A NEW PROPOSAL FOR SIMPLIFIED SEISMIC RESPONSE ANALYSIS OF PIPES IN GROUND WITH INCLINED BED-ROCK

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SUMMARY

Since, during the 1995 Earthquake in Kobe City, a large number of the buried pipelines were damaged, it was necessary to revise the seismic methods of analysis. Therefore two levels of the earthquake were defined as level 1 and level 2. A recommended standard method has been used for analysis of gas buried pipelines based on Displacement Response Method under the level 1 earthquake (here L1 Gas Standard). In this recommendation a simplified method is given for obtaining the maximum ground strain in a ground with inclined bed-rock. Recently a new simplified method for design of the Gas pipelines under the level 2 of ground motion is recommended based on the simplified method for level 1 (here L2 Gas Standard).

In this paper also a new method is proposed based on the modification of maximum strain considering the effects of inclination of the bed-rock and non-linear behavior of the soil. The effect of the inclined bed-rock is considered as a coefficient which is calculated from a curve using the natural period of the soil in the deeper part. The effect of the non-linearity is considered by a coefficient, which is obtained from another curve using the predominant period of the input earthquake wave. These two coefficients are multiplied by the maximum linear strain of the deeper part of the ground, which can be obtained from any one dimensional linear analysis (in this paper we have computed it using the SHAKE program). In this study we have used a developed FEM program to investigate the dynamic response of the soil. The results of the analysis are compared with the results of old and new proposals and discussed. For obtaining larger strains in the soil, the angle of the slope is selected as 45 degrees.

Two records of the Kobe Earthquake are selected for the analyses. One is recorded in Port Island with higher amplitude and the other one is recorded at East Kobe Big Bridge with very long period. The time-history response of accelerations, displacements and strains for both X and Y directions are obtained at the pipe location. Also, the contours of the mentioned responses have been output for the whole model in time domain. Since in design of buried pipeline, the axial strain is assumed as the main factor, the results of soil strain in X direction (pipe axial direction) at the location of the pipe are investigated and compared for different cases.

As some common points among the results of the cases of the analyses, it is clear that the maximum strains are concentrated near the sloped parts. The results of the non-linear cases are greater than the linear cases for each input wave. At the pipe location the maximum strains of soil are located almost on the top of the start point of slope. The results of the non-linear cases are greater than the linear cases. The results of the Port Island wave are less than the East Kobe Big Bridge input wave in non-linear analysis.

There is a good agreement between the results of the FEM analyses with those from proposed method. The obtained result from the L2 Gas Standard is lower than that obtained from the FEM results under the long period wave. Therefore, it needs more investigation and somehow modification for considering the effect of longer period waves of level 2 in the Gas Standard.

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INTRODUCTION

Since, during the 1995 Earthquake in Kobe City, a large number of the buried pipelines were damaged, it was necessary to revise the seismic methods of analysis and design of such structures for future. It became clear that the level of the ground motion in pervious methods was not enough for resisting against a strong one such as the quake in Kobe. Therefore two levels of the earthquake were defined as level 1 and level 2. The level 1 is the lower level of ground motion assumed by an expected earthquake with almost same return period as the period of buried pipelines in service. The level 2 is almost 5~8 times higher than the first level in which the earthquake occurrence would be expected within much more longer return period such as 1995 Kobe earthquake[1)].

For getting much more critical strains and stresses in buried pipes, a series of research [2,3)] have already started concerning the response analysis of buried structures in ground with inclined bed-rock, liquefied ground and those near or crossing active faults. This study is a part of such research.

A recommended standard method (here L1 Gas Standard) has been used for analysis of gas buried pipelines based on Displacement Response Method under the level 1 earthquake [4,5)]. In this recommendation a simplified method is given for obtaining the maximum ground strain in a ground with inclined bed-rock [4.5)].

Such a case has been investigated also by reference 4 under the level 2 earthquake. The most critical angle of slope for the inclined layer is about 45 degrees [4,5,6)]. However the maximum ground strains obtained by these two references are different from each other. Comparing their results, modifying the method of reference 2 was necessary considering the effect of the non-linear analysis during the earthquake level 2. Recently a new simplified method for design of the Gas pipelines under the level 2 of ground motion (here L2 Gas Standard) is recommended based on the simplified method for level 1. We have also proposed a simplified method for calculating the maximum ground strain at the pipe location by modifying the linear response of uniform soil due to the effect of inclination of bed-rock and non-linearity of the soil. A model is analysed by the FEM[.7)] program developed by the authors to have more clear understanding of the soil response and the comparison with results of proposed method.

THE OUTLINE OF PROPOSED SIMPLIFIED METHOD

Fig. 1 shows the general scheme of the ground with inclined bed-rock and those parameters which are used in the proposed method.

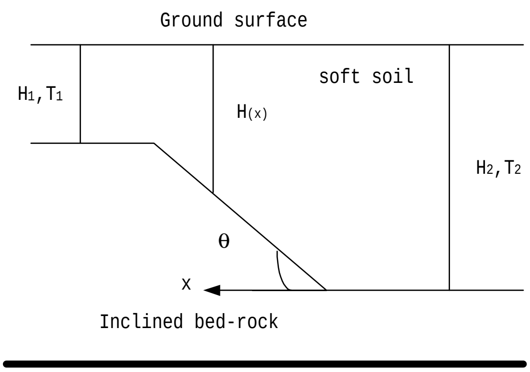

Fig. 1 Scheme of the ground with inclined bed-rock

The maximum strain of the ground at the location of the pipe will be obtained using the Eq. 1.

$$\varepsilon_{max} = \alpha.\beta.\varepsilon_{uL} \qquad (1)$$

Here, ε_{max} is the maximum ground strain at pipe location (almost surface of ground), ε_{ul}, the linear strain obtained by a linear one dimensional analysis, α the coefficient for the effect of the bed-rock inclination, and β is the coefficient for non-linearity modification .

For calculating the α, we follow the formulation as given in the recommendation standard for design of gas pipelines [4]. The soil natural period is given in Eq.2 , in which H(x) is as shown in Fig. 1 and *Vs* is the velocity of the shear waves in the soil.

$$T_{(x)} = \frac{4H_{(x)}}{Vs} \tag{2}$$

The displacement of the each column of the soil is calculated from Eq. 3. Here *Sv* is the velocity spectrum and K_{oh} is a coefficient related to the area and importance of the structure [4].

$$U_{h(x)} = \frac{2}{\pi^2}.T_{(x)}.S_v.K_{oh} \tag{3}$$

The maximum linear strain is as given in Eq. 4. The *L* is the length (equal to multiplication of apparent velocity of wave by the natural period *T*) of the wave and A is a constant.

$$\begin{aligned} \varepsilon_L &= \frac{\Delta u}{\Delta x} = \frac{\Delta(U_h.\sin(\frac{2\pi}{L}x)}{dx} = U_h.\frac{\Delta(\sin(\frac{2\pi}{L}x)}{\Delta x} + \frac{\Delta U_h}{\Delta x}\sin(\frac{2\pi}{L}x) \\ &= \frac{2\pi U_h}{L}\cos(\frac{2\pi}{L}x) + \frac{\Delta U_h}{\Delta x}\sin(\frac{2\pi}{L}x) = \sqrt{\varepsilon_{uL}{}^2 + \varepsilon_i{}^2}.\sin(\frac{2\pi}{L}x - A) \end{aligned} \tag{4}$$

The simple form of Eq.4 can be written as Eq. 5.

$$\varepsilon_L = \varepsilon_{uL}.\sqrt{1 + \left(\frac{\varepsilon_i}{\varepsilon_{uL}}\right)^2} = \alpha.\varepsilon_{uL} \tag{5}$$

ε_i which is additional strain due to the inclination of the bed-rock is calculated by Eq. 6, using Eq. 2.

$$\varepsilon_i = \frac{\Delta U_{h(x)}}{\Delta x} \tag{6}$$

After simplifying and computing all the above procedure, the coefficient α is calculated for different quantities of T_2 and T_2/T_1 and plotted in Fig. 2. This figure shows the proposed envelope curve for obtaining α using T_2 (safety side).

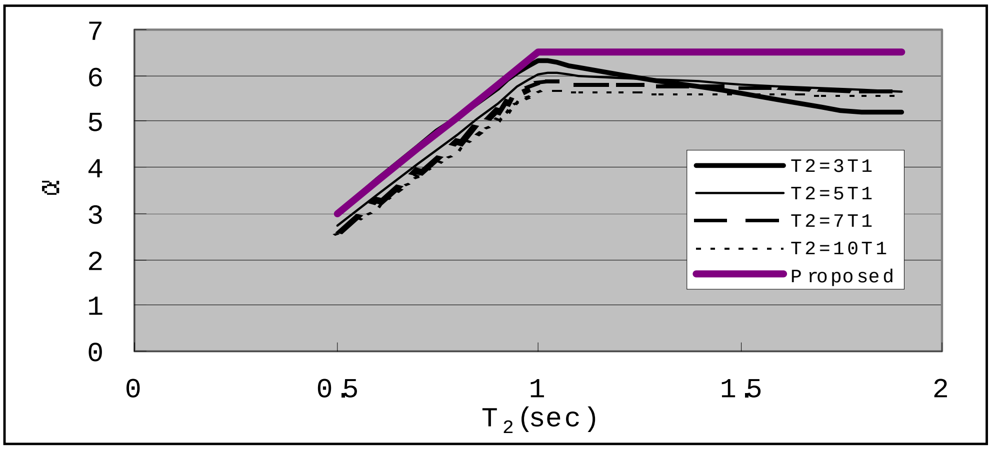

Fig. 2 Relation between α and T2

For obtaining the coefficient β , we have used the developed FEM program for both linear and non-linear dynamic. We found the relation shown in Fig. 3 between β and the ratio of wave predominant period (T_0) assuming T_1=0.2(sec) and T_2=1.0(sec) with an inclination of 45 degree. Although our investigation about this relation is under process for different cases of waves at near and far field, but the primary results show that it is almost similar to that shown in Fig. 3.

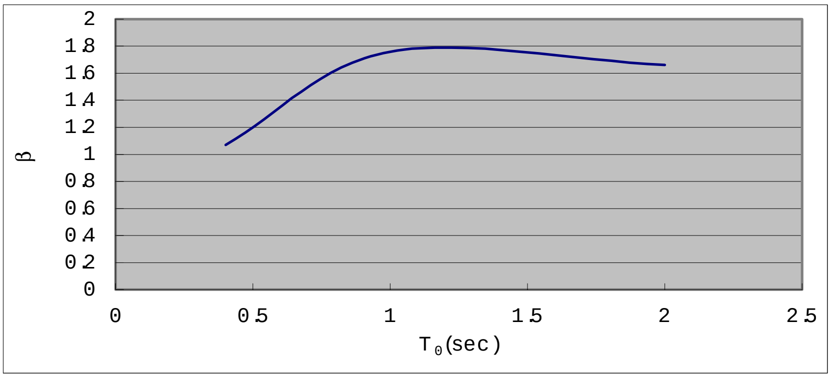

Fig. 3 Relation between β and T_0 for different types of the accelerations

ANALYTICAL MODEL

For obtaining larger strains in the soil, the angle of the slope is selected as 45 degrees. Considering practical cases the dimensions of the model are as shown in Fig. 4. The dashed line shows the location of the pipe, which is assumed been located 1.5m under the soil surface. The bottom of the model is fixed in vertical direction (the relative displacement is zero) while its left and right boundaries are assumed as the viscous boundaries using dash-pots with constant damping ratio. The bottom left of the model is assumed as the center of coordinate axes. The horizontal axis is X and positive from left to right and Y axis is upward positive.

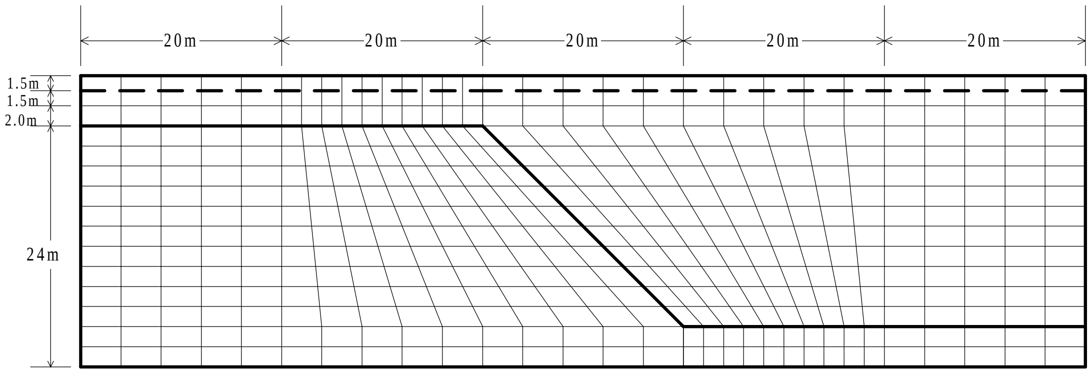

Fig. 4 Analytical model

The properties of hard (rock) and soft soil used in the analyses are given in Table 1. These data are selected according to the more practical cases.

Table 1 Soil input data

Soil type	N value	Unit weight (KN/m^3)	Vs (m/s)	Cohesion (KN/m^3)	Friction Angle (degree)	Poisson's ratio	Shear rigidity (KN/m^2)
Rock	50	19.6	300.0	2940.0	42.00	0.45	180000.0
Soft soil	10	14.7	100.0	0.0	27.00	0.49	15000.0

INPUT GROUND MOTION

The first acceleration, which is shown in Fig. 5, has the maximum amplitude of 679 gal and the predominant period of 0.4 second. The other one shown in Fig. 6, is recorded with a maximum amplitude of 446 gal and predominant period of about 2.0 second.

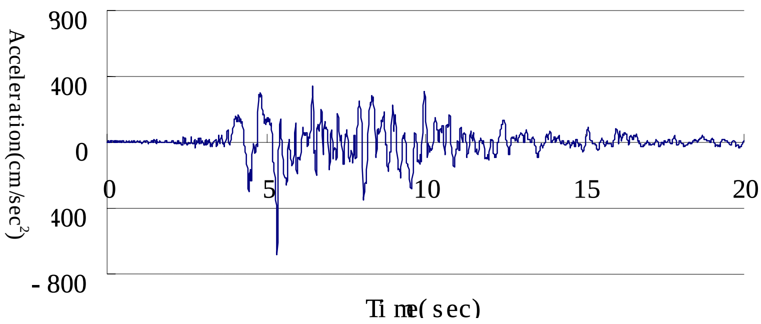

Fig.5 N-S component of horizontal acceleration recorded at Port island

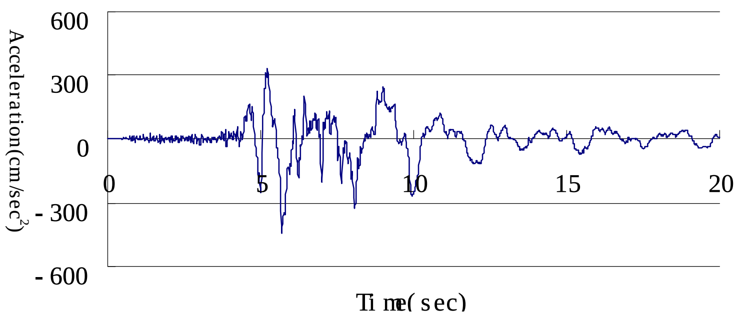

Fig.6 N-S component of horizontal acceleration recorded at East Kobe Big Bridge

Fig. 7 shows the velocity response spectra of these two waves. The maximum velocity in both of them is about 100 kine, which made them as the level 2 earthquakes.

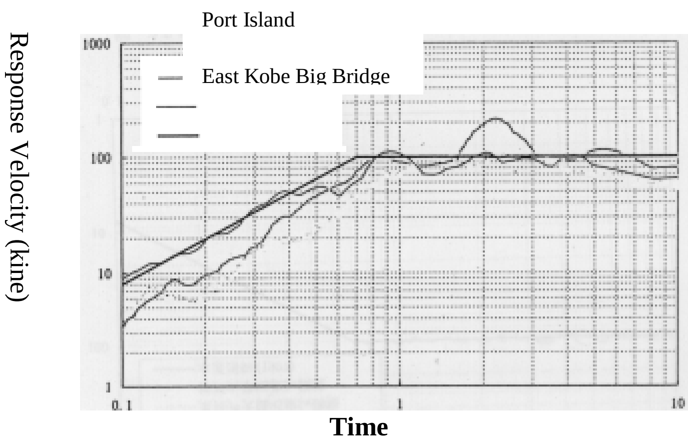

Fig. 7 Velocity response spectra of the input waves

The analyses have been done both in linear and non-linear cases for each input wave and the results are compared with each other.

NUMERICAL RESULTS

Since, there is not liquefaction in this study, the pore pressure and the settlement analysis options in the developed program have not been used and just linear and non-linear response of the soil are investigated. The time-history response of accelerations, displacements and strains for both X and Y directions are obtained at the pipe location. Also, the contours of the mentioned responses have been output for the whole model and in time domain. For getting more clear idea about the behavior of the model in the four cases of the analysis, the contours of the strains in X direction are shown in Figs. 8~11.

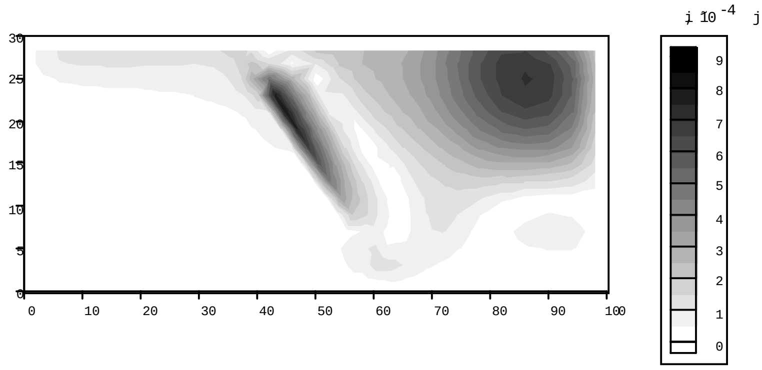

Fig. 8 Soil strain contours after 8 second for linear analysis (The Port Island input wave)

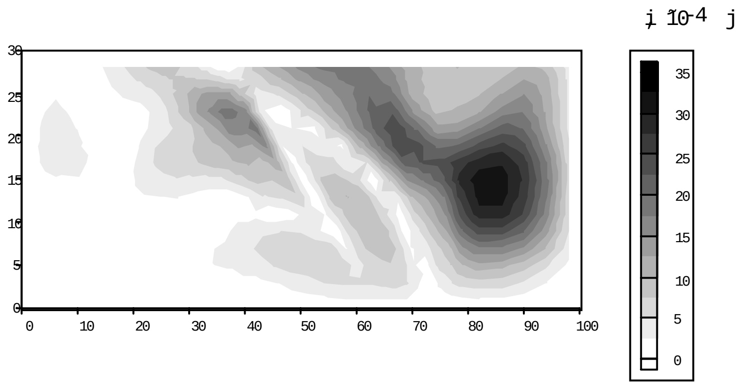

Fig. 9 Soil strain contours after 8 second for non-linear analysis (The Port Island input wave)

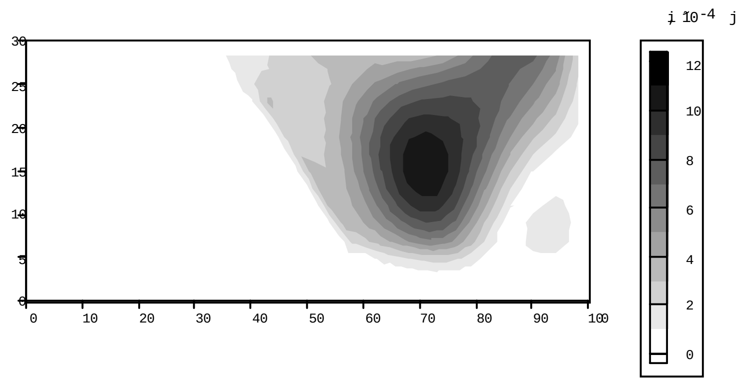

Fig.10 Soil strain contours after 8 second for linear analysis (The East Kobe Big Bridge input wave)

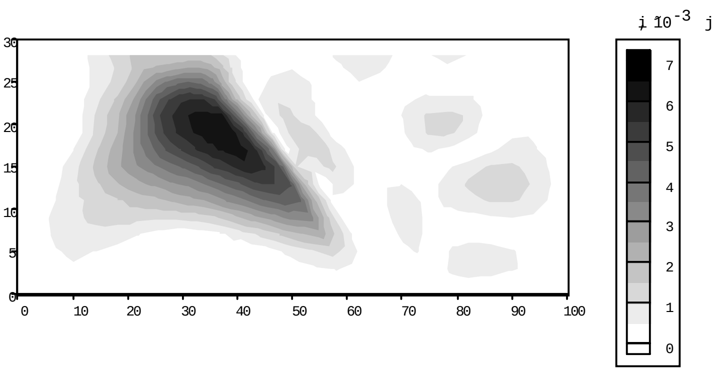

Fig.11 Soil strain contours after 8 second for non-linear analysis (The East Kobe Big Bridge input wave)

Since in design of buried pipeline, the axial strain is assumed as the main factor, the results of soil strain in X direction (pipe axial direction) at the location of the pipe are investigated and compared for different cases. The strains at pipe location are obtained by as the average strain of the element strains above and below of the pipe location. The maximum strains for the Port Island wave are given in Fig. 12 for linear and non-linear cases and those of the East Kobe Big Bridge are shown in Fig. 13.

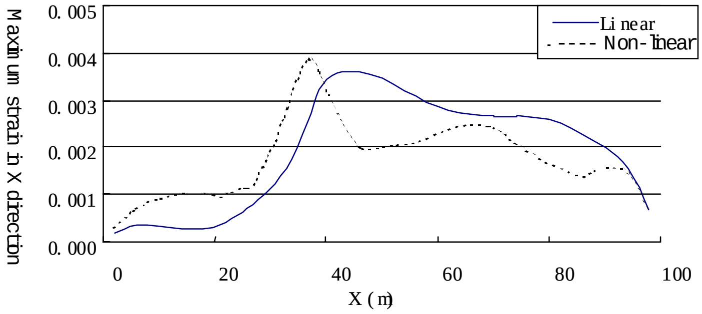

Fig.12 Soil maximum strain at location and axial direction of the pipe (The Port Island input wave)

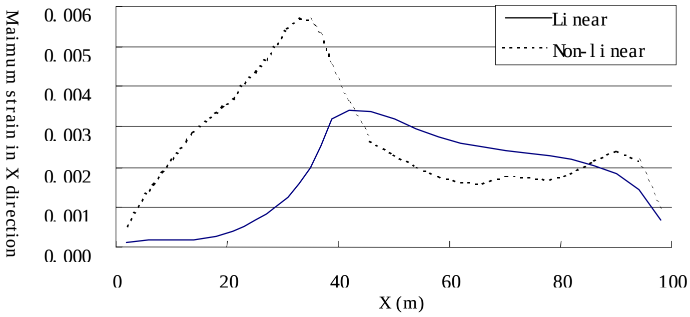

Fig.13 Soil maximum strain at location and axial direction of the pipe (The East Kobe Big Bridge input wave)

The maximum amounts of the soil strain in X direction at location of the pipe in four cases of analysis are summarized in Table 2.

Table 2 Maximum strain of the soil in X direction at pipe location

Input wave	Soil behavior	Maximum strain(%)	X of location (m)
Port Island	Linear	0.362	46.00
Port Island	Non-linear	0.389	37.00
East Kobe Big Bridge	Linear	0.341	42.00
East Kobe Big Bridge	Non-linear	0.569	33.00

DISCUSSION

As shown in Figs. 8 and 10, the maximum strains for the linear behavior of the soil and for both of the input waves are concentrated at the right side of the slope in the soil. It is also the case in Fig. 9, which shows the results of the non-linear analysis of the Port Island input wave. While, in Fig. 11, which is the result of the non-linear behavior under the input wave of the East Kobe Big Bridge with longer predominant period, the maximum strains have been concentrated on the left side of the slope. The results for the times after the 8 seconds are similar. This may be explained that, since the soft soil has been already completely plastic, so no more energy could be stored in the right side. Therefore, the remaining energy of the input wave has been accumulated in the hard one on the left side of the inclination and made it to be more strained for the remaining time.

As some common points among the results of the four cases of the analyses, it is clear that the maximum strains are concentrated near the sloped parts. The results of the non-linear cases are greater than the linear cases for each input wave. Also, it is found that the results in Fig. 8 is much more greater than all other cases, which mainly is due to the longer predominant of the input wave.

Figs. 12 and 13 show that in the pipe location the maximum strains of soil are located almost on the top of the start point of slope. Because this part is acting such as support for the soft soil on the right hand of the slope. But the X coordinate of the location is a little different for the different cases (as shown in Table 2). By increasing the maximum strain in a case, the X coordinate is decreased and shifted to the left side of the slope. For the linear analysis of the Port Island input wave it is most right, while for the non-linear analysis of the other input wave which has the largest strain that is on the most left of the slope. It means that stronger non-linear behavior make more of the left side (shorter period) of the slope to be strained.

Figs. 12 and 13 show also that the results of the non-linear cases are greater than the linear cases. The results of the Fig. 13 are larger than the ones in Fig.12. That is because of the difference in wave characteristics and the importance of the longer period in soil response.

Table 3 shows that there is a good agreement between the results of proposed method with those from FEM. The obtained results from the L1 and L2 Gas Standards are lower than the FEM results under the long period wave recorded at East Kobe Big Bridge. This may mean that in the Gas Standard, the effect of the ground motions with long periods is not enough investigated. Therefore, it needs more investigation and somehow modification for considering the effect of longer period waves of level 2.

Table 3 Comparison of the maximum strains obtained by simplified method and FEM analyses

Analysis	L1 Gas Standard [4)]	L2 Gas Standard [7)]	Proposed method (Eq. 1 with linear strain from SHAKE program)	FEM analysis Port Island wave (short period)	FEM analysis East Kobe B. Bridge wave (long period)
Maximum strain %	0.18%	0.39%	Port island: 0.06x6.5x1.1=0.429% Higashi Kobe B. B.: 0.055x6.5x1.7=0.607%	0.362 %(linear) 0.389% (non-linear)	0.342 %(linear) 0.569 %(non-linear)

CONCLUSIONS

A new method is proposed for computing the maximum ground strain at the pipe location (almost surface of the ground) in a ground with inclined bed-rock. The FEM analyses have been done both for linear and non-linear behavior of the soil and the obtained results are compared and discussed. The main conclusions can be summarized as followings:

1- The maximum soil strains in all cases of the analyses are concentrated around the slope part between the two non-uniform layers.
2-The maximum strains due to input wave with longer period are greater than the strains due to the input wave with higher amplitude.
3- There is a good agreement between the results of the FEM analyses with the maximum strain obtained from the proposed simplified method.
4- The obtained results from the Gas Standard is lower than the FEM result under the long period wave. This may mean that the effect of the ground motion at level 2 with longer period is not well considered.

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