at location PT1 was assumed equal to the value estimated using the results of numerical analyses as previously made for the comparisons in Figs. 13 and 14 (i.e., σ'_N at PT1 in Fig. 18*b* was assumed equal to that shown in Fig. 18*a*).

Normal stresses on pipe during pullout

The computed σ'_N from numerical modeling for the case simulating soil dilation (i.e., after expansion of the pipe by 0.7 to 1.0 mm) are presented in Fig. 19*a*. The stress distribution derived from numerical modeling under static conditions (already shown in Fig. 18) was also superimposed in

Fig. 18. Distribution of dimensionless normal stress (σ_N') around the pipe just prior to axial pullout based on (*a*) the results of numerical modeling and (*b*) pressure measurements using transducers. (Transducer locations: PT1, invert; PT2, haunch; PT3, spring line; PT4, shoulder; PT5, crown.) *, assumed value based on numerical modeling.

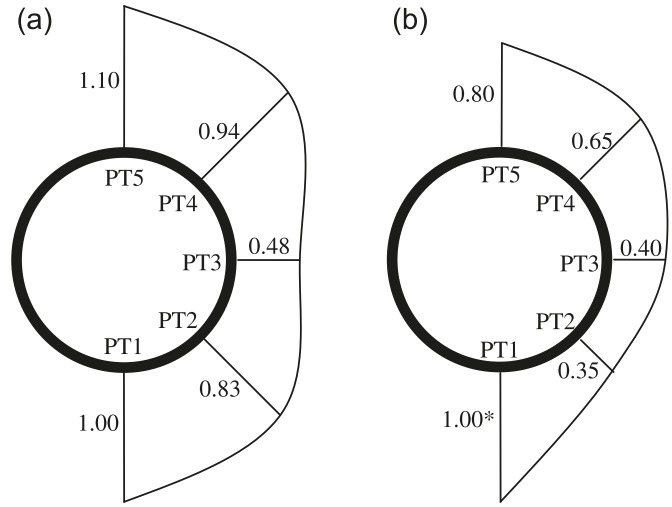
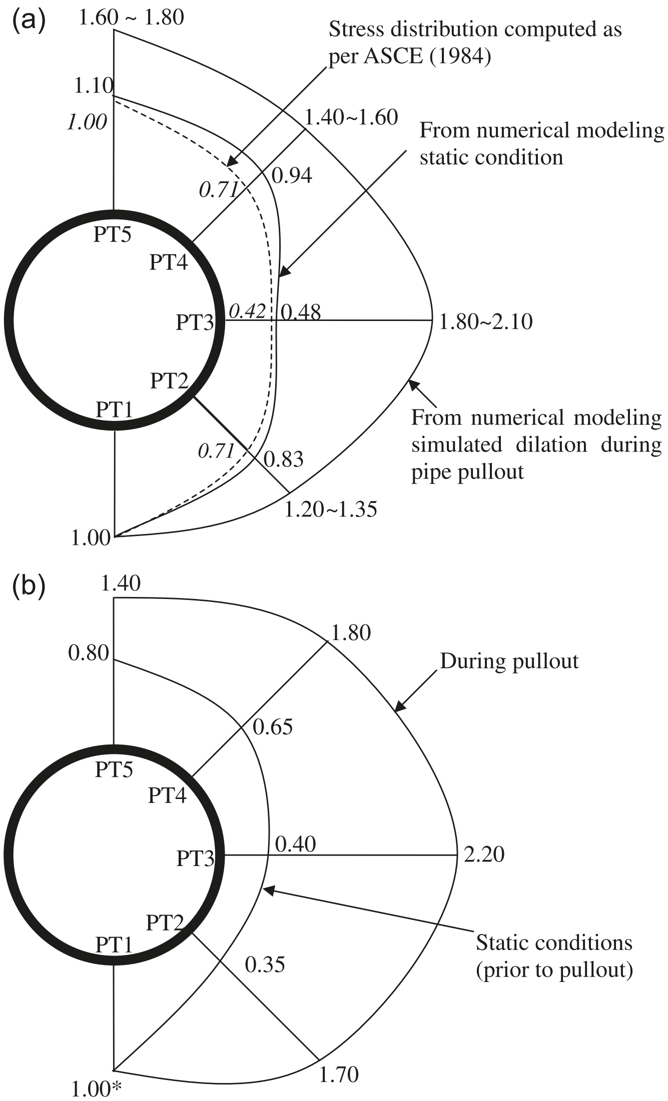


Fig. 19*a* for comparison purposes. As may be noted, numerical modeling has also predicted that the normal soil stress on the pipe would increase significantly due to dilation of sand within the shear zone. A normalized stress distribution around the pipe computed considering the ASCE (1984) guidelines (i.e., eq. [1]), and assuming linear variation of stress between the pipe crown and invert, is shown by the dashed curve in Fig. 19*a*. This stress distribution is clearly closer to that derived from the numerical modeling case simulating static conditions than that simulating pullout (dilation). It is also important to note that the computed normal stresses on the pipe in Fig. 19*a* are also in reasonable agreement with the values of σ'_N derived from direct measurement of normal stresses on the pipe given in Fig. 19*b*.

The computed normal stresses on the pipe perimeter in Fig. 19*a* allows back-calculation of the average σ'_N value and henc,e a value of K as the term $(1 + K)/2$ in eq. [1] represents average σ'_N value. The value of K estimated based on this approach is between 1.9 to 2.2 (for the dilation of the shear zone in the range of 0.7 to 1.0 mm, respectively), and this result is also included in Table 2 (fourth column). As may be noted, this result is in very good agreement with the value of K back-calculated from axial pullout loads, which is about 1.8 to 2.2 as per Table 2 (third column). Significantly smaller σ'_N (or K) values in comparison with the above are obtained if computations are made with the use of K_0 in eq. [1] as per ASCE (1984) and Honegger and Nyman (2004).

Clearly, the noted good mutual agreement between K values derived from numerical modeling, direct measurement of pipe stresses, and measured axial pullout loads corroborates the need to account for the stresses induced due to constrained dilation during relative axial soil movement. As the experimental observations from carefully controlled tests could be effectively represented by numerical analysis, it appears that the 2-D numerical modeling approach presented

Fig. 19. Distribution of dimensionless normal stress (σ_N') around the pipe during axial pullout based on the results of (*a*) numerical modeling and (*b*) pressure measurements using transducers. (Transducer locations: PT1, invert; PT2, haunch; PT3, spring line; PT4, shoulder; PT5, crown.) *, assumed value based on numerical modeling.



herein provides means of determining suitable "*K* values" instead of K_0 in eq. [1], under different soil dilation levels, pipe diameters, burial depth ratios, etc., in lieu of full-scale testing.

The finite difference numerical modeling approach considered herein to simulate soil dilation (i.e., the physical model) also provided an opportunity to assess the effect of boundary conditions on the soil loads on the pipe during axial pullout. In this regard, a finite difference mesh configuration representing a 6.0 m wide model, otherwise identical to that shown in Fig. 16 (i.e., identical pipe diameter, burial depth, chamber bottom depth, and soil parameters), was generated to assess the effect of side walls on the soil stress distribution during the simulation of dilation (Karimian 2006). Because of the selected 6 m width, the new configuration represented the conditions of a soil chamber where the distance to the side walls from the pipe surface was more than two times that corresponding to the 2.5 m wide real-life physical model. Based on this numerical modeling, Karimian (2006) reported that the overall stress distribution computed for the soil mass in the 6.0 m model (after numerically expanding the pipe by 1 mm) is very similar to that computed for the soil mass in the 2.5 m model. It was particularly noted that the normal stresses on the pipe computed from the two models were very similar, within about 5% of each other. These findings indirectly suggest that any boundary effects associated with the side wall boundaries are insignificant. In addition, measurements from the total earth pressure transducers, when mounted on the side walls of the soil chamber at approximately the same depth as the pipe spring line level, indicated no noticeable change in the transducer readings during axial pullout. This further supports the assumption that any boundary effects associated with the side walls are negligible.

Summary and conclusions

The performance of buried steel pipes subjected to relative soil movements in the axial direction was investigated using full-scale axial pullout loading of pipelines buried in a chamber filled with dry sand. In addition to the measurement of axial pullout loads and associated pipe displacements, the soil pressure on the pipe surface was monitored using pressure transducers mounted at several circumferential locations on the pipe test specimen. A 2-D plane strain model was developed to represent the experimental observations as a part of understanding the mechanisms involved in axial soil loading and evaluating the currently available methods for estimating axial soil loads.

It was noted that the measured axial soil loads from full-scale tests performed on pipes buried in loose dry sand are comparable to those predicted using the equation recommended in commonly used guidelines (ASCE 1984; Honegger and Nyman 2004). On the other hand, the peak axial resistance observed in pullout tests conducted on pipes buried in dense sand can be several-fold (in excess of 2 times) higher than those predicted from the commonly used guidelines.

The soil pressure measurements undertaken during pullout tests in dense sand indicated that the overall normal soil stresses on the pipe during pullout increased substantially in comparison with the initial values. This increase in normal stress is believed to be associated with the constrained dilation of the dense sand during shear deformations. It was determined that the use of a new parameter *K* (called equivalent lateral earth pressure coefficient) that is derived to represent an average of the normal stress distribution on the pipe is a more appropriate parameter than the "at-rest" lateral earth pressure coefficient (K_0) for use in eq. [1] proposed by ASCE (1984).

The 2-D plane strain model developed to represent the effect of dilation in the shear zone at the pipe–soil interface can provide the means of determining suitable "*K* values" for use in eq. [1] under different soil dilation levels, pipe diameters, burial depth ratios, etc., in lieu of full-scale testing. In the numerical model, the dilation of the shear zone was simulated by expansion at the interface. The computed normal stresses on the pipe after applying such expansion to the interface were in good agreement with normal stress measurements during axial pullout of the pipe. This further supports the notion that constrained dilation of sand in the shear zone is responsible for the increase in overall stress on the pipe.

The findings from this research suggest that caution is necessary when applying standard relationships (e.g., eq. [1]) for axial soil restraint on buried pipelines where dilative soil behaviour is possible. Such situations are most likely to exist in pipeline installations where the backfill is purposely compacted, such as those below road pavements or large flexible pipelines that require soil compaction to provide adequate lateral support of the pipe wall. It is important to note that the use of low K values in standard relationships, such as eq. [1], will result in a decrease of (*i*) the axial loads generated by ground movement and (*ii*) the rate at which axial soil restraint reduces axial pipeline loads outside of a zone of ground movement. As such, for a straight pipeline, underestimating the value of K can result in an unsafe condition as the maximum pipeline strains that typically occur within or at the boundaries of zones of ground movement will be underestimated. On the other hand, there are cases where the pipeline alignment outside of the zone of ground movement has bends or connections to pipeline equipment or other appurtenances that have lower resistance to axial loading compared with the pipe itself. The use of higher K values outside of a zone of ground movement will make the axial soil loads reduce at a faster rate over a given straight pipe length; this will result in underestimating the axial load experienced at the above-mentioned vulnerable locations (i.e., bends or connections) and, in turn, resulting in an unsafe condition. Where there is uncertainty regarding whether or not a high K value can develop, assuring adequate pipeline response for both upper and lower bound estimates of K is recommended. Additional research is necessary to provide more explicit guidance on methods to determine case-specific K values. In the interim, assessing the response of pipelines to ground displacement for installations that employ compacted sand backfill should consider a range of K values between K_0 and 2.5.

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