Importance of Taking Deformation of Substructure into Account for Bridge Design

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ABSTRACT: In Japan, conformity in the design of a bridge's superstructure and substructure has been realized by taking into account the correlation between the load and displacement of its foundation. This has enabled a rational seismic design to be achieved, but failure to take into account deformation of the foundation might invite a serious accident. The collapse of the Can Tho Bridge in Vietnam was caused by uneven settlement under the foundation of a bent supporting the concrete slab, which sank slightly during construction process due to the slab's step weights. In this paper, the author proposes a newly developed calculation method based on a revised Voigt Model for estimating the settlement of soft ground in the Mekong Delta.

1 INTRODUCTON

In the past, the design for a bridge's superstructure was developed independently the design for its substructure. A bridge's superstructure was designed according to the elastic theory, while the foundations of its substructure was designed according to the plastic equilibrium theory (which ensures that a balance is maintained between the load and the ground reactions) and the abutments and piers of its substructure were designed according to the elastic theory.

However, to make a rational design for a bridge, it is necessary to design the superstructure and the substructure according to the same criteria, especially, when constructing indeterminate structures. In Japan, conformity in the design of a bridge's superstructure and substructure has been realized by taking into account the correlation between the load and the displacement of its foundation. This in turn has enabled a rational seismic design to be achieved.

2 DESIGN CODE OF FOUNDATION OF HIGHWAY BRIDGE

2.1 *Types of foundation*

As shown in Fig.1 and Fig.2, there are three main types of foundations: spread foundations, caisson foundations and pile foundations

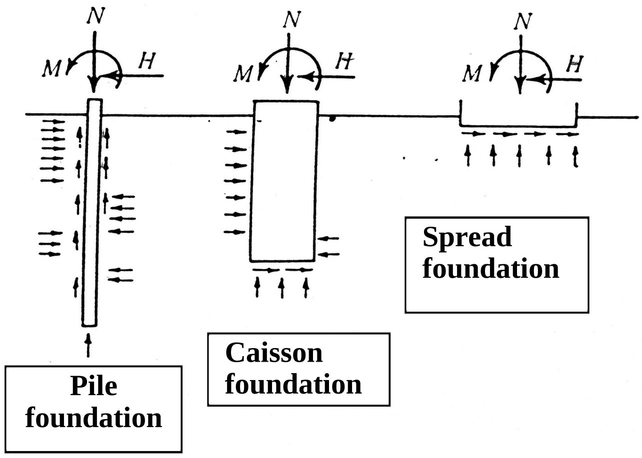

Fig. 1 Three main types of foundations

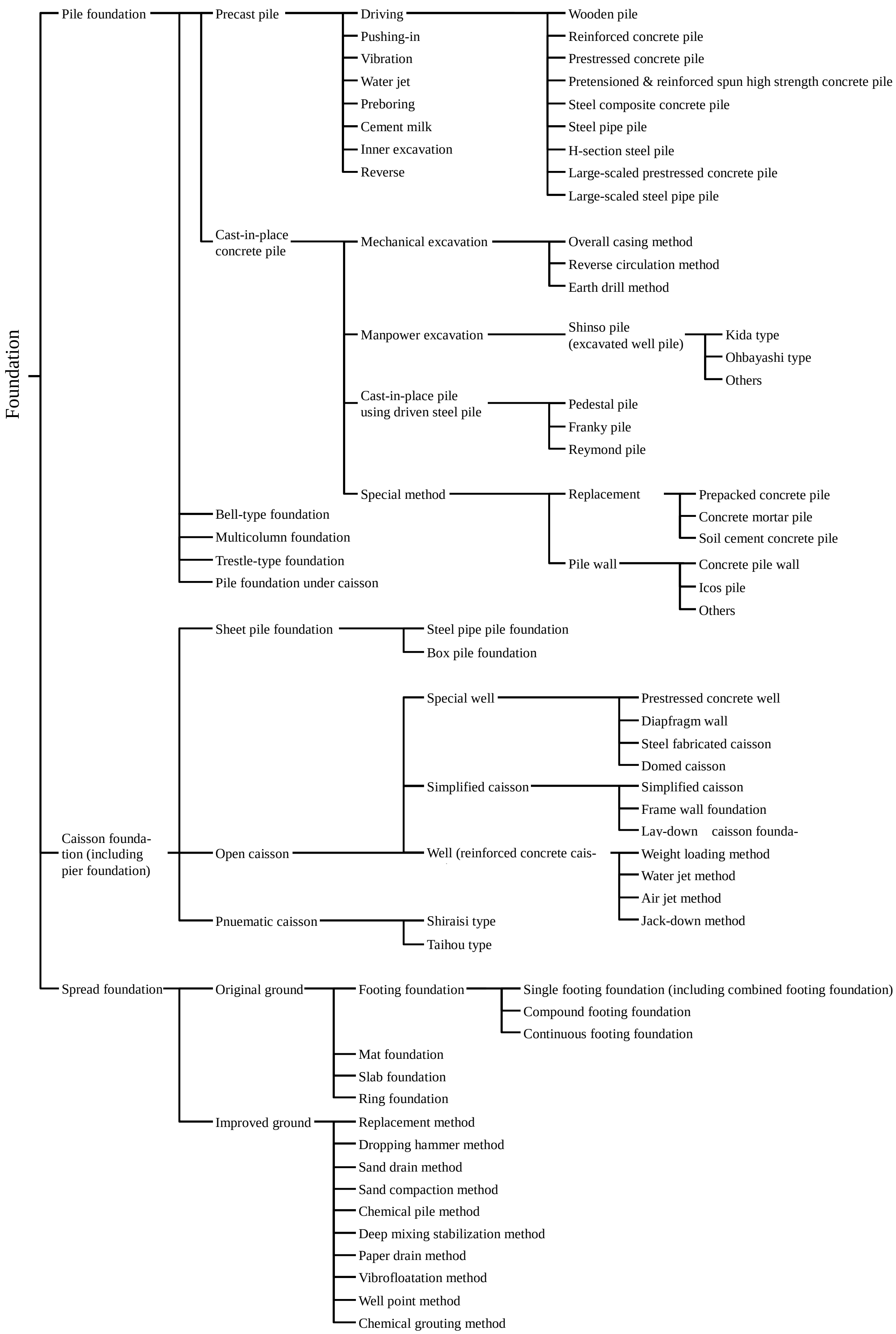

Fig.2 Classification of Foundation Methods

The design method used for spread and caisson foundations differs from that used for pile foundations. For a spread or caisson foundations the displacements and the rotational angles of the foundation, which is a rigid body, are calculated. For a pile foundation the displacement and the rotational angle of the foundation's footing are estimated using the ground reaction coefficient k_H (N/cm^3) and the axial spring constant Kv (N/cm) which varies according to the properties of the piles.

2.2 *Ground reaction coefficient*

The concept behind the linearization of k is shown in Fig.3. The angle of the secant line between the load and displacement, which is derived from the practical displacement limit (par S_0) serves as a kind of spring constant for the elastoplastic ground. The various spring constants are defined as follow: k_H for the horizontal force, k_V for the vertical force, k_S for the shear force and k_R for the moment.
For dynamic analysis, k represents the inclination angle of the line segment from A to C shown in Fig.4, while the equivalent damping ratio h_e is expressed by the formula $h_e = \Delta W / 2\pi W$ and the hysteresis curve of load-displacement shown in Fig.4.
These spring constants and the damping coefficients can be applied to the analytical models for the superstructure and the substructure of bridges as shown in Fig.8 (c substitutes for h_e).

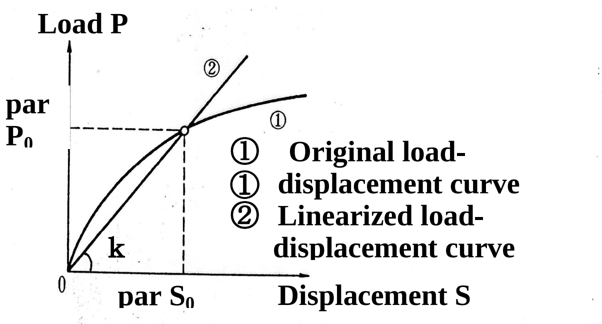

Fig.3 Concept behind the linearization of k

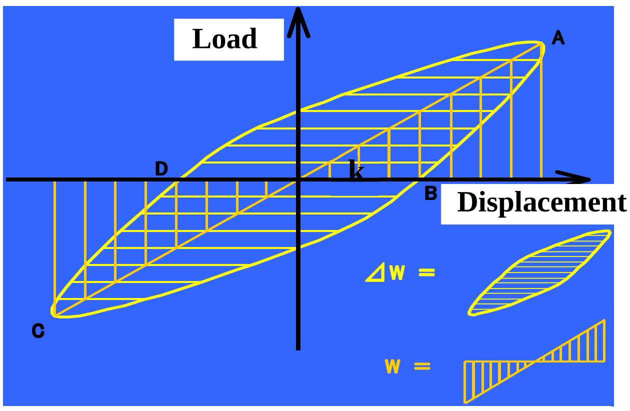

Fig.4 Hysteresis curve of load and displacement

2.3 *Static design*

The static design for a foundation is developed based on the correlation between the load and the displacement of a foundation. Fig.5 shows an analytical model of a pile foundation with the following spring constants K_1, K_2, K_3, K_4 and K_V. These constants combine the footing's horizontal displacement δ_x, its vertical displacement δ_y and its rotational angle α together with the vertical load V_0, the horizontal load H_0 and the rotational moment M_0 of the resultant force.

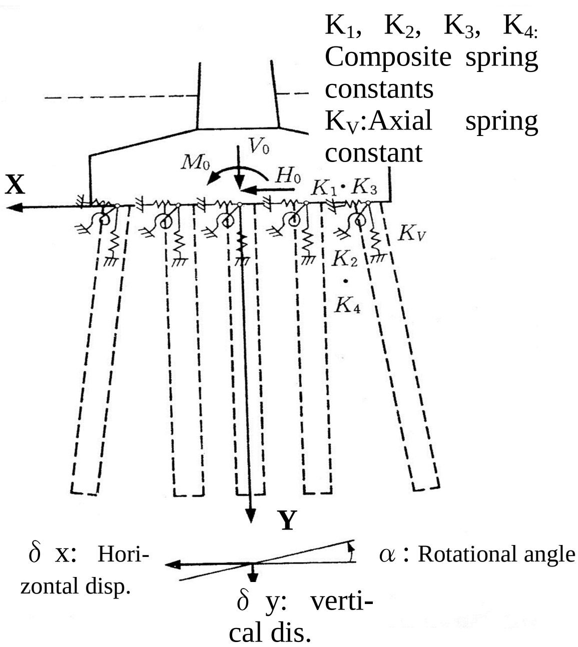

Fig.5 Schematic diagram of a pile foundation

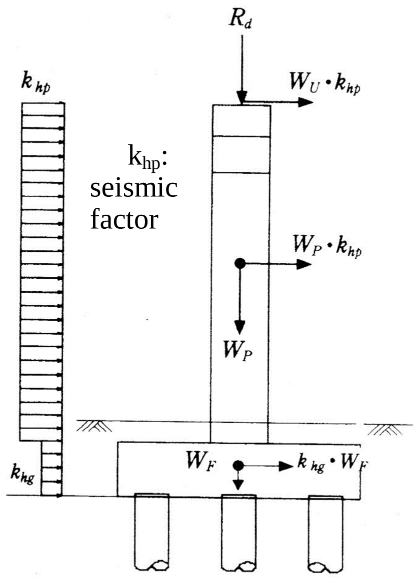

Fig.6 Seismic force to a pier

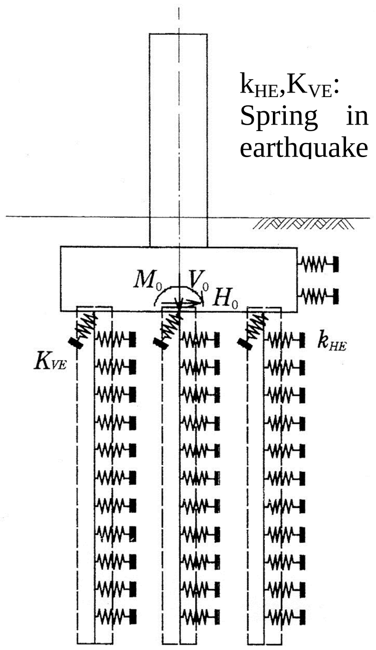

Fig.7 Static seismic design for a pile foundation

Fig.6 shows a static seismic design for a pier using the seismic factor method. Fig.7 shows the seismic design for a pile foundation subjected to the resultant force exerted by the substructure.

2.4 *Dynamic seismic design*

Thus, adoption of the above common coefficients makes it possible to develop the rational static and the dynamic designs of the whole structure that takes into account the influence of mutual movements between the superstructure and the substructure (Fig.8), though the coefficient values related to foundation are approximate. However, many models used for the dynamic analysis of seismic designs are composed of masses and springs as shown in Fig.9.

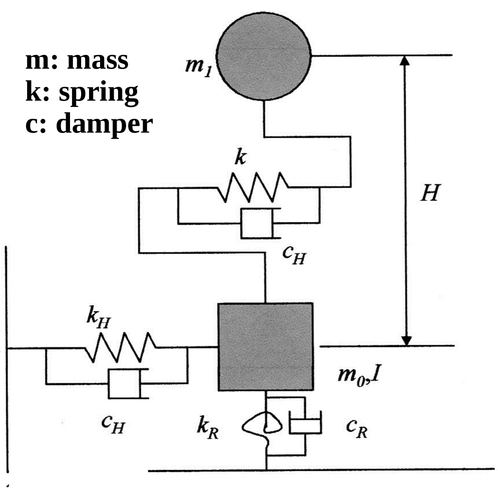

Fig.8 Simple model of a bridge

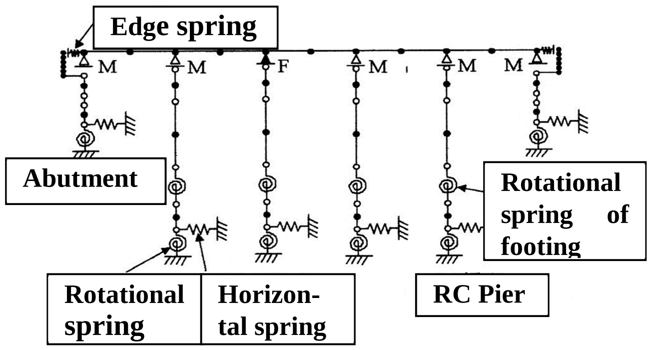

Fig.9 A model of a continuous bridge

Particularly, in the case of statically indeterminate superstructures, a bridge should be built on a firm, stable foundation because the forced deformation of panel points yields a substantial stress concentration and causes serious damage to the structure. It is important to note that if this concept is not thoroughly understood, deformation of the foundation might invite a serious accident.

3 COLLAPSE OF A SIDE SPAN OF CABLE STAYED BRIDGE BUILT ON VERY SOFT GROUND

3.1 *Overview of the bridge*

The forced deformation of panel points caused by differential settlement often yields a large moment in the girder used on the piers of a continuous bridge. With long span bridges or girders with high rigidity, it may be possible for the moment to be absorbed or the reactions and settlements to be redistributed between the supports on piers.

With a short and weak indeterminate girder, differential settlement gives rise to critical damage. Fig.10 shows a cable stayed bridge with a long pile foundation resting on very soft ground. During the construction of a concrete side span for this bridge, the temporary support (bent) collapsed resulting in many casualties and damage to the concrete girder which was supported by three piers and two bents forming four span continuous beams (Fig.11).

3.2 *Collapse of the concrete girder*

The concrete girder had two spans, each with a length of 40 m, a width of 26 m and a height of 2.7 m, and with concrete pile foundation of 75 m. The concrete portions of the girder were divided into upper and lower slabs (Fig.11), and into a total of 14 segments (as shown in Fig.12). The collapse occurred when work commenced the morning after the concrete for segments 10 and 11 had been laid. Fig.13 shows the situation before the side span's collapse, and Fig.14 shows the wreckage left after

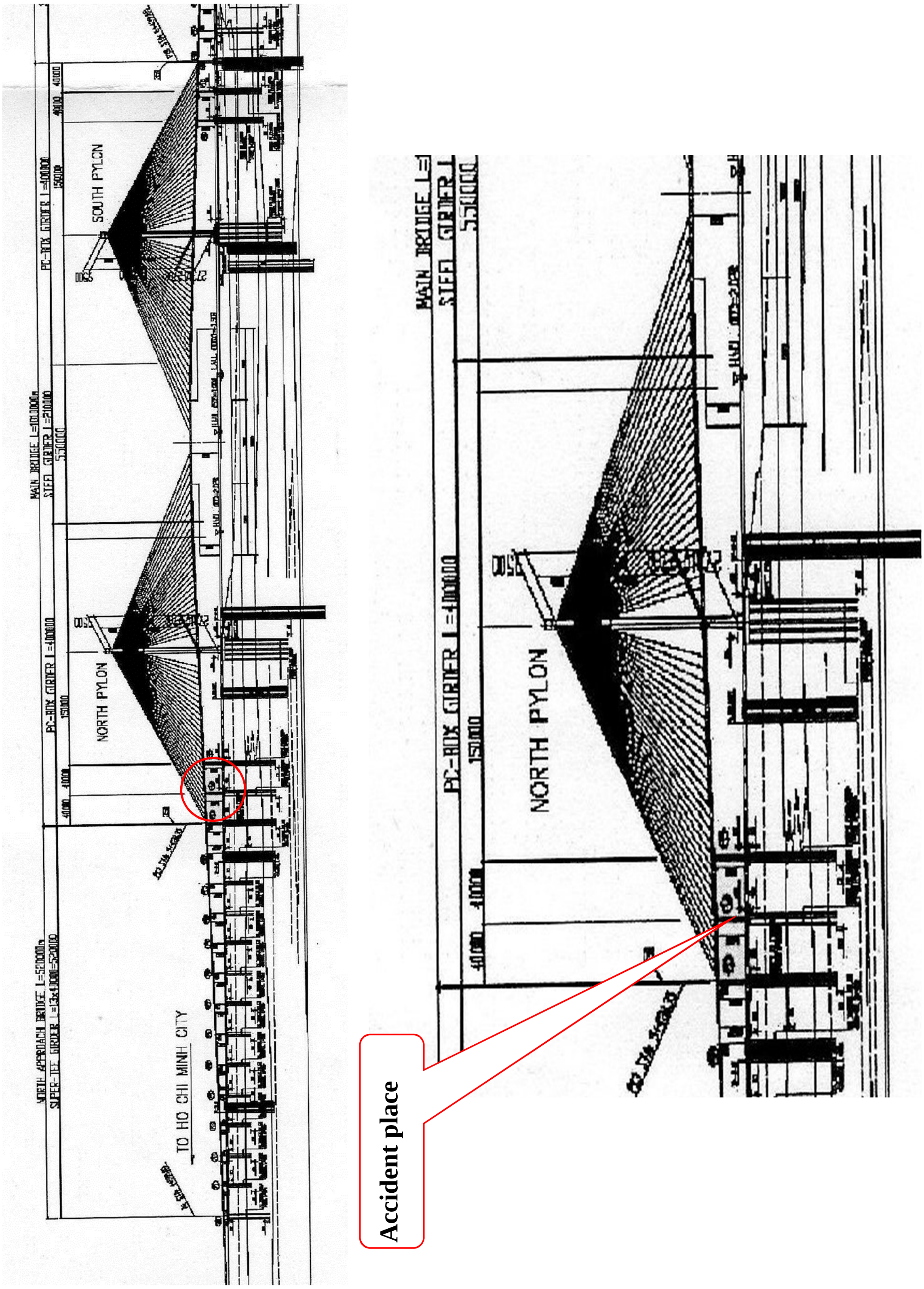

Fig.10 Side view of the Can Tho Bridge (a cable stayed bridge with the steel center span and the concrete side spans)

its collapse. According to a survey by the State Committee for Investigation of Two Side Spans of Can Tho Bridge, the members that collapsed were those shown in Fig.15, and the Committee determined that the bent-13 collapsed first.

3.3 *Causes of collapse*

A geological diagram of the site (Fig.16) indicates the presence of very deep sedimentary ground with weak sustaining layers (a fine sand layer located between 35 and 50 m below the ground surface is sandwiched between two cohesive layers).

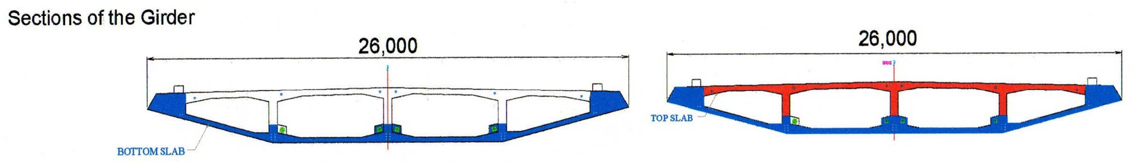

Fig.11 two stage construction of concrete girder

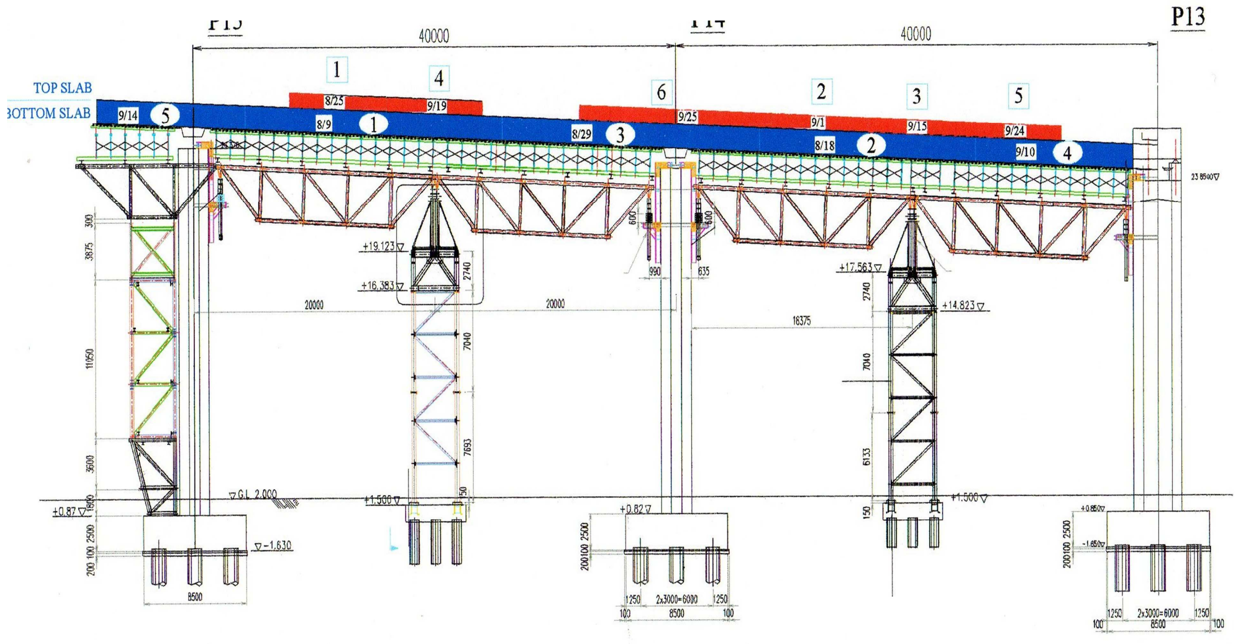

Fig.12 Scaffoldings and bents for prestressed concrete girder and order of segments to cast in

Fig.13 Construction view before collapse

Fig.14 Collapsed concrete girder

Furthermore, the material in each sustaining layer is very fine, with a grain size of less than 0.2 mm, and the sandy soils in particular are fairly uniform. Although the pile foundations for the pylons reached down to a sustaining layer at a depth of 70 m below the ground surface, the piles for the bents and piers were driven into the sandy layer about 35 m below the ground surface. This layer does not provide as reliable a degree of bearing capacity due to the presence of fine uniform sand particles with moderate N values.

Fig.17 shows the settlement of the bent-13, with twin pile foundations with 14 precast concrete piles of 37 m in length and a small base (footing), as well as cracks on the concrete girder that were witnessed by labors or were hypothesized to exist. Fig.18 indicates the differential settlement within one base (T13-U), which is sized into 4.5 x 4.5 x 1.0m, after the collapse and the first buckled members of the bent-13 based on calculation.

Based on these calculations, the cause of the collapse is presumed to have been a combination of the differential settlement and the buckling of the

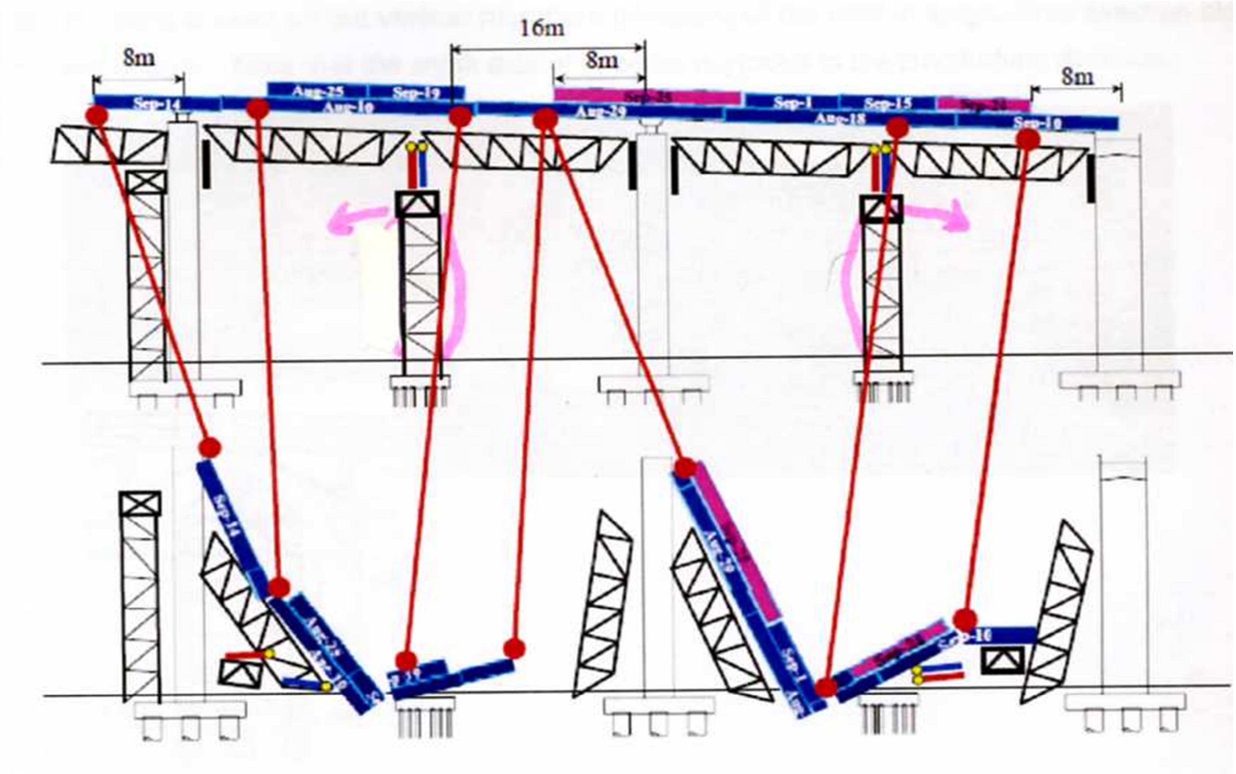

Fig.15 Collapsed concrete segments, scaffoldings and others

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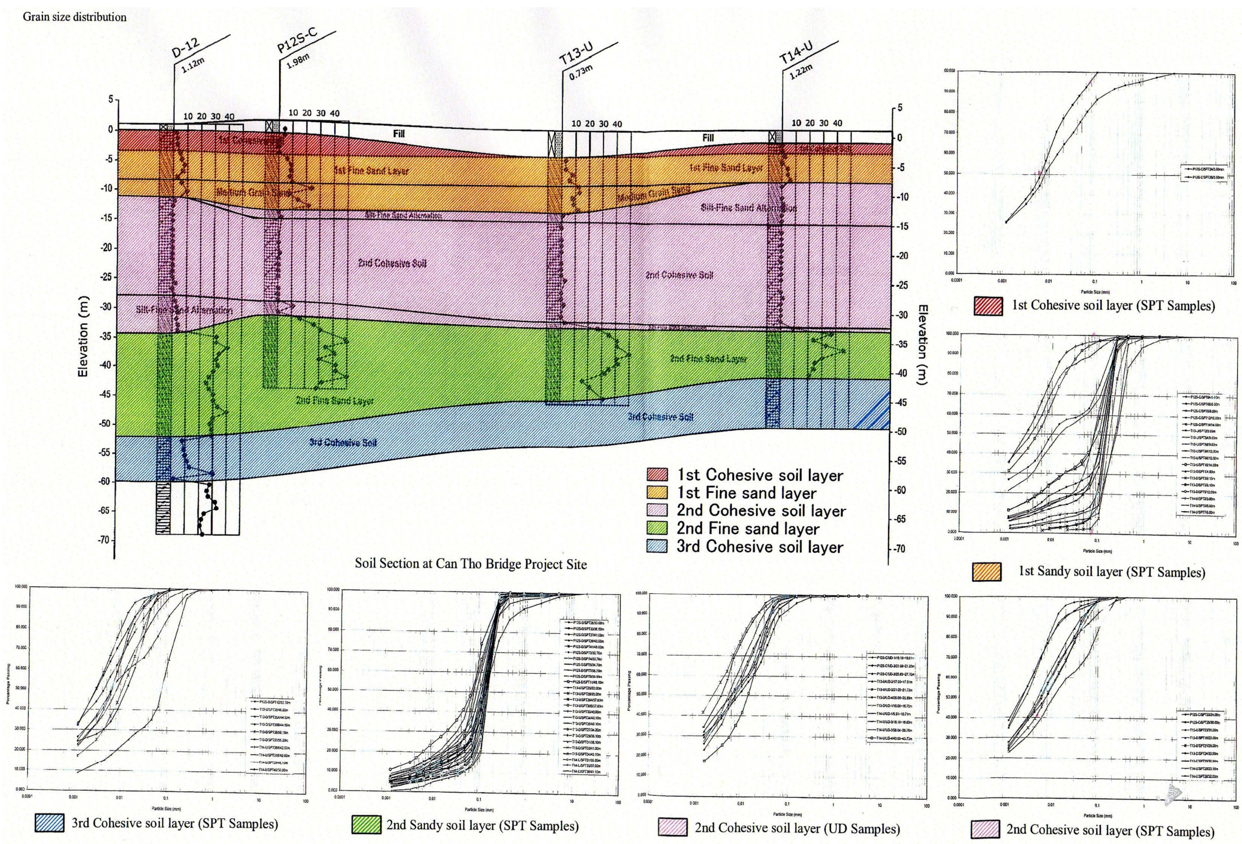

Fig.16 Geology at the site (STP blow account and grain size distribution of each layer)

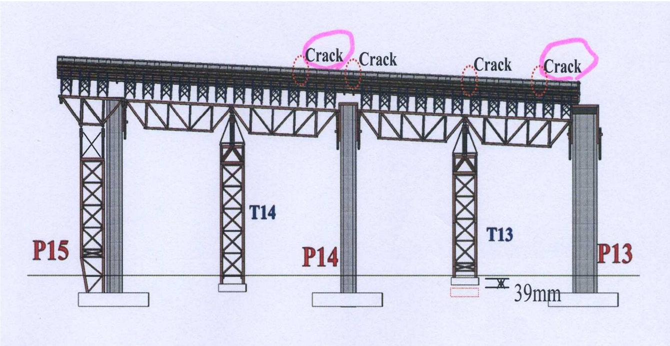

Fig.17 Cracks on slab and settlement of footing

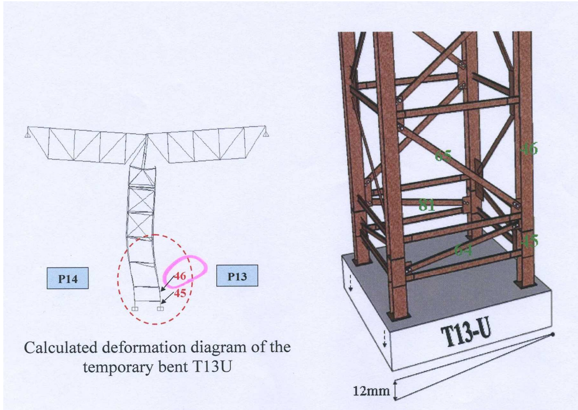

Fig.18 Buckled vertical members and differential settlement of base of bent-13

Fig.19 Can Tho Bridge after completion

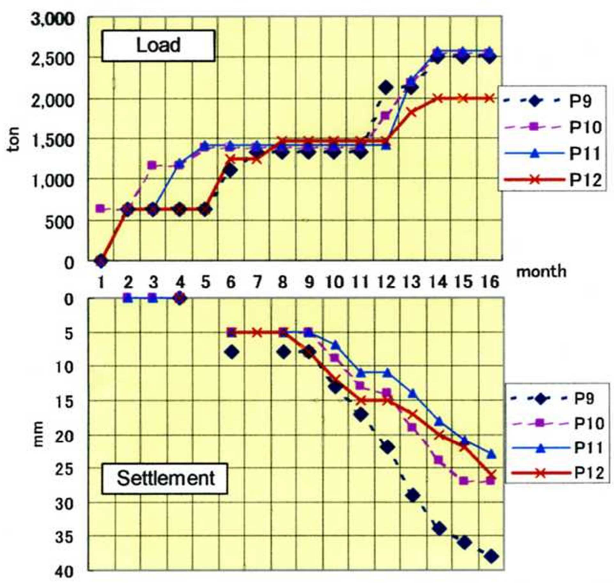

Fig.20 Relationship between load and settlement of viaduct

vertical members of the bent-13. In addition, because of the prestress that is introduced into the girder after the attachment of the main cables, the axial rigidity of the girder with less longitudinal reinforcement than transversal reinforcement is kept relatively small in order to redistribute the reactions on the tops of the bents and the piers.

After the conclusions of the State Committee were issued, a revised construction plan was drawn up and the bridge was beautifully completed without further incident, as shown in Fig.19.

3.4 *Technical problems*

Although the causes of the collapse have been clarified, a method estimating the amount of settlement (one of the main causes of the collapse) that can be expected to occur when constructing a bridge, need to be developed. Such a method would facilitate the development of the rational design for both the superstructure and the substructure that take into account the correlation between load and displacement at a foundation.

The ground at the Mekong Delta consists of very soft sedimentary layers. For a spread foundation, the amount of settlement can be calculated by using the consolidation settlement theory. For deep foundations with a large sustaining capacity such as caisson and pile foundations, the amount of settlement is actually quite small but calculating this is complicated.

Fig.20 shows the curious behavior of pile foundations for which settlements of piers increase under a constant load. Normally, the deformation of a deep foundation is limited to elastic deformation and long-term settlement might damage indeterminate structures. The continuous minute settlement that can be observed at the Mekong Delta is similar to the secondary consolidation or the creep deformation of soft rock.

Given this, a method that takes into account a long-term deformation of foundation as a result of settlement is required to ensure structural safety in the design. In addition, the design for an indeterminate structure with a foundation on a soft sustaining layer needs to provide for an axial rigidity that is strong enough to redistribute the reactions on supports.

4 DESIGN METHOD THAT TAKES INTO ACCOUNT FOUNDATION DEFORMATION ON SOFT GROUND

4.1 *Conventional settlement estimation method*

The settlement of a shallow foundation resting on clayey soil is calculated using Equation (1) below on the basis that it is a consolidation settlement. The measured or calculated settlement results are expressed as a hyperbolic curve in Fig.21.

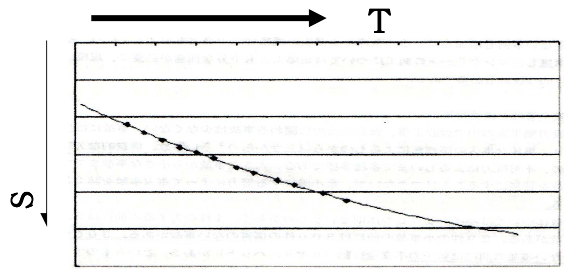

Fig.21 Consolidation settlement curve

$$S = t/(\alpha + \beta t) \tag{1}$$

Where, S : settlement(cm)
t : time(sec)
α, β : coefficient

However, because the hyperbolic curve approaches the asymptote over time and only represents settlement over a limited time scale. Namely, it means that the hyperbolic curve can not reflect the tendency for the settlement of clay to last continuously.

4.2 *New estimation method for settlement*

To estimate the settlement of soft ground, a new method of calculating plastic settlement has been developed that take in account the continuously lasting settlement of a pile foundation in such ground. The development of a new formula for viscoplastic deformation is also a practical objective.

Fig.22 shows a typical Voigt model which is composed of a spring and a dashpot. Equation (2), (3) and (4) below present the formulas used for this model. A linked Voigt models can express complicated settlement dynamics caused by the presence of various different types of sustaining layers (Fig.23).

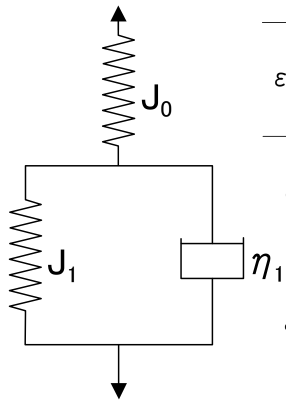

Fig.22 Voigt model

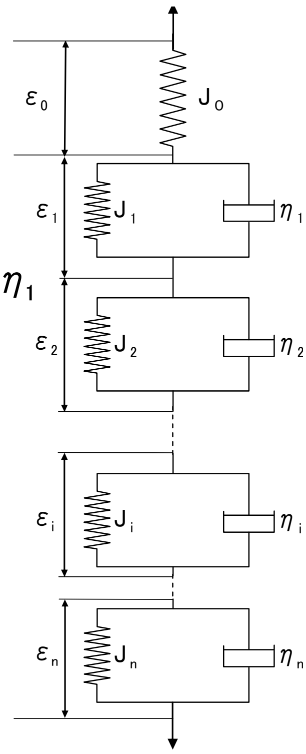

Fig.23 Linked Voigt model

$$\sigma(t) = \sigma_0 = \sigma_{J1}(t) + \sigma_{\eta 1}(t) = \frac{\varepsilon_1(t)}{J_1} + \eta_1 \frac{d\varepsilon_1(t)}{dt} \tag{2}$$

$$\varepsilon(t) = \varepsilon_0 + \varepsilon_1(t) = J_0\sigma_0 + \sigma_0 J_1\left[1 - \exp(\frac{-t}{T_1})\right]$$
$$= \sigma_0\left\{J_0 + J_1\left[1 - \exp(\frac{-t}{T_1})\right]\right\} = \sigma_0 J(t) \tag{3}$$

$$T_1 = J_1\eta_1 \tag{4}$$

Where, σ : stress
ε : strain
J : compliance (inverse of rigidity)
t : time
T : retardation time
J(t) : creep function
η : visco-elastic coefficient

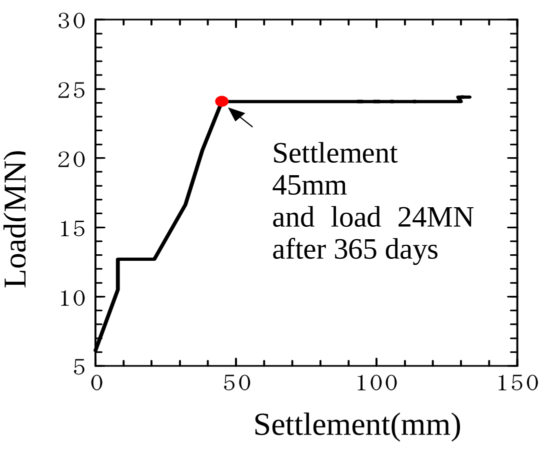

Fig.24 Relationship between load and settlement

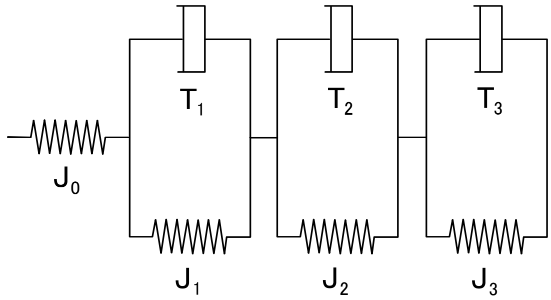

Fig.25 Seven elements model

This concept was applied to the settlement of a pier over a period of 750 days. The load was gradually increased until it was fixed at 24 MN from the 365th day onward as shown in Fig.24. The seven elements model was adopted as the calculation model (Fig.25). Equation 5 below present the calculation formulas used for this model and a series of values for J and T are also provided to simulate the observed settlement.

$$0 \le t \le 750 days$$

$$\varepsilon(t) = \varepsilon_0 + J_1\sigma_0(1-\exp\left[\frac{-t}{T_1}\right])$$

$$+ J_2\sigma_0(1-\exp\left[\frac{-t}{T_2}\right]) + J_3\sigma_0(1-\exp\left[\frac{-t}{T_3}\right])$$

$$= \sigma_0\left\{J_0 + \sum_{i=1}^{n=3} J_i(1-\exp\left[\frac{-t}{T_i}\right])\right\} \quad (5)$$

$$J_0 = 1.8680\frac{mm}{MN}, J_1 = 0.00858\frac{mm}{MN},$$

$$J_2 = 0.01263\frac{mm}{MN}, J_3 = 5.22819\frac{mm}{MN}$$

$$T_1 = 1day, T_2 = 20days, T_3 = 630days$$

The settlement curve generated by Equation (5) can not produce settlement estimates for beyond the first 1000 days because this equation (5) does not include the necessary constants and terms of influence in future (Fig.26). Observation data for after the first 750 days is required. Fig.27 shows the relationship between settlement and time on a semi-logarithmic sheet. This linear relationship can be expressed with Equation (6). Based on data given in Fig.27, a new calculation model, shown in Fig.28, was developed. For this model, a dashpot η * was used instead of a large number of Voigt elements. Equation (7) below presents the formulas used for this model. The curves generated by Equation (6) and (7) almost correspond to those shown in Fig.28, which mean that it is now possible to estimate viscoplastic settlement over the long term.

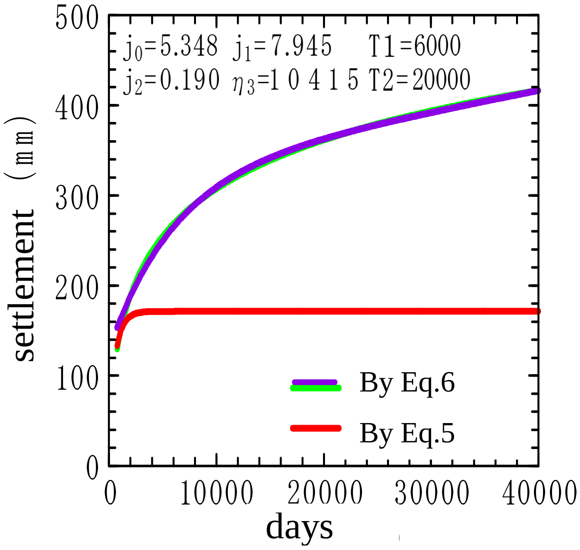

Fig.26 Estimated long term settlement

$$\varepsilon(t)=\sigma_0\cdot j_0+j_1\cdot\sigma_0(1-e^{-t/T1})+j_2\cdot\sigma_0(1-e^{-t/T2})+\sigma_0(1/\eta_3)\cdot t \quad (6)$$

$$750 days \le t$$

$$\varepsilon(t) = \varepsilon_{t=750} + J_1\sigma_0(1-\exp\left[\frac{-t}{T_1}\right])$$

$$+ J_2\sigma_0(1-\exp\left[\frac{-t}{T_2}\right]) + \frac{\sigma_0}{\eta^*}t$$

$$= \sigma_0\left\{\begin{matrix} J_0 + J_1(1-\exp\left[\frac{-t}{T_1}\right]) \\ + J_2(1-\exp\left[\frac{-t}{T_2}\right]) + \frac{t}{\eta^*}\end{matrix}\right\} \quad (7)$$

$$J_0 = 5.34769, J_1 = 7.94507, J_2 = 0.19021,$$

$$\eta^* = 10415.08597$$

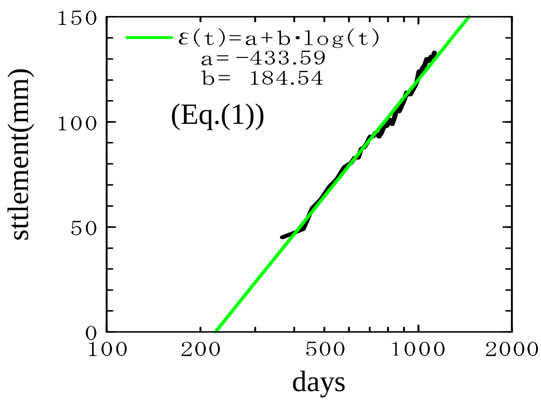

Fig.27 Relationship between settlement and time on semi-logarithmic sheet

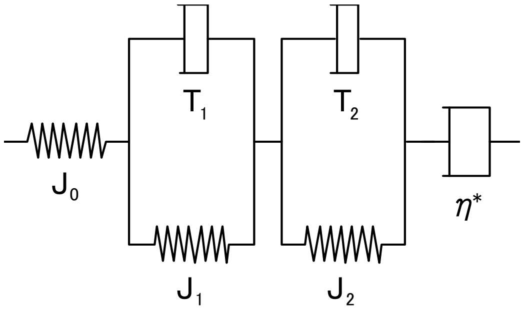

Fig.28 Newly developed model

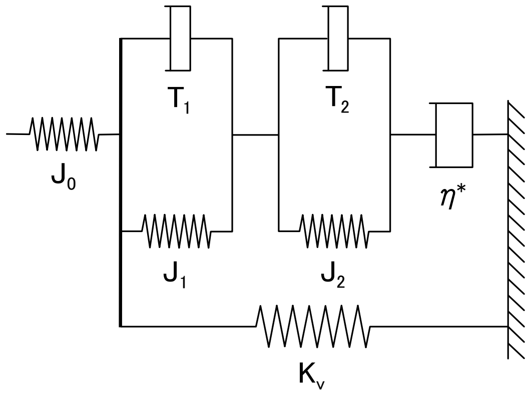

Fig.29 Revised model for enforcing pile foundation by using additional piles

4.3 *Application of new estimation method*

When a structure resting on very soft ground continues to deform continuously even if there has been no increase in the load, the construction work can prove difficult and dangerous.

A study was conducted to evaluate whether the use of additional piles would reinforce the pile foundation of a pier such as the one mentioned above and reduce its long-term settlement. Based on the results of this study, a revised calculation model that uses the axial spring constant for the pile (Kv) was developed shown in Fig.29. Equation (8) below presents the formulas used for this model. The calculation was performed for the following five cases: (1) zero pile, (2) one pile, (3) two piles, (4) three piles and (5) four piles. The results are illustrated in Fig.30. The effect of using additional piles is apparent, meaning that it is possible to use piles with a smaller diameter than that calculated by necessity of the minimum number of piles to take balanced disposition for stability,

$750\,days \le t$

$$\varepsilon(t) = \varepsilon_{t=750} + J_1\sigma_0(1-\exp\left[\frac{-t}{T_1}\right]) + J_2\sigma_0(1-\exp\left[\frac{-t}{T_2}\right]) + \frac{\sigma_0}{\eta^*}t$$

$$= \sigma_0\left\{J_0 + J_1(1-\exp\left[\frac{-t}{T_1}\right]) + J_2(1-\exp\left[\frac{-t}{T_2}\right]) + \frac{t}{\eta^*}\right\} \quad (8)$$

$J_0 = 5.34769, J_1 = 7.94507, J_2 = 0.19021, \eta^* = 10415.08597$

5 CONCLUSIONS

(1) Conformity in the design of a bridge's super-structure and substructure has been realized in Japan by taking into account the correlation between the load and displacement of its foundation.

(2) The static design for a foundation is developed based on the correlation between the foundation's load and its displacement using various spring constants under the resultant force.

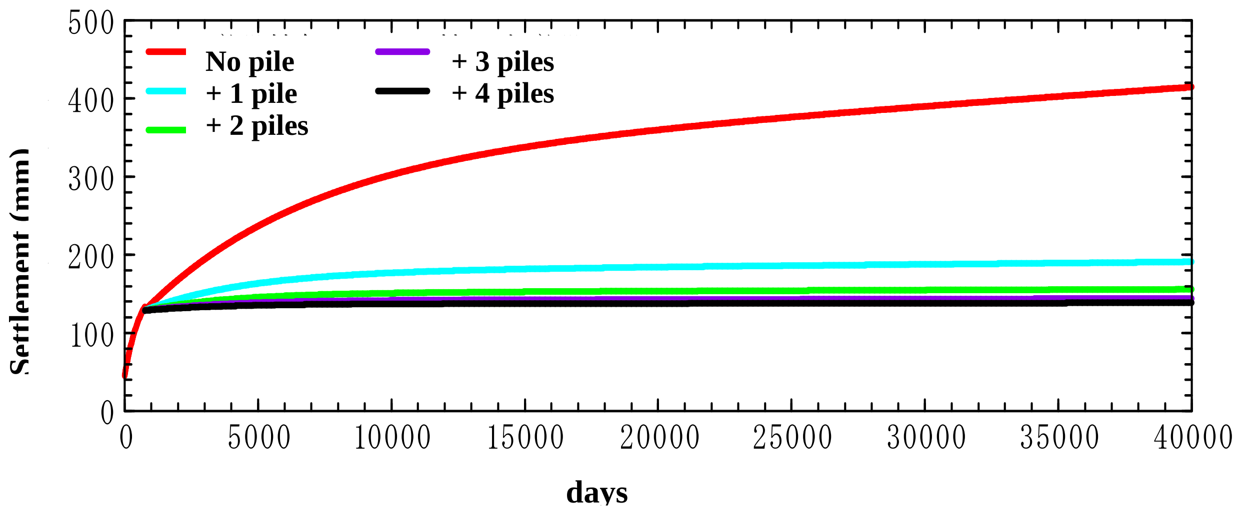

Fig.30 Reduction in settlement by use of additional piles (without live load)

(3) The adoption of common coefficients makes it possible to develop a rational static and dynamic design for the entire structure.

(4) Accidents are apt to occur when constructing indeterminate structures on very soft ground.

(5) A method to estimate the amount of long-term settlement is needed to facilitate the development of a rational design for both the superstructure and substructure that takes into account the correlation between the load and the displacement of a foundation.

(6) The design for indeterminate structures built on a weak foundation needs to provide for an axial rigidity that is strong enough to redistribute the reactions on supports of piers.

(7) The consolidation settlement theory can be applied for shallow foundations, while the linked Voigt model is effective for deep foundation.

(8) The linked Voigt model enables a deep foundation to be reinforced and its settlement to be reduced.

ACKNOWLEDGMENTS

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REFERENCES

Japan Road Association (1996), REFERENCE FOR HIGHWAY BRIDGE DESIGN--- SPECIFICATION FOR HIGHWAY BRIDGES---PART IV: SUBSTRUCTURES

Akagi.T, (1982), Application of generalized rheological model to time-dependent behavior of rock, Proceedings of the 4th International Conference on Numerical Methods in Geomechanics, Edmonton

Okada.Y et al,
doboku.metro.tokyo.jp/start/03-jyouhou/nenpo/17nenpo/... ·
PDF file:<http://doboku.metro.tokyo.jp/start/03-jyouhou/nenpo/17nenpo/1719.pdf>