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SEMINAR

XI

Influence of Soil Behaviour on Structural Design

Influence du comportement des sols sur le dimensionnement des structures

Einfluss des Bodenverhaltens auf die Bemessung von Bauwerken

Co-chairmen:	St. Soretz, Austria A. van Weele, Netherlands
Introductory Papers:	"Einfluss des Bodenverhaltens auf die Bemessung von Bauwerken" C. Veder, Austria "Influence of Soil Behaviour on Structural Design" S. Thorburn, Great Britain "Influence of Soil Behaviour on Structural Design" Y. Yoshimi, Japan
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(The Introductory Papers are published in the Introductory Report of IABSE 11th Congress in Vienna 1980)

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Summary

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Prof. C. V e d e r from Austria introduced the subject of the meeting by illustrating a number of interesting foundation solutions for various large building projects, such as the World Trade Centre in New York, arch-shaped bridges in the Taunus Mountains, Germany, the tower of the Latino Americana in Mexico City with its partly compensated foundation and the new UNO-City in Vienna. Although the subsoil conditions at the location of the last mentioned project are far from unfavourable, settlements have been observed close to 50 mm and the differential settlements amounted to nearly 25 mm. Especially the difference in loading by the high-rise and the low parts of the building have led to differential settlements of 26 mm. The building has behaved as expected and no damage was observed.

Prof. Y. Y o s h i m i emphasized the importance of the soil consultants contribution in the initial stages of a project design. He made his point clear in explaining the reclamation and preparation of the building site for the new Disney Land near Tokyo. The soilconditions below this site are very unfavourable so that piles of over 40 metre length would be required unless the compressible substrata could be improved. The latter solution was choosen and carried out in the mean time, by preloading the surface, in combination with vertical drainage. The time required for this solution, was incorporated right away in the overall planning of the project. The result obtained prooved to be successfull and has led to a considerable saving.

In the seminary seven contributions were presented from different parts of the world. Among them were purely theoretical approaches to the subject under discussion as well as results of field observations of structural behaviour.

Mr. T h o r b u r n of the U.K., who had been asked to present the concluding remarks, congratulated the authors on augmenting our stock of technical knowledge. He wanted to stimulate activities in order that new experiences and idea's can be presented during future conferences by drawing the audience's attention to:

- the real behaviour of structures,
- the real behaviour of soils.

He felt that there is still a lack of detailed information as to how the structural behaviour is influenced by the response of the completed structures to its environment and function.

Studies of structural behaviour over long periods of time are certainly fraught with difficulties (such as malfunction of instrumentation) but the information gathered is so important that further research in this direction must be encouraged.

Regional geological studies and the publication of information on the performance of particular structures present valuable design guides for designers. According to Mr. Thorburn, it is likely that in future greater emphasis will be laid on in-situ testing of soils.

It should be realized that the actual behaviour of a structure is what is to be predicted during the design stage and that as accurately as possible. In order to do so, a great many simplifications in the design considerations, in the soil profile and in the way of load distribution and load transfer has to be made. This is also the case when the help of a computer is obtained. Too often computer analysis is used to give the results a distinction of accuracy which in foundation engineering however hardly exists.

Accuracy is only reserved for the structure itself! But its actual behaviour is the clue to our better judgement of the theoretical models as well as to the better selection of soil parameters. Mr. Thorburn's suggestions are therefore to be taken serious by all of us who attended this interesting specialty session on soil-structure interaction.

XI

Local Unexpected Settlements on the Multy-Storey Structure SAB in Berlin

Tassements locaux imprévus de la superstructure SAB à Berlin

Unerwartete örtliche Setzungen bei der Überbauung SAB in Berlin

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SUMMARY
In West-Berlin for a stretch of about 600 m, the structure of a 15-storey building with two highway tunnels passing through has been completed in summer 1980. Local unexpected settlements required special treatments for the soil and the structure itself. By means of grouting the soil with cement injections the settlements could be stopped and by means of high pressure (soil fraction) a 150 MN heavy part of the structure was lifted. The structural response to the settlements has been treated theoretically and by field measurements.

RESUME
A Berlin-Ouest, l'autoroute a été recouverte sur une longueur de près de 600 m par 7 immeubles d'une hauteur de 15 étages. A la suite de tassements imprévus du terrain pour 2 des 7 immeubles, il a fallu prendre des mesures relatives au sol et à la superstructure. Des injections de ciment ont permis d'arréter les tassements du sol et de lever les immeubles de quelques millimètres. Le comportement de la construction a été étudié théoriquement et également mesuré.

ZUSAMMENFASSUNG
In West-Berlin wurde auf nahezu 600 m Länge die Stadtautobahn mit 7 Wohnblöcken mit Höhen bis zu 15 Stockwerken überbaut. Der Verkehr fliesst in 2 Tunnelröhren darunter hindurch. Unerwartet grosse und sprunghaft eingetretene Setzungen bei zwei von den insgesamt sieben Wohnblöcken erforderten zusätzliche Massnahmen für den Boden und die Überbauung. Durch Zementinjektionen im Boden konnten die Setzungen gestoppt und ein ca. 150 MN schwerer Teilabschnitt der Überbaukonstruktion mit einem Druck von 25 bar um einige Millimeter angehoben werden (soil fraction). Das Bauwerksverhalten unter den extremen Setzungsdifferenzen wurde rechnerisch und durch Messungen am Bauwerk untersucht.

1. General

In spring of 1980 as part of the city-freeway of West-Berlin two 600 meter tunnels were opened for traffic. The unique situation of this first project in Europe is that seven residential blocks, 46 meter high, with 15 levels and more than 1000 apartments rise above the autobahn-structure. The model fig. 1 gives an overall view of the housing project and the tunnel entrance.

This report deals with unexpected settlements in 2 of the 7 apartment-blocks where the absolute and the relative settlements started to increase after the structure itself was almost completed. With special methods, called soil-fraction, the settlements could be stopped while various measurements and treatments had to be given to the structure in order to learn which biggest differential settlements could be allowed without damaging the stiff wall-frames.

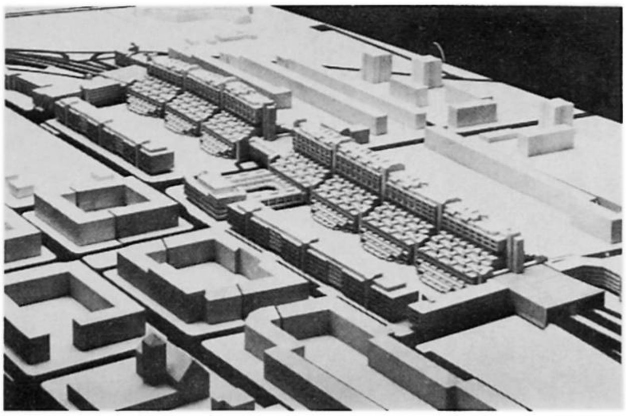

Fig. 1 Model of the SAB-Project

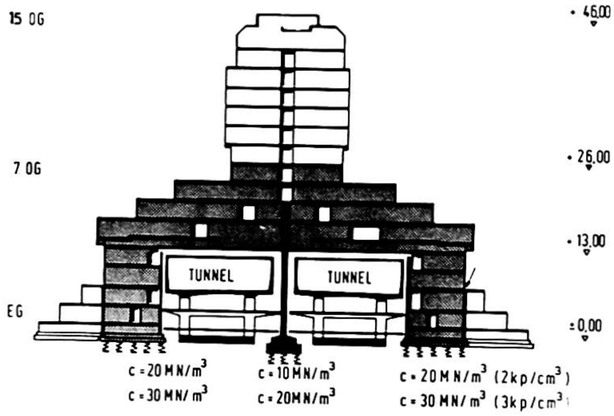

Fig. 2 Cross-section, foundation-moduli

2. Description of the Structure

The structure of the multy-storey apartment complex consists altogether of 77 three legged stiff wall-frames carrying a total load of 35 000 kN. Fig. 2 gives a cross-section of the housing structure. According to soil expertise the foundation mainly consists of sand and partly of loamy soil. The unsuitable soil at the top level was replaced by 1 to 2 m of sand. With this material exchange the soil condition could be represented by foundation moduli using upper and lower bounds over the total length of the structure. The figure also gives a view of the tunnels and the parkdecks.

The governing influence for the stiff wall-frames is the differential settlement of the soil.For estimating actual frame performance and the stresses,three loading conditions were selected and checked by different approaches. The frame, made up of plate-like elements was calculated on a finite element approach. It was also analysed by frame analysis and by linear theory. The influence of openings in walls on the deformation behaviour of the frame was checked by model analysis on a Plexiglass model.

The comparisons showed that the variation in action forces was small - 3 % to 6 %, the most critical forces being tension in the beams and compression in the outer joints of the frame.

Detailed calculation showed that, depending on the structural stiffness in the various stages of construction, the stiffness of the soil has different influences on the distribution of reactions in the frame.

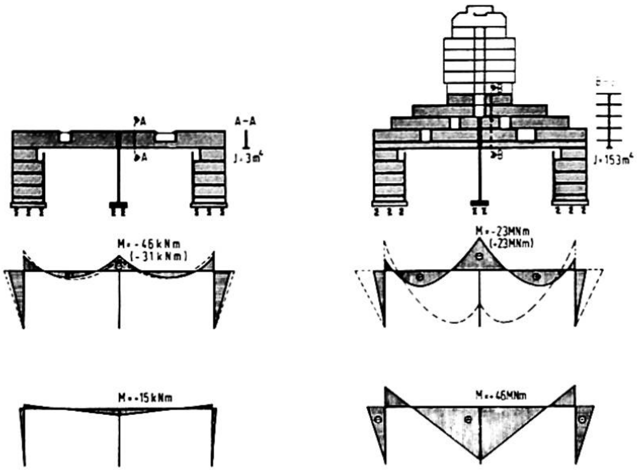

Fig. 3 Influence of elastic supports on frame action forces

Figure 3 shows that in the early stage when the strut only consisted of one storey, moments assuming rigid supports amounted to only 30 % of those assuming elastic supports. However, during the final stage when the structural stiffness of the frame was enlarged by a factor of about 50 there was a tremendously increased influence of differential settlements between the outer and the inner supports as shown on the right hand side of the figure. It may be mentioned that this relation is only valid for a completely elastical behaviour of the structure. [1, 2, 3]

3. Unexpected settlements in 2 blocks in the centre of the structure

In order to compare the design assumptions with the real structural behaviour field measurements were made for this comparison. There was a good accordance between field measurements and calculated values during construction. However, in the winter of 1978 to 1979, when construction was stopped due to cold weather condition, unexpected large non-linear settlements and differential settlements between inner and outer supports required special treatments for the structure.

First it could not be explained why these untypical and non-linear settlements developed. However, when additional deep soil controls down to 20 meters were made, it was found that the soil in this depth was soft and porous and therefore not any longer comparable with the data of the soil expertise given at the beginning of construction. From this a series of questions arose:

- What will be overall settlement and what will the differential settlement be between inner and outer supports?
- What will the reaction of the stiff wall frames due to differential settlements be and what can be done to avoid damage to the structure?
- One of the most important questions was the speed of settlements because the reduction of action forces in the structure due to creep and relaxation is mainly influenced by the speed of the settlement?
- What treatments could be used in order to stop the settlements as fast as possible and which were the possibilities to inverse differential settlements?

4. Treatments for the soil and the structure

Figure 4 shows the soil condition over the whole length of the structure and also the absolute and differential settlements of the 7 blocks. Evident is the untypical settlement in the centre part of the housing structure. Extensive investigations yielded in the decision to treat the soil and structure as follows:

4.1 Treatments for the soil

- Construction of pile walls on both sides of the centre foundation.
- Injection of the non-dense layers with cement mortar in order to reduce settlements.
- Injection of cement mortar under high pressure - called 'soil fraction method' - reducing extreme settlements to permissible values by lifting the foundation. From figure 5 the main steps and the concept of the soil fraction method can be observed.

4.2 Measurements on the structure

- Measurements of concrete deformation in extreme zones of tension and compression.
- Strain measurements on cast-in rebars in zones of high compression.
- Permanent observation of crack formation and crack width variation.

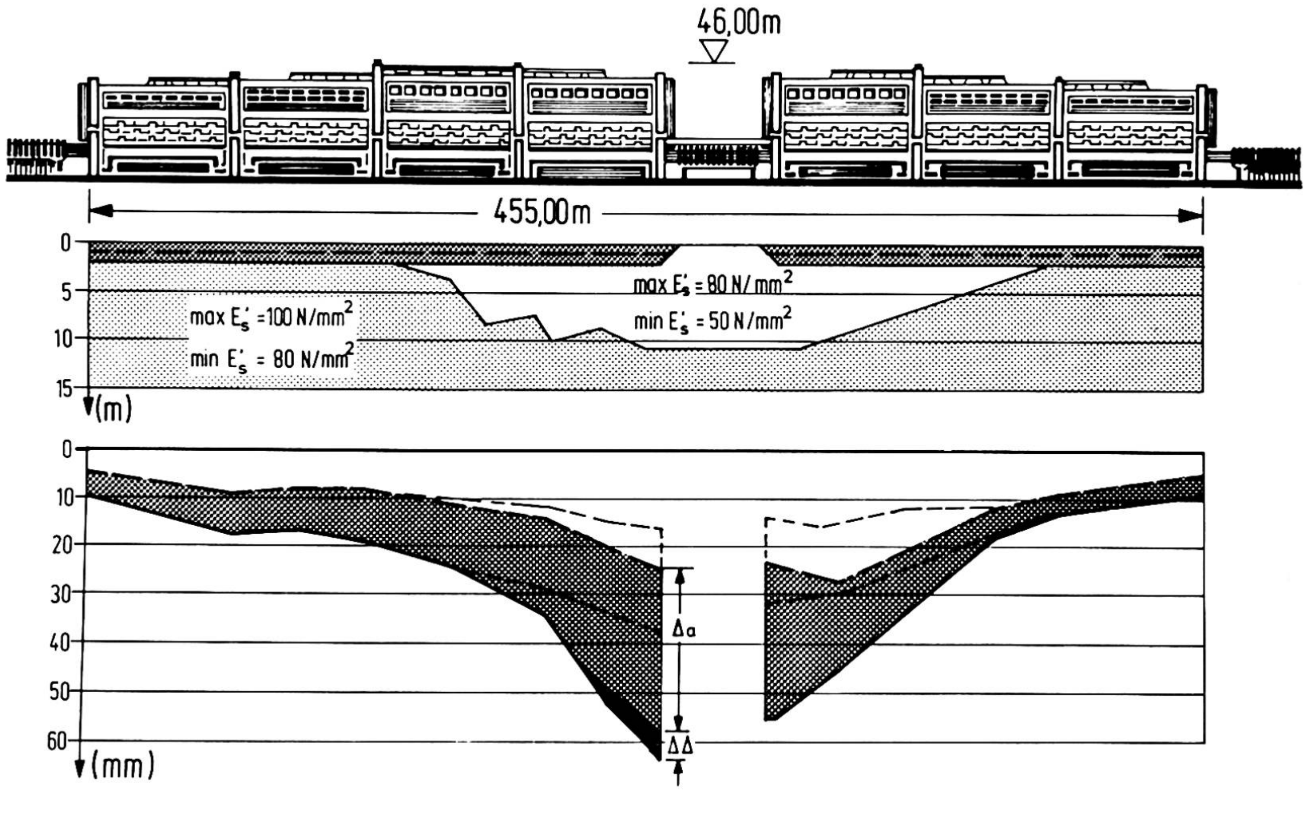

Fig. 4 Soil condition and settlements

4.3 Treatments for the structure

- Laboratory investigations to define the actual concrete tensile and compressive strength, the moduli of elasticity, and the behaviour of the concrete with respect to creep and relaxation. Figure 6 shows the development of restraint actions due to creep and relaxation under sudden and slow settlements assuming various conditions of the structure.[4]
- Theoretical calculations to define the permissible maximum deformation of the stiff wall frames taking into consideration the influence of bending, shear, of the cracked state II, as well as deformations due to creep and the reduction of actions due to relaxation. [5]
- Temporary installation of latteral props in walls to avoid instability.

5. Differential settlements and reactions in the frame

Reinforced concrete walls are generally very sensitive to differential settlements. Figure 7 gives possible responses of the structure to the actual measured settlements. In the uncracked elastic structure only a few millimeters of differential settlement would theoretically unload the centre support completely. The contribution of shear and a cracked state II increases the deformability by more than 200 %. However, the main effect will be gained by creep. Line cc represents the influence of creep established by laboratory tests. The main reason why it can be assumed that the reaction in the structure due to the settlement will follow the line of cc is based on the fact that the speed of the settlement being observed within two years corresponds more or less to the creep capability of the concrete used.

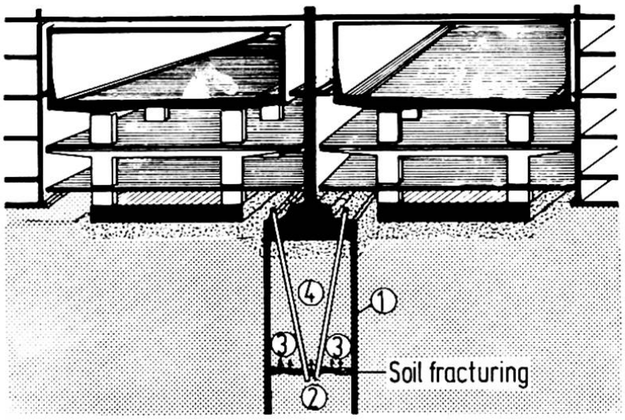

Fig. 5 Cement injection - soil fraction method

It was possible to stop the settlements in the region of the centre foundation with the method of soil fraction. The main question to be

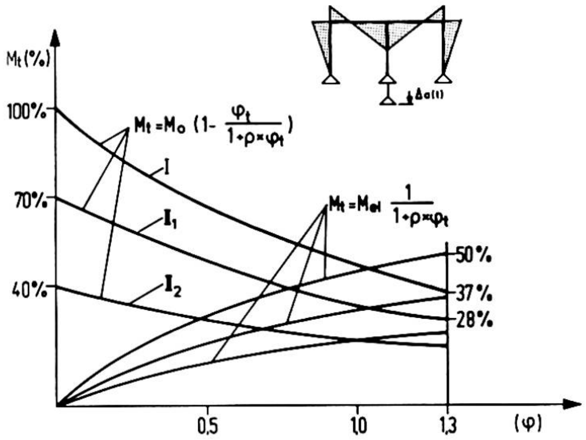

Fig. 6 Restraint forces due to settlements

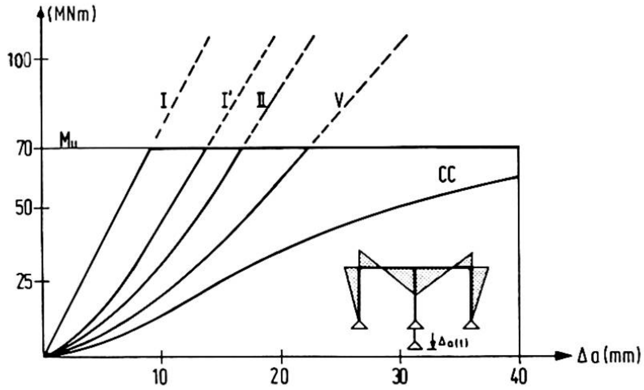

Fig. 7 Structural responses to settlements

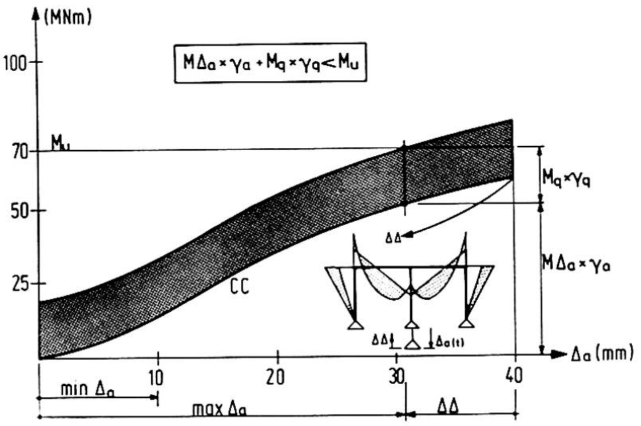

Fig. 8 Action forces to settlements and loads

answered is, how the required safety will be reached. In figure 8 possible residual actions in the structure due to settlements are superimposed onto those of the total load of the structure itself. Using a lower bound assumption of the relation between the reaction due to settlements and the reaction in the structure it seems that the present settlements should be reduced from 40 mm down to about 32 - 35 mm. It will be the task of the following months to investigate the long-time behaviour of the soil during a long obervation period.

6. Outlook

By means of soil fraction, that means injections with cement fluid, the local settlements of the appartment complex could be stopped. With high pressure it was possible to lift the centre foundation by about 3 mm. Further observations with respect to long time effect of the soil and the structural behaviour will result in a final definition of a stabalized settlement configuration.

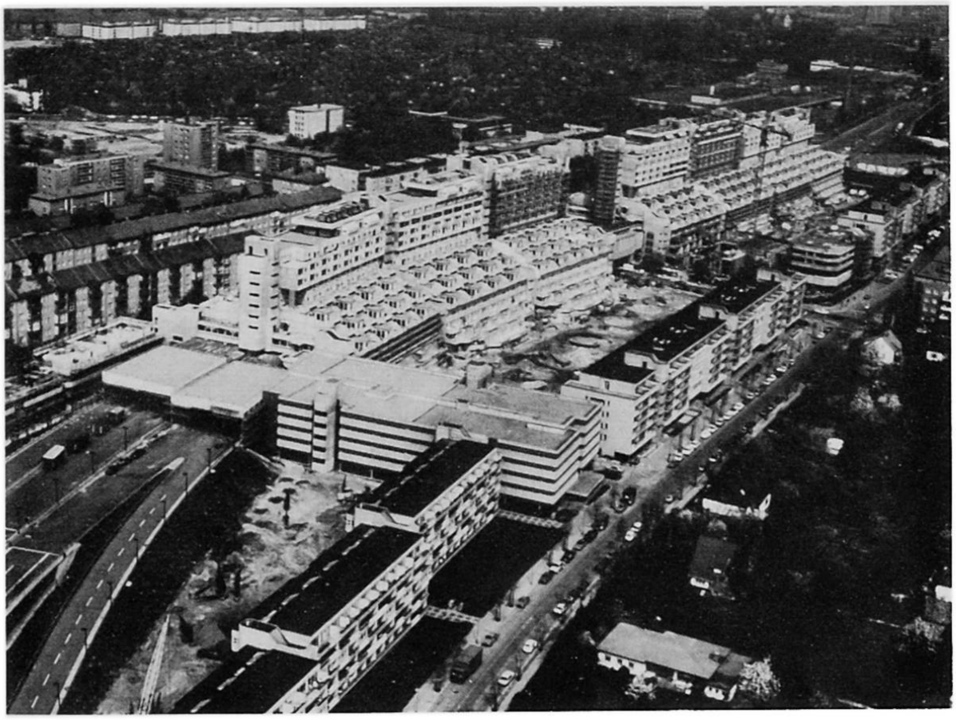

Fig. 9 SAB-Structure in summer 1980

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XI

Interaction between Panel Buildings and the Soil

Influence réciproque de bâtiments en panneaux préfabriqués et du sol

Wechselwirkung zwischen vorfabrizierten Grosstafelgebäuden und Boden

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SUMMARY
Simple discrete models are presented which are suitable for the unified analysis of systems composed of prefabricated large panel buildings and the soil. Another method takes the simultaneous development of the structure and the dead load during construction into consideration.

RESUME
L'auteur présente de simples modèles discrets aptes au calcul intégré de bâtiments en grands panneaux préfabriqués et du sol. Une autre méthode considère l'évolution de la structure et de son poids propre pendant la construction.

ZUSAMMENFASSUNG
Es werden einfache, diskrete Modelle aufgeführt, die für die gekoppelte Berechnung von aus vorgefertigten Grosstafelelementen ausgebildeten Gebäuden und dem Baugrundverhalten geeignet sind. Eine andere Methode zieht das gleichzeitige Entstehen des Tragwerkes und der Baugrundbelastung während der Erstellung des Gebäudes in Betracht.

1. INTRODUCTION

Recently, at the Technical University of Budapest theoretical research has been carried out in connection with the investigation of the interaction between the prefabricated large panel buildings and the soil [1,2]. In this work among others simple discrete models have been developed which are suitable for the unified analysis of the panel structure and the subgrade.

2. THE MODEL OF THE PANEL STRUCTURE

In the analysis of the structure each prefabricated large panel is considered a single rigid element /Fig.1./ These elements are interconnected at the corners and along the edges by elastic springs acting in tension or compression and in shear, respectively. The springs simulate the elastic behaviour of the panels and the joints. Their coefficients for panels with various dimensions and openings should be determined by separate analysis /e.g. by finite element method/ beforehand and are stored in the computer [2].

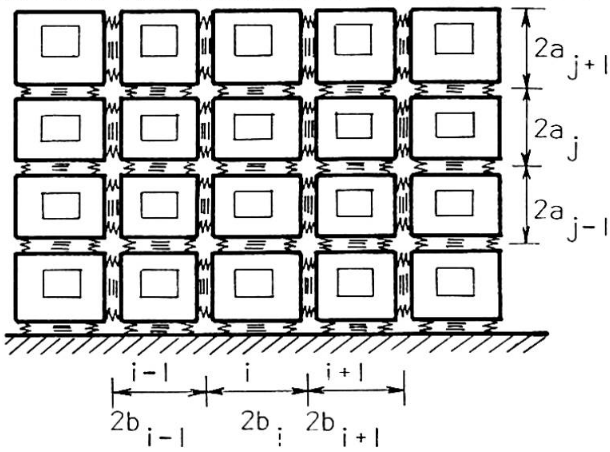

Fig.1. The model of the structure

When analysing a wall of a building /plane problem/ the equilibrium, compatibility and constitutive equations of the panel ij are expressed in the form

$$\begin{bmatrix} \underline{\underline{O}} & \underline{\underline{G}}^{*}_{ij} \\ \underline{\underline{G}}_{ij} & \underline{\underline{F}}_{ij} \end{bmatrix} \begin{bmatrix} \underline{\underline{u}}_{ij} \\ \underline{\underline{s}}_{ij} \end{bmatrix} + \underline{q}_{ij} = \underline{\underline{O}} \qquad /1/$$

or eliminating $\underline{s}_{ij}$

$$\underline{\underline{K}}_{ij}\, \underline{\underline{u}}_{ij} = \underline{\underline{q}}_{ij}\,. \qquad /2/$$

Here $\underline{\underline{G}}^{*}_{ij}$, $\underline{\underline{F}}_{ij}$ and $\underline{\underline{K}}_{ij} = \underline{\underline{G}}^{*}_{ij}\, \underline{\underline{F}}^{-1}_{ij}\, \underline{\underline{G}}_{ij}$ denote the equilibrium, flexibility and stiffness matrices and $\underline{\underline{q}}_{ij}$, $\underline{\underline{s}}_{ij}$ and $\underline{\underline{u}}_{ij}$ are the vectors of the external loads, the spring forces and the displacements of the element in question. The stiffness matrix $\underline{\underline{K}}$ of the whole wall can be derived from the matrices of the individual panels [2] and then from equation

$$\underline{\underline{K}}\, \underline{\underline{u}} = \underline{\underline{q}} \qquad /3/$$

the displacements $\underline{\underline{u}}$ of the elements and the spring forces can be determined. These latter form the basis of dimensioning of the joints.

3. THE MODEL OF THE ELASTIC SUBGRADE /SOIL/

The model of the subgrade consists of prismatical elements which transmit normal and shear stresses on their horizontal and vertical contact surfaces, respectively /Fig.2/a/ [3,4]. A proportion /2α/ of the force acting on the top of an element is transmitted by shear to the two neighbouring elements, while the reminding part /1-2α/ of the force loads the supporting element. Thus, the

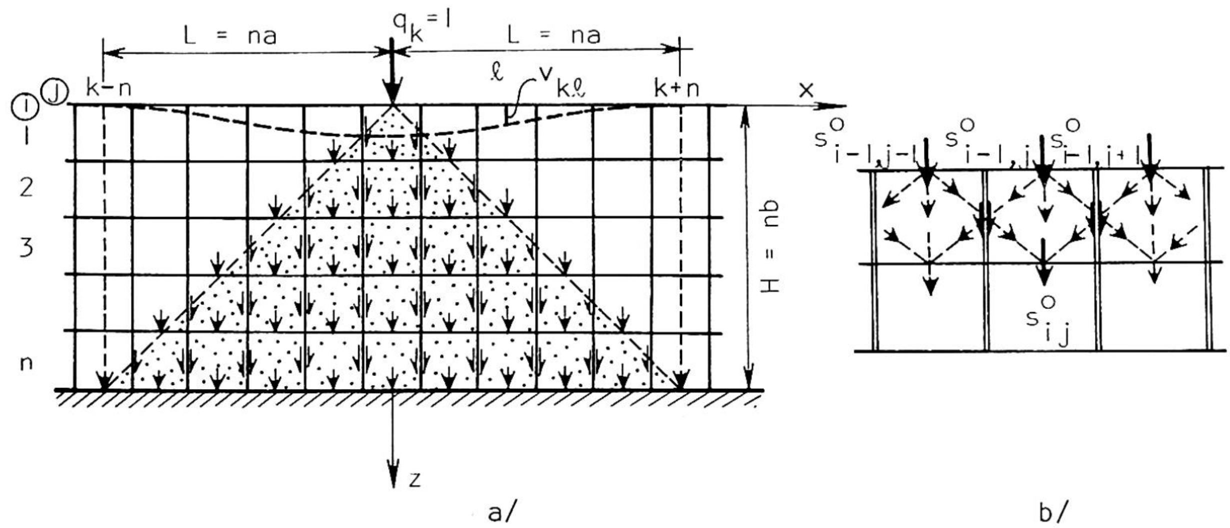

Fig.2. The model of the subgrade

pressure distribution of the whole subgrade caused by a unit force acting on the boundary is defined by the recurrent formula /Fig.2/b/:

$$s^o_{1j} = \delta, \qquad \delta = \begin{cases} 1, & \text{if } j = k \\ 0, & \text{if } j \neq k \end{cases} \tag{/4/}$$

$$s^o_{ij} = (1 - 2\alpha)\, s^o_{i-1,j} + \alpha\left(s^o_{i-1,j-1} + s^o_{i-1,j+1}\right)$$

From these the vertical displacements of the horizontal boundary of the subgrade can be obtained:

$$v_{k\ell} = \frac{1}{k} \sum_{i=1}^{n} s^o_{i\ell} \,. \tag{/5/}$$

These influence coefficients determine the elements of the flexibility matrix $\underline{\underline{F}}_s$ and the stiffness matrix $\underline{\underline{K}}_s = \underline{\underline{F}}_s^{-1}$ of the elastic subgrade. The parameters k and α of the model can be calculated from the material constants of the soil [4].

4. ANALYSIS OF THE SOIL-STRUCTURE INTERACTION

The combination of the stiffness matrices of the structure and the subgrade makes possible the unified analysis of the whole system which gives the internal forces in the joints of the panels and the settlements and stresses of the soil.

According to the experience of the numerical examples the computation time is relatively short and the accuracy of the results checked by the finite element method [7] is statisfactory for practical use.

The models have been extended to the investigation of spacial structures and to the limit analysis of systems composed of panel structures and the soil [5,6]. In this latter method the plastic deformations of the structure are concentrated in the joints and the plastic behaviour of the subgrade is characterized by the linearized Tresca yield condition. The analysis leads to linear programming and makes possible the ivestigation of the extremal limit states of panel buildings caused e.g. by gas explosion, cavitation of the soil and earthquake. The research on stochastical approach of problems described is in progress [8].

Previously, an other method has been developed which besides the interaction of the panel structure and the soil takes the effect of the simultaneous development of the structure and the dead load during construction into consideration [1]. Because of this latter phenomenon only the lower part of the building plays a dominant role in the interaction with the foundation and the soil /Fig.3/a/.

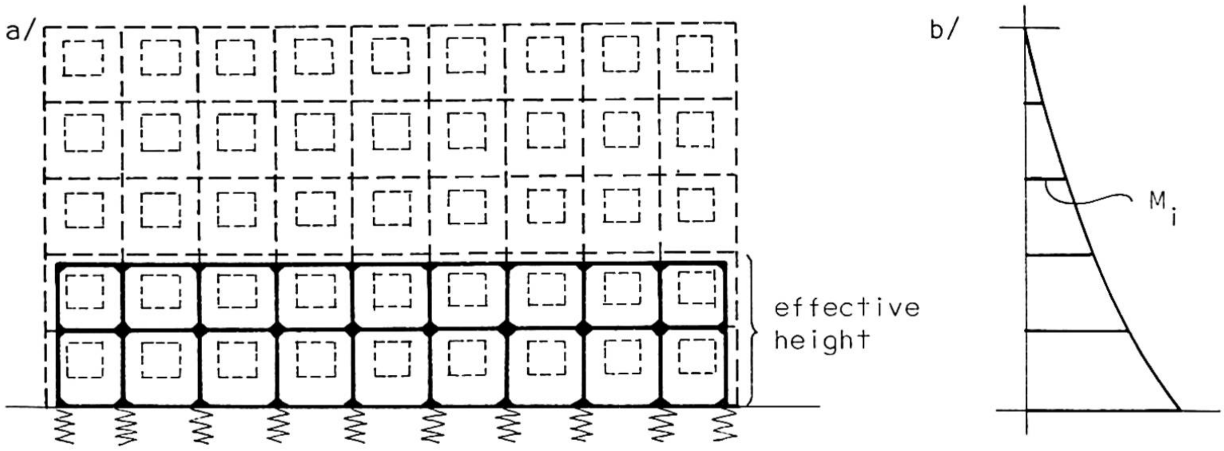

Fig.3. a/ Frame model, b/ bending moment distribution of the lintels

The height of the effective part depends on rigidity conditions and is given in diagrams. The analysis is based on the frame model for the structure and on the Ohde-method for the subgrade. At last, in each vertical section the sum of the beams obtained by these models should be devided among the lintels of the original building. According to the numerical experiences this distribution can be approximated by a second order parabola /Fig.3/b/.

The method described is suitable for the rapid estimation of the effect of soil-structure interaction on panel buildings.

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XI

Dynamic Response of Chimneys Interacting with Soil

Réponse dynamique d'une cheminée et du sol de fondation

Dynamisches Verhalten eines Schornsteines bei Zusammenwirken mit dem Baugrund

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SUMMARY
In the present paper, some results obtained by the authors are presented concerning dynamic soil-structure interaction in wind and earthquake analysis of slender chimneys. In particular, the influence of the soil flexibility is taken into account for both alongwind and crosswind response, and the characteristics of the earthquake response are discussed in general terms.

RESUME
La communication présente les résultats de l'analyse d'interaction sol-structure de hautes cheminées pour les actions du vent et des tremblements de terre. En particulier, l'influence de la flexibilité du sol a été prise en compte, soit pour la réponse dans le plan du vent soit pour la réponse dans le plan normal. Les caractéristiques générales de la réponse au séisme ont été également considérées.

ZUSAMMENFASSUNG
Der vorliegende Artikel behandelt das Problem der Wechselwirkung des Bodens und eines schlanken Schornsteins unter dynamischen Belastungen wie Wind und Erdbeben. Speziell wird auf den Einfluss der Bodenflexibilität bei Längs- und Seitenwind eingegangen. Das dynamische Verhalten bei Erdbebenerregung wird kurz erörtert.

1. INTRODUCTION

The recent tendency of realizing more and more slender chimneys, is due to the need of ejecting the smokes in the higher atmosphere, so that the smoke dispersion and the ash fall-out will take place as far as possible from the source. The reason of this need is based on the fact that industrial plants are often poles of aggregation of minor activities and it is therefore desirable to prevent pollution in the neighbouring areas.
By the other hand, progressing design and construction technologies allow very light structural solutions thus making dynamic analyses more and more important. The Authors did some research activities in this field, dealing especially with soil-structure interaction effects when wind and earthquake loads are of concern. The present paper can be regarded as a synthesis of such researches and also contains some additional considerations.

2. SOIL-STRUCTURE INTERACTION EFFECTS

In the context of dynamic analyses of slender structures, the presence of the soil is usually taken into account by applying, at the base of the structure itself, frequency dependent springs and dash-pots. This idealization is based, of course, upon the assumption that the structures have direct foundations; that is, the effects of moderately flexible soils only can be modelled in this way. The stiffness and damping values can be computed in the frequency domain according to a variety of techniques. Analythical methods are available, indeed, for rigid circular or elliptical footings, embedded or not, resting on half-spaces or strata, of a homogeneous linearly visco-elastic nature. Some of the most popular solutions are available in Refs [1-3] .
Semianalytical procedures can be derived, for instance, from Ref. [4], in which, by means of one and two-dimensional Fourier transforms, combined with a transfer matrices solution of the wave propagation problem [5], stiffness and damping functions can be obtained for strip or rectangular rigid surface footings, respectively, on layered soils.
Rectangular foundations can be treated, under the same conditions, by an ad hoc finite element technique [6].
It should be pointed out that the two latter methods can also be applied to the stiffness analysis of foundation systems, composed by two or more footings of the same kind [7].
More general cases can obviously be treated by means of general pourpose finite element procedures.
Several alternative techniques are also available to analyze the structural behaviour, including soil flexibility.
Continuum and discrete approaches can be used to this pourpose both in time and in frequency domain.
Some more comments are due to the extension of modal analysis to soil-structure interaction problems because, as already mentioned, the foundation stiffness and damping coefficients are frequency dependent and therefore the soil-foundation--structure system does not possess classical normal modes. Indeed, it has been shown in Refs. [8,9] that modal superposition can still be used giving rise to rather small errors but relevant simplifications.
Substantially the presence of the soil induces the following three main effects:

a) rigid-body motions are added to the state of displacement of the structure
b) the natural frequencies of the coupled system are different from the ones belonging to the structure alone
c) the energy dissipation in the coupled system is the sum of the energy dissipation within the structure plus the energy dissipation in the soil due to the mechanisms of hysteresis and radiation.

The consequences on the dynamic behaviour of the structure can be different with respect to the kinds of external loads; in the next paragraphs these consequences will be discussed in more detail.

3. WIND LOADS

It is well known that, when the wind is acting on a chimney, the resultant force can be split in two components: a drag force, in the plane of the wind, and a lift force, in the normal plane. Usually, the dynamic analysis is performed separately for the two kinds of forces and the corresponding results are termed alongwind response and crosswind response, respectively.
This traditional approach is justified by the fact that the strongest vibrations in the wind plane and in the cross plane are due to winds of different characteristics. The former ones are consequence of strong gusty winds while the second ones are produced by winds characterized by moderate velocities.

3.1 Alongwind Response

A general formulation of the problem of the dynamic alongwind response of structural systems including soil flexibility is given in Ref.[10]. Essentially, the theoretical formulation can be summarized as follows.
When the wind is idealized as a stationary Gaussian random process, the alongwind displacement, at height z, may be written as:

$$Y(z) = \overline{Y}(z) + g_y(z)\ \sigma_y(z) \tag{1}$$

where $\overline{Y}(z)$ is the mean displacement, $g_y(z)$ is the peak factor and $\sigma_y(z)$ is the root mean square of the fluctuating displacement, expressed by the integral over the frequency domain of the spectral density of the displacement S_y:

$$\sigma_y^{\,2}(z) = \int_0^\infty S_y(z;n)\ dn \tag{2}$$

In Ref.[10], by utilizing modal analysis, it was shown that, also when soil-structure interaction is to be taken into account, the contribution of the higher modes than the first can be neglected as it is already usual in the analysis of clamped structures.
In Ref.[11] it was also proved that a linear shape can be assumed for the first mode still getting good approximations in the analysis of soil-structure systems.
By the above considerations, together with the hypothesis of small damping, eq. (2) can be simplified and takes at the top of the structure, that is at the height H, the following form:

$$\sigma_y^{\,2}(H) = \frac{1}{(2\pi n_1)^4\ m_1^{\,2}}\left\{\int_0^\infty S_p(n)\ dn + \frac{\pi n_1}{4\xi(n_1)}\ S_p(n_1)\right\} \tag{3}$$

in which n_1 is the first natural frequency; m_1 is the first modal mass; S_p is the spectral density of the alongwind excitation; ξ is the damping computed at the frequency n_1.
Simple expressions to calculate n_1 and $\xi(n_1)$ are contained in Ref.[11].
In Ref.[10] it was shown that the key parameter for understanding the correct influence of soil flexibility is the ratio between the energy dissipated internally in the structure and the energy dissipated in the soil by both radiation and hysteretic damping. More precisely, when the former energy is prevailing on the latter energy, as in the case of large structural damping, the amplification of the response due to rigid-body motion and to the decrease of n_1, is prevailing on the damping effects. In all the other cases, these effects are the most relevant.
The first situation is typical of reinforced concrete chimneys, while the second one is typical of unlined welded steel stacks.
Within the practical limits of direct foundations, it should be observed that in absence of large rigid-body motions, the variation of the fundamental frequency does not originate significant effects on the response.
By the other hand, when the structural damping is small, also relatively good soils can give rise to an energy dissipation which can be very large. This does not mean, however, that the structural response will be significantly modified as the resonant part only of the response (second term in brackets in eq.(3)) will be affected by such a large soil damping.
For instance, being 0.2% a typical value of the hysteretic damping of a 100 m height unlined welded steel stack and assuming a 5% hysteretic damping in a soil with G=1000 daN/cm^2, it will be found that the equivalent damping is 0.43% while the reduction in the top displacement will be about 10%.

3.2 Crosswind response

It is well known that a bluff chimney sheds alternating vortices whose primary frequency n_s is, according to the Strohual relation:

$$\frac{n_s D}{V} = \mathcal{S} \tag{4}$$

in which the Strohual number $\mathcal{S}$ depends on the Reynolds number, while D is the diameter of the chimney and V is the velocity of the wind.
Under the action of vortex shedding the structure oscillates in a plane normal to the wind direction; in particular, when the frequency n_s approaches the fundamental frequency of the chimney n_1, a typical resonant excitation arises. In this case the wind velocity V_c is defined critical velocity:

$$V_c = \frac{n_1 D}{\mathcal{S}} \tag{5}$$

It is easy and immediate to observe that, when taking the soil flexibility into account, the decrease of the fundamental frequency n_1 determines a decrease of V_c, leading to a reduction of the dynamic external loads.
It is well known that the key parameter of this dynamic behaviour is the damping. When damping is large, which is the case of reinforced concrete chimneys, it is possible to notice that the crosswind response of the structure is a random

process, and that the displacement is proportional to $\xi^{-1/2}$.
This kind of behaviour can be correctly estimated by utilizing a prediction procedure based on a random excitation model. Then the maximum displacement in the crosswind direction may be expressed in the usual form

$$X(z) = g_x(z)\ \sigma_x(z) \tag{6}$$

A general formulation to predict the standard deviation σ_x is contained in [12]. However, by introducing the same hypotheses already used in the evaluation of the alongwind response, the top value of σ_x may be expressed in the following form:

$$\sigma_x^2(H) = \frac{1}{(2\pi n_1)^4\ m_1^2}\ \frac{\pi n_1}{4\xi(n_1)}\ S_c(n_1) \tag{7}$$

S_c being the spectral density of wake excitation.
By the other hand, at low values of damping, which are typical of unlined welded steel stacks, displacement dependent lock-in excitation become significant, giving rise to a large increase in cross wind excitation forces. In other words the increase in crosswind response causes an essential interdipendence between the crosswind excitation and the response process. In this case two main effects can be observed: the crosswind forces become well-correlated along the chimney, while the response becomes proportional to ξ^{-1} ; consequently a sinusoidal excitation model must replace the random model to predict the response.
Giving to the lift forces the following expression:

$$F_L(z,t) = C_L\ \frac{\rho V_c^2 D}{2}\ \sin 2\pi n_1 t \tag{8}$$

the maximum crosswind top displacement becomes:

$$X(H) = \frac{C_L \rho D^3 H}{2\xi(n_1)\ m_1\ (4\pi \mathscr{S})^2} \tag{9}$$

in which C_L is the lift coefficient and ρ the air density.
In conclusion it should be pointed out that the crosswind response is completely controlled by the amount of damping present in the system.
From eqs. (7) and (9) indeed it can be seen that the resonant term is the only one appearing in the expression of the response.
In the same example mentioned while analyzing the alongwind response, the same damping increase will now produce a 50% reduction in the top displacement.

4. EARTHQUAKE LOADS

The major difference between earthquake and wind analysis is that in the former one also modes higher than the first are to be taken into account. Indeed the first natural frequency of such structures is generally very low and subsequent frequencies only fall into the most significant range of frequency content of an earthquake. Due to this fact the influence of soil flexibility has different

implications as in the wind case.
First of all, if the first natural frequency is not greatly modified by the presence of the soil, higher natural frequencies can differ very significantly from the clamped situation. For instance, still referring to the already mentioned example, the third natural frequency is reduced by a 20%.
Secundly the variation of the system damping associated with higher modes may be essential but quite impredictable by simple formulas.
In conclusion, it can be affirmed that soil-structure interaction effects are important for a realistic evaluation of the dynamic structural behaviour, although they cannot be described and categorized as simply as in the wind case.
However, a phenomenon which is significant in earthquake excitation and which should be mentioned is the following. Let us consider the presence of a large heavy building in the neighborouds of a chimney. As shown in Ref. [13] this condition can significatly alter the resonant amplification of the chimney itself. For values of the structural parameters typical of conventional power plants the second natural frequency of the chimney is very close to the first frequency of the other building, thus generating a very high mutual interaction effect.

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XI

Déformabilité des appuis et de leur fondation dans le calcul des structures

Berücksichtigung der Verformbarkeit der Auflager und deren Setzung bei der Bemessung von Tragwerken

The Flexibility of Supports and Foundations in the Design of Structures

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RESUME
La déformabilité des fondations et des appuis intervient dans le comportement d'un grand nombre de structures. La présente communication rappelle les méthodes permettant d'évaluer la déformabilité des fondations, et établit les formules donnant les coefficients de souplesse en tête d'un appui quelconque. Elle traite en particulier le cas des appuis formés de voiles souples ou de lignes d'appuis dédoublés en néoprène.

ZUSAMMENFASSUNG
Die Verformbarkeit von Auflagern und Setzungen des Baugrundes wirken auf das Verhalten zahlreicher Bauwerke ein. Der vorliegende Beitrag untersucht dieses Problem und stellt Formeln auf, welche die Verformungsbeiwerte für irgend ein Auflager wiedergeben. Dieser Beitrag behandelt besonders den Fall von biegsamen Stützwänden und von Neopren-Linienlagern.

SUMMARY
The flexibility of supports and foundations has an effect on the behaviour of a great number of structures. The present communication presents in detail the methods giving the foundation flexibility, and establishes the formulas giving the flexibility matrix at the top of any support. It deals with the case of piers made of two thin walls, or of two lines of neopren shores.

1. INTRODUCTION

Les *appuis* sont les éléments d'une structure destinés à transmettre les charges appliquées au terrain sur lequel elle repose. Ce sont, par exemple, les piles ou les culées d'un pont, les poteaux ou les murs-porteurs d'un bâtiment.

Un appui se compose essentiellement de trois parties :

– une *fondation* (semelles, caissons, pieux, etc.),
– le *corps de l'appui,*
– éventuellement des *appareils d'appui.*

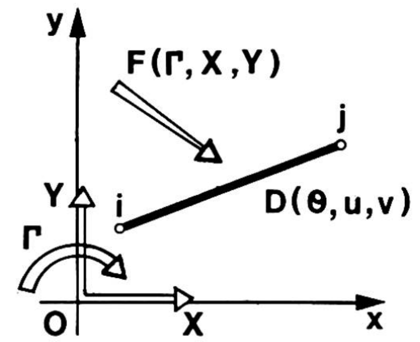
L'étude d'une structure nécessite de tenir compte de la *déformabilité des appuis*, qui interviennent dans la répartition des efforts verticaux et horizontaux mais participent aussi à la résistance à la flexion des éléments porteurs principaux (tablier ou plancher), quand ils en sont solidaires mécaniquement (portiques).

La *déformabilité* des appuis se traduit par les équations linéaires reliant les déplacements en tête de l'appui aux efforts qui lui sont appliqués. Elle est caractérisée par la *matrice d'élasticité de l'appui.*

2. EXPRESSION DE LA MATRICE D'ELASTICITE D'UN APPUI

2.1. - Expression générale

Les composantes θ, u, v du déplacement D de l'extrémité j d'un appui rectiligne quelconque, de fibre moyenne ij, soumis en j à un système de forces F, dont les éléments de réduction par rapport aux axes o x y sont un moment résultant Γ par rapport à O et une résultante générale de composantes X et Y suivant Ox et Oy, sont données par (fig. 1) :



(Fig. 1)

$$\theta = S_{11}\ \Gamma + S_{12}\ X + S_{13}\ Y$$

$$u = S_{21}\ \Gamma + S_{22}\ X + S_{23}\ Y$$

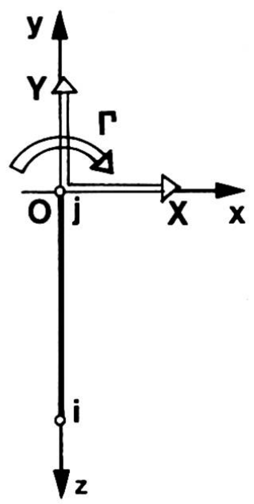
$$v = S_{31}\ \Gamma + S_{32}\ X + S_{33}\ Y$$

ou sous forme matricielle, par : (D) = (K) (F)

La matrice carrée (K) est la matrice d'élasticité de l'appui. Elle est symétrique :

$$S_{12} = S_{21} \qquad S_{23} = S_{32} \qquad S_{31} = S_{13}$$

2.2. - Matrice d'élasticité d'un appui vertical



(Fig. 2)

Un appui est le plus souvent vertical. Si l'appui i j est orienté suivant l'axe o y, l'origine O étant confondue avec l'extrémité j, sa matrice d'élasticité se réduit alors à (fig. 2) :

$$(K) = \begin{pmatrix} S_{11} & S_{12} & 0 \\ S_{21} & S_{22} & 0 \\ 0 & 0 & S_{33} \end{pmatrix}$$

Les coefficients S_{11}, S_{12}, S_{22} et S_{33} sont généralement appelés :

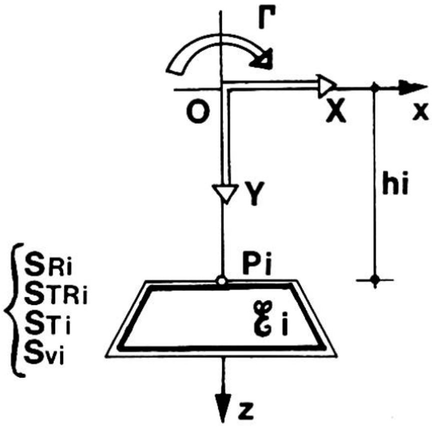
– *souplesse de rotation* pour S_{11} et notée S_R
– *souplesse de translation* pour S_{22} et notée S_T
– *souplesse de translation-rotation* (ou souplesse croisée) pour $S_{12} = S_{21}$ et notée S_{TR}
– *souplesse verticale* pour S_{33} et notée S_V.

Dans la suite de cet exposé nous compterons les composantes verticales Y et v suivant l'axe Oz, dirigé de j vers i.

Le déplacement D d'un appui s'obtient par superposition des déplacements relatifs aux différents éléments qui le constituent. Si l'appui comporte plusieurs éléments $\mathcal{E}_i$ dont les coefficients de la matrice d'élasticité, calculée en un point P_i situé à la distance h_i du point d'application O des efforts Γ, X, Y, sont S_{Ri}, S_{Ti}, S_{TRi} et S_{Vi}, les coefficients de la matrice d'élasticité de l'ensemble de l'appui sont (fig. 3) :

$$S_R = \sum^{i} S_{Ri} \qquad S_{TR} = \sum^{i} S_{TRi} + \sum^{i} S_{Ri}\ hi$$

$$S_T = \sum^{i} S_{Ti} + 2 \sum^{i} S_{TRi}\ hi + \sum^{i} S_{Ri}\ h_i^{2} \qquad S_V = \sum^{i} S_{Vi}$$



(Fig.3)

Par conséquent, à partir des valeurs des coefficients de la matrice d'élasticité de la *fondation*, du *corps d'appui* et des *appareils d'appui*, on peut obtenir les coefficients de la matrice d'élasticité de *l'appui* en un point quelconque, par exemple au niveau G de la fibre neutre de la structure.

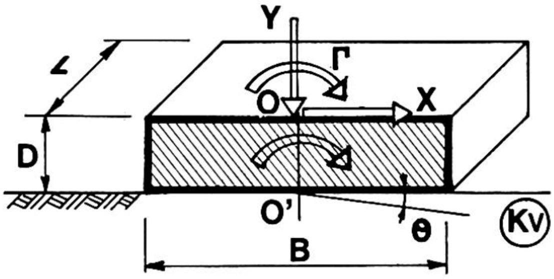
3. INFLUENCE DE L'INTERACTION SOL-APPUI

Le comportement du *sol*, qui intervient de façon importante dans la déformabilité de l'appui, dépend de la nature de la *fondation.*

3.1. - Semelles superficielles

La rotation d'une semelle superficielle, supposée indéformable, de section S (S = B x L) sous l'effet d'un couple C, appliqué en O', est égale à (fig. 4) :

$$\theta = \frac{c}{K_v\ I_f}$$



(Fig. 4)

K_v étant le module de réaction verticale du sol de fondation et I_f le moment d'inertie de la section d'appui de la semelle sur le sol : ($I_f = \frac{B^3\ L}{12}$). K_V peut être obtenu à partir des essais pressiométriques (1).

Les coefficients de la matrice d'élasticité, calculée en O, au niveau supérieur de la semelle, sont :

$$S_R = \frac{1}{K_V\ I_f} \qquad S_{TR} = \frac{D}{K_V\ I_f} \qquad S_T = \frac{D^2}{K_V\ I_f} \qquad S_V = \frac{1}{K_V\, S}$$

3.2. Fondations profondes massives

C'est le cas d'une fondation constituée d'un massif en béton enterré, d'un caisson havé à l'air libre ou à l'air comprimé ou d'une colonne de grande dimension. La largeur B de la fondation est importante vis-à-vis de sa profondeur D. Sa section est S = B x L.

3.2.1. - Méthode simplifiée

On peut supposer que la fondation, considérée comme indéformable, pivote autour du milieu O' de sa base (fig. 5). Sa rotation, sous l'effet d'un couple C appliqué en O' est alors égale à :

$$\theta = \frac{C}{K_V\ I_f + K_H\ I_\ell}$$

avec :

- K_V et K_H les modules de réaction vertical et horizontal du sol
- I_f le moment d'inertie de la section d'appui de la fondation sur le sol
- I_ℓ le moment d'inertie de la surface de contact latérale de la fondation avec le sol.

(1) $$K_V = \frac{9\,E}{\alpha\, B\, \lambda' + 1{,}5\,(1+\nu)\, Bo\, (\lambda \frac{Bo}{B})^{\alpha}}$$

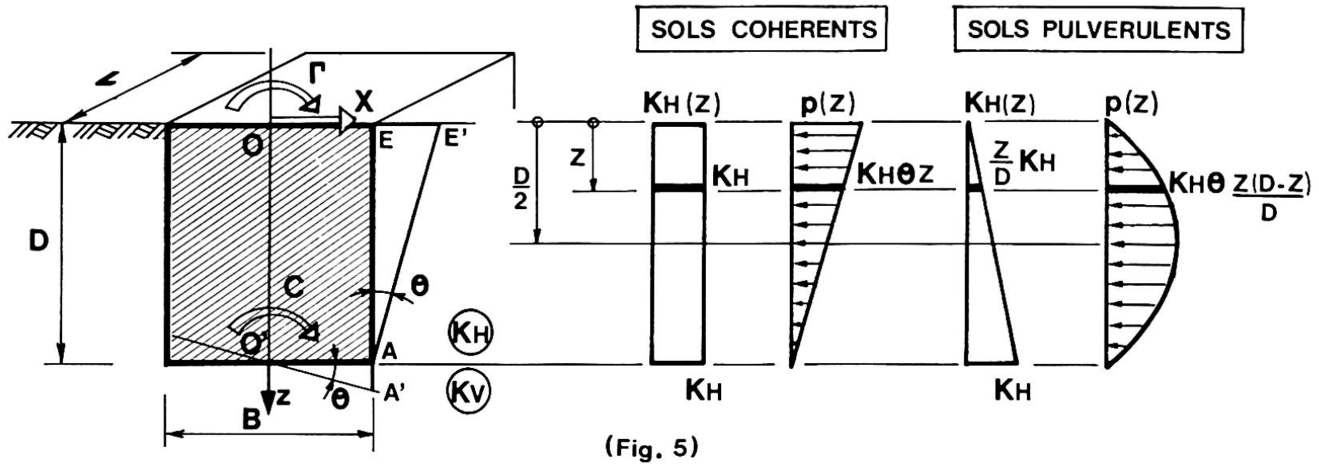
Si on admet que le module de réaction horizontal K_H est constant sur la profondeur D (sols cohérents)

$$I_\ell = \frac{D^3 L}{3}$$

Si au contraire ${}_H$ varie linéairement avec la profondeur (sols pulvérulents), $I_\ell = \dfrac{D^3 L}{12}$

Les coefficients de la matrice d'élasticité de la fondation, calculée en O, au niveau supérieur de la fondation, sont alors :

$$S_R = \frac{1}{K_V I_f + K_H I_\ell} \qquad S_{TR} = \frac{D}{K_V I_f + K_H I_\ell} \qquad S_T = \frac{D^2}{K_V I_f + K_H I_\ell} \qquad S_V = \frac{1}{K_V S}$$



(Fig. 5)

On remarquera que le moment de flexion agissant à la base de la fondation, sous l'effet des éléments de réduction Γ, X appliqués en O, est égal à :

$$(\Gamma + D\,X)\;\frac{I_f}{I_f + \dfrac{K_H}{K_V}\, I_\ell}$$

La réaction latérale du terrain a donc pour effet de *réduire* le moment de flexion dans le rapport $\dfrac{I_f}{I_f + \dfrac{K_H}{K_V}\, I_\ell}$

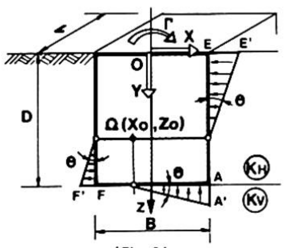
Cette réduction est très sensible quand la fondation est profonde.

Pour une fondation, dont la profondeur est supérieure au double de sa largeur ($D \geqslant 2\,B$) et pour $K_H \# K_V$, la fraction du moment de flexion transmis au sol est faible (de l'ordre de quelques % du moment agissant en l'absence de terrain latéral) et sa portance est alors conditionnée essentiellement par l'effort normal appliqué.

3.2.2.- Méthode générale des rotations (1)

Dans cette méthode les efforts appliqués Γ, X, Y sont équilibrés uniquement par les réactions du sol, qui ne peuvent être que des *compressions* (fig. 6).

Sous l'effet des éléments de réduction Γ, X, Y, la fondation va alors tourner d'un angle O autour d'un *centre instantané de rotation* Ω. Mais, Ω dépendant alors des efforts appliqués, la méthode n'est pas adaptée à la recherche de la matrice d'élasticité de la fondation.



(Fig. 6)

Elle présente par contre l'avantage, connaissant les efforts appliqués à la fondation, de vérifier de façon plus exacte sa stabilité. Elle est en particulier couramment utilisée pour l'étude des fondations des piles de pont soumises aux chocs des convois fluviaux.

(1) Méthode développée par M. CASSAN « Les essais in situ en Mécanique des Sols » — Editions Eyrolles Paris 78

3.3. - Fondation sur pieux

3.3 1.- Matrice d'élasticité d'un pieu isolé

On admet que le pieu est situé dans un *milieu élastique continu*, caractérisé par le module de réaction K_H du sol. On appelle *longueur élastique* (ou longueur de transfert) du pieu

$$\ell_o = \sqrt[4]{\frac{4E}{K_H i}}$$

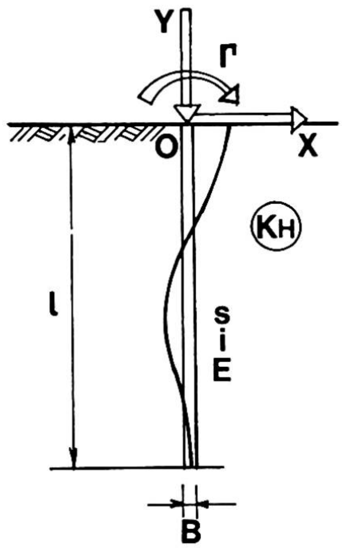
E étant le module d'élasticité du matériau constitutif du pieu et i le moment d'inertie de sa section droite.

Un pieu, dont la longueur ℓ est supérieure à 3 ℓ_o, (ℓ_o généralement inférieur à 5 m) peut être assimilé à un pieu de *longueur* infinie.

En supposant le module de réaction K_H du sol *constant* avec la profondeur, les coefficients de la matrice d'élasticité d'un pieu isolé de *longueur infinie*, sont alors :

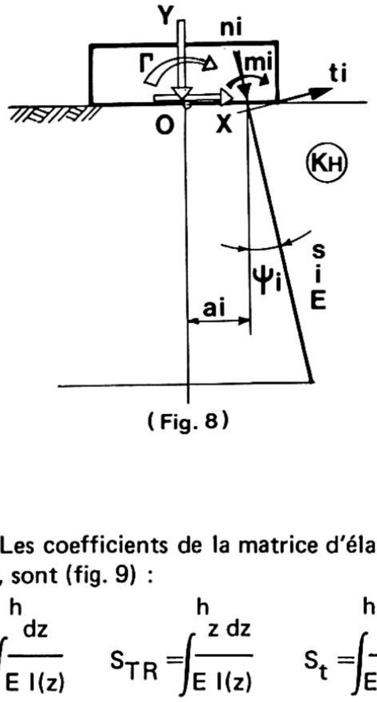
$$S_R = \frac{4}{\ell_o^{\,3} K_H B} \quad S_{TR} = \frac{2}{\ell_o^{\,2} K_H B} \quad S_T = \frac{2}{\ell_o K_H B} \quad S_V = \frac{\ell}{E s}$$

s étant la surface de la section droite du pieu.



(Fig. 7)

3.3.2 - Matrice d'élasticité d'un système de pieux



(Fig. 8)

Sous l'effet des éléments de réduction Γ, X, Y appliqués en O, chaque pieu, incliné de ψ_i sur la verticale et situé à la distance a_i du point O, est soumis en tête aux efforts m_i, n_i, t_i.

Les coefficients de la matrice d'élasticité du système de pieux s'obtiennent en écrivant :

- les équations d'équilibre ;
- les équations de déformation des pieux (3.3.1.) ;
- les conditions de comptabilité des déformations exprimant que les têtes des pieux sont solidaires de la semelle.

On admet généralement que les pieux de gros diamètres sont *encastrés* en tête dans la semelle. Pour les pieux de faible diamètre (B < 0,8 m) il est plus réaliste de les supposer *articulés* en tête ($m_i = O$).

4. INFLUENCE DU CORPS DE L'APPUI

4.1. - Expression générale de la matrice d'élasticité

Les coefficients de la matrice d'élasticité d'un corps d'appui, de hauteur h, sont (fig. 9) :

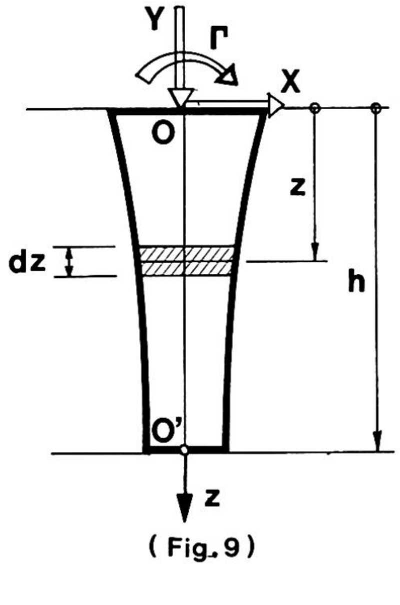
$$S_R = \int_o^h \frac{dz}{E\, I(z)} \quad S_{TR} = \int_o^h \frac{z\, dz}{E\, I(z)} \quad S_t = \int_o^h \frac{z^2\, dz}{E\, I(z)} \quad S_V = \int_o^h \frac{dz}{ES(z)}$$

avec :

- E module d'élasticité du matériau de l'appui
- S (z) et I (z) la surface et le moment d'inertie de la section du corps d'appui, de cote z.

Dans le cas d'un appui, de surface et de moment d'inertie constants :

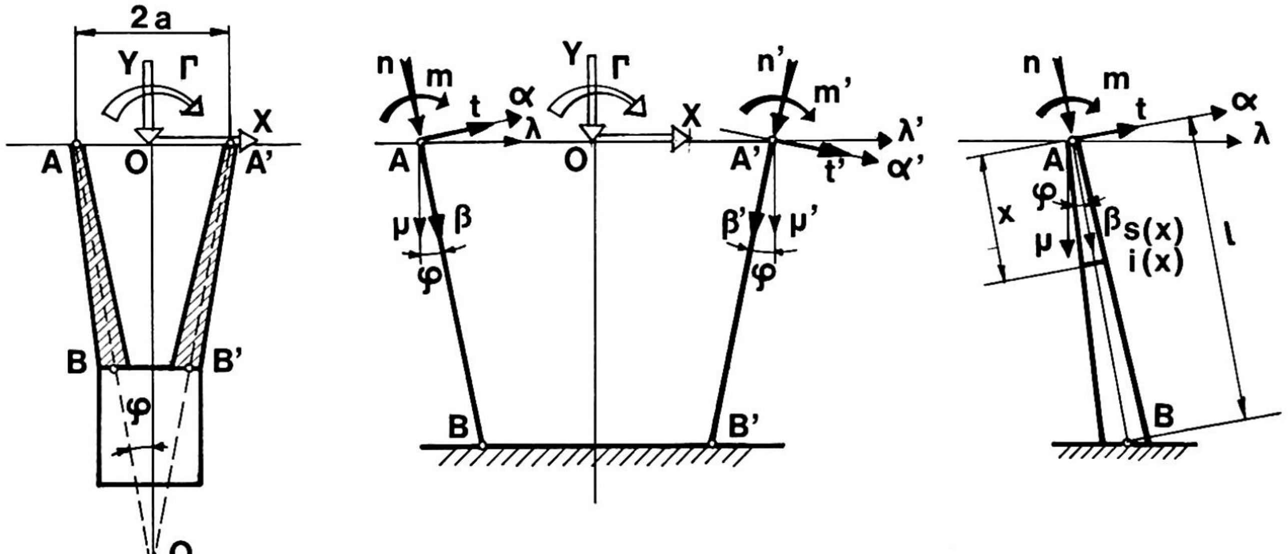
$$S_R = \frac{h}{EI} \quad S_{TR} = \frac{h^2}{2\,EI} \quad S_T = \frac{h^3}{3\,EI} \quad S_V = \frac{h}{ES}$$



(Fig. 9)

4.2. - Cas particulier d'un corps d'appui comportant des voiles souples

On suppose les voiles, de longueur ℓ, identiques et inclinés symétriquement de φ sur la verticale.



(Fig 10)

La surface et le moment d'inertie de la section droite située à la distance x du sommet A (ou A') de chaque voile sont respectivement s(x) et i(x). On pose :

$$\frac{1}{\sigma} = \frac{1}{\ell}\int_0^{\ell}\frac{dx}{s(x)} \qquad \rho_o = \frac{s(o)\,a^2}{2\,i(o)} \qquad I = 2\,i(o)\,(1 + 2\rho_o)$$

$$U = \int_0^{\ell}\frac{dx}{i(x)} \quad ; \quad V = \int_0^{\ell}\frac{x\,dx}{i(x)} \qquad W = \int_0^{\ell}\frac{x^2\,dx}{i(x)}$$

Soient Γ, X, Y les éléments de réduction par rapport au point O (milieu de AA') et θ, u, v les composantes du déplacement de la section AA' (fig. 10). En désignant par m, t, n, et m', t', n' les efforts dans les voiles en A et A', on peut écrire les relations suivantes :

a) Conditions d'équilibre

$$\begin{cases} \Gamma = m + m' + a\sin\varphi\,(t + t') + a\cos\varphi\,(n - n') \\ X = (t + t')\cos\varphi - (n - n')\sin\varphi \\ Y = (t - t')\sin\varphi + (n + n')\cos\varphi \end{cases}$$

b) Conditions de déformation

Les déplacements ω, α, β du point A sont donnés par

$$\begin{cases} \omega = \omega_o + \dfrac{mU}{E} + t\,\dfrac{V}{E} \\ \alpha = \omega_{o\ell} + \dfrac{mV}{E} + \dfrac{tW}{E} \\ \beta = \dfrac{n\ell}{E\sigma} \end{cases}$$

ω_o étant la rotation du voile A β en β et E le module d'élasticité longitudinale du béton. Des expressions analogues donnent les déplacements ω', α', β' du point A'.

On obtient les déplacements θ, λ, μ et θ', λ', μ' en effectuant les changements de coordonnées correspondants.

c) Conditions de compatibilité

Les conditions de compatibilité entre les déplacements des points A, A' et O s'écrivent :

$$\begin{cases} u = \lambda = \lambda' \\ \mu = v + a\theta \\ \mu' = v - a\theta \end{cases}$$

L'élimination des inconnues entre les différentes relations ci-dessus permet de calculer θ, u, v en fonction de Γ, X, et Y et d'obtenir par conséquent les coefficients de la matrice d'élasticité des voiles. (1)

Dans le cas de voiles *verticaux* de *section constante*, les calculs se simplifient et on trouve en particulier :

voiles encastrés aux deux extrémités

$$\begin{cases} S_R = \dfrac{\ell}{EI} \\ S_{TR} = \dfrac{\ell^2}{2\,EI} \\ S_T = \dfrac{\ell^3}{6\,EI}(2 + \rho_o) \end{cases}$$

voiles encastrés en tête et articulés à la base

$$\begin{cases} S_R = \dfrac{\ell}{EI}(1 + \dfrac{1}{2\rho_o}) \\ S_{TR} = \dfrac{\ell^2}{EI}(1 + \dfrac{1}{2\rho_o}) \\ S_T = \dfrac{\ell^3}{3\,EI}(1 + \dfrac{1}{2\rho_o})(3 + 2\rho_o) \end{cases}$$

voiles articulés en tête et encastrés à la base

$$\begin{cases} S_R = \dfrac{\ell}{EI}(1 + \dfrac{1}{2\rho_o}) \\ S_{TR} = o \\ S_T = \dfrac{\ell^3}{3\,EI}(1 + 2\rho_o) \end{cases}$$

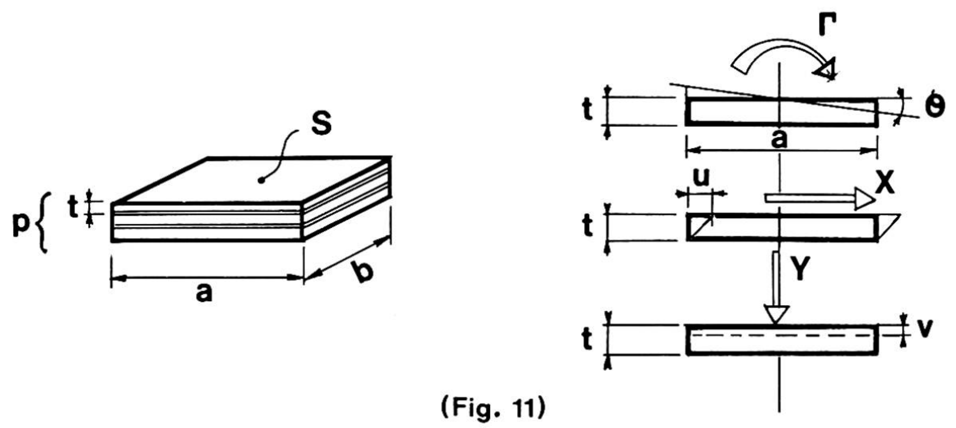
et dans le cas des voiles articulés aux deux extrémités :

$$S_R = \frac{\ell}{E\,I}(1 + 2\rho_o) \quad S_{TR} = o \quad S_T = \infty$$

5. INFLUENCE DES APPAREILS D'APPUI

5.1. - Appareils d'appui en élastomère fretté

Les appareils d'appui en *élastomère fretté* sont constitués par un empiècement de feuillets d'élastomère associés à des plaques d'acier (inoxydable ou non) assurant leur frettage (fig. 11).



(Fig. 11)

Les coefficients de la matrice d'élasticité d'un appareil d'appui en élastomère fretté, de surface S (a x b), comportant p feuillets élémentaires ayants chacun une épaisseur t de néoprène, sont :

$$S_R = p\,\frac{c'\,t^3}{GS\,a^4} \qquad S_{TR} = o \qquad S_T = p\,\frac{t}{GS} \qquad S_V = p\,\frac{ct^3}{GS\,a^2}$$

avec :

— G module d'élasticité transversal du néoprène

— c et c' coefficients de forme dépendant du rapport $\dfrac{b}{a}$

b/a	0,5	0,6	0,7	0,75	0,8	0,9	1	1,2	1,4	1,5	2	3	4	5	10	∞
c	5,83	4,44	3,59	3,28	3,03	2,05	2,37	2,01	1,78	1,70	1,46	1,27	1,18	1,15	1,01	1
c'	136,7	116,7	104,4	100,0	96,2	90,4	86,2	80,4	74,7	75,3	70,8	66,8	64,9	63,9	61,9	60

(1) Cf. « Constructions par encorbellement des ponts en béton précontraint » par J. MATHIVAT (Editions Eyrolles - Paris 78) Chapitre III - Conception des appuis

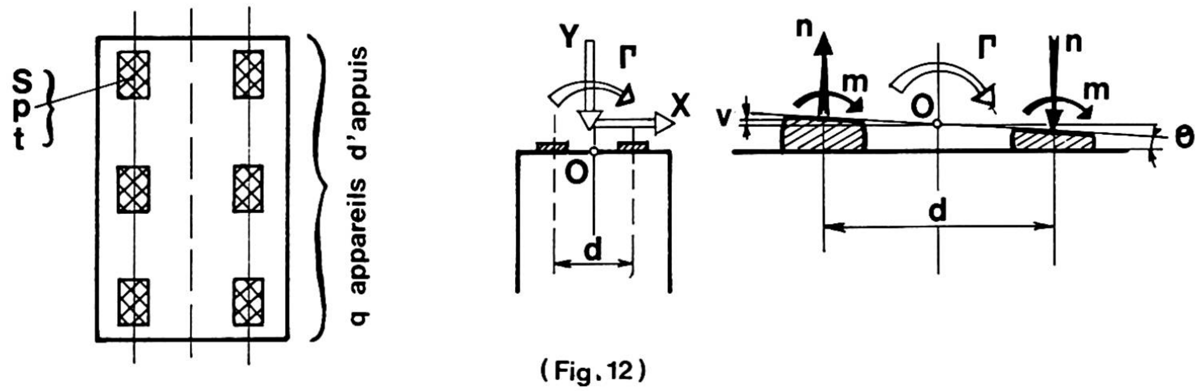
5.2. - Appuis comportant une seule file d'appareils d'appui en élastomère fretté

Si q est le nombre d'appareils d'appui, les coefficients de la matrice d'élasticité sont :

$$S_R = \frac{p}{q} \frac{c' t^3}{GS a^4} \qquad S_{TR} = 0 \qquad S_T = \frac{p}{q} \frac{t}{GS} \qquad S_V = \frac{p}{q} \frac{c t^3}{GS a^2}$$

5.3. - Appuis comportant une double file d'appareils d'appui en élastomère fretté

La présence de deux files d'appareils d'appui distantes de d, crée un encastrement partiel des éléments principaux de la structure sur leurs appuis (fig. 12).



(Fig. 12)

Soient n et m l'effort normal et le moment transmis aux appareils d'appuis sous l'effet du couple Γ appliqué en tête de l'appui, on a :

$$\Gamma = n d + 2 m \qquad \theta = \frac{2 v}{d}$$

$$\Gamma = \frac{p}{q} \frac{G S a^2}{t^3} \left(\frac{d^2}{2c} + \frac{2a^2}{c'} \right) \theta$$

q représentant le nombre d'appareils d'appuis par file.

La raideur des appareils d'appuis vis-à-vis des rotations peut généralement être négligée ($\frac{2a^2}{c'}$ faible eu égard à $\frac{d^2}{2c}$)

Les coefficients de la matrice d'élasticité sont alors :

$$S_R = \frac{p}{q} \frac{2 c t^3}{GSa^2 d^2} \qquad S_{TR} = 0 \qquad S_T = \frac{p}{2q} \frac{t}{GS} \qquad S_V = \frac{p}{2q} \frac{c t^3}{Gsa^2}$$

6. CONCLUSIONS

Malgré les nombreuses incertitudes (caractéristiques du sol, fonctionnement des appareils d'appui) que comporte la détermination de la déformabilité des appuis, il est nécessaire d'en tenir compte dans l'étude des structures hypersta-tiques, en particulier dans celle des ouvrages d'art.

Il est alors recommandé d'introduire dans le calcul des structures deux valeurs de la déformabilité des appuis (appui souple et appui raide) encadrant leur valeur probable, afin de se prémunir contre les incertitudes précitées.

XI

The South Abutment at Kessock Bridge, Scotland

La culée sud du pont de Kessock en Ecosse

Das südliche Widerlager der Kessock Brücke in Schottland

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SUMMARY
The predictions of movement of a large bridge abutment, based on a consideration of the forces in the piles, indicated that it would move outwards and tilt forwards. However, taking into account the ground displacement due to the large mass of fill which was placed behind the abutment, inwards and backwards tilting movements were predicted. Since this latter mode of movement could have introduced undesirable stresses, the abutment movements were measured.

RESUME
La prévision des déplacements d'une culée de pont de grande taille, basée sur l'analyse conventionnelle des forces appliquées, indiqua que la culée se déplacerait et basculerait en avant. Cependant, en considérant toutes les forces, la grande masse de remblai placée derrière la culée étant inclue, un déplacement et un basculement vers l'arrière furent prévus. Comme ce dernier type de mouvement pouvait introduire des contraintes indésirables, les déplacements effectifs de la culée furent mesurés.

ZUSAMMENFASSUNG
Bezüglich der Bewegung eines grossen Widerlagers der Kessock Brücke in Schottland wurde vorausgesagt, es würde nach aussen gleiten und nach vorn kippen. Trotz Berücksichtigung der Bodenbewegungen und der grossen Füllmasse, die hinter dem Widerlager angebracht wurde, wird vermutet, dass das Widerlager nach innen gleiten und nach hinten kippen wird. Da dies unerwünschte Spannungen auslösen würde, sind an dem Widerlager Messungen durchgeführt worden.

1. INTRODUCTION

The Kessock Bridge when completed will span the Beauly Firth at Inverness, Scotland. As shown in Figure 1 the abutment to the southern approach viaduct retains a selected granular fill embankment some 12m high, and supports one end of the first 64m span. The bearings of the viaduct are designed to transmit vertical load only, longitudinal forces being absorbed in the piers and at the northern abutment.

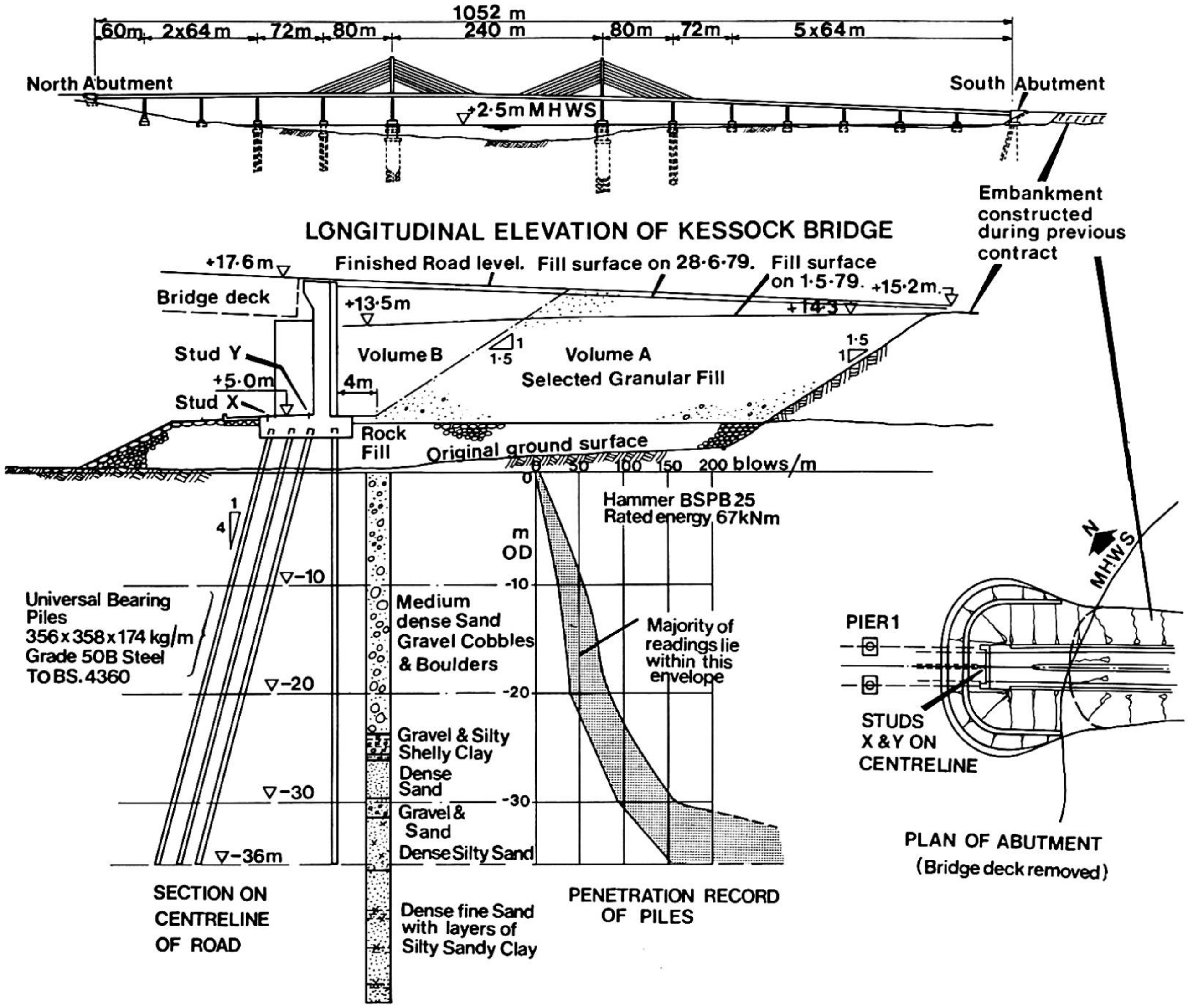


Figure 1 Location and general details

2. DESIGN OF FOUNDATIONS

2.1 Soil conditions

The great depth of alluvial deposits at the site have influenced the choice of foundations; a borehole for a pier adjacent to the navigation channel reached 95m below bed level, and a borehole 1 km away reached 89m below ground level, both encountering only alluvial deposits. The alluvial deposits increase in density with depth and pad foundations have been adopted where maximum bearing pressures can be economically kept to within 250 kPa. Where bearing pressures exceed this, or the depth of construction below water exceeds 10.5m, steel H-section bearing piles have been driven into the alluvial deposits.

2.2 Abutment foundations

Conventional analysis of the southern abutment foundation indicated that three rows of piles raked at 1 in 4, and a rear row of vertical piles were required to provide adequate support for the loads shown in Figure 2. The driving record of piles is given in Figure 1; test loading of a vertical pile gave 7mm settlement at the design working load of 1600 kN.

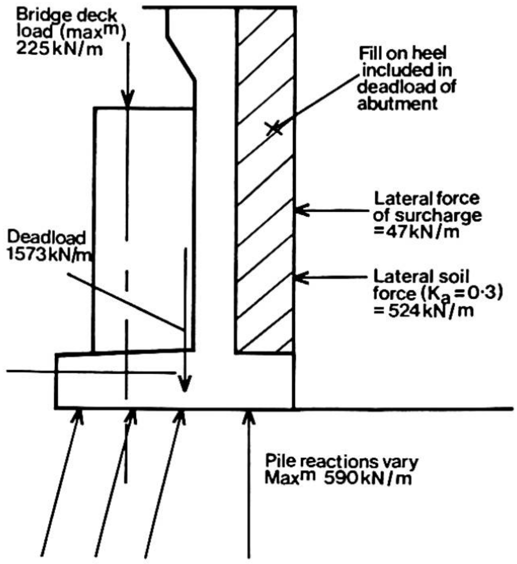


Figure 2 Loading diagram of forces on abutment (non-seismic loads)

3. MOVEMENT OF THE FOUNDATIONS

3.1 Analysis treating only forces on abutment

Analysis of the movement of the abutment, taking account of the forces shown in Figure 2, and treating the piled foundation as elastic springs, showed that abutment movements would tend to be downwards and forwards, the base of the abutment moving some 40mm towards the north. However, it was evident that the analysis of movement of the abutment could not be considered in isolation from the ground movements that would be generated by the construction of the short length of granular fill embankment between the already placed embankment and the abutment.

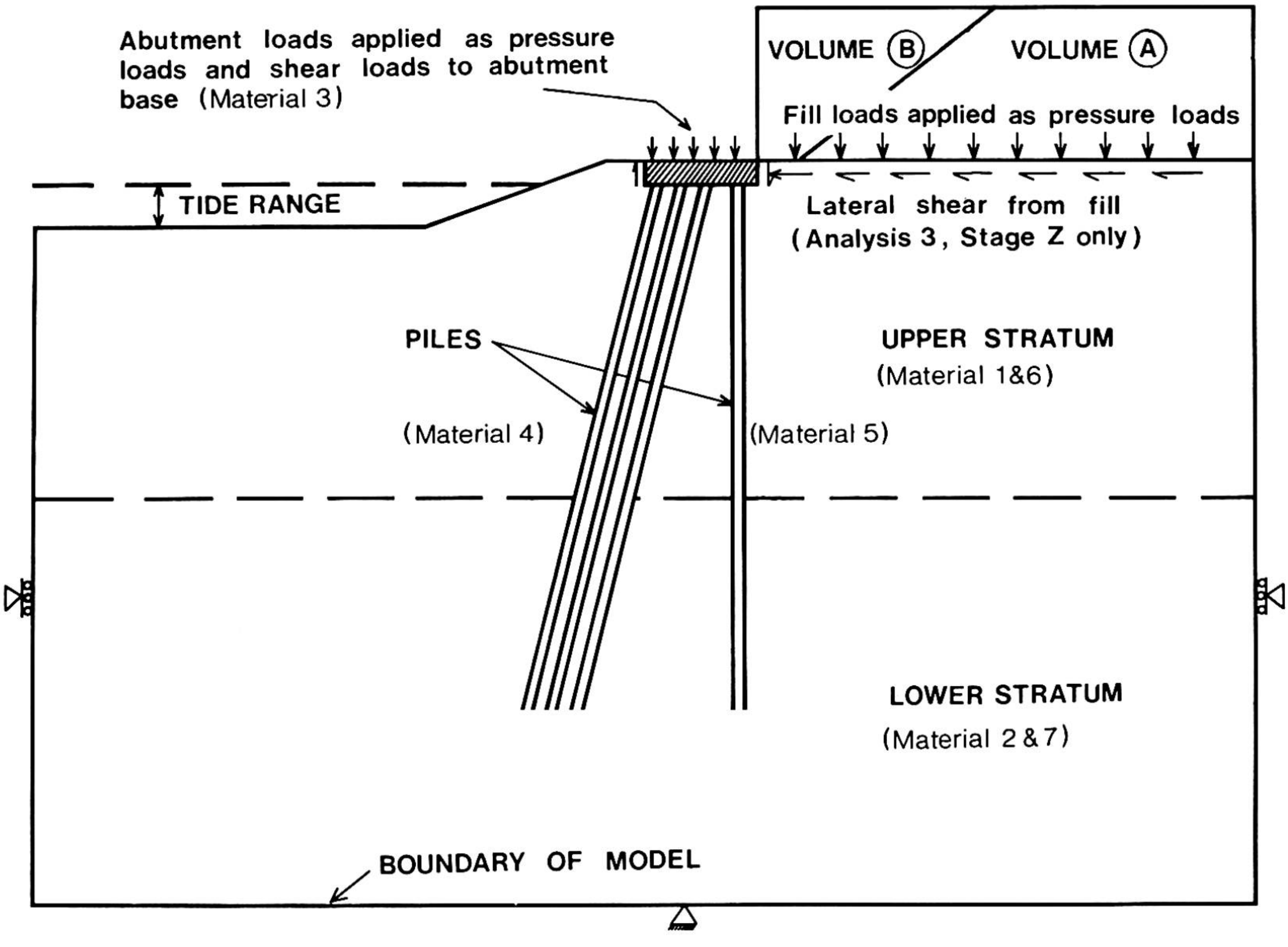


Figure 3. Finite element model. For properties of materials 1 to 7 see Table 1.

3.2 Analyses of interaction of soil and structure

A finite element programme, SAFE, (Reference 1) incorporating facilities to take account of soil and structural material properties was used to compute ground movements and stresses in the structural elements. Previous computations using the SAFE programme including modelling of anchored retaining walls, had shown that the programme gave reasonably accurate predictions of movements when compared with field measurements.

A two dimensional finite element model of the abutment and the soil was established, see Figure 3, and a series of analyses undertaken, the assumed soil and structural material properties for Analyses 3, 4 and 5 being shown in Table 1.

Table 1 "SAFE" finite element input data of material parameters.
All strength and modulus properties in MN/m^2

Material parameters	Soil (Analyses 3 & 4)		Concrete (All analyses)	Piles (Analyses 3 & 5)		Soils (Analysis 5)	
Material number	1	2	3	4	5	6	7
Material type	LE	LE	LE	LE	LE	LE(s)	LE(s)
Drainage	Drained	Drained	Drained	Drained	Drained	Drained	Drained
E_1	50	30	30 000	50	50	50	30
ν_{12}	0.3	0.3	0.1	0.3	0.3	0.3	0.3
E_2	50	30	30 000	4809	7642	50	30
ν_{12}	0.3	0.3	0.1	0.3	0.3	0.3	0.3
G_{12}	19.20	11.54	13636	19.20	19.20	19.20	11.54
α	0	0	0	-14.036	0	0	0
c'	–	–	–	–	–	0.017	0.017
ϕ'	–	–	–	–	–	0	0

Note

LE = linear elastic model

LE(s) = linear elastic model with strength limitation

Drained = excess pore water pressures assumed dissipated

E_1, E_2 = principal Young's modulii in x, y plane, suffix 1 denoting horizontal (in plane), 2 denoting vertical and 3 denoting horizontal (normal to plane) directions when $\alpha = 0$

G_{12} = shear modulus between directions 1 and 2

α_1 = anticlockwise inclination of direction 1 to horizontal (degrees)

c' = cohesive resistance (limiting value)

ϕ' = frictional resistance (limiting value)

The soils were given linear elastic properties and assumed to be drained, since their rate of consolidation was expected to be rapid. The assumptions concerning the pile elements are stated in Figure 5; for Analysis 4 the piles were assumed to be absent in order to demonstrate the general validity of the method. The main defect of the model was that the pile elements had to be considered as having the same unit "depth" (of 1 metre) as the soil and concrete elements, thus prohibiting relative movement in the line of the bridge between the piles and adjacent soil.

3.3 Loading sequence

The loading sequence used is shown in Figure 4, the construction process being simulated by four stages, A to D. To check on the effect of the simplifying assumption that the embankment load applied only a vertical load, Stage Z in which horizontal shear loads from the embankment were applied was run in addition to Stage B in Analysis 3.

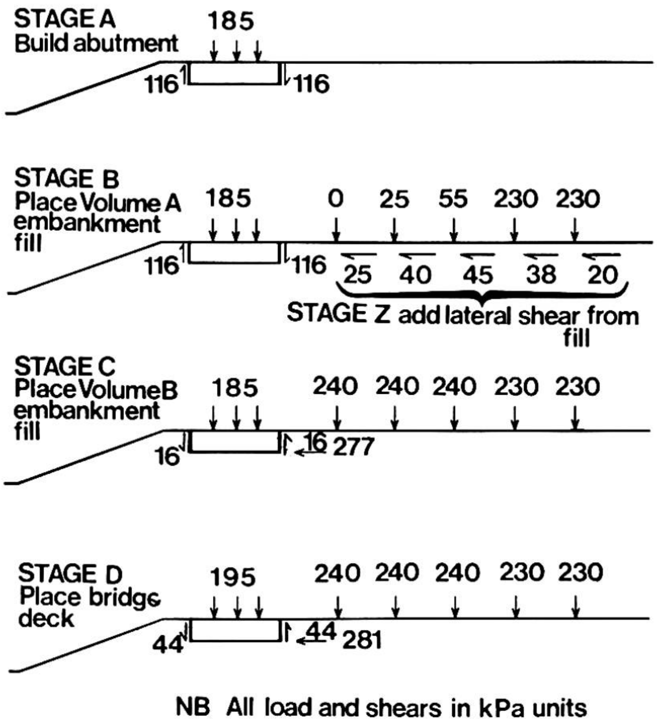
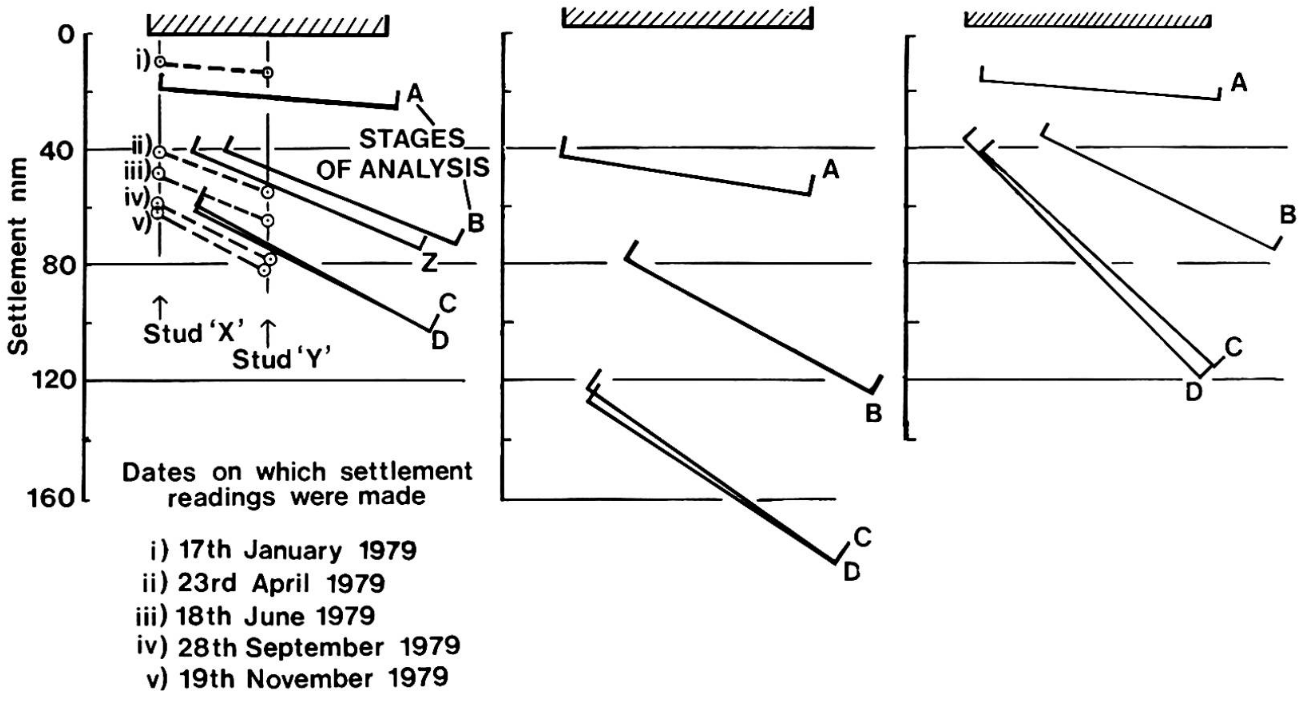


Figure 4 Loading diagram for finite element model



ANALYSIS 3

Assumptions
Pile elements have correct axial stiffness, are anisotropic and are "connected" to adjacent soil elements.

ANALYSIS 4

Assumptions
Pile elements deleted and replaced by soil.

ANALYSIS 5

Assumptions
Pile elements have correct axial stiffness, are anisotropic and "connections" to adjacent soil elements have shear strength limitation cohesion = 170 kPa, $\phi = 0$

Figure 5 Computed and measured movements.

3.4 Computed and measured movements

The results of the computations of movement are presented in diagrammatic form in Figure 5, with the measured results to date shown on Analysis 3. Studs X and Y on which level readings have been made are located on the road centreline at the positions shown in Figure 1. Site measurements of movement of the abutment in the line of the road were attempted, but showing little if any movement and being difficult to make with sufficient accuracy they were discontinued. The abutment movements have therefore been presented on the assumption that only vertical movement took place.

3.5 Movements were time dependent

The diagram giving settlement against time, Figure 6, shows that realisation of full settlement at each stage of loading was delayed by classical consolidation involving the dissipation of excess pore water pressures from the fine grained soils, complete dissipation for each stage taking about 6 months. The specified sequence of embankment filling was that Volume A should be completely placed in Stage B (Figure 4) before the placing of Volume B was started in Stage C, the object being to eliminate the pressure of fill material against the stem of the retaining wall as the base took up the major part of its backward rotation during the placing of Volume A, see Figure 5. In the event, the fill surface was raised uniformly over the entire fill length from about 5m above datum to 13m above datum in one continuous lift, followed by further filling to road base level at 17m above datum after a delay of two months.

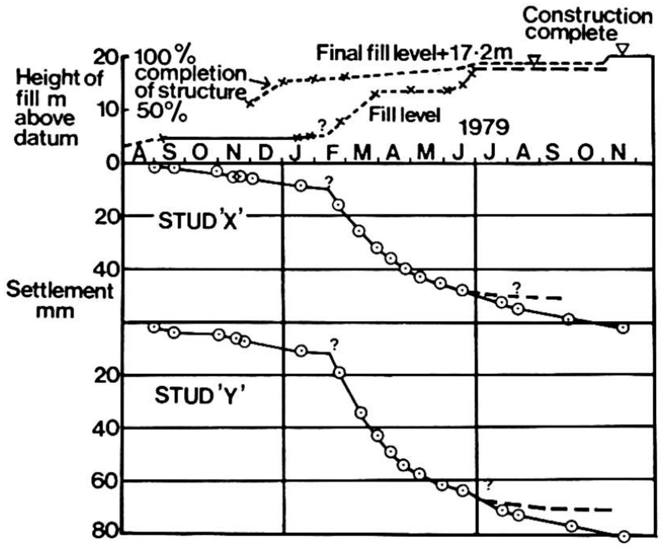


Figure 6 Loading and settlement with time.

4. EFFECT OF MOVEMENTS ON STRUCTURE

4.1 Restraint on backward rotation

As a consequence of following the described sequence of filling the lateral pressure of retained soil restrained the tendency of the base to tilt backwards by developing the bending strength of the connection between stem and base, the measured average angular rotation between the stem and base being about 1 in 900 after the filling to 13m above datum. During the subsequent filling this rotation increased to about 1 in 525, It is thought that the average vertical movements of base will have not been affected by the difference between the intended and the actual sequences of filling, but the backward tilt of the base may have been reduced.

4.2 Stiffening effect of piled foundation

From Figures 5 and 6 reasonably good correlations are apparent between the predicted settlement from Analysis 3 and the actual settlement on 17th January 1979, when the loading was the abutment base and stem (Stage A) and on 18th June 1979 when all loads except the bridge deck had been placed and consolidation under those loads was virtually complete (Stage C). It is evident that the piles moved together with the surrounding soil to give much greater settlements than could have been predicted from the results of the pile test loading and considerations of group action; the movements were of the same form but smaller than those that would have occurred if the foundation had not had piles. The smaller actual backward tilt than predicted by Analysis 5 indicates that the soil adjacent to the rear row of piles did not move differentially to the piles, and that shear failure played little part in the overall behaviour.

5. ACKNOWLEDGEMENTS

The Client is the Scottish Development Department who kindly gave their consent to the publication of this paper.

The Contractor is a consortium of Cleveland Bridge and Engineering Company and Redpath Dorman Long (Contracting) Ltd. The bridge was designed by Dr. Ing H. Homberg in association with Cleveland, and the substructure by Trafalgar House Engineering Services in association with R.D.L.

The Joint Engineers for the Client are Crouch and Hogg of Glasgow and Ove Arup and Partners of London

Acknowledgement is made of the dedication of the site staff in obtaining the readings which made possible this contribution.

6. REFERENCE

A computer model for the analysis of ground movements in London Clay. Simpson B., O'Riordan N.J. and Croft D. D. (1979). Geotechnique 29, No. 2, 149 -175.

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XI

Fundierung und Sicherung von Brücken in Rutschhängen

Foundation and Protection of Bridges on Sliding Slopes

Fondation et stabilisation des ponts sur talus glissants

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ZUSAMMENFASSUNG
Beim Bau von Gebirgsautobahnen ist man in zunehmendem Masse gezwungen, Talübergänge und Brücken in übersteilten und rutschgefährdeten Böschungen zu errichten. Zur Erzielung einer ausreichenden Standsicherheit der Brückenobjekte sind sowohl Hangsicherungen im Gelände als auch besondere konstruktive Massnahmen an den Fundamenten und Pfeilern erforderlich; deren gegenseitige Beeinflussung ist in bodenmechanischer, statischer und konstruktiver Hinsicht zu berücksichtigen.

SUMMARY
Planning highways in the alpine regions one is forced more and more to construct bridges over deep valleys and on steeply inclined slopes which are extreme slide areas. To gain sufficient stability of the bridges, both safety measures in the slope and special design details of the foundations and the piers become necessary; their mutual influence has to be considered in connection with soil mechanics, statics, construction and erection method.

RESUME
La construction des autoroutes de montagne exige de plus en plus la fondation des viaducs et des ponts sur des versants très raides et instables. Pour des raisons de stabilité, il est non seulement nécessaire de stabiliser le terrain, mais aussi de prendre des mesures constructives pour les fondations et les piles. Il y a lieu de considéres l'incidence réciproque des mesures de stabilisation sur les mesures constructives du point de vue de la mécanique des sols, du calcul statique et de la méthode de construction.

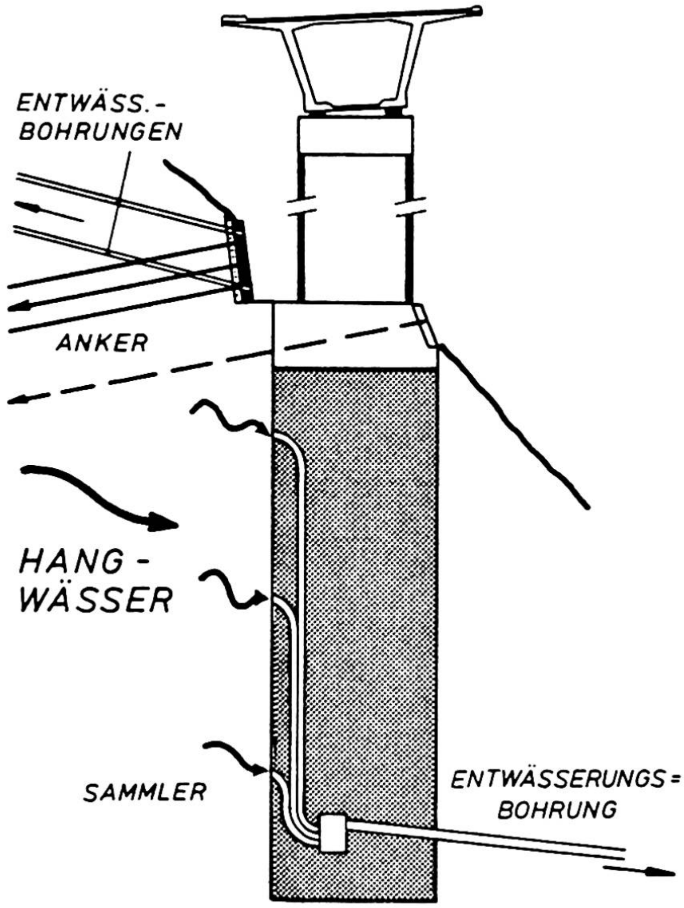

Abb.1:
Stützmaßnahmen und Entwässerungen für einen auf elliptischem Brunnen fundierten Brückenpfeiler in steilem durchnäßten Rutschhang

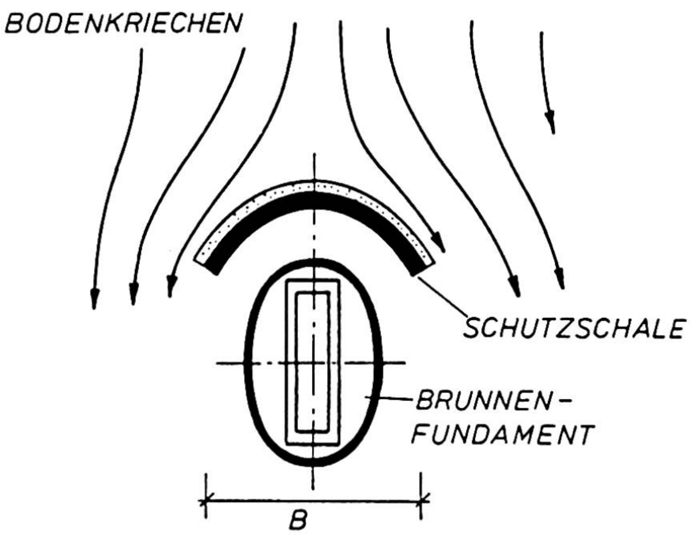

Abb.2:
Schutzschale (auf Filterbeton; mit Vorspannankern rückverhängt) für einen Brückenpfeiler im Rutschhang - flexibles System

1. HANGSICHERUNGEN

Entwässerungen können in den meisten Fällen die Hangstabilität entscheidend verbessern. Neben Oberflächendränagen, Sickerschlitzen und Tiefendräns haben sich Dränagebohrungen bewährt (bis ca. 50 m Länge), welche die Resultierende des Strömungsdruckes wirkungsvoll versteilen und auch örtliche artesische Wässer entspannen.

Wird z.B. beim Abteufen von Schächten für Brunnenfundamente Hangwasser angetroffen, so ist dieses nach Möglichkeit zu fassen und schadlos abzuleiten. Bei großen Schachtdurchmessern in steilem Gelände haben sich Entlastungsbohrungen aus der Brunnentiefe nach außen bewährt (Abb.1).

Schutzwände bzw. -schalen, welche die Brückenpfeiler und -fundamente von den Kräften rutschender Hangmassen abschirmen sollen, sind so zu konzipieren, daß das eigentliche Bauwerk seitlich "umflossen" wird, ohne daß in ihm unzulässige Verformungen oder Zwängspannungen auftreten (Abb.2).
Da diese Stützelemente in Etagen von oben nach unten hergestellt und sofort verankert werden, sinkt die Gefahr einer Rutschungsauslösung schon von Baubeginn an. Je nach Untergrundverhältnissen kommen auch Kombinationen zwischen geschlossenen Ankerwänden und Ankerrippen sowie bewehrtem Spritzbeton infrage (Abb.3).

Bei der Bemessung derartiger (verankerter) Schutzwände ist zu beachten, daß ein Kriech- bzw. Staudruck auftreten kann, der wesentlich über dem Grenzwert des aktiven oder Erdruhedruckes liegt.

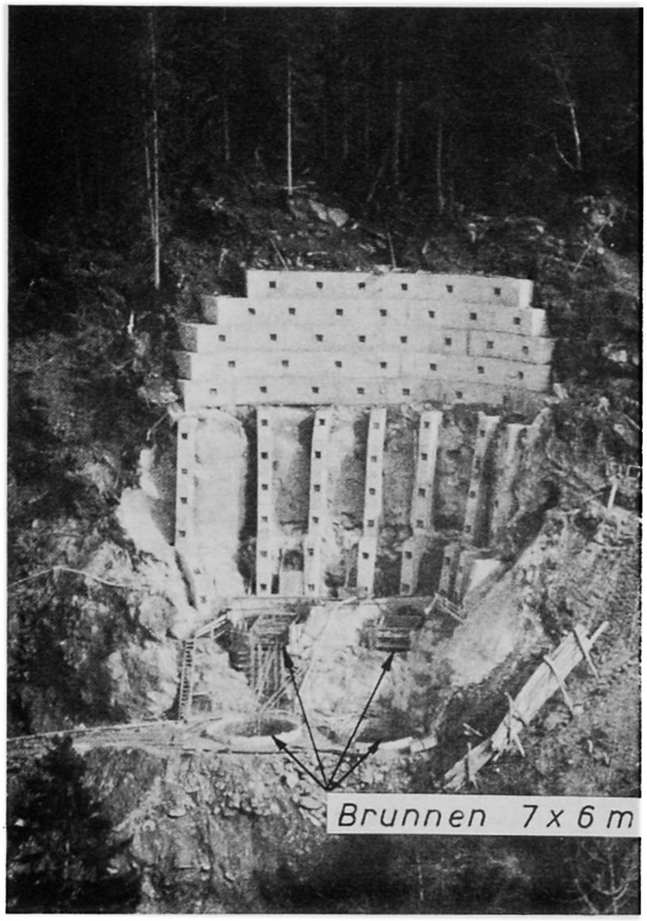

Abb.3 :
Ca.30 m hohe Hangsicherung für die 4 ellipt.Brunnen des Stützenpaares einer Autobahnbrücke (Festhaltepfeiler)

Bewährt hat sich die semiempirische Dimensionierung, indem zunächst nur ein theoretisch noch vertretbares Minimum an Vorspannankern versetzt wird. Im Zuge der Kontrollmessungen (Meßanker, Extensometer, geodätisch) zeigt sich dann meist schon während der Bauzeit, ob Zusatzmaßnahmen erforderlich sind. Dadurch ist es möglich, sich mit relativ geringem Aufwand an ein technisch - wirtschaftliches Optimum heranzutasten. Aufgrund der in solchen Hängen meist sehr stark streuenden Boden- und Felsparameter und der Unsicherheiten über die ungünstigsten Strömungsverhältnisse können erdstatische Berechnungen ohnehin nur grobe Anhaltspunkte liefern.

2. FUNDIERUNGSBEMESSUNG

Zur Ermittlung der Schnittkräfte in den Gründungskörpern und der Boden- (bzw. Fels-)reaktionen können grundsätzlich sowohl das Bettungsziffer- und Steifemodulverfahren als auch die Erddrucktheorie herangezogen werden.

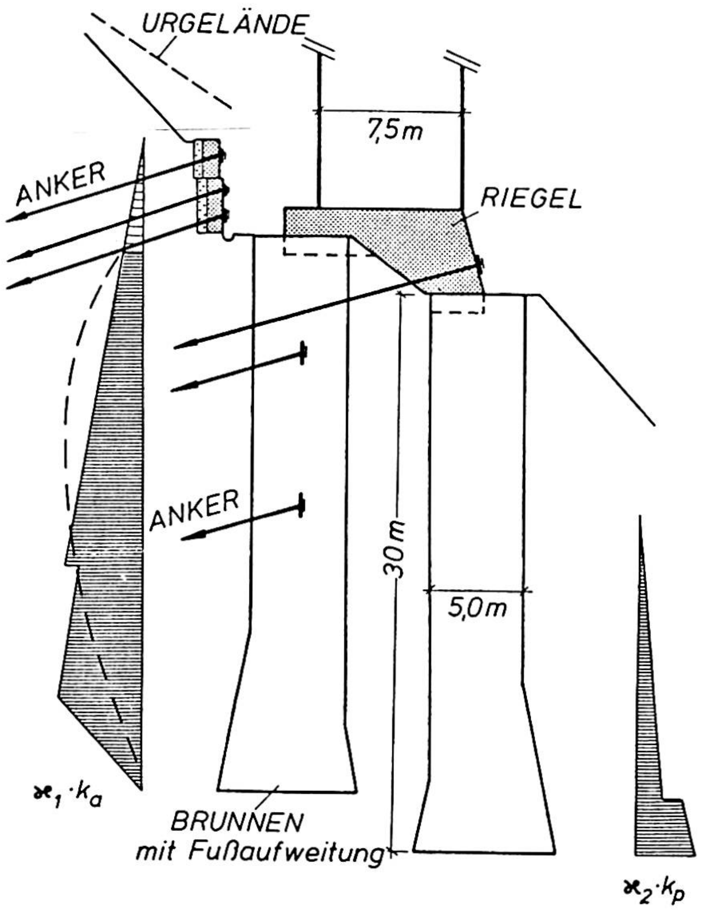

Abb.4:
Hangsicherung und Fundierung einer Brückenstütze in extrem ungünstigen Untergrundverhältnissen (Ankerlängen l_A = 37 - 45 m, schematisch angedeutet); Brunnenpaar und Riegel bilden steife Rahmen. Erddruckansätze (Umlagerung strichliert)

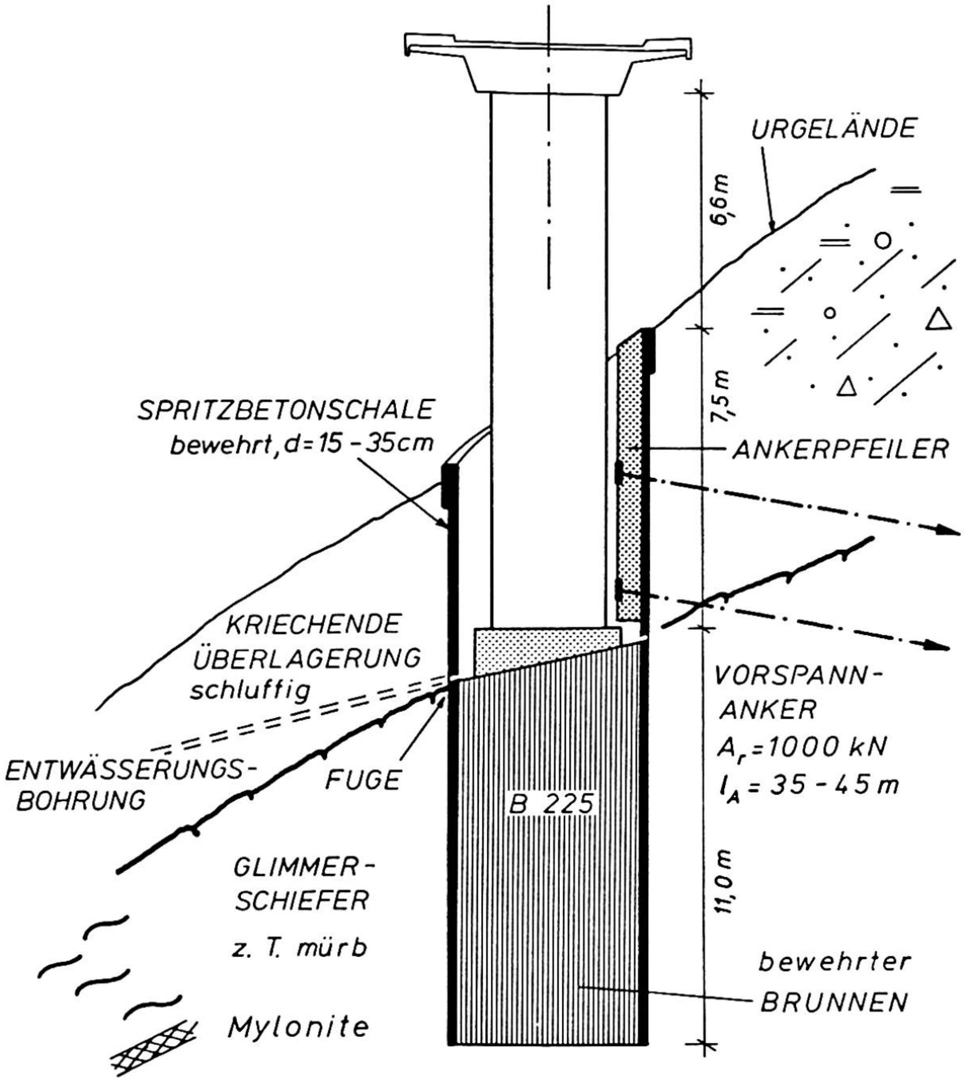

Abb.5: Brunnenfundierung eines Brückenpfeilers in mürbem Glimmerschiefer mit Zerrklüften. Erddruckabschirmung von kriechender Überlagerung durch Hohlellipse aus Spritzbeton ("Knopfloch"); zusätzliche Verankerung (stehende Ankerpfeiler)

Das Bettungszifferverfahren weist allerdings theoretische Schwächen auf (z.B. keine Schubübertragung im Untergrund), zudem ist die Bettung c_b keineswegs eine Bodenkonstante. Berechungen nach dem Steifemodulverfahren liefern wiederum Bodenspannungen, welche de facto nicht auftreten können. Dies ist darauf zurückzuführen, daß die Steifezahlen des Untergrundes spannungsabhängig sind, den elastischen und plastischen Bodenbereichen nicht genügend angepaßt werden können und gegen die Geländeoberfläche nicht gegen Null auslaufen.

Somit ist für die praktischen Berechnungen die Erddrucktheorie am zweckmäßigsten. In den meisten Fällen kann als hinreichende Näherung eine dreiecksförmige Erddruckverteilung angenommen werden.

Verankerte Schutzwände oberhalb des Pfeilerkopfes bedingen eine Erddruckabschirmung, verankerte Brunnen eine Erddruckumlagerung. Gemäß dem Beispiel der Abb.4 ist bergseits ein erhöhter "aktiver" Erddruck anzusetzen ($\varkappa_1 > 1$), talseits ein reduzierter Erdwiderstand ($\varkappa_2 < 1$); für die Aktivierung des vollen Erdwiderstandes ($\varkappa_2 = 1$) sind nämlich in der Regel Verformungen erforderlich, welche für das Brückentragwerk nicht mehr verträglich sind (vor allem bei hohen Pfeilern). Nur wenn die Hangstabilität deutlich über F = 1 liegt, kann der auf die Gründungskörper wirkende räumliche Erddruckbeiwert $\varkappa_1$ auch kleiner 1,0 werden.

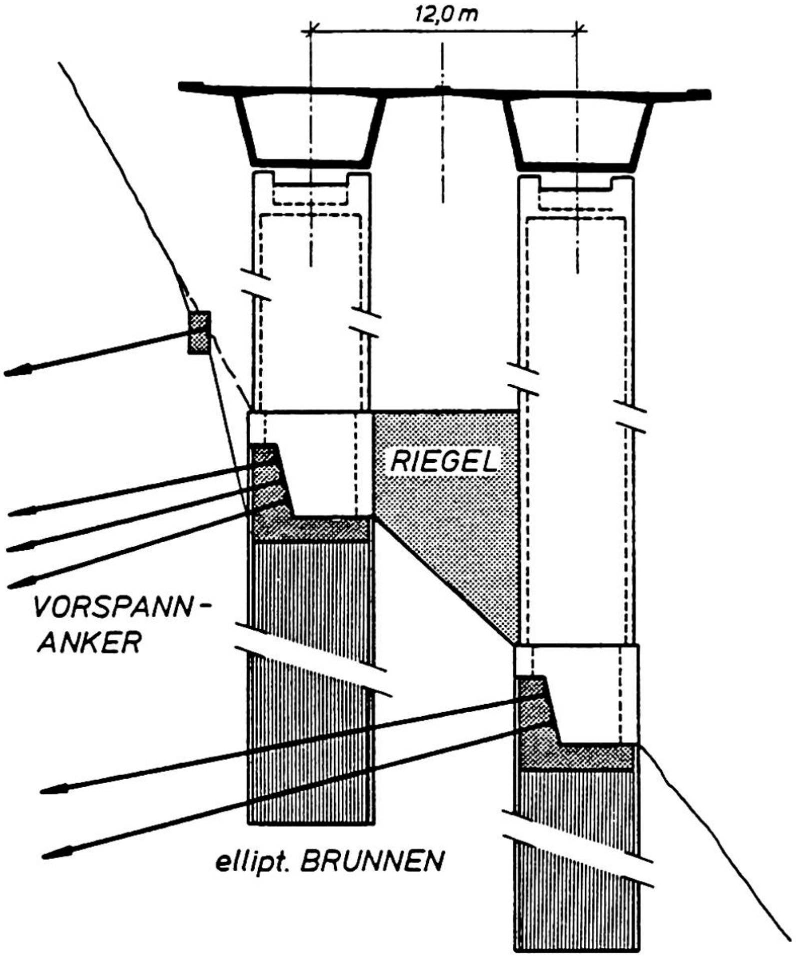

Abb.6:
Brückenfundierung mit Hangsicherung direkt von den Brunnenköpfen aus - - starres System; bergseits nur lokale Ankerrippen

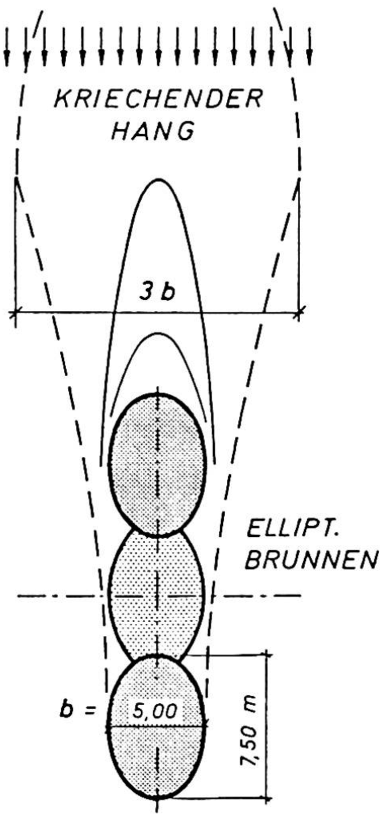

Abb.7:
Brückenfundierung (Stützenpaar) auf Brunnenscheibe aus 3 ellipt.Brunnen; Grenzlinien des umströmten Körpers bzw. fikt. Einflußbreite (3b)

3. BEISPIELE AUS DER BAUPRAXIS

Falls keine echte Rutschgefahr mit progressiver Bruchbildung besteht (geringer Restscherwinkel φ_r), sondern es sich nur um einen langsam kriechenden Hang handelt, können die Erddruckkräfte von der Brückenstütze durch "Knopflöcher" abgeschirmt werden: hiebei wird der Pfeiler im Schutz einer Hohlellipse aus bewehrtem Spritzbeton tiefer geführt. In kritischen Fällen haben sich Verstärkungen der Schale durch stehende Ankerrippen bewährt (Abb.5).

In der Regel sollten die eigentlichen Hangsicherungen (z.B. Ankerwände) und Brückenfundierungen (z.B.Brunnen) konstruktiv voneinander getrennt werden. Abb.4 stellt eine ziemlich aufwendige Gründung in einem übersteilten tiefgreifenden Rutschhang dar: Die Brunnenfundamente sind am Kopf mit einem massiven Stahlbetonriegel biegesteif verbunden; die so entstehende Rahmenkonstruktion besitzt ein sehr großes Widerstandsmoment in der Fallinie.

Falls die Hangsicherungen mittels langer Vorspannanker direkt vom Brückenobjekt aus erfolgen (Abb.6), ist ein größerer Erddruck auf die nunmehr starre Stützkonstruktion in Rechnung zu stellen als bei

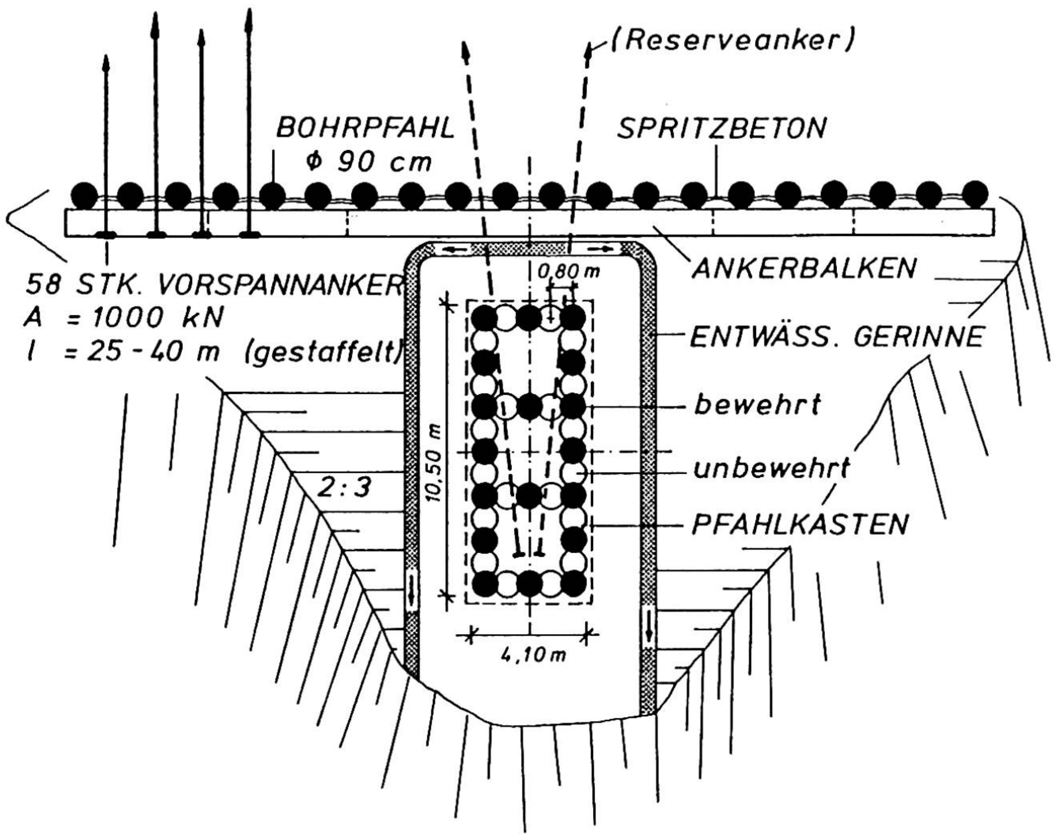

Abb.8:
Brückenfundierung in durchnäßtem weichen Rutschhang auf Bohrpfahlkasten (l x b = 10,5 x 4,1 m); Hangsicherung: verankerte Pfahlwand

flexiblen Ankerwänden. Überdies sind die Meßkontrollen zu verschärfen.

Anstelle von Fundamentverankerungen sind auch Gründungsscheiben möglich, indem in der Fallinie mehrere Brunnen hintereinander hergestellt werden; durch biegesteife Verbindungen wird ein statisch gemeinsam wirkendes Widerstandsmoment erzielt. Gemäß Abb.7 hängt der in Rechnung zu stellende Erddruck von den möglichen Gleitflächen im Kriechhang ab. Anstelle der seitlichen Reibungskräfte wird z.T. auch mit einer fiktiven Einflußbreite des ebenen Erddruckes gerechnet.

Wenn der talseitige Untergrund von sehr weichen, wasserführenden rutschgefährdeten Böden größerer Mächtigkeit überlagert ist, können Brunnenschächte nur unter hohem Aufwand und Risiko abgeteuft werden. In solchen Fällen haben sich Pfahlkästen (ev.Schlitzwandkästen) mit hohem Widerstandsmoment gegenüber Hangschub und Fließ- bzw. Staudruck bewährt (Abb.8). Hiebei handelt es sich um vertikale Gründungskästen aus überschnittenen, allenfalls tangierenden Bohrpfählen (Ø $\geq$ 90 cm) mit aussteifenden Querschoten.

Bei sehr steilem Gelände und breiten Autobahnquerschnitten oder mehreren parallel verlaufenden Verkehrswegen sind sogenannte Halbbrücken meist die wirtschaftlichste Lösung: Hiebei liegen die bergseitigen Fahrbahnen im Anschnitt bzw. auf Schüttungen (mit Stützmauer), die talseitigen auf einer Brücke. Auf diese Weise werden übermäßig hohe Hanganschnitte vermieden und der Erddruck gestaffelt übernommen.

XI

Soil-Structure Interaction on Multispan Continuous Bridge

Influence réciproque du sol et de viaducs autoroutiers

Wechselwirkung von Baugrund und Tragstruktur bei mehrfeldrigen Autobahnbrücken

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SUMMARY
The highway construction office of Nihon Doro Kodan is trying to adopt multispan continuous viaducts with 10 to 20 spans. This type of bridge has a problem of soil-structure interaction. The problem was solved by analyzing many trial cases; it confirmed the propriety of the design method by a long term observation on actual behavior of a 10 span continuous bridge constructed 4 years ago.

RESUME
Le bureau des autoroutes Nihon Doro Kodan a un programme des construction de viaducs autoroutiers par tranches continues de 10 à 20 travées; ce programme est en cours d'exécution. Ce type de viaduc présente un problème d'interaction sol-structure. Le problème a été résolu par l'analyse de nombreux cas et la méthode d'étude justifiée par des mesures à long terme effectuées sur un viaduc de 10 travées continues qui avait été construit 4 ans auparavant.

ZUSAMMENFASSUNG
Das Autobahnbüro Nihon Doro Kodan begann mit der Praxis, Autobahnbrücken mehrfeldrig, nämlich 10- oder 20-feldrig durchlaufend zu bauen. Bei diesen Brücken stellt sich das Problem des Zusammenwirkens von Baugrund und Tragstruktur. Mit dem Problem wurden die Autoren durch systematische Untersuchungen fertig und bestätigten die Richtigkeit der Entwurfsmethode durch langjährige Messungen an einer vor 4 Jahren ausgeführten, 10-feldrigen durchlaufenden Stahlbetonbrücke.

INTRODUCTION

Standard types of viaducts adopted for expressways in Japan are reinforced concrete slab bridge with hollows as shown in Fig.1. This standard bridge is consisted of 5 continuous spans and the span length is varied from 15 to 20 meters, in general.

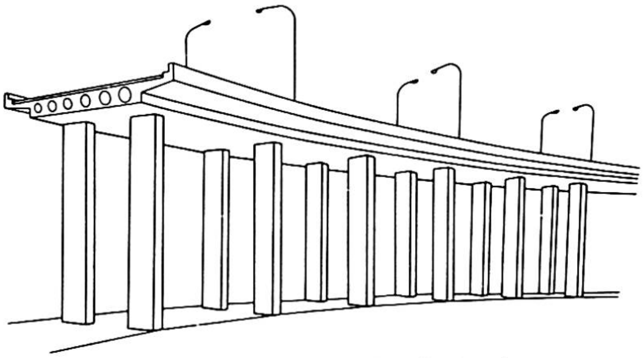
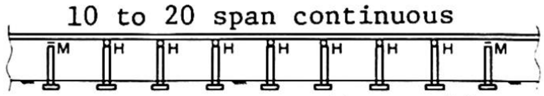


Fig. 1 Standard Viaduct

Nihon Doro Kodan (Japan Highway Public Corporation) is trying to adopt a bridge with many more continuous spans such as 10 to 20 (Fig.2) for the purpose of lessening expansion joints on the surface of viaducts. These joints are the weakest point in bridges and cause a problem of frequent maintenance. By lessening them, the maintenance of bridge becomes easier and also car driving becomes more comfortable. One of distinctive problem of the multispan continuous bridge in Japan is a problem of soil-structure interaction. However, the design of this type of bridge becomes possible if an advantage of soil behavior is taken into account.

This paper describes a basic concept in designing this type of structure and introduces the actual behavior obtained from a long term field measurement for a 10 span continuous bridge constructed 4 years ago.

10 to 20 span continuous



M: movable
H: hinged

Fig.2 Multispan Continuous Bridge

OUTLINE OF STRUCTURE

Since inertia force of superstructure due to earthquake increases in a continuous bridge as the number of spans increases, the design of the substructure in a multispan continuous bridge becomes almost impossible due to too much concentration of horizontal force if bearings of only one or two piers are hinged. Therefore, dispersion of the horizontal seismic force is required by increasing the number of hinged piers in designing a multispan continuous bridge. However, the increase in the number of hinged piers results in the increase in the horizontal forces to the piers caused by deformation of the superstructure due to temperature variation or shrinkage of concrete. The amount of these horizontal forces is also a function of stiffness of the substructures including the soil foundation. The less the stiffness of the substructures, the smaller the horizontal force to be induced. Therefore, continuation of bridges can be done if the flexibility of soil foundation is taken into account in the design. In conventional bridges, their superstructures and substructures have been designed individually. However, in designing a structure such as multispan continuous bridge, analytical solutions obtained through a model together with the superstructure, substructure and soil foundation should be employed, referring to their interaction effects.

INFLUENCE OF SOIL FOUNDATION

A structural model employed for a design considering the interaction effects of the structure and the soil foundation should, of course, include a factor representing a mechanical behavior of the soil. It is usually represented by a spring. Since the properties of soil used in this type of design should represent the behavior of the soil foundation properly, more precise survey of the soil is required prior to the design for this type of bridge, comparing with a conventional one. Steel and concrete used for the superstructure and substructure are considered to be homogeneous and the mechanical properties can be found easily and accurately by testing their specimens. Moreover, their moduli of elasticity remain almost constant under the ordinal loading condition. On the contrary, the values obtained for the mechanical behaviors of soil do not always have a good accuracy for design although they are estimated by testing or surveying. Moreover, test results of soils sampled from different sites usually differ each other even if they belong to a similar tyep of soil. In addition, the load-deformation relation of soil is usually non-linear even if under the ordinal loading condition. Therefore, in actuality, performance of non-linear calculation is required to evaluate the soil behavior. However, it could be practical to design such a structure with selected upper and lower boundary values of soil properties, considering that the reliability of data for soil is uncertain.

FEATURE OF A STATICALLY INDETERMINATE STRUCTURE

In case of a highly indeterminate structure such as multispan continuous bridge with many hinged piers, the entire bridge still remains stable due to stress redistribution even if the loading capacity of some part of piers or foundations exceeds their limiting values. This feature is quite unique comparing to the case of conventional bridges with only one or two piers hinged. Therefore, it is not always appropriate to adopt design values used in conventional bridges, such as displacement or uplift of pile foundation, as the limiting values in this type of structure.

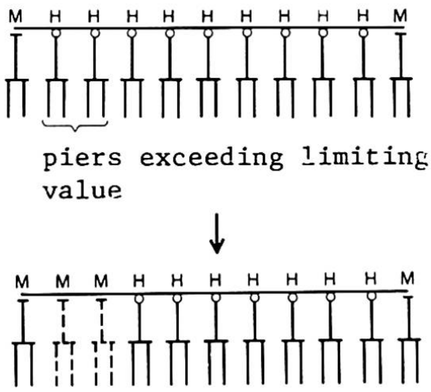


Fig. 3 Structure Model

If the deformation of the pier near the end of the bridge exceeds its limiting value due to temperature variation or earthquake, the soil of the foundation may be considered to be yielded in design. However, it is difficult to find the real yield point of the soil and the real behavior of the yielded soil, so that one of the realistic design of the bridge in the case above mentioned may be done by considering two types of structure models (Fig.3). In the first model, the pier is considered still to resist the horizontal force assuming the soil foundation to behave elastically in spite of its yielding. In the second model, the pier is assumed not to resist any horizontal force in the longitudinal direction. The safety of the structure is considered to be guaranteed if the safety of the two distinct models is obtained.

OBSERVED BEHAVIORS OF AN ACTUAL BRIDGE

A multispan continuous reinforced cocrete slab bridge with 10 spans was

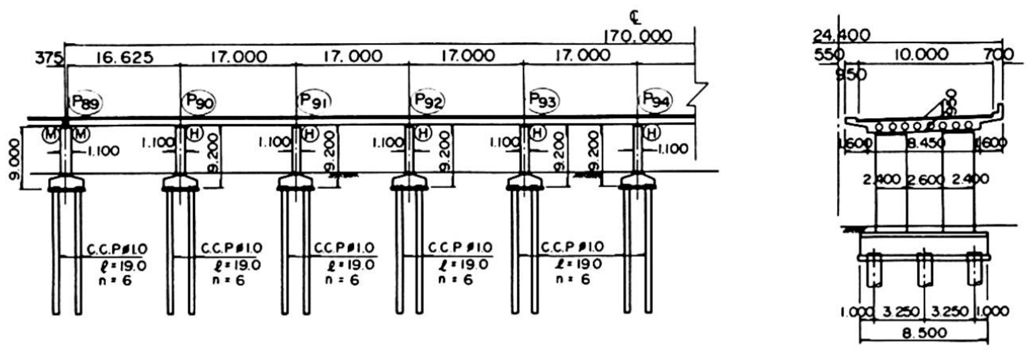


Fig. 4 General View of 10 Span Continuous Bridge

constructed in 1976 at Kanazawa in Japan (Fig. 4). Hinged bearings are installed on top of all piers but both ends of the bridge where movable bearings are installed. Concrete piles cast in site with 1.0 meters in diameter are used for the foundation of piers.

As above mentioned, influences of temperature variation and shrinkage of slab concrete become an important factor for design of multispan continuous bridges. A long term field measurement of actual behavior of the bridge has been carried out for the purpose of finding the influences mentioned above and also confirming the effects of soil behavior. The measurements were done successfully. Fig. 5. to Fig. 10 are the part of the results.

Fig. 5 shows temperature variation of the slab concrete two years after placing.

Fig. 6 shows movements of both ends of the bridge in two years.

Fig. 7 shows the amount of shrinkage of the slab concrete estimated from elongation of the slab.

Fig. 8 shows the relation between elongation of the slab and strain of the second pier from the bridge end.

Fig. 9 shows the relation between elongation of the slab and strain of the pile of the second pier from the bridge end.

Fig. 10 shows temperature variation of the slab concrete a day.

Concluding results obtained from the long term field measurements of the structure are as follows:

1. The temperature variation of the slab concrete depends nearly on that of the atmosphere. The minimum temperature of the slab concrete was 0 °C and the maximum was 35 °C (Fig. 5).
   Although the range of temperature variation taken in the design was 10 °C up and down from the average temperature, it would have been more rational to take the range as 15 °C.
2. The coefficient of thermal expansion of the slab concrete calculated from the measured relation between the temperature variation and the elongation at the end of the slab was 0.9×10^{-5} 1/°C, while it was assumed to be 1.0×10^{-5} 1/°C in the design.
3. Temperature difference between the top and bottom of the slab reached up to 10 °C during summer due mainly to direct sunshine (Fig. 5).
   Stress caused in the slab by this temperature difference is quite large and is not always negligible.
4. Shrinkage of the slab concrete calculated from the measured value of elongation or strain of the slab was found to be approximately 8×10^{-5} within the age of one year and it remained almost undeveloped since then (Fig. 7).
   This value corresponds to temperature variation of -8 °C, although it was assumed to be -15 °C in the design.
5. Behavior of the piers and foundations caused by the elongation of the slab due to temperature variation or shrinkage was almost linear elastic (Fig. 8, Fig. 9). This result seems to give a guarantee the assumption that the soil foundation may be represented as a linear elastic spring. In addition, over all stiffness of the substructure including the soil foundation obtained from the measurement was found to agree reasonably with that used in the design.

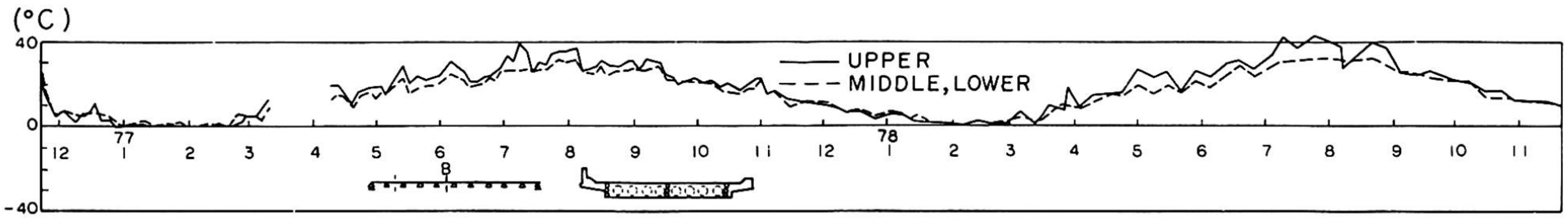


Fig. 5 Temperature Variation of Slab Concrete

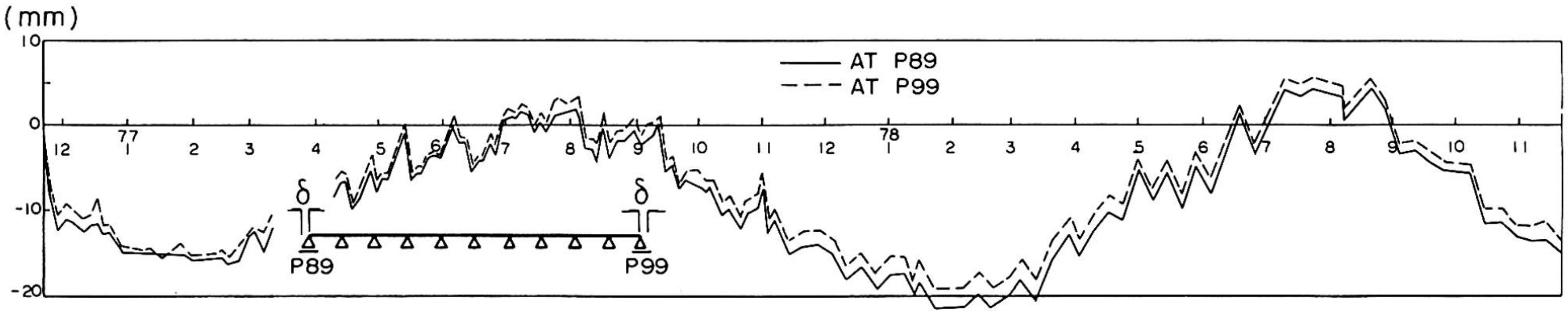


Fig. 6 Elongation of Slab

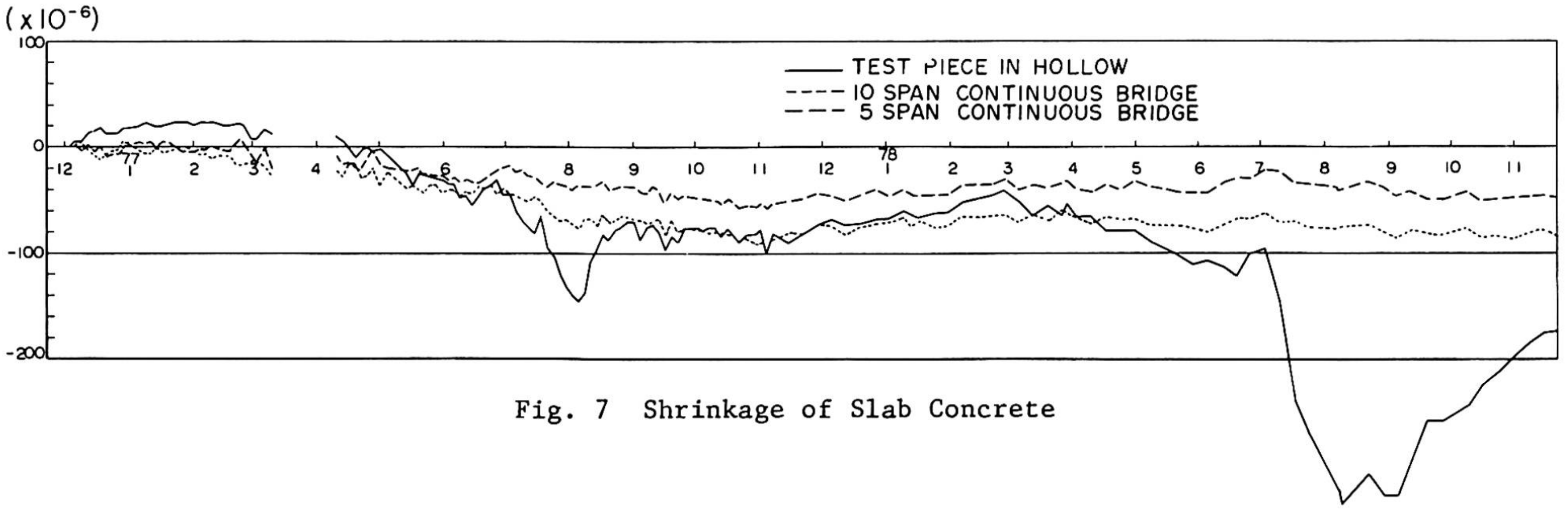


Fig. 7 Shrinkage of Slab Concrete

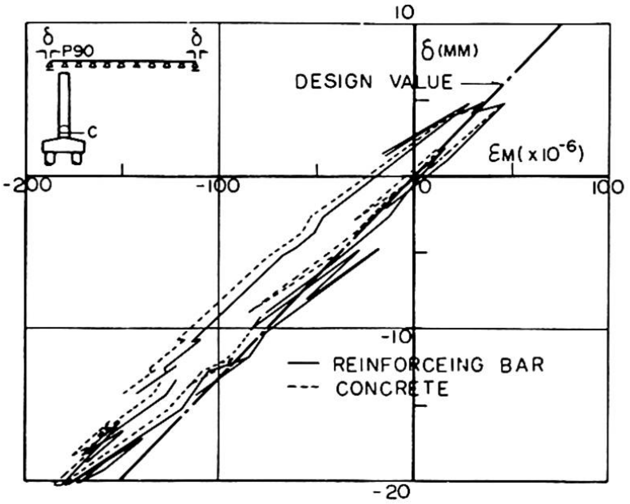


Fig. 8 Relation between Elongation of Slab (δ) and Strain of Pier (εM)

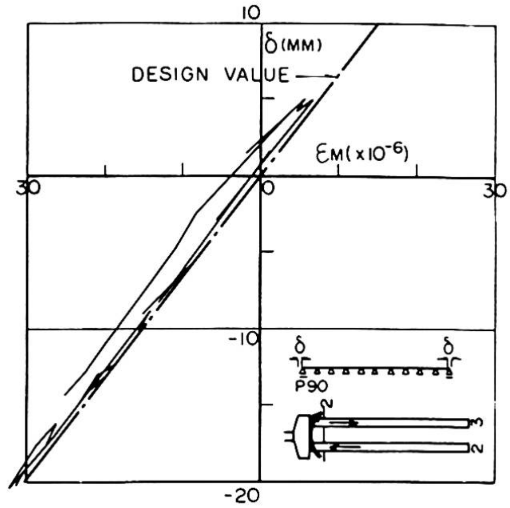


Fig. 9 Relation between Elongation of Slab (δ) and Strain of Pile (εM)

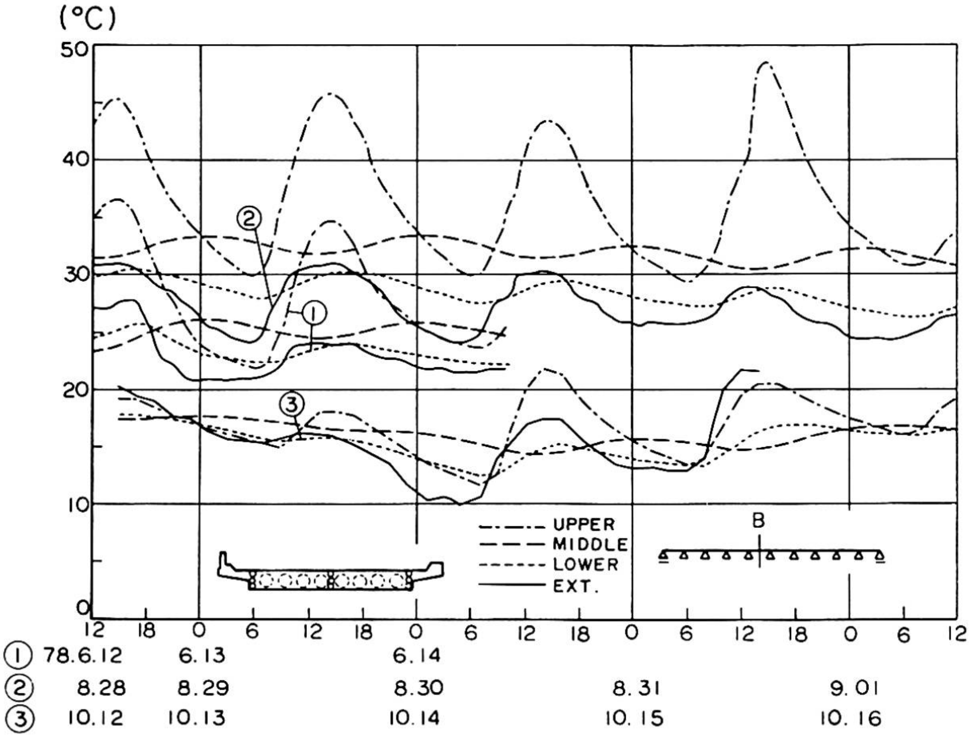


Fig. 10 Temperature Variation of Slab Concrete a Day

CONCLUDING REMARKS

Multispan continuous bridges can be designed in Japan by taking the preferable behavior of soil foundation into account. Adoption of this type of bridges enabled expansion joints to lessen and also proved to be economical as a whole. In addition, installation of many hinged piers could increase the load bearing capacity against earthquake.

Multispan continuous bridge would be very suitable structure as an expressway bridge in Japan which is often forced to be constructed on the very soft ground and yet must be taken the effect of earthquake into account.

Nihon Doro Kodan has established a design manual for multispan continuous bridge using various theoretical approaches and results obtained from a long term field measurement. We would like to apply this type of bridge more to future highway construction.