

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

A new design method for buried pipes

Une nouvelle méthode de dessin des conduits enterrés

J. Tohda – Osaka City University, Osaka, Japan
 H. Yoshimura – Konoike Construction Co. Ltd., Tsukuba, Japan

ABSTRACT: A rational method is proposed for design of buried pipes which was validated based on extensive experimental and numerical (finite element) results. The proposed design method is based on a new design concept, reported at the 14th ICSMGE, that employed elastic analysis of this soil-structure interaction phenomenon using a continuum model. A design chart that accounts for mechanical properties of soil, flexibility of pipe, and type of pipe installation reasonably well is represented for practical applications. The chart can be applied for almost all construction conditions and is shown to be reliable as opposed to widely used conventional design procedures based on Marston-Spangler theory. The proposed design method is hoped to play a key role in preventing damage to buried pipes, in particular, for ditch-type of pipe installation with sheet-piling that has caused many problems in the field.

RÉSUMÉ: Nous proposons une méthode rationnelle de dessin des conduits enterrés, validée par les résultats d'une série d'essais et de l'analyse numérique (méthode des éléments finis FE). Le concept de cette méthode de dessin, comme nous l'avons exposée au 14e ICSMGE, utilise l'analyse de la théorie d'élasticité des interactions des conduits et du sol selon le modèle du continuum. Nous présentons pour les applications pratiques un graphique qui figure rationnellement la nature du terrain, la flexibilité des conduits et leur mode d'enfouissement. Ce graphique peut s'adapter aisément à la plupart des conditions de construction et s'avère plus fiable que la procédure conventionnelle de dessin basée sur la théorie de Marston-Spangler. On peut espérer que ce nouveau système de dessin jouera un grand rôle dans la prévention des ruptures des conduits enterrés, notamment celles qui ont causé beaucoup de problèmes dans la procédure d'installation en digue avec une palée.

1 INTRODUCTION

It is well known that current design procedures for buried pipes are inadequate because they are usually based on Marston-Spangler theory (M-S theory, Spangler 1948) which is based on unreasonable assumptions, especially the unrealistic distribution of earth pressures. As a result, buried pipes continue to fail or get damaged throughout the world.

At the 14th ICSMGE, the authors presented a new design concept which employed elastic analysis to simulate this soil-structure interaction phenomenon (Tohda & Yoshimura 1997). This paper proposes a rational method for a design of buried pipes based on the abovementioned concept. The proposed method is validated based on extensive experimental and numerical (finite element) results. A total of thirty-six centrifuge model tests and their FE elastic simulations were conducted in order to validate the numerical model. The paper describes simplifications that were applied to the FE model. A design chart developed using this simplified FE model is then represented. The current design procedure and proposed design method are compared to demonstrate the reliability of the proposed method as opposed to the M-S theory-based current design procedures. Finally, elastic constants (deformation moduli and Poisson's ratios) of different backfill soils with various densities are represented to assist in utilizing the design chart.

2 CENTRIFUGE MODEL TESTS AND FE ANALYSIS

2.1 Centrifuge model tests

Earth pressure on and deformation behavior (bending strains and vertical deflections) of buried pipes were first investigated by conducting thirty-six centrifuge model tests. Three model pipes of different flexibilities were buried under various conditions, and effects of pipe flexibility, type of pipe installation, type of backfill soil, and burial dimensions were quantified.

The model aluminum pipes were named as Rigid-pipe,

Medium-pipe, and Flexible-pipe according to their flexibilities (Tohda et al. 1990). External diameters (D) and wall thicknesses (t) of these model pipes varied as 90 mm and 3.5 mm for Rigid-pipe, 86 mm and 1.5 mm for Medium-pipe, and 90 mm and 0.95 mm for Flexible-pipe, for their flexural stiffnesses (S_p) to be 3.2 MPa, 0.29 MPa, and 64 kPa. S_p is defined by the authors as $S_p = E_p t^3 / \{12(1 - \nu_p^2) R^3\}$, where E_p and ν_p : Young's modulus and Poisson's ratio of pipe material (72.5 GPa and 0.33 in the model pipes), respectively, and R: neutral radius of pipe ($=D/2 - t/2$).

The tests were carried out using models shown in Fig. 1 for three types of pipe installations: ditch type with sheet-piles involving sheet-pile extraction process (Ditch-S), embankment type (Embk.), and ditch type without sheet-piling (Ditch-0). Ditch-S type of pipe installation method is typically adopted in urban areas. The table in Fig. 1 shows the prototype dimensions (external diameter of pipe D, cover height H, thickness of foundation H_b , ditch width B_d , and width of the ground B), where the rectangular enclosure marks denote standard dimensions.

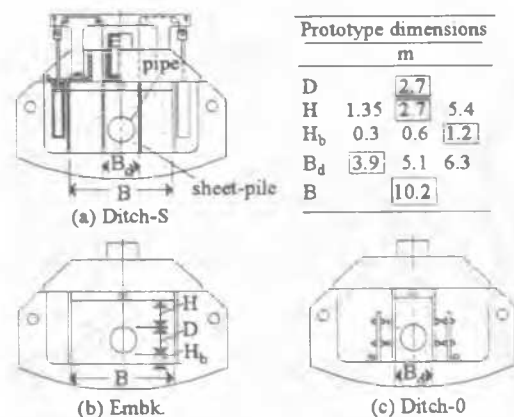


Figure 1. Centrifuge model configurations for three types of pipe installations.

Three types of sandy soils: dry silica sand (S0), decomposed granite (S16), and silty sand (S30), with primary properties as shown in Table 1 of Section 5, were used in the tests. The model ground was prepared by pluviating the S0-soil, and by compacting the S16- and S30-soils. The secondary properties (dry density and water content) of the model grounds are listed as S0- I, S0- III, S16- I, and S30- I in Table 2 of Section 5.

The models were subjected to a centrifugal acceleration of either 30 g or 31.4 g. In Embk. and Ditch-0 models, a load equivalent to a design load due to several 20-ton vehicles was applied at the center of the ground surface by means of a strip loading plate. In Ditch-S model, sheet-piles were modeled with plane aluminum plates, with thicknesses that were adjusted to be 15 cm in prototype scale which considered the soil that usually sticks to the extracted sheet-piles. A pair of model sheet-piles was simultaneously extracted to simulate the most severe field condition, and the surface load was applied afterwards.

Earth pressures, bending strains in the pipes, and vertical pipe deflections (change in the pipe diameter in vertical direction) were measured. The accuracy in these measurements was confirmed in all tests (Tohda et al. 1994) by establishing an excellent agreement between measured and calculated bending strains (based on measured earth pressures) and deflections (based on measured bending strains), and also by satisfying force equilibrium (based on measured earth pressures). The tests revealed that earth pressure concentrations on tops and bottoms of rigid pipes and their relaxations on flexible pipes occur, and their magnitudes strongly depend on the type of pipe installation.

2.2 FE elastic analysis

All thirty-six centrifuge model tests were analyzed under plain strain condition using FEM, in which the pipe, soil, and sheet-piles were modeled to be linearly elastic. Fig. 2 shows FE mesh used in the analysis. The analysis for Ditch-S model included the following five stages: before and after the sheet-pile extraction, and at the instances when the lower ends of the sheet-piles passed the levels of bottom, mid-height, and top of the pipe.

The unit weights of the model pipes were assigned uniformly to the pipe elements. The weights of the sheet-pile elements in Ditch-S model were neglected, because the sheet-piles were extracted by means of wires connected to a pair of hydraulic cylinders in the tests. The apparent unit weights of the soil elements were determined based on the measured frictional force on the inner walls of the test container.

Young's modulus E_s and Poisson's ratio ν_s of the soil were determined from K_0 -compression tests described later in Section 5. According to the test results (Table 2 and Fig. 7), E_s was dependent on applied axial stresses σ_1 , whereas ν_s was independent of σ_1 . Thus, different values of E_s were assigned to the soil elements with different depths to account for the stress-dependency. These values of E_s were determined from Fig. 7 as the ones corresponding to the ground stresses of the soil elements calculated

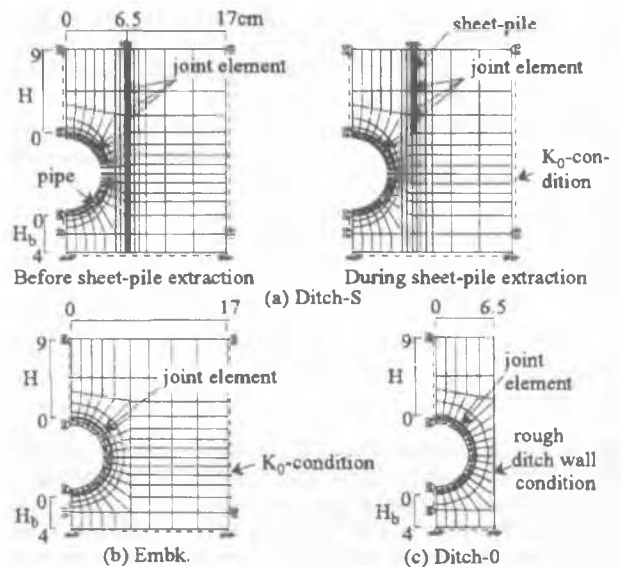


Figure 2. FE models for three types of pipe installations.

by using the apparent unit weight of the soils. On the other hand, a constant value of ν_s (Table 2) was assigned to all soil elements. In the analysis of Ditch-S model, very small values of E_s , $\nu_s=0$, and γ (unit weight)=0 were assigned in the spaces that were initially occupied by the extracted sheet-piles. Furthermore, the E_s values were reduced in the backfill zones which were loosened by the sheet-pile extraction. The values of E_s and the extent of loosened zones were determined at each stage based on the ground deformation observed in the photographs taken during the extraction of the sheet-piles in the experiment.

K_0 -conditions were applied to the lateral boundaries of the model ground in Ditch-S and Embk. models. A rough wall condition allowing the separation, slip failure, and frictional resistance (Tohda et al. 1988) was given at the ditch walls in Ditch-0 model. Both vertical and horizontal nodal displacements at the bottom of the model ground were fixed. The separation and small value of frictional resistance were allowed at the interface between the pipe and soil by introducing joint elements. In Ditch-S models, similar joint elements were introduced at the interface between the sheet-pile and soil to consider the separation and frictional resistance.

2.3 Comparison between the tests and analysis

Figs. 3 and 4 compare the measured and computed earth pressures for different pipe flexibilities, types of pipe installations, and types of soils. The figures are presented using polar coordinates. Compressive normal (σ) and downward tangential (τ) earth pressures are defined as positive. The symbols and curves in the figures denote the measured and computed earth pressures.

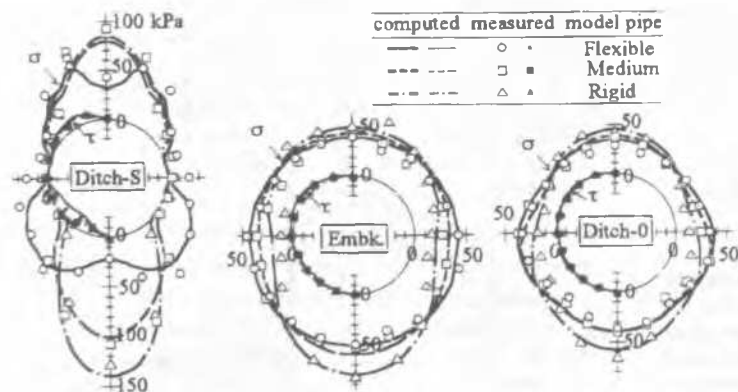


Figure 3. Measured and computed earth pressures for different pipe flexibilities and types of pipe installations (backfill soil: S0-IV).

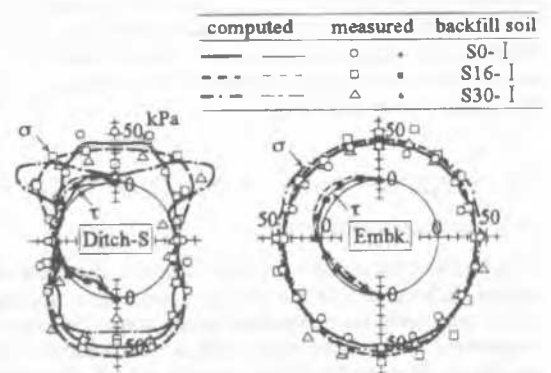


Figure 4. Measured and computed earth pressures on Flexible pipe for different backfill soils and types of pipe installations.

tures, respectively. The data from Ditch-S model correspond to the instance at which the lower ends of the extracted sheet-piles passed the level of the top of the pipe. The computed earth pressures conform very well to the measured ones in all cases. Computed vertical deflections and bending strains of the pipes also compared well with measured ones in all cases. In general, the numerical results compared very well with the experimental measurements from all thirty-six model tests, involving the surface loading stages in the three installation models and five stages of the sheet-pile extraction in Ditch-S models.

3 SIMPLIFIED FE MODEL AND DESIGN CHART

The FE model used in the former section was simplified for the practical applications as follows:

First, among the three types of soils used in the model tests, the decomposed granite (S16) was selected as a standard backfill, because this soil is commonly used as a backfill in Japan, and the model tests that employed this soil generated the greatest pipe deflection for any type of pipe installation. The following simplifications were further applied to the FE model: (1) The pipe-soil interface was changed to be of full-slippage with a possibility for some separation; (2) A constant value of E_s corresponding to the ground stress level at the mid-height of the pipe was assigned to all soil elements; (3) In Ditch-S model, only the instance, just after the sheet-pile extraction, when the measured pipe deflection was greatest during the sheet-pile extraction process was selected for the calculation; and (4) In addition, for Ditch-S model, it was assumed that the spaces initially occupied by the sheet-piles became completely empty, and the backfill soil remained intact without loosening as a result of the sheet-pile extraction.

This simplified model generated the distribution of earth pressures with somewhat higher concentration at the top and the bottom of the pipe. As a result, the experimental measurements of pipe deflections and bending strains were closer to those based on the simplified model than those with the model described earlier. Based on these results, it was concluded that the adopted simplifications were acceptable.

Using the simplified model, a design chart shown in Fig. 5 was developed to predict the vertical deflection ratio ($\delta=AD/(2R)$, AD : vertical deflection of pipe) and maximum bending moment (M) of the pipe. In Fig. 5, the relationships between the normalized design values (ω and K) and the flexibility index (κ) were presented for three types of the pipe installation methods, with different cover heights (H). The quantities ω , K , and κ are defined as: $\omega=\delta E_s/(\gamma H)$, $K=M/(\gamma H R^2)$, and $\kappa=E_s/S_p$, respectively. Here, γ and E_s : unit weight and Young's modulus of the backfill soil, and S_p : flexural stiffness of the pipe ($=E_p I_p / \{12(1-\nu_p^2)R^3\}$).

The value of ν_p of the backfill soil was selected to be constant

and equal to 0.3, because this ν_p value can generally be applied to any backfill soil as indicated by Table 2. A thorough examination showed that this chart can be applied to any pipe (regardless of different materials, weights, and dimensions) and any backfill soil (regardless of different soil types and values of E_s and γ). Therefore, it was concluded that the design chart can be applied to almost all field construction conditions.

4 COMPARISON: CURRENT AND PROPOSED DESIGNS

A hypothetical design problem is now considered. A concrete pipe ($t=17.4$ cm, $E_p=44$ GPa, $\nu_p=1/6$, and $S_p=9.8$ MPa) and a steel pipe ($t=2$ cm, $E_p=206$ GPa, $\nu_p=0.3$, and $S_p=63$ kPa) with the same external diameter (D) of 2.7 m were designed for the three types of pipe installations according to a typical M-S theory-based current design standard (JMAFE) and the proposed design method. The burial dimensions are the standard ones shown in the table of Fig. 1, that is, $H=2.7$ m, $H_s=1.2$ m, and $B_s=3.9$ m. Backfill soil is the decomposed granite with $\gamma=16.2$ kN/m³ and $w=10\%$ (S16- I shown in Tables 1 and 2).

For the present design conditions, the JMAFE standard specifies 90 degrees as the bedding angle and 2.74 MPa as the coefficient of subgrade reaction. It also specifies different design loads (intensities and distribution shapes, primarily based on M-S theory) for a combination of three factors: flexibility of the pipe (rigid and flexible), type of pipe installation (Ditch-S, Embk., and Ditch-0), and existence of equal settlement plane that depends on an empirical coefficient named as equal settlement ratio. The concrete pipe and the steel pipe are respectively classified into the rigid-pipe and flexible-pipe categories according to type of pipe material only. δ and M for both pipes are calculated by using the respective design loads.

In the present design method, δ and M for both pipes are obtained as follows. The deformation modulus of the backfill soil (E_s) is determined to be 0.78 MPa from K_σ -compression test (cf. S16- I curve in Fig. 7) as the value corresponding to the ground stress level at the mid-height of the installed pipes ($\sigma_1=65.7$ kPa). Poisson's ratio (ν_s) is determined as 0.3 (cf. S16- I in Table 2). Using E_s and S_p , $\kappa (=E_s/S_p)$ is calculated as 0.08 for the concrete pipe and 12.5 for the steel pipe. Thus, ω and K for each installation type can be obtained by using the curves of $H/D=1$ in Fig. 5. Substituting the values of ω and K in their definitions: $\omega=\delta E_s/(\gamma H)$ and $K=M/(\gamma H R^2)$, δ and M can be calculated.

Fig. 6 compares δ and M obtained in accordance with the JMAFE standard and proposed method. The figure shows that the proposed design method generated δ and M of both pipes to be definitely greater (2 to 4 times) in Ditch-S type, considerably greater in Embk. type, and quite smaller in Ditch-0 type, than those based on the JMAFE standard. The design values for

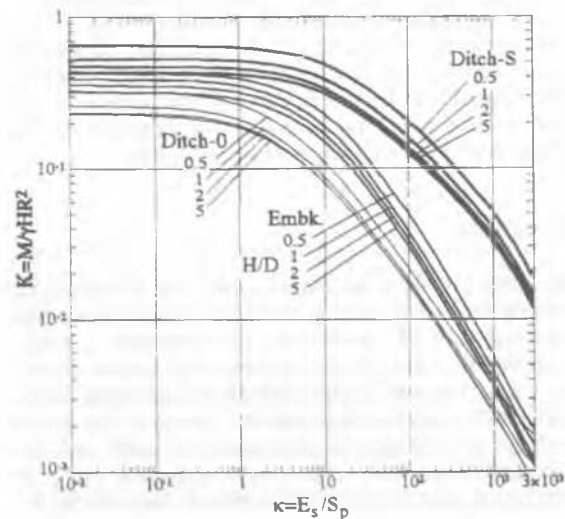
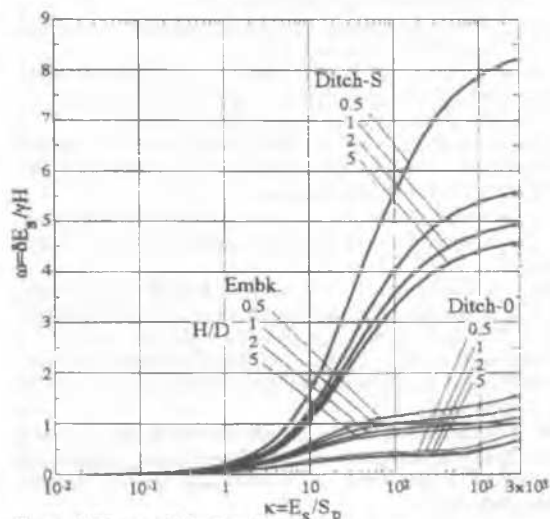


Figure 5. Proposed design chart.

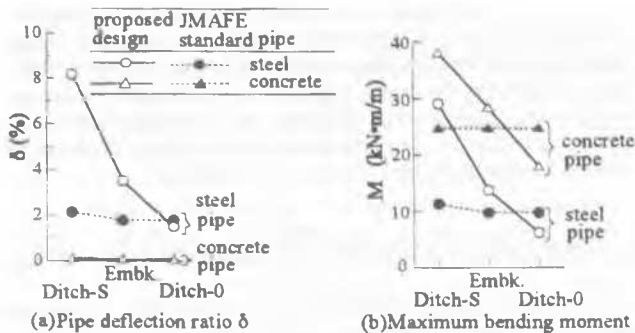


Figure 6. Comparison between proposed design method and JMAFE Design standard.

Ditch-S type predicted by the proposed method are adequate in the light of the authors' experience and knowledge (Tohda et al. 1985), as well as several reports (for example, Noppadol et al. 1991 and Werner et al. 1991).

The above comparison revealed that the procedure of the proposed design method is considerably simple and accounts for the mechanical properties of soil, flexibility of pipe, and type of pipe installation reasonably well. As a result, as opposed to the M-S theory-based current design procedures, the proposed method predicts adequate design values, in particular, for Ditch-S type that has historically caused many problems in the field.

5 ELASTIC CONSTANTS OF BACKFILL SOILS

In order to use the design chart (Fig. 5), it is necessary to determine the elastic constants E_s and ν_s of backfill soils. Table 2 and Fig. 7 show ν_s and E_s for four types of soils with different densities, whose properties are shown in Tables 1 and 2. The E_s and ν_s were obtained through K_0 -compression tests using a rectangular box (12 cm \times 12 cm \times 10 cm in height), in which axial strain ϵ_1 and lateral stress σ_3 ($=\sigma_2$) at each level of applied axial stress σ_1 were measured under the K_0 -condition ($\epsilon_2=\epsilon_3=0$). E_s and ν_s were calculated from Hooke's law through $\nu_s=(\sigma_3/\sigma_1)/(1+\sigma_3/\sigma_1)$ and $E_s=(1-\nu_s^2)/(\sigma_1/\epsilon_1)$.

ν_s of all soils were independent of σ_1 and could be taken as 0.3 as indicated by Table 2. E_s of the silica sand (S0) increased monotonously with increasing σ_1 as shown in Fig. 7. According to the test results, however, E_s of three c- ϕ type soils (S16, S30, and S44) increased initially with increasing σ_1 and reached peak values when σ_1 reached yield stresses (σ_y) that were induced by compaction. After the yield point in σ_1 was passed, E_s generally decreased. For practical purposes, the changes in E_s seen in the range of $\sigma_1 \leq \sigma_y$ can be ignored with an assumption that the soils behave as linear elastic bodies under overconsolidated conditions. Therefore, constant E_s that are equivalent to the peak values obtained at $\sigma_1=\sigma_y$ are assigned in the range of $\sigma_1 \leq \sigma_y$ in Fig. 7.

In order to use the design chart, a designer may choose the properties of backfill soils first from Tables 1 and 2 closest to those given. Using the respective E_s curve shown in Fig. 7, E_s for design can be determined as the value corresponding to the ground stress level at the mid-height of a buried pipe.

6 CONCLUSIONS

A rational design method for buried pipes that is thoroughly validated based on the results of an extensive series of centrifuge experiments and their FE simulations was proposed. A simple design chart that accounts for the mechanical properties of soil, flexibility of pipe, and type of pipe installation reasonably well is represented for the practical applications. The chart can be applied for almost all construction conditions very easily, and can predict adequate design values. The chart is especially useful for the design of ditch-type of pipe installation with sheet-piling, the

Table 1. Primary properties of backfill soils.

| Soil | G_s | D_{max} mm | Sand % | Fine % | U_o | ρ_{dmax} g/cm ³ | ρ_{dmin} g/cm ³ | W_{opt} % |
|------|-------|-----------------|-----------|-----------|-------|------------------------------------|------------------------------------|----------------|
| S0 | 2.65 | 1.4 | 100 | 0 | 1.75 | 1.58 | 1.32 | — |
| S16 | 2.71 | 2.0 | 84 | 16 | 70 | 1.92 | 1.42 | 11.4 |
| S30 | 2.67 | 2.0 | 70 | 30 | 115 | 1.86 | 1.29 | 13.5 |
| S44 | 2.67 | 2.0 | 56 | 44 | >500 | 1.79 | 1.31 | 16.2 |

Table 2. Secondary properties (w and ρ_d) and Poisson's ratios (ν_s) of backfill soils.

| Soil | w % | ρ_d (g/cm ³) | | | | ν_s | | | |
|------|----------|-------------------------------|------|------|------|---------|------|------|------|
| | | I | II | III | IV | I | II | III | IV |
| S0 | 0 | 1.43 | 1.47 | 1.50 | 1.55 | 0.29 | 0.29 | 0.29 | 0.28 |
| S16 | 10 | 1.50 | 1.60 | 1.70 | 1.80 | 0.30 | 0.30 | 0.30 | 0.27 |
| S30 | 12 | 1.50 | 1.60 | 1.70 | 1.80 | 0.32 | 0.31 | 0.30 | 0.30 |
| S44 | 14 | 1.40 | 1.50 | 1.60 | 1.70 | 0.33 | 0.33 | 0.33 | 0.32 |

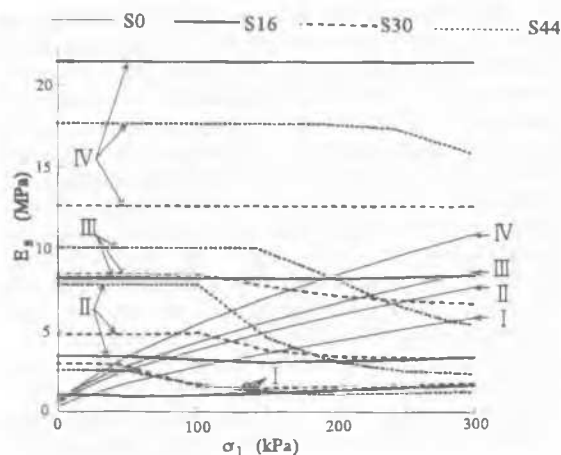


Figure 7. Stress dependent deformation moduli of backfill soils to be adopted in design.

method that has caused many problems in the field. Therefore, the proposed design method is hoped to play a key role in preventing damage to buried pipes in future.

REFERENCES

- JMAFF (Japanese Ministry of Agriculture, Forestry and Fisheries). 1988. Design standard of pipeline.
- Noppadol, P. et al. 1991. Geotechnical experiences from construction of large diameter underground water pipe in soft Bangkok clay. *Developments in geotechnical aspects of embankments, excavations and buried structures*. Rotterdam: Balkema.
- Spangler, M.G. 1948. Underground conduits—an appraisal of modern research. *Trans. of ASCE* 113: 316-373.
- Tohda, J., Horita, K., Shinozaki, W. & Mikasa, M. 1985. Earth pressure on underground concrete pipe in a field test. *Proc. Intern. Conf. on advances in underground pipeline engg.*: 576-585. ASCE.
- Tohda, J., Mikasa, M. & Hachiyu, M. 1988. Earth pressure on underground rigid pipes: Centrifuge model tests and FEM analysis. *Centrifuge 88*: 395-402. Rotterdam: Balkema
- Tohda, J., Yoshimura, H., Morimoto, T. & Seki, H. 1990. Earth pressure acting on buried pipes in centrifuged models. *Intern. Conf. on Design and Installation*: 17-31. ASCE.
- Tohda, J., Li, L. & Yoshimura, H. 1994. Analysis of the factors in earth pressure and deformation of buried flexible pipes through centrifuge model tests. *STP 1222, Buried pipe technology*: 180-194. ASTM.
- Tohda, J., Yoshimura, H. & Li, L. 1997. Mechanical behavior and design of buried pipes as a soil-structure interaction. *Proc. of 14 th ICSMGE*: 1049-1052.
- Werner, H.U. & Eichstadt, F. 1991. Large pipes buried in soft soil pre-fabricated pipes in River Nile Sediments. *Developments in geotechnical aspects of embankments, excavations and buried structures*. Rotterdam: Balkema.