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RATE EFFECT ON LATERAL SOIL RESTRAINT OF PIPELINES

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ABSTRACT

The purpose of this study was to determine the pipeline lateral soil restraint due to relative movement between the pipeline and the dry sand. A large scale drag device with dimensions of $6' \times 6' \times 4'$ (1.8 m × 1.8 m × 1.2 m) had been fabricated to study the primary variables such as sand density, pipe diameter, pipe burial depth and relative velocity on the pipe lateral soil restraint. All 120 test results indicated that the dimensionless maximum soil restraints and the corresponding displacements exhibited the power law relationship with the pipe velocity, V/D. In dense sand, the pipe velocity exponents, n, were found constant and be irrespective of pipe burial depth, however, values of n increased with burial depth in loose sand and emerged with the dense sand constant exponent at the burial depth of 10.5 times of pipe diameter. Also, the generalized maximum soil restraints increased with depth of burial, until the burial depths with H/D of 10.5 in loose sand and of 12.5 in dense sand were reached, at which points, the soil restraints became constant. Force-displacement relationship of pipe-soil interaction could be represented by a two-constant hyperbolic equation. These two constant values of a and b were found to have the power law relationship with the pipe velocity, V/D.

Key words: pipeline, rate effect, soil restraint (**IGC:**D7/E5/H8)

INTRODUCTION

An ever increasing search for oil and gas resources has led to explore a variety of adverse environment. Pipelines serve as an efficient and reliable means of transporting oil and gas from their deposit to the consumption places. In the region of seismic area, ground movement such as landslide and differential motion, could cause excessive soil restraint on pipelines and damage them. Generally, pipes are installed in the excavated trench and covered with well compacted sand backfills. Current design analysis were emphasized on the maximum soil restraint on the pipes due to the soil-pipe relative movement. However, the relative velocity in the soil-pipe interaction which is estimated as a very important factor in the seismic region is not taken into account. Therefore, it becomes pertinently important to take the relative velocity factor into consideration for computing the soil restraint on the pipes.

The soil restraint on the pipes due to the relative motion between the pipe and soil may depend on several variables such as the burial depths of the pipe, soil density, pipe diameter and relative velocity between them. These

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variables are the usual and real situation occurring in the field. However, currently few available documents or literatures considering velocity effect are attainable for design purposes. For these reasons, extensively experimental study of the soil restraint on the pipes due to the relative motion becomes appropriate. In view of this pertinent essential, a large scale drag device with dimensions of 6 ft × 6 ft × 4 ft (1.8 m × 1.8 m × 1.2 m) has been fabricated. Pipe diameters of 1.5, 3, 6 and 9 in. (38.1, 76.2, 152.4 and 228.6 mm) with burial depths covering shallow and deep situation were performed to study the velocity effect ranging from 0.001 dia./sec. to 0.1 dia./sec. on the soil restraint of the pipes both in loose and dense sands. The velocity was in the range of static to impact loading conditions according to Vesic et al. (1965).

LITERATURE REVIEW

There are several published information available for lateral soil restraint on the pipe. Hansen (1961) used the passive Rankine state approach to evaluate the soil restraint in which he assumed that the vertical pile was bounded by two failure planes and the direction of the failure planes was the same as in the plane strain case. However, the model is not compatible with the real soil-pipe interaction due to the unsufficient displacement between the soil and pipe to fully mobilize the interface friction. Neely et al. (1973) developed surcharge method and equivalent free surface method which are based on plasticity theory to predict the soil resistance of the anchor plate. These two methods which disregarded the active pressure behind the anchor plate and assumed the fully mobilized friction between the soil and the plate had led to the overprediction of soil restraint. Ovesen (1964) developed a method by considering vertical equilibrium to estimate the lateral resistance of vertical anchor plate. Thomas (1978) used this model to predict the soil restraint on the pipe. He found that the soil restraint was obtained by multiplying the passive pressure with the reduction factor which was a function of backfilled soil height and pipe diameter. Nevertheless, the anchor plate was vertically restrained, interface friction might be fully mobilized and this could not be entirely existed in the soil-pipe interaction. Das and Seeley (1975) performed the pullout test on the rectangular plate anchor in loose sand and developed a normalized force-displacement relationship. Audibert and Nyman (1977) conducted a series of tests on model conduits with diameters of 25 mm, 60 mm and 114 mm with embedded depths H/D ranging from 1 to 24 in sand to study the soil restraint on the pipe. The results were also compared with the in-situ test of conduit diameter of 230 mm. However, the relative velocity between the pipe and soil was not considered in the report. Akinmusuru (1978) tested rectangular plate in loose sand with embedded ratios ranging from 1 to 10 and found the relationship between the dimensionless pullout capacity and embedded ratio. In his findings, the pipe was regarded as a deeply buried failure mechanism for the burial ratios beyond 6.5. Trautmann and O'Rourke (1983) reported that the pipe soil restraint increased with burial depth, until a embedded depth H/D of 8 was reached, at which point, the soil restraint became constant. But that finding only occurred in loose sand, no definite embedded depth with constant soil restraint was obtained in dense sand. The ultimate displacements associated with the maximum soil restraints were found to be about $0.13H$, $0.08H$ and $0.03H$ for loose, medium and dense sand, respectively. However, the velocity effect was not taken into account in the paper. Vesic (1971) reported that in the model test, beyond loading rate of 5.08×10^{-3} cm/sec, the bearing capacity of footing in dense sand increased as the loading rate increased. As stated by Georgiadis (1991), an increase in mudslide velocity resulted in higher drag forces on the pipelines and the relationship between velocity and drag force was found to be represented by a simple power law equation. Hsu (1987) used the model pipe of 0.75-in. (19.05-mm) diameter to study the drag forces in the marine sediment and found that drag forces on the pipe exhibited the power law relationship with the pipe velocity. However, these approaches were dealt with marine sediment which is classified as Mont morillonite clay and not for the sandy materials. Hence, for seismic design purposes, velocity effect study on the pipeline lateral soil restraint in sand becomes appropriate.

INSTRUMENTATION

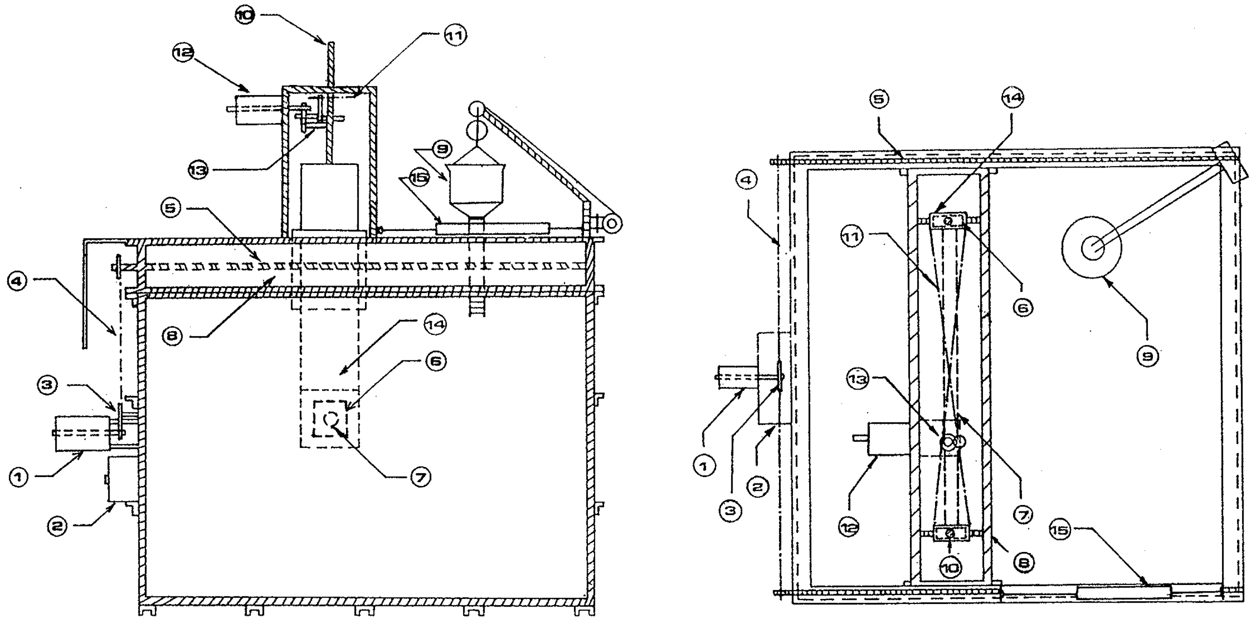
The experimental tests in this research program required the development of test equipment to provide enough space for studying the relative velocity effect between the pipe and soil, also to accomodate instruments to measure the soil restraint on model pipe with diameters ranging from 1.5 in. (38.1 mm) to 9 in. (228.6 mm) and pipe velocities covering from 0.001 dia./sec. to 0.1 dia./sec., in which pipe velocity was expressed in terms of pipe diameter per second, as well as burial depths, H/D, ranging from shallow to deeply buried pipes where H is the recess depth of the pipe measured from the bottom of the pipe to the sand surface and D is the pipe diameter. Hence, the equipment should be large enough to accomodate model pipes of adequate size without producing end effect, besides, it could also supply a wide range of pipe velocity to examine the velocity effect on the soil restraint without generating boundary effect. Roscoe and Poorooshasb (1964) found that for sandy soils, changes in stress can be modelled with adequate accuracy as long as geometrical similarity between the model and the prototype is maintained. In this study, different sizes of pipe with diameters of 1.5, 3, 6 and 9 in. (38.1, 76.2, 152.4 and 228.6 mm) were also tested to evaluate the scale effects. The test equipment consists of the following components: (1) large scale drag box; (2) instrumentation for measuring forces and displacements; (3) soil compaction control unit (4) data acquisition system; and (5) the model pipes. All of these components are briefly described in the following paragraphs:

Large Scale Drag Box.—This box is made of heavy steel with internal dimensions of 6-ft × 6-ft in plane, 4-ft high (1.8-m × 1.8-m × 1.2-m) and 0.25-in. (6.35-mm) thick. It was reinforced on the sides with angle steel to withstand 6 tons of sand with minimal deflection as shown in cross section in Fig. 1. The 6-ft (1.8-m) wide box provides 1-ft (0.305-m) clearance between the edge of the box and the end of the pipe to eliminate the possible edge effect and allows 4-ft (1.2-m) long pipe installed on the two sides of the vertical plates which mount on the platform. A 3-hp horizontal drive motor equipped with automatic switch gear drives the steel chain connecting leadscrew which is mounted on the top of box. Rotation of the leadscrew is translated to the motion of the platform mounted on the leadscrew; this provides the linear horizontal movement of the pipe in the soil. The gear box could provide the platform velocities ranging from 1.4 mm/min. to 726 mm/min. by changing the gears inside the gear box. Vertical drive motor actuates two vertical plates by using the same horizontal technique to provide vertical pipe movement.

Instrumentation for Measuring Forces and Displacements.—Pipe soil restraint was measured by the strain gage type transducers which were installed on the lower end of each vertical plate. Production with 350-ohm quarterly bridge single gages were placed on the transducers. The pipe horizontal displacement was measured by LVDT (Linear Variable Differential Transducer) which was mounted on the top right corner of the box. It was 350-ohm full bridge type with maximum displacement of 30 cm and precision of 0.1 mm.

Data Acquisition System.—Digital data acquisition system was used for data recording and processing. The analog signals from the load measuring transducer and displacement LVDT were taken with a sampling rate of 0.06 second per a channel. The digitized data were transmitted to a personal microcomputer through the interface GP-IB (IEEE-488) for storage on dual disks. A computer program was written which allowed the data to be output and printed in engineering unit by the digital plotter.

Soil Compaction Control Unit.—The sand was delivered through the spreading hopper. It was supported and governed by the hanger frame. The maximum uplift capacity of



(a) Side View

(b) Plane View

1. Horizontal Drive Motor
2. Horizontal Drive Gear Box
3. Change Gears for Variable Horizontal Speed
4. Horizontal Drive Chain
5. Lead Screw for Horizontal Movement
6. Force Measurement Transducer
7. Model Pipe
8. Platform
9. Sand Spreading Hopper
10. Lead Screw for Vertical Movement
11. Vertical Drive Chain
12. Vertical Drive Motor
13. Change Gears for Variable Vertical Speed
14. Vertical Plate
15. Linear Variable Differential Transducer

Fig. 1. Cross Section (a) side and (b) plane views of the large scale drag box

hanger frame was 200 kg and the hopper volume capacity was 0.06 m^3. The density of loose sand was controlled by the pouring distance of the sand through the flexible tube which is connected on the spreading hopper, whereas the density of dense sand was directed by the number of plate hammer compaction which corresponded to the same compaction energy of the Standard Proctor test. The dimensions of the plate hammer were 25 cm × 40 cm with the weight of 12.4 kg.

Properties of Sand.—Local sand of Taiwan Da-Du river bed was used throughout this investigation. The grain size distribution together with the physical properties of the sand were shown in Fig. 2. To study the effect of soil density on the pipe soil restraint, both loose and dense sands were conducted in the series of experimental tests. Densities of 1.55 and 1.75 g/cm^3 (relative densities of 21 and

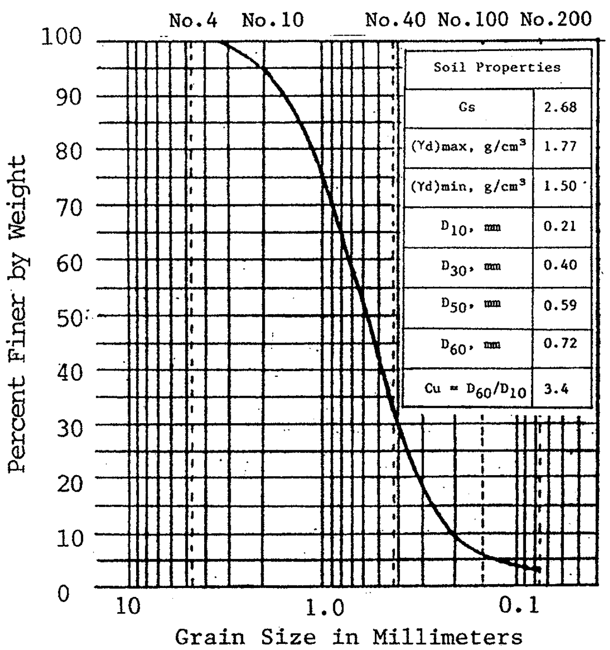


Fig. 2. Grain size distribution and physical properties of the test sand

94%), corresponding with direct shear internal friction angles of 33° and 42°, were found in loose and dense sand, respectively.

The Model Pipes.—Four sizes of pipes with diameters of 1.5 in. (38.1 mm), 3 in. (76.2 mm), 6 in. (152.4 mm) and 9 in. (228.6 mm) were used. All the pipes were 4-ft (1.2-m) long which were galvanized steel with 0.25-in. (6.35-mm) thickness. The exterior of the pipes was not machined, and the surface roughness was that of a normal exposed pipe.

EXPERIMENTAL PROCEDURE

For each test, a minimum of 30 cm sand was placed beneath in the box. The test procedure for each experiment was identical and briefly described below:

1. Lower the two vertical plates to the desired position, and install the pipe properly on the two sides of the plate. The embedded depth of the pipe in the sand was accurately obtained by measuring from the platform to the sand surface and to the top of the pipe.
2. The sand was uniformly spread in every 50-mm thick lift, the soil density control was previously described in the Instrumentation. During the preparation of the test, the soil density was carefully checked for every 10-cm thick lift at various location inside the box. A thin wall tube with 2-in. (50.8-mm) diameter and 4-in. (101.6-mm) height has an inside diameter of about 1⅞ in. (47.63 mm). The tube was gently pushed into the sand, all the sand inside the tube was carefully taken away with the spatula. The soil density could be obtained by weighing the removed sand and measuring the volume of the tube. Precision of 2% or better could be considered as homogeneous.
3. The desired pipe velocity was set in the gear box and the data acquisition units were adjusted for recording the test results.
4. The experiment was started by simultaneously triggering the data acquisition units and the gear box.
5. After the experiment was completed, remove all the sand out of the box due to the disturbance of the pipe during the test and be ready for the next test following procedures from step 1 to step 4.
6. A series of one size diameter pipe were conducted by changing the pipe velocity, embedded depth and sand density. Subsequently, another size of diameter was then installed for similar tests.

A total of approximate 120 tests were performed. First, 3-in. (76.2-mm) pipe was tested with burial depths, H/D, of 1, 1.5, 2.5, 4.5, 6.5, 8.5 and 10.5 at both loose sand (density=1.55g/cm^3) and dense sand (density=1.75 g/cm^3). Four velocities ranging from 0.001 dia./sec. to 0.1 dia./sec. were selected for each test. Owing to the limitation of the box dimensions, 3-in. (76.2-mm) pipe can only be tested at the burial depth down to 10.5. Hence, 1.5-in. (38.1-mm) pipe was performed to extend the results to the deeply embedded depths H/D from 10.5 to 20.5, as well as several duplicate tests at shallow depths to check 3-in. (76.2-mm) pipe results. Also, 6-in. (152.4-mm) and 9-in. (228.6-mm) pipes were conducted at shallow depths H/D of 2.5 and 1 separately to compare with the 3-in. (76.2-mm) pipe results for evaluating the scale effects. As mentioned above, each test was conducted at four velocities to estimate the velocity effect on the soil restraint.

TEST RESULTS

Maximum Force.—All tests were performed at a constant strain rate. Typical curves for 1.5-in. (38.1-mm) pipe in loose sand and 3.-in. (76.2-mm) pipe in dense sand were shown in Figs. 3(a) and 3(b), respectively. In the figures, the results are expressed in terms of the dimensionless force $F'=F/\gamma HDL$ versus dimensionless displacement $Y'=Y/D$, in which F=the total horizontal force exerted on the pipe; γ=the sand density; L=the pipe length; Y=the pipe displacement; and H and D are as previously defined. At this point, it is important to distinguish between the behavior in the various densities. In loose sand, a less definite point was observed without exhibiting a pronounced peak (Fig. 3(a)). Therefore, the maximum dimensionless force was selected at the point where the force-displacement curve has the maximum curvature. However, in dense

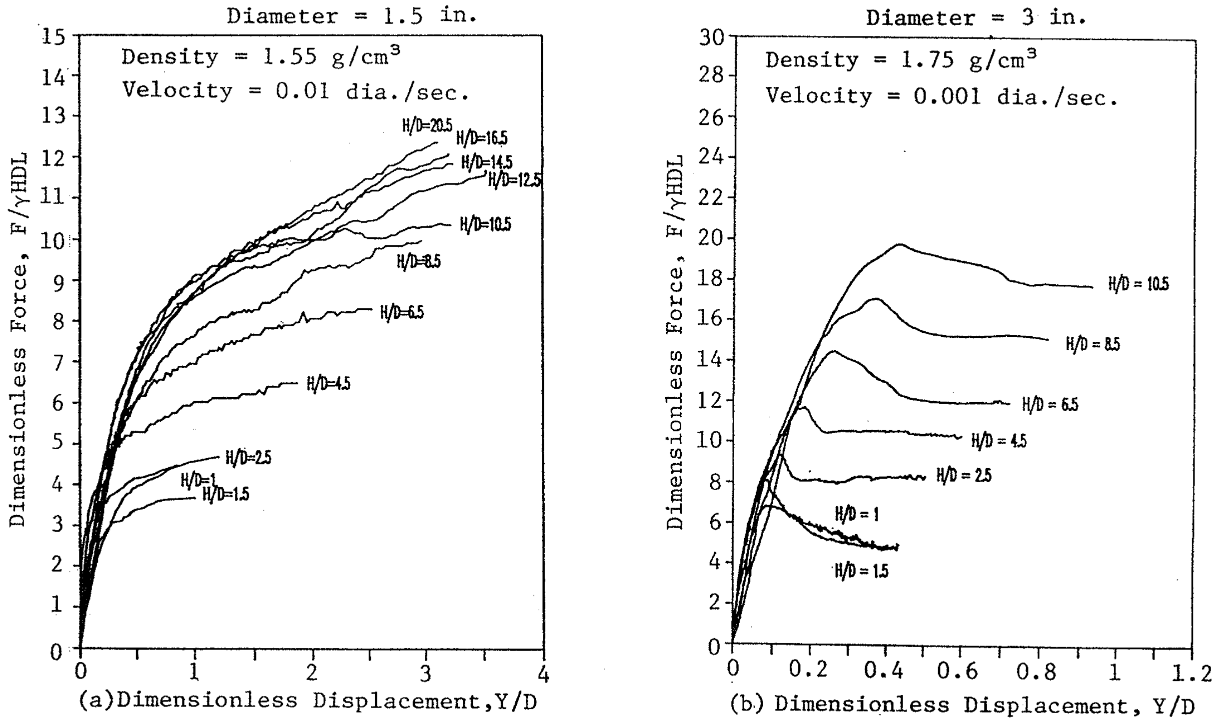


Fig. 3. Dimensionless force $F/\gamma HDL$ versus dimensionless displacement Y/D for (a) 1.5-in. pipe in loose sand and (b) 3.0-in. pipe in dense sand

sand, there was a definite peak point at which the force reached a maximum and then decreased (Fig. 3(b)). The maximum dimensionless force was denoted as F_m' and defined as $F_m' = F_{max}/\gamma HDL$, in which F_{max} is the maximum force. The corresponding dimensionless displacement was $Y_f' = Y_f/D$, where Y_f is the corresponding displacement associated with the maximum force. The results of the values F_m' for diameter 1.5-in. (38.1-mm) and 3-in. (76.2-mm) pipes in both loose and dense sands are shown in Figs. 4(a) and 4(b), respectively. As expected, F_m' increased as the burial depth increased. However, for 1.5-in. (38.1-mm) pipe, F_m' reached a constant value at H/D of approximately 12.5 in dense sand and of 10.5 in loose sand as shown in Fig. 4(a); and for 3-in. (76.2-mm) pipe, F_m' still increased as the burial depth approaches 10.5 as shown in Fig. 4(b). The findings confirmed the previous work done by Trautmann and O'Rourke (1983) that F_m' did not reach a constant value for H/D less than 11.5 in dense sand. Also, as shown in the figures, F_m' increased with the pipe velocity increasing. The detail of this rate effect on F_m' will be discussed in the subse-

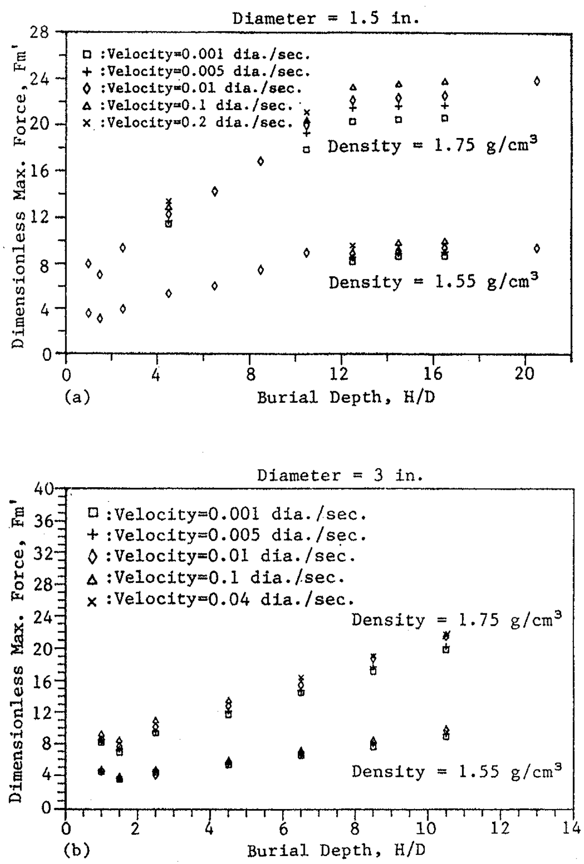


Fig. 4. Dimensionless maximum force F_m' versus burial depth H/D for (a) 1.5-in. pipe and (b) 3.0-in. pipe

quent section.

Displacement Associated with F'_m.—The dimensionless displacements corresponding to maximum force, Y_f/D, are replotted as the ratio Y_f/H versus burial depth HD and shown in Figs. 5(a) and 5(b) for 1.5-in. (38.1-mm) and 3-in. (76.2-mm) pipes, respectively. As shown in the figures, values of Y_f/H decreased as burial depth increased. The average values of Y_f/H for loose and dense sand are approximately equal to 11% and 6%, respectively. In loose sand, Y_f/H becomes an average constant value of 9% as the burial depth reaches 10.5 for 1.5-in. (38.1-mm) pipe, however, it approaches the same constant value of 9% as the burial depth reaches approximately 5 for 3-in. (76.2-mm) pipe. At these burial depths, 1.5-in. (38.1-mm) and 3-in. (76.2-mm) pipes are subjected to the same overburden pressure. It indicates that the burial depth at which Y_f/H reaches the constant value in loose sand depends upon the overburden pressure where the pipes are subjected to. In dense sand, Y_f/H becomes constant at an average value of 5% as the burial depth approaches approximately 5 regardless of the pipe diameter. This constant displacement, Y_f/H, is consistent with the results done by Trautmann and O'Rourke (1983) in which displacement of 3% for dense sand was found, whereas displacement of 9% in this experimental study less than 13% comparing with their findings in loose sand was tributed to the difficulty in selecting the point of maximum force. Besides, Y_f/H increases with the pipe velocity increasing. The discussion will be shown in next.

Rate Effect on Maximum Force and Corresponding Displacement.—Four different strain rates ranging from 0.001 dia./sec. to 0.1 dia./sec. were used in the series of tests for 3-in. (76.2-mm) pipe. Results were plotted on a log-log scale in terms of dimensionless maximum force, F'_m, versus pipe velocity, V/D. Fig. 6 shows the linear relationship was obtained from these tests. The behavior of this plot suggests that the dimensionless maximum force could be described by a simple power law function of the form:

$$F'_m = K(V/D)^n \tag{1}$$

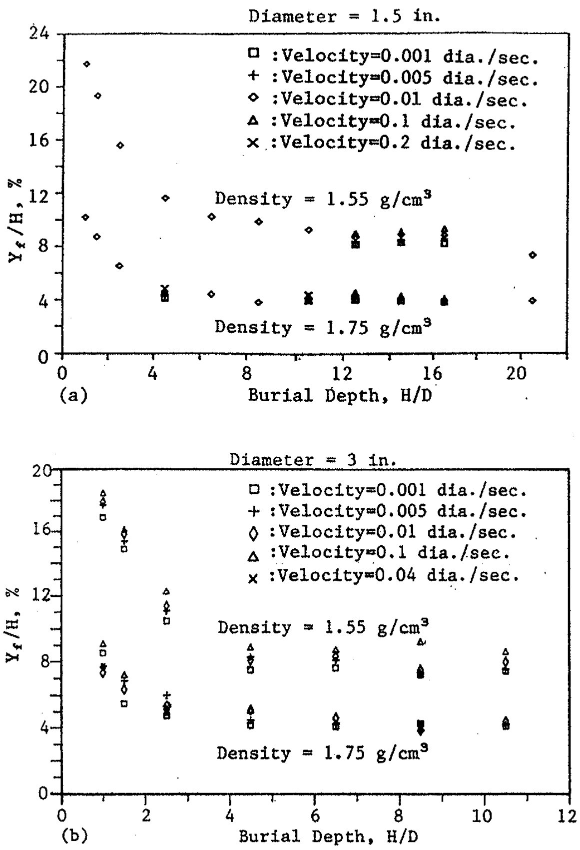


Fig. 5. Dimensionless displacement Y_f/H versus burial depth H/D for (a) 1.5-in. pipe and (b) 3.0-in. pipe

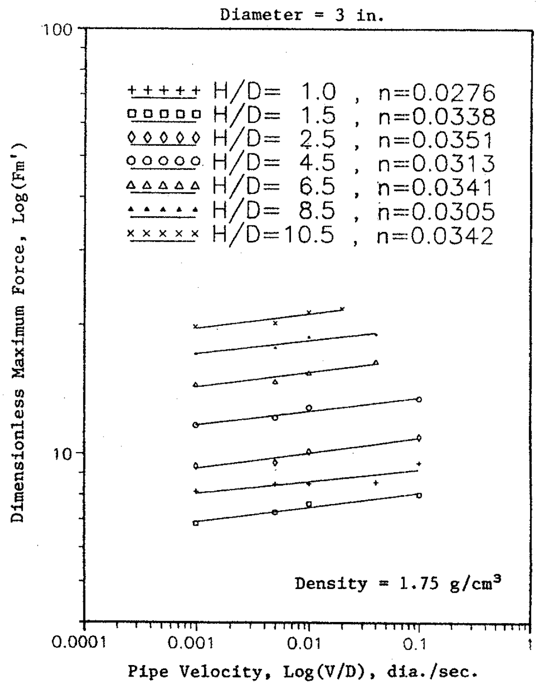


Fig. 6. Dimensionless maximum force F'_m versus pipe velocity (V/D) for 3.0-in. pipe

where K and n=experimental parameters which depend on the pipe burial depth and soil density. The velocity exponents, n, remain constant essentially indepedent of pipe burial depth in dense sand, whereas for loose sand, values of n increase with burial depth and approach the dense sand constant exponent value at H/D of 10.5 as shown in Fig. 7. Surprising findings indicated that this burial depth was just the burial depth at which maximum forces start remaining constant in loose sand. Based on the power law relationship, the dimensionless maximum forces could be expressed in terms of the generalized maximum force coefficients, $F_m'/(V/D)^n$, which in turn are independent of the pipe velocity. The corresponding generalized displacement coefficients, $(Y_f/H)/(V/D)^n$, could also be obtained. Plotting generalized maximum force coefficients and corresponding generalized displacement coefficients versus burial depths result in more condensely data as shown in Figs. 8 and 9, respectively. From these graphs, the rate effect on maximum forces and corresponding displacements could be explicitly expressed in terms of pipe velocity, V/D, in the form of power law.

Scale Effects of Pipe Diameter.—The scale effects of pipe diameter were determined by two series of tests. First, 1.5-in. (38.1-mm) pipe was used with burial depths H/D of 4.5 and 10.5 at various velocities with V/D= 0.001, 0.005, 0.01, 0.1 and 0.2 dia./sec. to check the 3-in. (76.2-mm) pipe results. After that, additional tests were performed to ex-

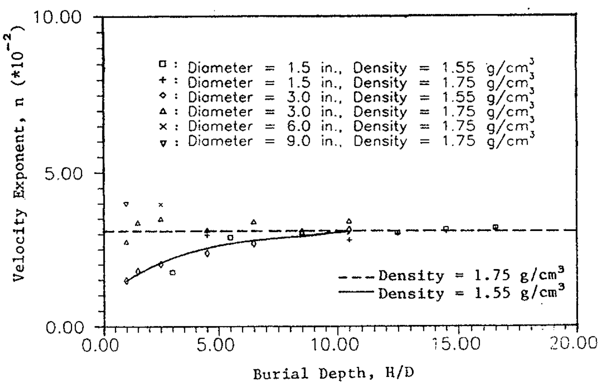


Fig. 7. Velocity exponent n versus burial dcpth H/D for different pipe velocity

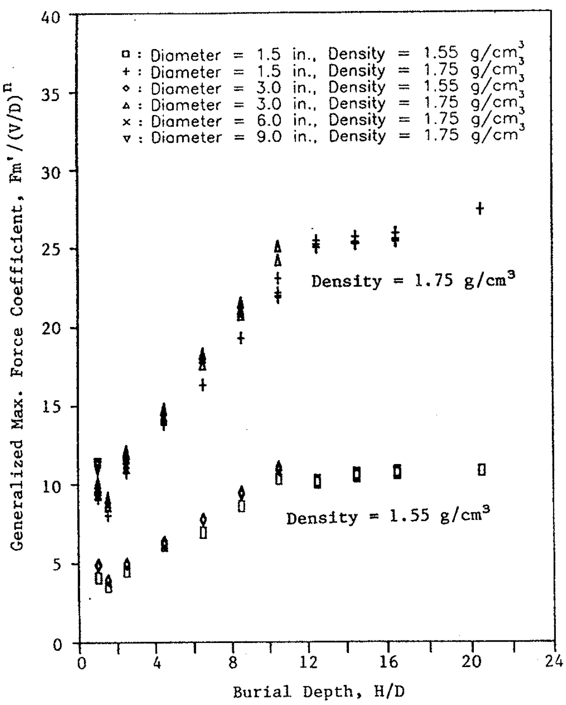


Fig. 8. Generalized max. force coefficient $F_m'/(V/D)^n$ versus burial depth H/D for pipe loading tests

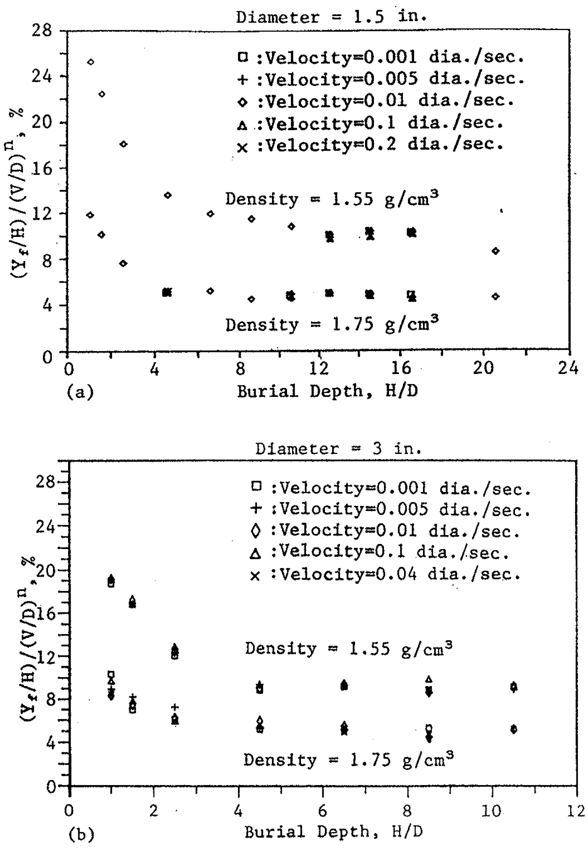


Fig. 9. Generalized displacement coefficient $(Y_f/H)/(V/D)^n$ versus burial depth H/D for pipe loading tests

tend the results to the deeply burial depths with $H/D=12.5$, 14.5 and 16.5 at the same above velocities, as well as the burial depths with $H/D=1$, 1.5, 2.5, 4.5, 6.5, 8.5, 10.5 and 20.5 at one velocity with $V/D=0.01$ dia./sec. Second, 6-in. (152.4-mm) and 9-in. (228.6-mm) pipes were conducted at shallow depths with H/D of 2.5 and 1 separately under various velocities with $V/D=0.001$, 0.005, 0.01 and 0.05 dia./sec. to check the scale effects. Results shown in Fig. 8 indicate that the scale effects of the pipe diameter on the soil restraint are not significant. This findings are consistent with the report by Audibert et al. (1978) and Thomas (1978), in which the scale effects were minor for the pipe diameter up to 2 ft (609.6 mm). Besides, the velocity exponents of 6-in. (152.4-mm) and 9-in. (228.6-mm) pipes were slightly higher as compared with 3-in. (76.2-mm) pipe in Fig. 7.

MATHEMATICAL EXPRESSION OF FORCE-DISPLACEMENT RELATIONSHIP

Based on the extensively experimental study, the explicit form of the force-displacement relationship can be modelled as a two-constant hyperbolic equation which was originally suggested by Kondner (1963), and reported by Audibert and Nyman (1977), Das and Seeley (1975) and Trautmann and O'Rourke (1983). The generalized form for this expression is:

$$F''=\frac{Y''}{a+bY''} \tag{2}$$

in which $F''=(F/\gamma HDL)/(F_m/\gamma HDL)$; $Y''=(Y/D)/(Y_f/D)$; and F and Y=the measured force and displacement in the experiment. Both F'' and Y'' are normalized and ranged from 0 to 1. Explicit forms of two constant values a and b for each pipe velocity were found by using regression analysis with the coefficients of correlation given as:

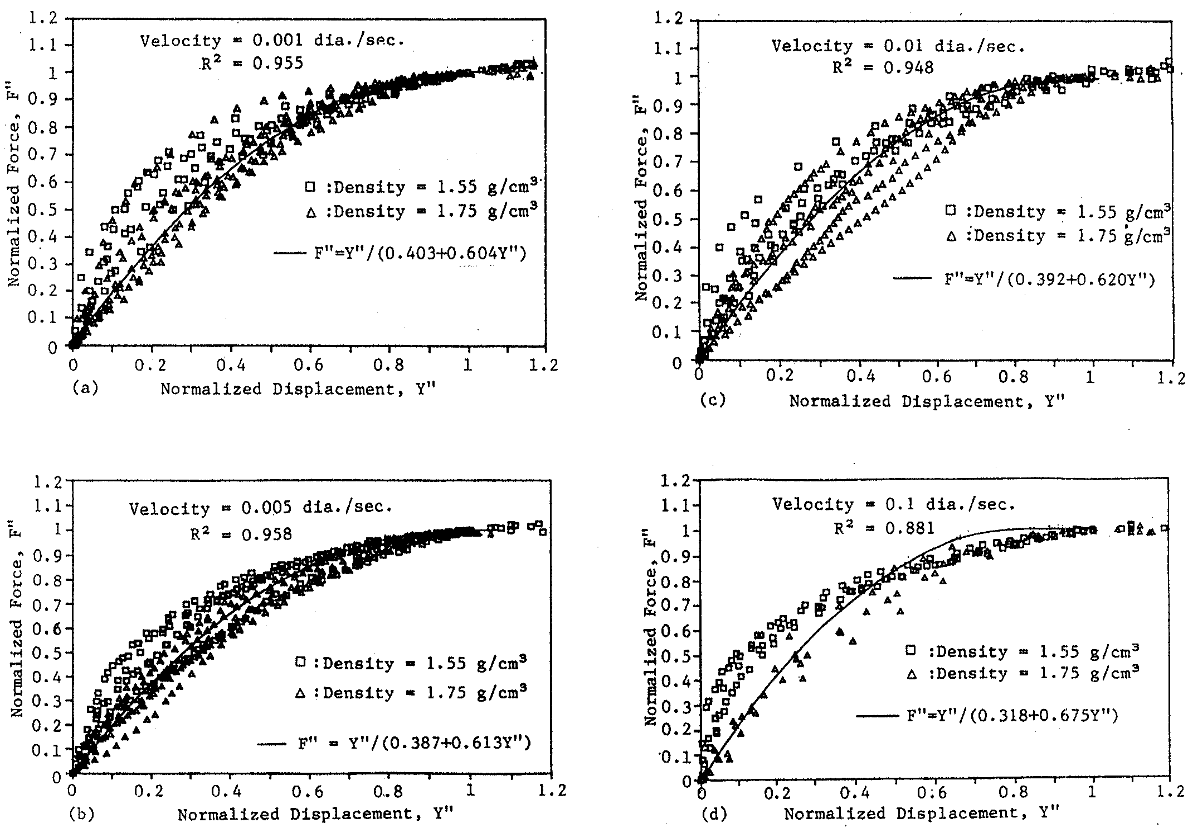


Fig. 10. Normalized force F'' versus normalized displacement Y'' for different pipe

For

$V/D=0.001$ dia./sec.;

$$F''=\frac{Y''}{0.403+0.604Y''} \quad R^2=0.955 \qquad (3)$$

$V/D=0.005$ dia./sec.;

$$F''=\frac{Y''}{0.387+0.613Y''} \quad R^2=0.958 \qquad (4)$$

$V/D=0.01$ dia./sec.;

$$F''=\frac{Y''}{0.392+0.620Y''} \quad R^2=0.948 \qquad (5)$$

$V/D=0.1$ dia./sec.;

$$F''=\frac{Y''}{0.318+0.675Y''} \quad R^2=0.881 \qquad (6)$$

Selected experimental points from each velocity test and the best fit of the rectangular hyperbola are shown in Fig. 10. Plotting logarithm of these two constant values a and b versus logarithm of pipe velocity, V/D, respectively resulted in a linear relationship. Thus, the hyperbolic two constant values of a and b after regression could also be expressed in the form of a power law function as:

$$a=0.29^*(V/D)^{-0.052} \quad R^2=0.981 \qquad (7)$$

$$b=0.71^*(V/D)^{0.025} \quad R^2=0.983 \qquad (8)$$

CONCLUSIONS

This paper has presented extensively experimental study on the pipe lateral soil restraints due to the various velocities between the pipe and the surrounding dry sand. Based on the experimental findings, conclusions can be drawn as follows:

1. The dimensionless maximum soil restraints and the corresponding displacements exhibit the power law relationship with the pipe velocity, V/D. In dense sand, the pipe velocity exponents, n, are essentially constant and be irrespective of pipe embedded depth, however, values of n increased with burial depth in loose sand and emerged with the dense sand constant exponent at the burial depth of approximately 10.5 times of pipe diameter.
2. The generalized maximum force coefficients increase with depth of burial, as it might be expected, until the burial depths with H/D of 10.5 in loose sand and of 12.5 in dense sand are reached, at which points, the soil restraints become constant. The corresponding displacements decrease with burial depth increasing until at the burial depth with H/D approximately equal to 5 in dense sand, at which point, the displacement remains constant. However, in loose sand, the burial depth at which the displacement reaches constant is affected by the overburden pressure where the pipes are subjected to. The generalized displacement coefficients at these depths were found at the values of 5% in dense sand and of 9% in loose sand.
3. Force-displacement relationship of pipe-soil interaction could be represented by a two-constant hyperbolic equation. These two constant values of a and b were found to have the power law relationship with the pipe velocity, V/D.
4. Test results show that the experimental data exhibit no significant differences among various sizes of pipe diameter up to 9 in. (22.86 cm). It indicates that scale effects are minor and agree closely with the findings of Audibert et al. (1978) and Thomas (1978).

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