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SYNOPSIS : First, an empirical formula to predict the magnitude of permanent ground displacement is proposed based on the observed data at the past earthquake events. Next, a simplified procedure to estimate the failure probability of buried pipes is proposed, in which the model of non-linear beam supported by ground spring elements is used to calculate the strain in the pipe by following the response displacement method. Finally, a simplified method to obtain the probability considering failure modes expected to occur in the pipe is proposed and some numerical example showing the probability and discussions follow.

INTRODUCTION

The past large earthquakes have caused various types of failures and malfunctions of lifeline facilities[1]. Among them, extensive damage of such structures as buried manholes and pipes in particular have been designated to liquefaction-induced Permanent Ground Displacement (PGD) of surrounding ground soil and uplift due to buoyancy force. Considerable research efforts to find the relationship between the PGD and uplift and the consequences of these buried structures have been made by many researchers. However, in most of the past researches, these major load effects have been treated separately when considering the failure modes of these buried structures. In this paper, a practical procedure to probabilistically assess the safety of buried lifeline facilities such as buried pipes and manholes is proposed considering those PGD and uplift due to liquefaction of surrounding soil.

MAGNITUDE AND DISTRIBUTION OF PGD

Hamada et al.[2] conducted a regression analyses by using ground deformation data obtained from the 1964 Niigata, the 1971 San Fernand and the 1983 Nihonkai-Chubu earthquakes and proposed an empirical formula to estimate the maximum displacement from the thickness of the layer (H) which is thought to liquefy and the gradient (Θ) of either the ground surface or the bottom of the layer. However, the measured ground deformation data are actually scattered to the variables used there. In particular, this formula shows a comparatively low correlation between D and Θ. Furthermore, it is practically difficult to correctly determine Θ, since the gradient is usually less than a few percent. Therefore, using the data obtained from the area in Niigata and Noshiro Cities where the gradient of the ground surface is less than 1 %, a new regression analysis was conducted to develop a simplified but more practical formula. In this formula, only the thickness of liquefiable layer was taken into account as a practical but major factor for predicting the maximum displacement of liquefaction-induced PGD of such area consisting of alluvial flat plains as Metropolitan Tokyo.

It was found clearly from the regression analysis that a comparatively good correlation was in the maximum value of magnitude of the ground deformation(D_{max}) and the thickness of liquefiable layer(H). However, as is shown in Figure 1, the correlation coefficient of the PGD to the thickness varies depending upon the areas under study. In the vicinity of the Niigata Railroad Station DGP value becomes larger with the thickness than the cases of the Ebigase-Shitayama area in Niigata City and the area in Noshiro City. The vicinity of the Niigata Railroad Station was already urbanized at the time of the earthquake, and many underground structures such as basements and foundation piles of buildings might have had a significant influence to constrain the magnitude of the ground deformations due to liquefaction, whereas the other areas have not been developed yet at that time. Considering the situation, the following two formulae were proposed for this study:

$$D_{max} = 0.34\,H \qquad (1)$$

$$D_{max} = 1.2\,H \qquad (2)$$

Where, D_{max}: maximum value of magnitude of ground deformation in the horizontal direction (m)
H : estimated thickness of the liquefied layer (m)

and Eqs.(1) and (2) are to be applied to predict PGDs for the urbanized and non-urbanized area, respectively.

The pattern of the ground deformation is considered to depend essentially on the local topographical and geological condition of a site. Figures 2(a) and (b) show the distributions of the length of compressive and the tensile zones respectively observed from the data of Niigata and Noshiro Cities. The length in the compressive zone ranges from 150 to 450 meters

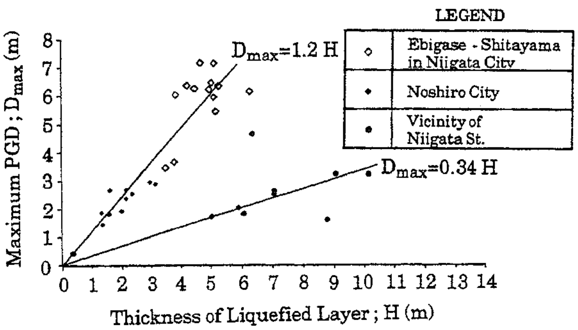


Figure 1 Relationship between Thickness of Liquefied Layer and Maximum Deformation

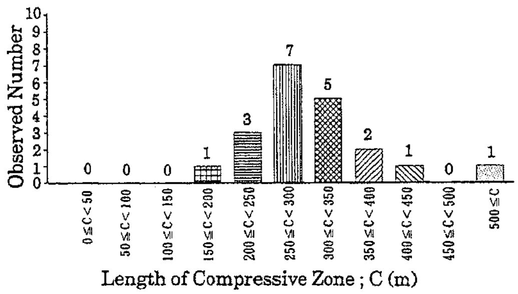


Figure 2(a) Diagram of Frequency versus Slope Length in Compression Strain Zone

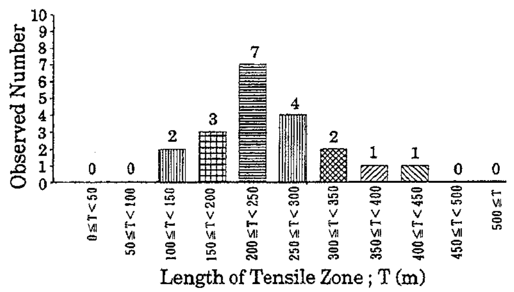


Figure 2(b) Diagram of Frequency versus Slope Length in Tensile Strain Zone

with the mean value of 320 meters and in the tensile zone from 100 to 450 meters with the mean value of 240 meters, respectively. The width of the ground deformation zone for the both cases ranges from 100 to 400 meters with the mean value of 240 meters. Based on these observed facts, the deformation pattern shown in Figure 3 was assumed and used for this study; a sinusoidal and trigonometric deformation loading patterns were assumed in subsequent analyses of the pipe for laterally orthogonal components of ground deformations.

FAILURE PROBABILITY OF EACH FAILURE MODE

Both compressive and tensile failures associated with the axial and bending deformations acting perpendicularly to a pipe are considered to be significant modes of failure of the pipe due to PGD. Besides, the bending failure of the pipe due

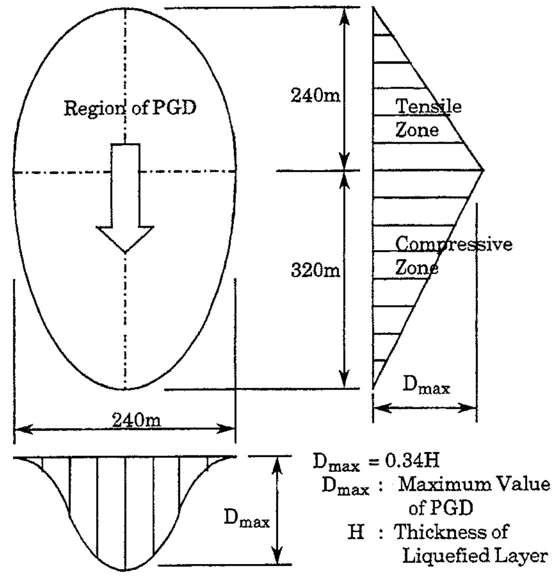


Figure 3 Idealized Pattern of Permanent Ground Deformation Used for the Study

to uplift of manholes in the vertical plane is also considered as another significant mode of failure in case of liquefied zone of soil. Thus, in this study, the failure probability for each mode above described is estimated by the following methods:

Generally, the principally influential factors for the failure of structures during an earthquake are the characteristics of earthquake motions, and the deformation and strength characteristics of the ground and structure. There will be a statistical variation in the values of all of these quantities. Clearly it is required that any assessment of structural failure probability be made considering all of these quantities with the relevant variations. Primarily, in order to carry out such complete assessment as above described, any study should start first from the investigation of the cross-correlations among these quantities with variation. However, in this study, only the variation of maximum accelerations in earthquake motions is considered as the controlling factor on failures, and the variations of other quantities are assumed as not so critical for the assessment and neglected.

A Performance Function (Z), in assessing whether a particular mode of failure will occur or not, is given as :

$$Z = \alpha_r - \alpha_s \qquad (3)$$

Z ≦ 0 : Non-failure

Z < 0 : Failure,

Where, α_r : critical acceleration of each point of the structure at which a limit status occur defined at the ground surface

α_s : peak ground acceleration at the ground surface

The failure probability (P_f) is then given by:

$$Pf = P(Z < 0) = P(\alpha_r < \alpha_s) \qquad (4)$$

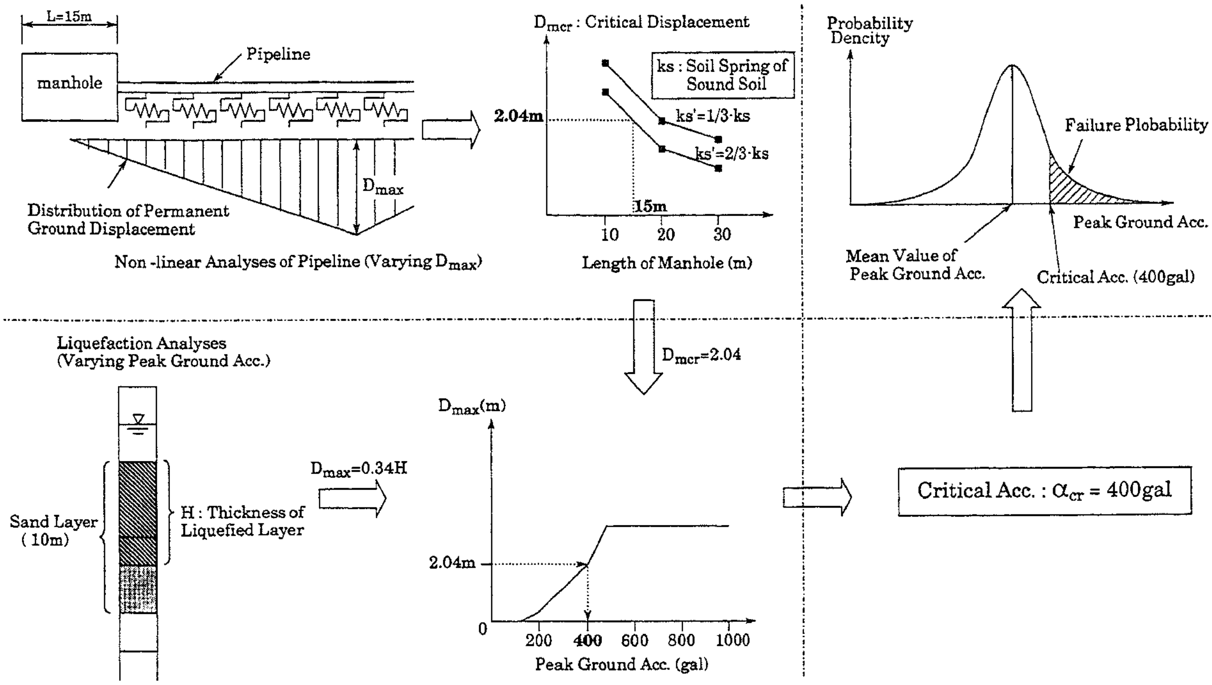


Figure 4 Flow Chart of Procedure for Estimation of Failure Probability Regarding PGD

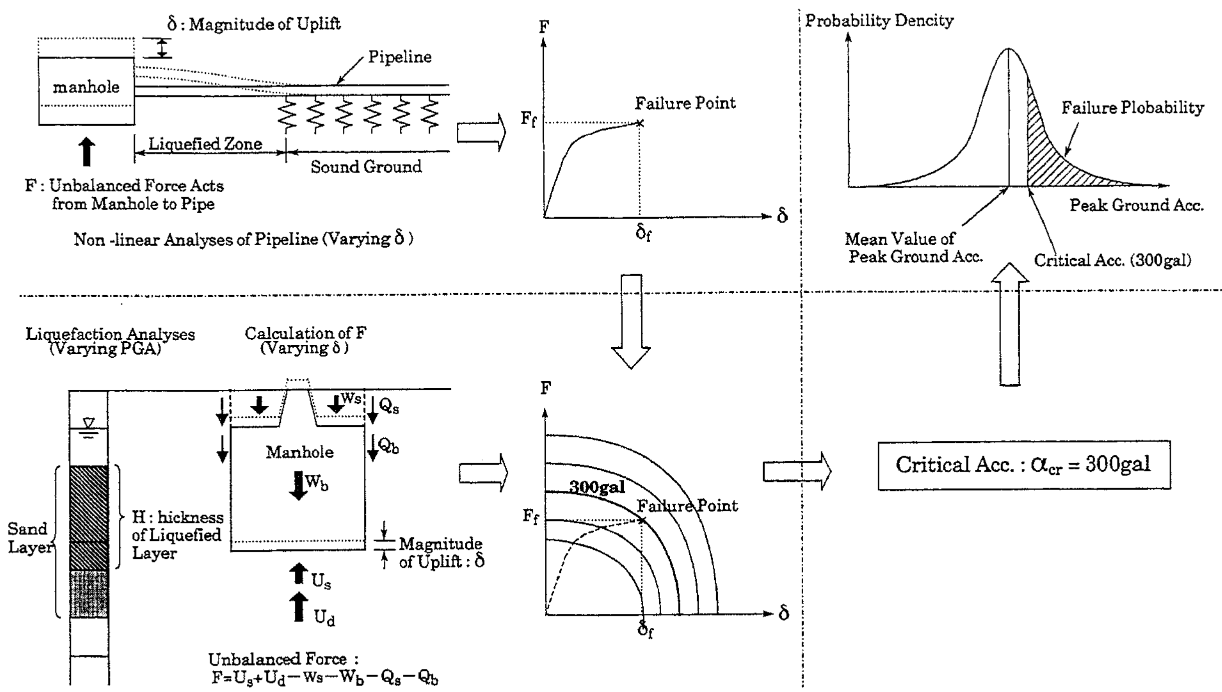


Figure 5 Flow Chart of Procedure for Estimation of Failure Probability Regarding Uplift

In this study, since all quantities except the peak ground acceleration are assumed deterministically, the quality of the critical acceleration then becomes deterministic.

The failure probability of a particular failure mode of a pipe due to any given a PGD may be determined by following the process as shown in Figure 4[3)], and similarly of the pipe by the uplift of a manhole due to liquefaction in Figure 5. As previously shown, it is first necessary to evaluate the critical acceleration in order to calculate the failure probability of buried pipes due to PGDs and uplifts of a manhole. In this study, the critical acceleration was evaluated by using the response displacement method as follows:

The interactive effect of PGD on buried pipes has been recently investigated in various research organizations, and has been becoming clearer gradually. Since there exists not yet any accepted method applicable for a practical evaluation at present, it is therefore a subject yet to be examined in the future whether the behavior of buried pipes under any given PGD can be properly analyzed by so-called response displacement method. However, in this study, it is assumed that the surrounding soil of a pipe and manhole has reached the liquefied state and the analysis to evaluate the critical acceleration for a buried pipe due to PGD can be performed properly with using reduced ground stiffness and strengths. In the response displacement method, the buried pipes and manholes are modeled as a beam with ground spring having a bi-linear stress-strain relation. As is shown in Figure 4, the critical value of maximum deformation (D_{mcr}) at which the strain of a pipe exceeds ultimate strain, is determined varying the axial length of the manholes and the spring stiffness of the surrounding soil. In this manner, relationship between the D_{mcr} and the physical conditions of the buried pipe was produced.

A liquefaction analysis of the ground soil was conducted in accordance with Japan High Way Bridge Code[6], in which the dynamic stress ratio causing liquefaction (R) is estimated from SPT blow count (N), mean grain size (D_{50}) and fine contents (F_c), and shear stress ratio (L) in the ground soil during earthquake is estimated from seismic coefficient (k_s) at ground surface as:

$$L = r_d \cdot k_s \cdot \frac{\sigma_v}{\sigma_v'} \qquad (5)$$

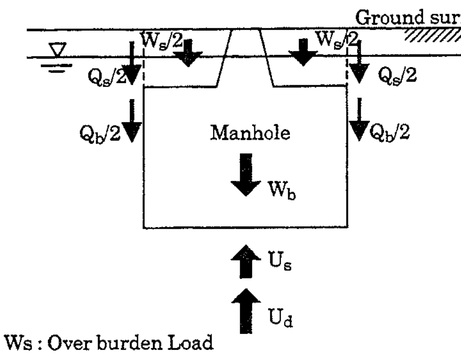
where, r_d : reduction factor of L (r_d=1-0.015Z)
σ_v' : effective over burden pressure (kgf/cm^2)
σ_v : over burden pressure (kgf/cm^2)
Z : depth (m).

Then, the factor of liquefaction resistance (F_L) is determined as below:

$$F_L = \frac{R}{L} \qquad (6)$$

In this study, the ratio of the peak ground acceleration at the ground surface(α_{max}) to gravity (g), α_{max}/g, is used instead of k_s in Eq.(5). By calculating F_L value for each sand layer using the above Eq.(5), the layer possible to liquefy is identified with its thickness. Thus, the thickness of liquefied layer(H) is related to α_{max} and using this H in Eq.(1), D_{max} is finally obtained. The liquefaction analysis is performed by varying α_{max} and D_{max} is estimated. Finally, comparing the critical displacement (D_{mcr}) and the above α_{max}~D_{max} relation, the critical acceleration (α_{cr}) for a particular buried pipe is evaluated.

Fundamental concept of the procedure for obtaining the failure probability of pipes due to the uplift of the manhole is as same as that of the PGD, while, the process to determine the critical acceleration is different. At first, as shown in Figure 5, liquefaction analyses are carried out varying α_{max}, the relationships between the magnitudes of uplift of the manhole and the unbalanced forces acting to the manhole are



Ws : Over burden Load
Wb : Dead Load of Structure
Qs : Shear Resistance of Overburden
Qb : Friction Resistance of Sides of Structure
Us : Uplift Force Due to Hydrostatic Pressure
Ud : Uplift Force Due to Excess Pore Water Pressure

Figure 6 Forces Acting to Manhole During Liquefaction

calculated for various values of PGA based on the equilibrium relation as is shown in Figure 6[5]. On the other hand, an analytical model of ground-pipe system is constructed, in which the beam lain in the liquefied zone has no ground spring. By using this model, the relationship between the maximum displacement δ, at which the strain of the pipe exceed ultimate strain, and the shearing force F at the end of the beam is estimated. Comparing these two relationships, as shown in Figure 5, the critical acceleration for the failure mode of the pipe due to the uplift can be determined.

If following the above methods, through one time of very laborious non-linear analysis of the pipe-ground system, it is possible to evaluate, irrelevantly to the ground conditions and various parameters for each failure mode, the critical acceleration not only for PGD by simply establishing the relationship between the PGA and the thickness of liquefied layer but also for the uplift by establishing the relationship between magnitude of uplift of the manhole and the unbalanced force acting to the manhole. As the result, this method can be applied over a wide range of similar problems to the above, and the method is relatively easy to be followed even if a large number of design conditions need to be studied.

TOTAL PROBABILITY OF FAILURE

The probability previously described is designated only to one particular mode of failure. For example, the failure probability calculated from the critical acceleration in an axial compression analysis refers to a conditional probability of failure at the point of interest under permanent deformation in the axial direction, provided that the point lies within a compression zone. Therefore, it is necessary to calculate the probability, total probability, which considers the occurrence of each possible failure mode.

To determine this total probability, we consider separately the occurrence of the deformation under liquefaction in each direction relative to the pipe. In practice, the deformation will occur in the direction intercepting the pipe with some angle.

However, this situation is simplified here to assume the deformation to occur either at 0 or 90 degree(s) to the pipe, with a probability of occurrence of 0.5. In addition, among the cases in which a deformation occurs in the direction along the pipe axis, there will be such cases that the pipe lies either within a compression or a tension zone. The probability of this case is evaluated by taking the ratio of the deformation pattern length in compression zones (Lc=240m) or length in tensile zones (Lt=320m) to the sum of these two lengths (Lc+Lt=560m). According to this approach, the probabilities that the pipe will lie in a deformation pattern in a compressive zone (Rc), or in a tensile zone (Rt), or the pipe is subject to deformation in the direction perpendicular to the pipe axis (Rm), are determined by the equations as follows:

$$Rc = 0.5\ \frac{Lc}{Lc{+}Lt}\quad (= 0.286) \tag{7a}$$

$$R_t = 0.5\ \frac{L_c}{L_c{+}L_t}\quad (= 0.214) \tag{7b}$$

$$R_m = 0.5 \tag{7c}$$

In addition to the failures because of PGD, failure due to uplift of manhole must be taken into consideration. The union of these failure probabilities leads to the total probability of failure. Since PGA, which is only random variable in the procedure, is common to both phenomena of PGD and uplift, the union of these failure probabilities of each case is equal to the larger value of failure probability due to PGD or uplift. Therefore, if the failure probabilities of compression, tension and bending due to PGD are expressed as P_{fc}, P_{ft}, and P_{fm} respectively, and failure probability due to uplift is expressed as P_{fu}, the total failure probability P_f can be obtained as below:

$$P_f = R_c P_{fc}' + R_t P_{ft}' + R_m P_{fm}' \tag{8}$$

where, P_{fc}' : larger probabilities comparing P_{fc} with P_{fu}
P_{ft}' : larger probabilities comparing P_{ft} with P_{fu}
P_{fm}': larger probabilities comparing P_{fm} with P_{fu}.

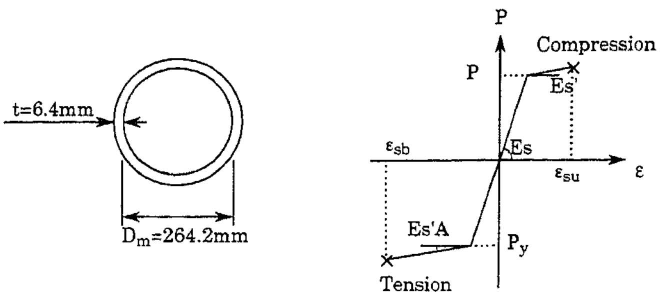
NUMERICAL MODEL OF PIPE

Physical properties of the pipe are shown in Figure 7(a) and (b). A bi-linear load-strain relationship was employed. The limiting tensile strain for the pipe was taken as 8% with a consideration of fatigue failure. Compressive failure strain was the buckling strain for the pipe determined from Eq.(9).

$$\varepsilon_{sb} = 44\ \frac{t}{D_m} \tag{9}$$

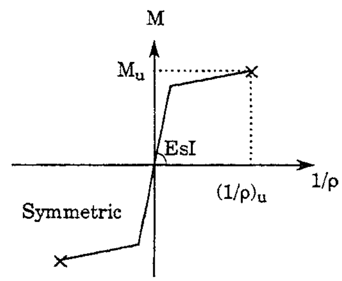
where, ε_{sb}: pipe buckling strain (%)
t : pipe thickness (cm)
D_m : average pipe diameter (cm)

Eq.(9) is the same as that in the code for aseismic design of high-pressure gas pipes[6] for the allowable buckling strain in



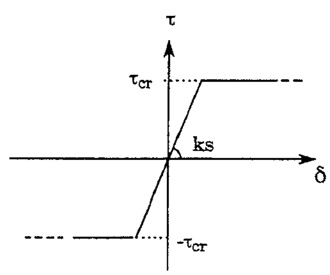
A : Section area of pipe
Es : Young modulus of steel (Es=2.1×10^6 (kgf/cm^2))
Es : Second modulus of steel (Es'=Es×10^{-7} (kgf/cm^2))
Py : Yield load of pipe (Py=Aσ_{sy})
σ_{sy} : Yield stress of steel (σ_{sy}=2900 (kgf/cm^2))
ε_{su} : Critical tensile strain of steel (ε_{su}=8 (%))
ε_{sb} : Critical compressive strain of steel (ε_{sb}=44t/D_m(%))

Figure 7(a) Assumed Physical Property of Steel Pipe (In the Case of Axial Force Load)



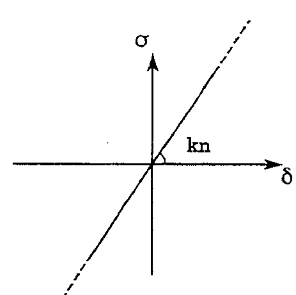
Es : Young modulus of steel
I : Inertia moment of Section area
Mu : Failure moment of pipe
$(1/\rho)_u$: Failure bending strain of pipe
ρ : Radius of curvature

Figure 7(b) Assumed Physical Property of Steel Pipe (In the Case of Bending Load)



τ : Shear stress
k_s : Soil spring constant per unit area in longitudinal direction (ks=0.6 (kgf/cm^3))
τ_{cr} : Critical shear stress (τ_{cr}=0.1 (kgf/cm^2))
δ : Relative displacement between pipe and ground

Figure 8(a) Spring Characteristics of the Surrounding Ground (In the Case of Axial Force Load)



σ : Normal stress
k_n : Soil spring constant per unit area in orthogonal direction (kn=3ks)
δ : Relative displacement between pipe and ground

Figure 8(b) Spring Characteristics of the Surrounding Ground (In the Case of Axial Force Load)

Table 1 Reduction Factor for Soil Spring During Liquefaction

Depth (m)	$F_L \leqq 0.6$	$0.6 < F_L \leqq 0.8$	$0.8 < F_L$
$0 \leqq Z < 10$	(1/3)*	1/3	2/3
$10 \leqq Z < 20$	1/3	2/3	2/3

* : 0 is adopted in the Code for this portion

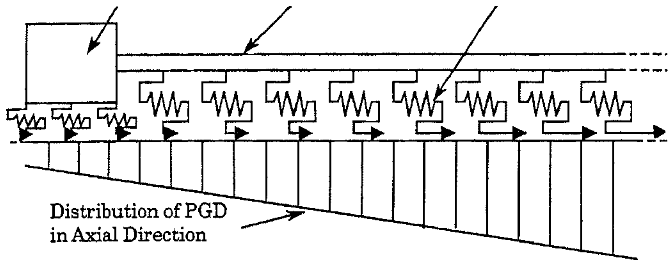


Figure 9(a) Analytical Model for Axial Direction of Pipe

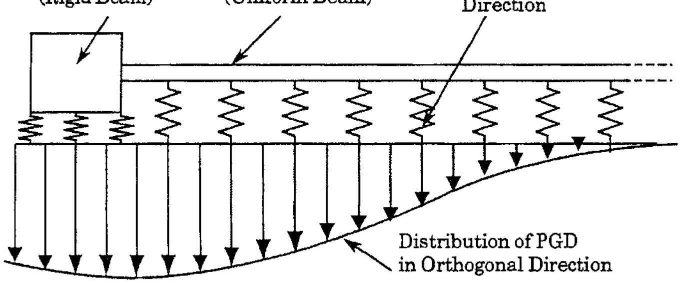
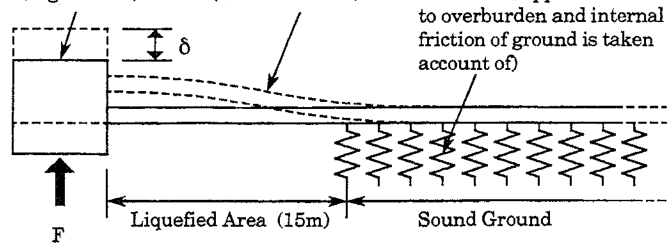


Figure 9(b) Analytical Model for Orthogonal Direction of Pipe

straight pipes with the safety factor being unity. The bi-linear load-strain relationship was formed using a straight line between the origine and the yield point as the primary gradient, followed by a secondary gradient after yield taking a value of the primary gradient multiplied by a coefficient of 7×10^{-3}. This coefficient is determined from the code for aseismic design of high-pressure gas pipes [6]. For the bending properties of the pipe, a bi-linear moment-strain relationship was used on the basis of the properties of the pipe material. Bending failure was assumed to occur when the compressive strain at the ends of pipe reached the buckling strain.

Among the ground springs of the model, those acting in the pipe axis direction were assumed to have a bi-linear stiffness as shown in Figure 8(a), based on the code for aseismic design of high-pressure gas pipes [5]. The ground stiffness (k_s = 0.6 kgf/cm3) and critical shear stress (τ_{cr} = 0.1 kgf/cm2) are approximately in the average values obtained from load tests on gas pipes. Spring supporting the pipe perpendicularly in the horizontal direction have a linear relation as shown in Figure 8(b). These values of ground stiffness and critical shear stress were modified by the coefficients shown in Table 1



δ : Magnitude of Uplift
F : Unbalanced Force Acts from Manhole to pipe
Due to Liquefaction

Figure 10 Analytical Model for Uplift of Manhole

depending on the FL value and depth of the pipe. Table 1 is taken Japan Highway Bridge Code [4].

Since the stiffness of the manholes is much greater than that of the pipe, stress concentrations occur at the connection between pipe and manhole. In this study, it is assumed that failure only occurs at the connections. The typical distance of neighboring two manholes is 200 to 300 meters. For this reason, the design point of one pipe-manhole junction would not be influenced by other manholes, and the analysis was made using one manhole with a connected pipe only. The analytical models for the permanent deformations in the pipe axis direction and in the perpendicular direction are shown in Figure 9(a) and (b). For the analysis of the pipe in the perpendicular direction, a deformation pattern was used where the point of maximum curvature of the distribution coincided with the manhole junction, as shown in Figure 9(b).

The analytical model for uplift of manhole is shown in Figure 10. The length of liquefied area in the model is decided based on the results of the study on equivalent diameters of liquefied zones, which is carried out in Noshiro city after Nihonkai-Chubu earthquake by Tanabe[7]. As shown in Figure 11, liquefaction is seemed to occur mainly in the area from 10m to 40m of equivalent diameter. Based on this study, mean value of equivalent diameter of liquefied zone is assumed 30m, and it is assumed that the connecting point between pipe and manhole is just at the center of the liquefied zone. Consequently, the length of liquefied area in the model assumed to be 15m. Assuming that unliquefied ground is perfectly sound, no reduction factor is taken into account to the stiffness of ground springs.

CRITICAL GROUND DEFORMATIONS

The critical ground deformations for the pipe in the axial direction are shown in Figure 12, with respect to the manhole length. Similar results from the analysis of the pipe in the perpendicular direction are shown in Figure 13. For the pipe under axial tensile load, no failure occurs, because the limiting tensile strain of 8% is large enough. From Figure 12 and 13, for the same critical deformation, failure is more likely to occur in cases when the ground deformation occurs in the

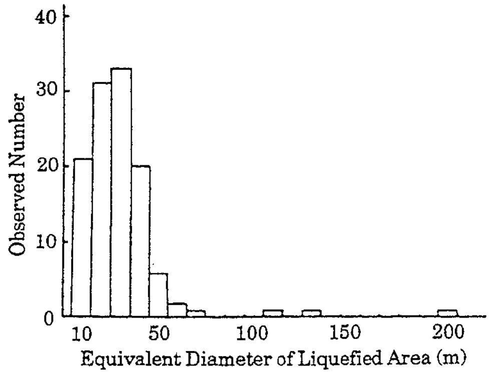


Figure 11 Size of Liquefied Area and Observed Number[7]

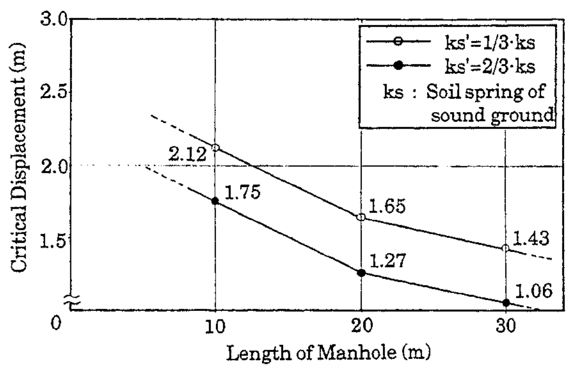


Figure 12 Relationship between Length of Manholes and Critical Ground Deformation (Axial Direction Force Load, Compressive Failure)

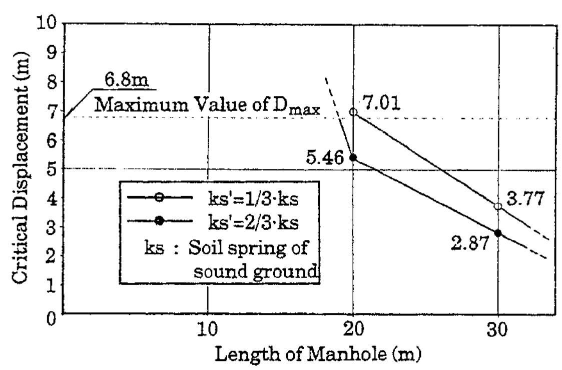


Figure 13 Relationship between Length of Manholes and Critical Ground Deformation (Lateral Direction Force Load, Bending Failure)

direction of axial compression in the pipe than in cases when the deformation is in the perpendicular direction.

The relationship between displacement, which corresponds to the magnitude of uplift of manhole, and shear force act to the end of pipe obtained by using the model of Figure 10 is shown in figure 14. From Figure 14, it can be found that the pipe fails when the manhole is uplifted 84cm, and in this condition 1.61tons of downward force is acted on manhole by the pipe.

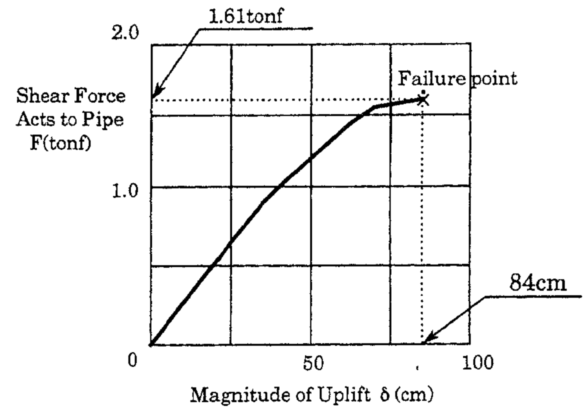
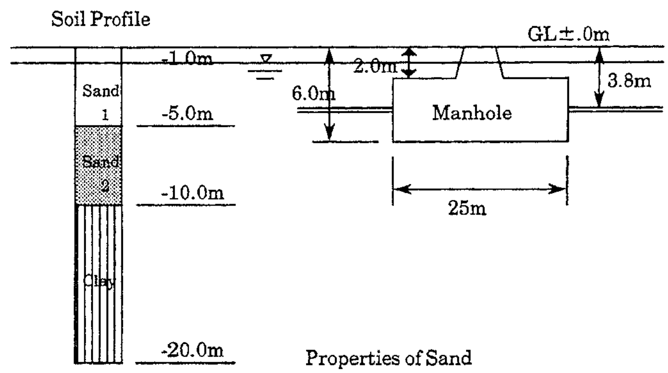


Figure 14 Relationship between Magnitude of Uplift δ and Shear Force F Acts to Manhole from Pipe

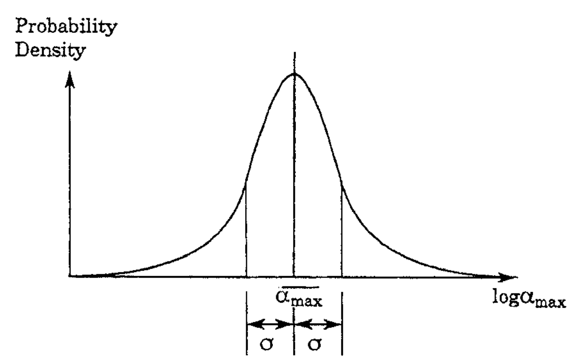


Properties of Sand

	N-Value	Unit Weight γ (gf/cm^3)	Mean Grain Size D_{50} (mm)
Sand 1	5	1.7	0.1
Sand 2	20	1.8	0.2

Fc is presumed less than 40%

Figure 15 Situation of Pipe Attached to Manhole and Assumed Ground Condition for Estimation of Failure Probability



α_{max} : Maximum Ground Acceleration at Surface
$\overline{\alpha_{max}}$: Mean Value of α_{max} ($\overline{\alpha_{max}}$=100, 200 and 300 gal)
σ : Standard Deviation (σ=0.269)

Figure 16 Distribution of Probability Density of the Maximum Acceleration at Ground Surface

Table 2 Results of Estimation of Failure Probability

Mean Value of PGA: α_{max} (gal)	Failure Mode of Pipe		Critical PGA: α_{cr} (gal)	Failure probability for Each Failure Mode: $P_{fc}, P_{ft}, P_{fm}, P_{fu}$	Larger Probabilities Comparing P_{fc}, P_{ft} and P_{fm} with P_{fu}: $P_{fc}', P_{ft}', P_{fm}'$	Total Failure Probability: $P_f=R_cP_{fc}'+R_tP_{ft}'+R_mP_{fm}'$
100 (53.8 to 185.8)	PGD	compression	220	$P_{fc}=0.101$	$P_{fc}'=0.101$	$P_f=0.078$
		tension	non-failure	$P_{ft}=0$	$P_{ft}'=0.069$	
		bending	non-failure	$P_{fm}=0$	$P_{fm}'=0.069$	
	uplift (bending)		250	$P_{fu}=0.069$		
200 (107.7 to 371.6)	PGD	compression	220	$P_{fc}=0.440$	$P_{fc}'=0.440$	$P_f=0.382$
		tension	non-failure	$P_{ft}=0$	$P_{ft}'=0.359$	
		bending	non-failure	$P_{fm}=0$	$P_{fm}'=0.359$	
	uplift (bending)		250	$P_{fu}=0.359$		
300 (161.4 to 557.3)	PGD	compression	220	$P_{fc}=0.692$	$P_{fc}'=0.692$	$P_f=0.638$
		tension	non-failure	$P_{ft}=0$	$P_{ft}'=0.616$	
		bending	non-failure	$P_{fm}=0$	$P_{fm}'=0.616$	
	uplift (bending)		250	$P_{fu}=0.616$		

Notice : (-- to --) denotes log variation of $-\sigma$ to $+\sigma$

$R_c=0.286$, $R_t=0.214$, $R_m=0.5$

AN EXAMPLE OF NUMERICAL RESULTS

The assumed pipe, manhole, and ground conditions are shown in Figure 15. As seen in the figure, the pipe is assumed to be installed at 3.8 meters below the ground level, attached to manholes with 25 meters in axial length. The thickness of liquefaction was assessed in accordance with the method given in the "Japan Highway Bridge Code[6]", with liquefaction taken to occur in sand layers. The probability distribution of maximum ground surface acceleration was taken as a log normal distribution as shown in Figure 16. The mean value (α_{max}) was varied at 100, 200 and 300 Gals, and the failure probability was determined. The standard deviation of $\log\alpha_{max}$ was assumed as $\sigma=0.269$ in this example estimation.

Summary of the failure probability are shown in Table 2. In this table, there was no failure due to permanent deformation in the direction perpendicular to the pipe axis. As a result, the value of the total failure probability was considerably influenced by failure probability of uplift of the manhole, because there is neither probability of tensile failure nor bending failure due to PGD. As indicated in the example, if critical PGA at surface α_{cr} is determined for each failure mode only once, total failure probability is easily obtained for any supposed PGA at surface.

CONCLUSION

The methodology to assess the safety of buried pipes against Permanent Ground Displacement and uplift of manhole has been proposed. And an example of the calculation in one case was presented. Although the calculated values of failure probability are regarded not to have meanings, from the results of the calculation, the proposed procedure may be a practical method to assess the safety of many buried pipes in different ground conditions and under different earthquake intensities. Furthermore, the concept of the proposed procedure may be applicable to other types of buried lifeline structures such as the pipe which has flexible joint or socket joint, or reinforced concrete duct, by considering appropriate analytical model for each structure.

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