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Buried Pipeline Damage Caused by Soil Liquefaction under the Slope

Hong SUN[1], Masakatsu MIYAJIMA[2] and Lei QIAO[3]

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Abstract

Soil liquefaction does major damage to buried pipelines during earthquakes. The laboratory shaking table experiments and numerical analysis for buried pipeline under the slope were carried out. The pore water pressure buildup, sloping ground deformation, and deformation of pipe were studied. It is available to use the nonlinear method to simulate the soil-structure interaction. It is necessary to find a simplified analysis method for predicting pipe damage.

Key Words: Liquefaction, Pipeline, Damage, Shaking table, Three-dimensional analysis

Ⅰ. Introduction

Liquefaction-induced ground deformations are generally observed close to open faces, or in gently sloping ground. Liquefaction-induced ground deformation is a potential source of major damage to buried pipelines during earthquakes. During the 1994 Northridge earthquake, several pipelines were broken due to large permanent ground deformation caused by soil liquefaction. They were numerous during the 1995 Hyogoken-Nanbu earthquake (Hamada *et al*., 1996a and 1996b) and caused substantial damage to lifelines and other facilities along the Kobe shoreline (Hamada *et al*., 1996a; Karube and Kimura, 1996; Matsui and Oda, 1996; and Tokimatsu *et al*., 1996).

For buried pipelines, seismic damages can be classified into wave propagation damages and permanent ground deformation damages. There have been some events where pipe damage has been due only to wave propagation. More typically, pipeline damage is due to a combination of hazards. However large ground deformation caused damage typically occurs in isolated areas of ground failure, with high damage rates, while wave propagation damage occurs over much larger areas, but with lower damage rates.

The laboratory shaking table experiments and numerical analysis for buried pipeline under the slope were carried out.

Ⅱ. Shaking Table Experiments

Shake-Table experiments used a model of slope ground and a pipe buried under the crest of the slope in a box 1,800mm long by 600mm wide by 800mm high. The model ground consisted of 400mm thick fully-saturated liquefiable No.5 silicon sand made by water-pouring method, overlaid by free-falling 200mm thick dry No.5 silicon sand with a 2H:1V slope with its crest at the center of the box. Model pipe was a 25mm diameter and was buried crossing the full box width and 100mm

[1]Department of Civil Engineering, Shanghai Jiao Tong University, 800 Dongchuan Road, Shanghai, China
[2]School of Environmental Design, College of Science and Engineering, Kanazawa University, Kakuma-machi, Kanazawa 920-1192, Japan
[3]Jiangsu Provincial Geo-Engineering Investigation Institute, Nanjing City, Jiangsu Province, China

Fig.1　Configuration of model pipe.

Fig.2　A photo showing the test model before shaking.

below the crest with two ends fixed, shown in Fig.1. A horizontal accelerometer was fastened on the pipe at the central location. Fig.2 presents a side view of the model. The model ground was applied by different amplitudes of horizontal input sine waves varying from 100 gal to 250gal with a frequency of 5Hz. The shaking time during each case was 20seconds.

Ⅲ. Test Results and Observations

Sand at the crest of slope slipped and the slope had large deformation, shown in Fig.3. The maximum displacement at the crest was 10cm in horizontal direction, 7.7cm in vertical direction. Liquefaction phenomena, such as sand boiling, were observed during shaking, and significant amount of water appeared above the toe of the slope right after the shaking, shown in Fig.4.

Time histories of the excess pore water pressures of 250gal at different depth shown in Fig.5. Hydrostatic pressures were zeroed before shaking. Sand boiling at the toe areas was extensive. Hence, the excess pore pressure during sand boiling after about 2 to 4 seconds of shaking reached the effective overburden pressure indicating the occurrence of the complete liquefaction. Some portion of the dry sand close to the original water level became wet. This rise of moisture might be due to dissipation of pore water pressure upward and capillary phenomena.

The model pipe was embedded 10cm below the crest of the slope, and a horizontal accelerometer was used. Oscillatory displacement due to shaking can be estimated by double integration of the recorded acceleration of the model pipe.

The displacement time history, corresponding to 250gal input motions after baseline correction, is shown

Fig.3　Post-shaking deformation at the crest due to liquefaction.

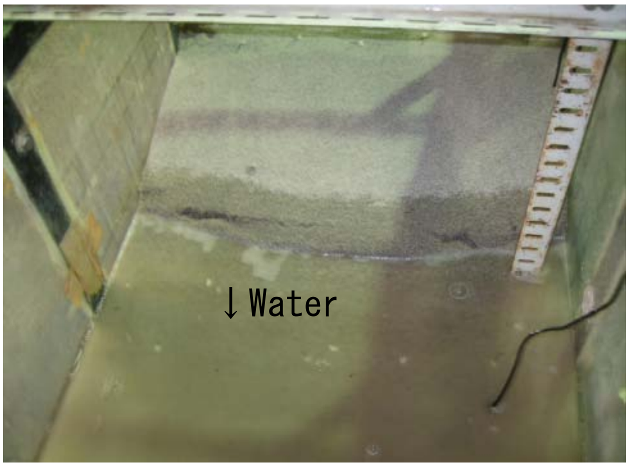

Fig.4　Presence of water above the toe due to liquefaction (dark portion is wet.).

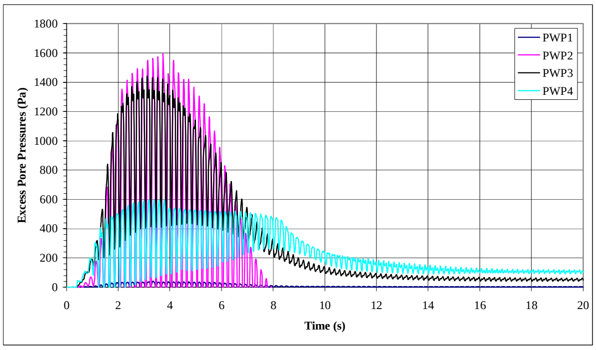

Fig.5　Excess pore water pressures at 250 gal sine wave.

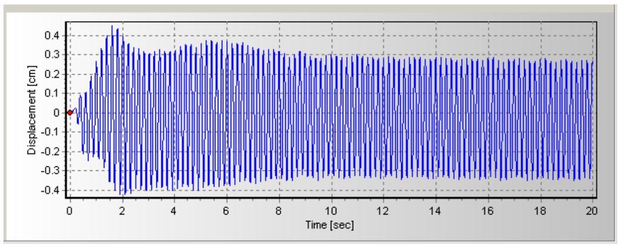

Fig.6　Time history of displacement of the model pipe (250 gal input).

in Fig.6. The maximum oscillatory displacement of the model pipe was 0.45cm during the 250gal input motion. The maximum permanent deflection of the model pipe should be more than the estimated oscillatory displacement, due to permanent ground deformation.

Ⅳ. Three -dimensional Dynamic Analysis

In order to analyze the damage to pipe due to liquefaction, three-dimensional analysis was carried out using FLAC finite difference meshes. Considering that the dimension of mesh and boundary conditions have a strong influence on the numerical results, the scale of numerical model was chosen to be 100 times of the model of shaking table experiment.

The saturated sand was modeled using a Mohr-Coulomb soil model coupled with Finn model which is the pore water pressure generation model. The foundation of dry sand was also modeled as Mohr-Coulomb model, without the pore water pressure generation model. The input parameters for the saturated and dry sands are summarized in Table1.

The iron pipe was modeled using structural beam-type elements interacting with surrounding soil via shear and normal coupling springs. Elastic modulus of the pipe was 1.2 $\times 10^{11}$ Pa, Poisson's ratio was 0.25 and density was 7,000kg/m^3, the diameter was 300mm. Soil-pipe interaction is bilinear elastic, and elastic modulus before liquefaction is 3 $\times 10^{7}$ Pa, 3 $\times 10^{4}$ Pa after liquefaction. A small (0.5%) Rayleigh damping was also assigned.

Base boundary was rigid boundary. In the initial static analysis, in order to compute gravity stresses, the base boundary was fixed both horizontally and vertically, and the side boundaries were only fixed horizontally. In the dynamic analysis, free-field boundaries were used. The numerical mesh for the problem is presented in Fig.7.

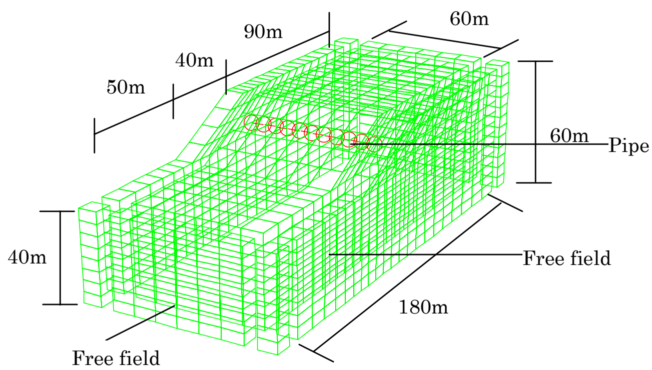

Fig.7　Numerical mesh and boundary of model.

Table 1　Soil properties used in numerical modeling.

Soil layer	Soil type	Total density (kg/m^3)	Friction angle (deg)	Shear modulus (Pa)	Bulk modulus (Pa)	Coefficient of permeability (m/s)
1	Dry sand	1600	36	1.5 $\times 10^{7}$	2.40 $\times 10^{7}$	-
2	Saturated sand	1900	35	2.00 $\times 10^{7}$	3.00 $\times 10^{7}$	1 $\times 10^{-10}$

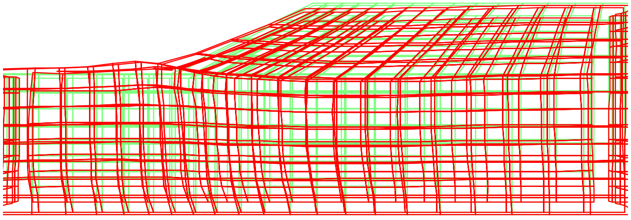

Fig.8 Deformed grid with sine wave velocity of amplitude=0.5m/s, frequency=5 Hz at 10 seconds.

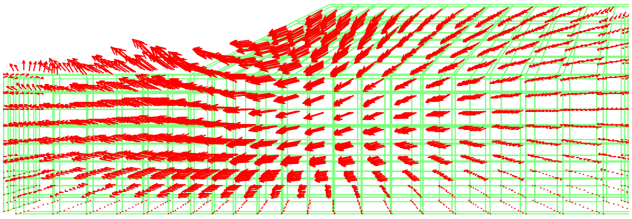

Fig.9 Displacement vectors with sine wave velocity amplitude=0.5m/s, frequency=5 Hz at 10 seconds.

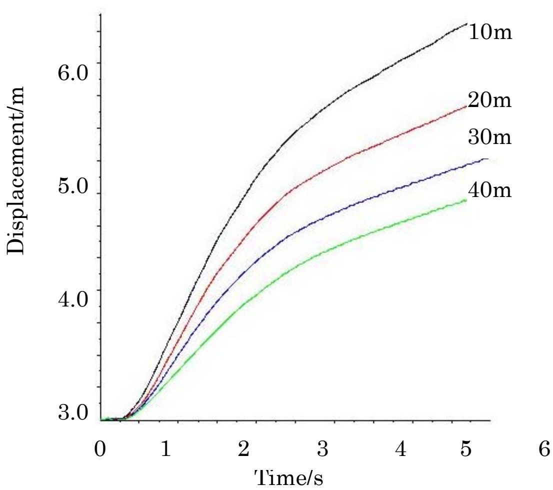

Fig.10 Displacement of different depth below the toe of slope with velocity of sine wave of amplitude=0.5m/s, *f*=5 Hz

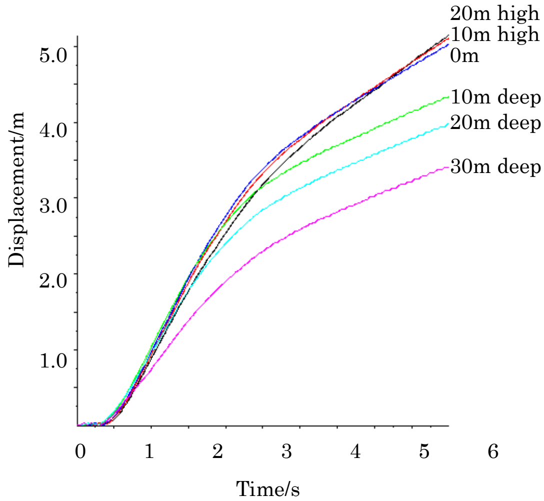

Fig.11 Displacement of different depth below the crest of slope (Surface of foundation is at 0m) with velocity of sine wave of amplitude=0.5m/s, *f*=5 Hz

Ⅴ. Dynamic Analysis Results

After computing static stress conditions, time history of dynamic analysis was carried out for sine wave velocities with different frequency and amplitude.

1) Displacement and pore pressure

The deformed grid and displacement vector from the numerical modeling are respectively presented in Fig.8 and Fig.9. The upper slope and foundation settled. The down slope and the area between the toe and the left side moved upward.

Fig.10 and Fig.11 are respectively displacements of different depth below the toe and the crest of slope. It was shown that the displacement increased with time. The displacement below the toe of slope was bigger than that below the crest of slope.

Fig.12 and 13 showed the pore pressure increased with time initially, finally reached to maximum and becomes constant. That was to say, liquefaction occurred. At the same depth, the maximum pore pressure below the crest of slope was larger than that below the toe of slope, because the overburden weight below the crest of slope was larger.

2) Pipe Dynamic Response

The permanent pipe displacement, caused by permanent ground deformation, increased with shaking time, and changed with amplitude and frequency of velocity.

Fig.14 and Fig.15 are the horizontal and vertical displacement of nodes of pipe. The displacement increased with the time. The displacement of pipe increased linearly at first stage, and then increased nonlinearly with the increase in damage.

Fig.16 is relationship between displacement and velocity amplitude of sine wave of *f*=0.5Hz. It was shown that displacement increased with amplitude, nonlinearly.

Fig.17 is relationship between displacement and velocity frequency of sine wave of *A*=1m/s. It was shown that displacement changed with frequency nonlinearly.

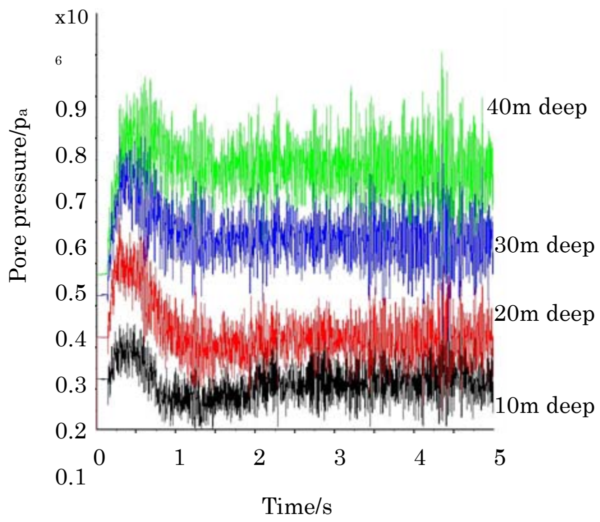

Fig.12 Pore pressure versus time below the toe of slope with velocity of sine wave of amplitude=0.5m/s, *f*=5 Hz.

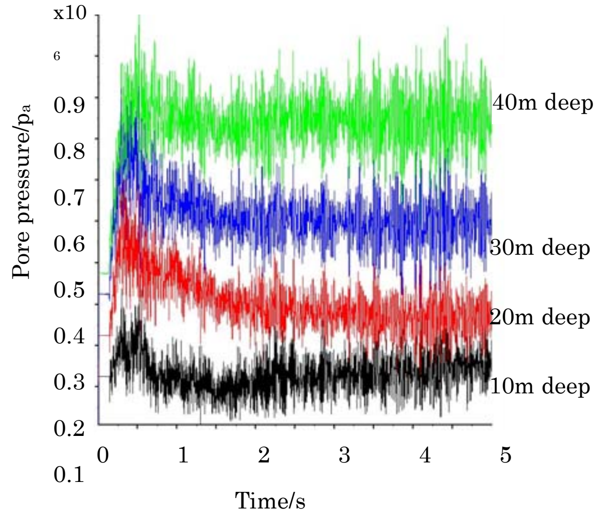

Fig.13 Pore pressure versus time at foundation below the crest of slope with velocity of sine wave of amplitude=0.5m/s, *f*=5 Hz.

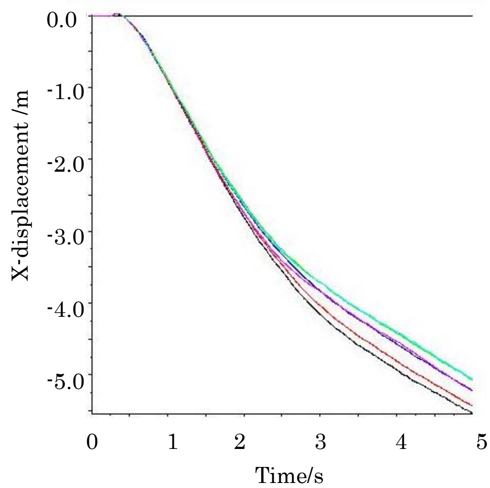

Fig.14 X-displacement of pipe nodes versus time with velocity of sine wave of amplitude=0.5m/s, *f*=5 Hz.

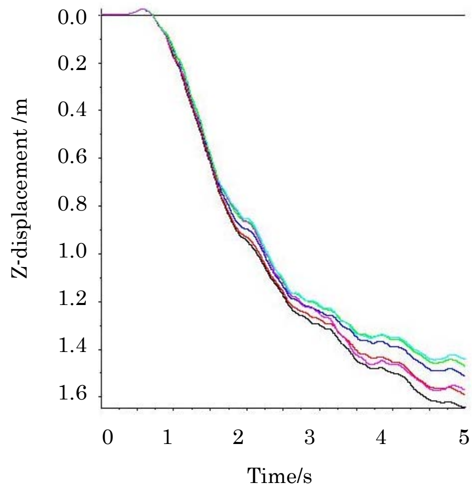

Fig.15 Z-displacement of pipe nodes versus time with velocity of sine wave of amplitude=0.5m/s, *f*=5 Hz.

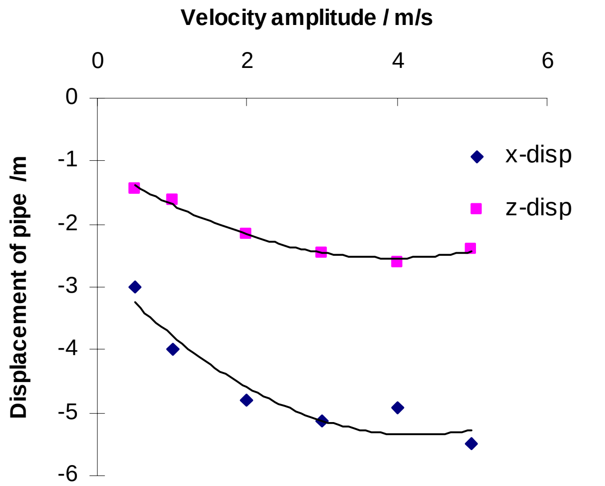

Fig.16 Displacement versus velocity amplitude with sine wave of *f*=0.5Hz.

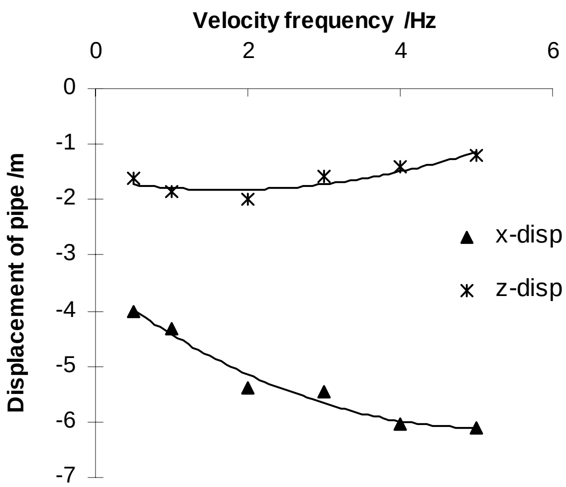

Fig.17 Displacement versus velocity frequency with sine wave of *A*=1.0m/s

Ⅵ. Concluding Remarks

The laboratory shaking table experiments and numerical analysis for buried pipeline under the slope were carried out. Damage of buried pipeline is caused by soil liquefaction. It is available to use the nonlinear method to simulate the soil-structure interaction. It is necessary to find a simplified analysis method for predicting pipe damage.

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斜面に埋設された管路の地盤液状化による被害

孫　紅[1]・宮島昌克[2]・喬　磊[3]

要　旨

地盤の液状化は地中埋設管に大きな被害を及ぼす。本論文では，液状化に伴う斜面の変形が地中埋設管に及ぼす影響を，室内振動実験と3次元動的解析を通して検討した。特に，飽和砂地盤内の過剰間隙水圧の蓄積，液状化に伴う斜面の変形状況，それに伴う埋設管の変形に注目した。振動実験では，模型地盤内の液状化に伴い，斜面の表層が大きく変形することが再現でき，このときの埋設管模型の変形を把握することができた。また，3次元個別要素法解析プログラムFLACを用いて動的解析を行ったところ，過剰間隙水圧の上昇過程や斜面の変形など，振動実験と対応する結果を表現することができた。さらに，埋設管の挙動も定量的に評価することができ，本解析プログラムによる地盤―構造物間の相互作用を表現する非線形手法の妥当性が確認できた。

[1]上海交通大学土木工程系　中国上海市東川路800号
[2]金沢大学理工研究域環境デザイン学系　〒920-1192　石川県金沢市角間町
[3]江蘇省岩土工程勘察院　中国南京市