Session 5

Soil — Structure Interaction

Interaction Sol — Structure

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P. Habib, Chairman

INTRODUCTION

Avec 58 communications, la session 5 Interaction Sol/Structure est, au moins par le nombre, l'une des plus importantes du Congrès. Elle n'est dépassée que par les sessions 4 (Essais de Laboratoire) et 8 (Pieux), qui ont respectivement 64 et 63 communications. La session 5 représente ainsi 11,6% des publications du Congrès au lieu de 8% pour une session moyenne.

Il est vrai que le thème proposé était vaste : il s'agit, en fait, de tous les problèmes de Mécanique des Sols ayant des conditions aux limites. Avec une telle définition, nous n'aurons donc pas aujourd'hui de communications relatives aux problèmes de stabilité des pentes, aux problèmes d'hydraulique souterraine, enfin aux essais de laboratoire, trois sujets pour lesquels les conditions mécaniques aux limites ont en principe peu d'importance.

Les sujets qui ont été présentés pour cette session du Xème Congrès de Mécanique des Sols sont extrêmement différents : Fondations rigides, Fondations profondes, Puits et Caissons, Parois, Ancrages, Tuyaux enterrés, Module de réaction, Modes de calcul des interactions, etc... Tous les auteurs n'ont pas également insisté sur l'interdépendance de la raideur du sol et de celle de la structure pour l'évaluation des contraintes dans la structure et pour l'évaluation des déplacements dans le sol. Ce sujet est, à l'évidence, le domaine privilégié du calcul numérique et plus particulièrement de la méthode des éléments finis. On peut dire au moins deux choses à propos des méthodes numériques. Il apparaît d'abord que, pour le moment, les calculs numériques restent coûteux pour les problèmes de routine, dès qu'on quitte le domaine bidimensionnel élastique qui lui, par contre, est parfaitement maîtrisé. Mais, les calculs numériques permettent aussi des études paramétriques et permettent ainsi de préciser l'effet de certaines grandeurs dont le rôle n'est pas évident et qui, sous prétexte qu'elles étaient difficiles à mesurer, n'étaient pratiquement pas recherchées systématiquement au laboratoire comme "in - situ". Je prendrai, comme simple exemple, les contraintes résiduelles, classiquement définies en Mécanique des Sols par le coefficient K_0. Le calcul montre que leur connaissance est tout à fait essentielle pour la détermination du déplacement horizontal lors de l'ouverture d'une fouille. C'était pourtant un paramètre à peu près complétement négligé il y a vingt ans et son importance a été mise en évidence grâce au calcul. De la même façon, le calcul numérique nous montre l'importance des raideurs relatives du sol et des structures : c'est d'autant plus instructif que dans la plupart des cas la raideur de la structure est définie par l'Architecte indépendamment de toute connaissance du sol et sans que le projeteur ait demandé au spécialiste des sols dans quel sens il aurait intérêt à modifier cette raideur. Ceci est d'autant plus regrettable que le cortège des désordres dans les constructions est lié à la souplesse et à la fragilité des structures. Les définitions suivantes peuvent être données pour les bâtiments :

- Constructions souples (constructions métalliques, charpentes en bois sans remplissage entre poteaux, c'est-à-dire avec façades suspendues et bardages) ;
- Constructions moyennement souples (béton armé avec poutres élancées) ;
- Constructions rigides, non fragiles (béton armé bien calculé, avec longrines permettant à la structure d'agir comme un monolithe) ;
- Constructions rigides et fragiles (Maçonneries modernes à haut module d'élasticité, liées par des mortiers de ciment très raides).

Des définitions analogues peuvent être données pour d'autres structures que des bâtiments.

Ces définitions permettent une classification des dangers, mais ne sont évidemment pas assez mathématisées pour permettre une évaluation précise de l'interaction entre le sol et les structures. Il est certainement difficile de définir la raideur d'une structure, et il y a là un problème mécanique délicat. Il y en a d'ailleurs un autre pour la détermination de la déformabilité globale du sol, c'est-à-dire du coefficient E_s à introduire dans le calcul numérique avant de comparer les déformations mesurées et les déformations calculées : la notion de module d'élasticité du sol est à peu près correctement définie, mais curieusement le module global semble s'écarter des mesures sur échantillons. C'est très désagréable pour l'évaluation des déplacements et la prévision des désordres. On relie généralement les uns et les autres. Pourtant, ce problème important n'a été abordé ici que dans une seule communication, avec pour référence le tassement différentiel rapporté à la distance entre appuis, c'est-à-dire un angle de rotation exprimé en radian. Je signale que tout récemment en France le Professeur Kérisel a proposé des critères de désordres exprimés à partir du rayon de courbure de la déformée du sol. Pour des rayons de courbure supérieurs à 2000m, il a constaté peu ou pas de désordres. Entre 1000m et 2000m, des incidents se produisent dans les vieux bâtiments. Pour des rayons de courbure inférieurs à 1000m, des dommages peuvent apparaître dans toutes les structures dépendant, bien entendu, du mode de construction. L'expérience des dégâts de surface provoqués par les travaux miniers, où le paramètre rayon de courbure est normalement employé, montre que ce critère est certainement préférable à celui du tassement différentiel.Or, il est différent, puisque le tassement différentiel s'écrit

$$\theta = \frac{w}{L}$$

et le rayon de courbure :

$$R = \frac{L^2}{2w} \quad .$$

La courbure introduit donc une expression de la forme:

$$\lambda \frac{w}{L^2} \quad .$$

Il serait intéressant si l'on pouvait avoir, au cours de cette session, quelques informations sur ce sujet.

Parmi les sujets abordés au cours de cette session par les différents auteurs, une petite moitié a été consacrée aux fondations qui forment ainsi la partie principale de la session 5. Mais, les autres sujets posent des problèmes extrêmement importants et tout à fait vivifiants : conditions aux limites, problèmes tridimensionnels, entrée dans le domaine plastique, et j'en passe. Tout ceci ne rend pas facile l'organisation de la discussion, mais l'excellent Rapport Général du Professeur Poulos permet une mise en ordre du sujet et permet de distinguer les activités essentielles et la hiérarchie des difficultés, car il a fait un énorme travail de synthèse des études et de la bibliographie des quatre dernières années puis un travail précis pour l'analyse des communications présentées au Xème Congrès de Mécanique des Sols.

J.A.Diaz-Rodriguez, Panelist

AN APPROACH TO SOIL-STRUCTURE INTERACTION PROBLEMS OF RAFT FOUNDATIONS

Une Aproximation aux Problèmes d'Interaction Sol-Structure de Fondations sur Radier

1. INTRODUCTION

This paper intends to describe two models developed at the Universidad Nacional Autónoma de México (National University of Mexico).

Soil-structure interaction may be understood as the reciprocal action that is exerted between a structure and its supporting soil. This reciprocal action is established through soil-foundation interfaces, and is defined by the movement of such interfaces. The movement may be mutual by assuming compatibility at the interfaces, relative slip, debonding, and rebonding.

Many problems in soil mechanics are related with soil-structure interaction such as: continuous footings, raft foundations, compensated foundations, deep-piled foundations, retaining walls, and underground structures such as piers, cofferdams, buried conduits, tunnels, anchors, etc.

Equilibrium equations resulting from the model of soil-structure interaction may have a great variety of forms, depending on the fashion used to model its elements, since they are governed by the magnitude of the considered displacement (small or large), the nature of the considered geometry (uni-bi or three-dimensional) and the type of material considered (linear or nonlinear). Moreover, the formulation of the equations is closely related to the method of solution.

Owing to the fact that each of the concepts involved in the equilibrium equations have an empirical base, as well as the existence of numerical simplifications, the range of validity of an analytical model must be calibrated with experimental models both of small scale (physical models) and of natural scale (prototypes).

The soil-structure interaction model include the following:

1. Model of the superstructure
2. Model of the foundation
3. Model of the soil
4. Model of the interfaces

Mat foundations may be modelled as a three dimensional, bidimensional (flat plate) or one-dimensional (beam) solid. The behaviour of the material may be linear or non linear and may include viscous effects. The deformed shape may include small or large displacements.

The soil should be considered as a two-phase material when it contains water. There exist one, two, or three-dimensional formulations in terms of effective or total stresses. The solid phase (skeleton) may be modelled with linear or non linear behaviour and may include viscous effects. The movement of the liquid phase (water) may be laminar or not, and the liquid may be compressible. The various distributions of the materials (stratification) may also be taken into account.

The coupling and mutual influence of foundation and soil may be modelled by compatibility of deformation, relative slip, debonding and rebonding.

2. CALCULATION METHOD WITHOUT THE USE OF A DIGITAL COMPUTER

2.1 General Aspects

A method was developed by Prof. Zeevaert, and first published in 1973, (Refs. 5,6). This method permits the calculation of stresses and displacements at the contact surface between a rectangular mat foundation and the bearing soil which is characterized as a modified semi-infinite space.

2.2 Model of the Superstructure

This approach does not consider the foundation-superstructure interaction; although it maybe included. Mainly, it considers the loads that act upon the foundation.

2.3 Model of the Foundation

It supposes a beam of width 2B, length L, and height t, supported by the soil on a finite number of predefined points. Superstructure loads may be distributed or concentrated. The beam's equilibrium equations only consider deflections owing to flexure. The supports of the beam over the soil are idealized as vertical springs whose characteristics are determined with the interaction model.

2.4 Model of the Interface

The generation of shear stresses is not considered at the soil-foundation interface, and, therefore, interaction takes place through uniformly distributed vertical stresses at previously determined locations.

2.5 Model of the Soil

The soil is modelled as a modified semi-space. That is, the distribution of stresses and displacements is calculated using linear elastic theory, but secant parameters are determined from laboratory stress-strain-time tests that simulate field conditions, (Zeevaert, 1973).

2.6 Soil-Structure Interaction Model

The soil-structure interaction model is constructed by establishing compatibility of displacements at common points of the soil and the foundation. The way to construct and to solve the model leads to an iterative process.

2.7 Design Conditions

Zeevaert applies the procedure described above for the following conditions:

a) Expansion owing to load removal

b) Recompression owing to load applications

c) Compression without previous expansion

d) Transitory loads

e) Dynamic loads

3. A NONLINEAR NUMERICAL MODEL TO ANALYZE SOIL-STRUCTURE INTERACTION OF MAT FOUNDATIONS

3.1 Introduction

The model described in this section is being developed at the Graduate Division of the School of Engineering, UNAM (Ref. 2).

This model will allow the study of the behaviour of the elements that define the soil-structure system under the action of static loads.

3.2 Model of the Superstructure

The superstructure considered corresponds to a building such as:

a) The building is modelled by substructures formed by plane frames whose elements are beam-columns with three degrees of freedom per nodal point.

b) The plane frames are linked by rigid diaphragms (slabs) in such a way as to ensure compatibility of the desired displacements.

c) The inelastic behaviour of the bars of the building is established by several yield interaction surfaces.

The equations of equilibrium are established in an incremental fashion. Their discrete form uses the stiffness method.

3.3 Model of the Foundation

Two different models are being worked out:

3.3.1 Plate model

This model is based on Mindlin's plate theory. Its characteristics are:

a) Thick or thin plates may be modelled
b) The material behaves as an elasto-viscoplastic solid

c) Viscoplastic behaviour is governed by either von Misses, Tresca, Mohr-Coulomb, or Johansen's yield function.

d) Equilibrium equations are established incrementally based on the finite element method (displacement formulation).

3.3.2 Shell model

The model used corresponds to that developed at Swansea, University of Wales by Prof. Zienkiewicz and his group. Linear shell theory is built as a particular case of a three dimensioal solid, and it is based on the finite element method. The model is being adapted to fit the characteristics indicated in the plate model.

3.4 Model of the Interface

Interface models should simulate the behaviour of the interface between two continuous media that possess very different strength properties and that may provoke deformed shapes such as those observed at field.

The model used was developed by Prof. Desai from

Virginia Polytechnical Institute and Virginia State Univ., Blacksburg, Va. (Ref. 3). Its characteristics are:

a) The interface is considered as a solid of finite thickness.

b) Constitutive equations for normal and shearing stresses are established through elastoplasticity. Either the Mohr-Coulomb or the strain-hardening models may be considered.

c) Equilibrium equations are established incrementally using the finite element method (displacement formulation).

3.5 Model of the Soil

The soil is considered as a two phase continuous medium: a solid phase formed by the porous skeleton and a liquid phase that completely fills the interstitial pores of the skeleton.

3.5.1 Local equilibrium equations

The equilibrium equations for a two phase saturated continuous medium with a linear elastic skeleton were established by Biot for static loadings (Ref. 1). In the non linear model of the soil skeleton the constitutive equations are expressed incrementally.

3.5.2 Discrete form of the equilibrium equations

The mathematical model of the local equilibrium equations, for a specific constitutive equation, may be integrated if boundary and initial values are provided. The integration of the model with respect to the spatial variables is done with the finite element method (displacement formulation). The expressions for the resulting discrete model corresponds to an initial value problem and its solution may be achieved with several step by step numerical methods.

3.5.3 Constitutive equations of the soil skeleton

Nonlinear constitutive equations model, with good approximation, the behaviour of soils. The most successful have been the ones developed within the context of elastoplasticity and elastoviscoplasticity. Both theories make use of yield surfaces (F) and plastic or viscoplastic potential surfaces (Q).

Mohr-Coulomb and critical state cap models are used to model soil skeleton's yield surfaces. Plastic or viscoplastic potential surfaces are equal to yield surfaces in the associative case ($Q=F$), and different, but of a similar form to yield surfaces, in the nonassociative case ($Q\neq F$).

3.6 Soil-Structure Interaction Model

The model of soil-structure interaction is built by establishing the equilibrium of the system formed by the superstructure, the foundation, the interface, and the soil, by considering that each of the elements in the system gives rise to one or several substructures.

The equations are formulated in an incremental form and solved by the method of residual loads due to material nonlinearities. Integration with respect to time is performed with a finite difference recurrent method.

4. RECOMENDATIONS FOR ADDITIONAL RESEARCH IN THE SOIL-STRUCTURE INTERACTION PROBLEM

Taking into consideration the current analytical soil-structure interaction models and those that are being developed, it is necessary to recommend that applied research should be carried out simultaneously in three major areas.

4.1 Soil Properties and Measurement Techniques

Laboratory testing is an integral part of soil-structure interaction problems. Many of our Analytical Models need to be fed with the proper parameters of the soil support and interfaces.

Specific areas requiring research are:

a) To develop reliable constitutive equations consistent with experimental data

b) To develop multidimensional testing techniques

c) The development of apparatus capable of accomodating more generalized stress states.

4.2 Experimental Modeling and Simulation

Experimental modeling and simulation are vital for a better understanding of basic elements of soil-structure interaction problems.

Specific areas requiring research are:

a) To develop proper instrumentation for the reliable measuring

b) The development of large facilities for testing models.

4.3 Measurement Techniques in the Field

Field measurement offer the most direct method for obtaining useful data of the parameters governing the soil-structure phenomena and to verify the accuracy of analytical models.

Specific areas requiring research are:

a) The development of field instrumentation

b) The development of field techniques for evaluating *in situ* static and dynamic state of stress

c) The investigation and evaluation of more case histories.

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M. Bozozuk, Panelist

TOLERABLE MOVEMENTS OF BRIDGE FOUNDATIONS
Movements Admissibles des Fondations de Ponts

In 1975 the Transportation Research Board's Committee on Foundations and Bridges and Other Structures, Washington, D.C., conducted a performance survey of highway bridges in the United States of America and Canada to determine the magnitude and nature of foundation movements that the structures could tolerate. A questionnaire was prepared and circulated to all highway and bridge departments, public works agencies and research organizations in every state and province in the two countries. The following information was requested for each bridge: type of bridge (steel, concrete, continuous or single-span, etc.), description of foundations for abutments and piers, description of subsoils, magnitude and description of foundation movements, kind of maintenance required, and whether or not the movements were tolerable.

It was clear from the replies that where little maintenance was required the performance was rated "tolerable" no matter how small or large the movements were. On the other hand, they were "not tolerable" where considerable maintenance or repairs were required.

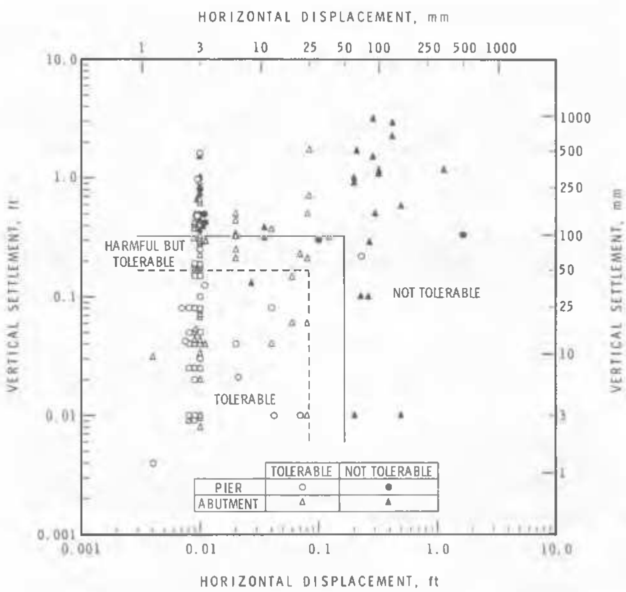

Figure. 1. Engineering performance of bridge abutments and piers on spread footings.

An analysis of the survey was carried out to relate performance to type of foundation (Bozozuk, 1978). Type or size of bridge was not considered at this time.

Three basic types of foundation were used to support the abutments and piers:

1. spread footings placed on fill, natural ground, or bedrock;

2. friction piles, all types (wood, concrete, steel pipe or H) driven through fill or natural ground;

3. end-bearing piles, all types bearing on rock or in a resistant soil formation.

The recorded vertical and horizontal movements and the rated tolerable, harmful but tolerable, and not tolerable movements reported in the questionnaires for each type of foundation are shown in Figures 1 to 3. These include about 120 cases of abutments and piers on spread footings, 60 on friction piles, and 90 on end-bearing piles. Vertical settlements of abutments and piers on spread footings varied from 0 to over 1 m, on friction piles from 0 to over 1.2 m (including up to 1.1 m of frost heave), and on end-bearing piles from 0 to 1.1 m. Horizontal movements of abutments and piers on spread footings ranged from 0 to over 0.5 m, and of those supported on friction and end-bearing piles from 0 to 0.5 m.

A consistent pattern of tolerable and non-tolerable movements is evident in the three figures, permitting the following classification, which applied to 85% of all the cases reported in the questionnaire.

Tolerable or acceptable	$S_V < 50$ mm
	$S_H < 25$ mm
Harmful but tolerable	$50 \text{ mm} \leq S_V \leq 100$ mm
	$25 \text{ mm} \leq S_H \leq 50$ mm
Not tolerable	$S_V > 100$ mm
	$S_H > 50$ mm

where S_V and S_H are vertical and horizontal movements, respectively.

The following observations on movements of abutments and piers were noted:

1. Large vertical and horizontal uniform movements were often tolerated.

2. Differential or rotational movements were more damaging than uniform movements.

3. Bridges are more sensitive to large horizontal movements than to large vertical movements.

The Federal Highway Administration (FHWA) of the United States supported a detailed study of bridge movements, taking into account type and size of bridge, at West Virginia University. Preliminary reports by GangaRao and Moulton (1980) and Moulton et al. (1980 a, b) provided the following conclusions:

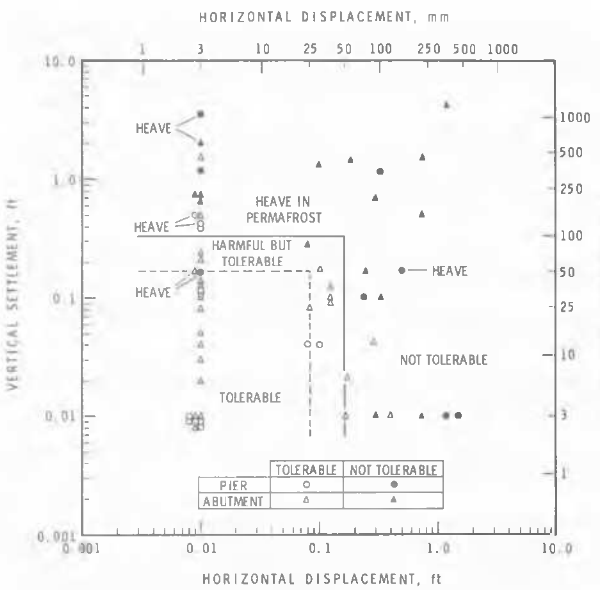

Figure 2. Engineering Performance of Bridge Abutments and Piers on Friction Piles.

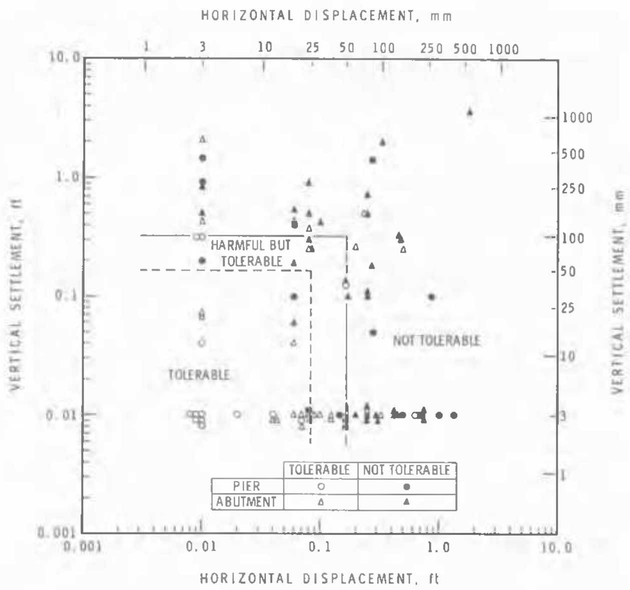

Figure 3. Engineering Performance of Bridge Abutments and Piers on End-bearing Piles.

1. In general, abutments moved more frequently than piers.

2. Depending on type, length, and stiffness of spans, many bridges can tolerate significant total and differential vertical settlement without becoming seriously overstressed.

3. Horizontal and rotational movements of abutments and piers are more serious than vertical movements.

4. Tolerable limits of longitudinal angular distortion for continuous and simply supported bridges are 0.004 and 0.005, respectively.

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M. Adam (Written discussion)

INCIDENCE DU FLUAGE SUR LE COMPORTEMENT DES OUVRAGES
Influence of Creep on Behaviour of Constructions

Commentaire sur les Exposés du Dr Bozozuk et du Prof. Kérisel

Il me parait surprenant que les tassements tolérables pour les ponts come pour les autres édifices soient rapportés uniquement à des critères géometriques: *le temps joue aussi un rôle important, et il faut tenir compte du fluage qui, dans le comportement des structures, est un phénomñe lent.*

Autrefois certaines constructions anciennes ont toléré des déformations importantes (exemples classiques à Mexico, Amsterdam. Paris...) sans qu'il y ait eu d'autres d'adaption à effectuer que des remises en jeu des menuiseries, ceci tout simplement parce que les affaissement se produisaient lentement.

Par contre aujourd'hui, à l'occasion de travaux souterrains souvent rapides, comme le creusement d'un tunnel pour métro en site urbain, la structure n'a pas le temps de fluer et se fissure de manière parfois dramatique.

Je suggère que le critère concernant les possibilités de fluage des structures soit pris en considération par les chercheurs.

M.C. Goel (Written discussion)

Discussion on "SOIL-STRUCTURE INTERACTION SETTLEMENT, PREDICTION FOR WELL FOUNDATION"

During the panel discussion, a question was raised as to what should be the permissible settlement for bridges and the adequacy of the adoption of distortion criteria(δ/L).The writer has a chance to perform the prototype load test on 1.25 m thick R.C.C. circular well of 5 m outer diameter sunk to the foundation level of Gomti Aqueduct in India for predicting the settlement and load bearing capacity of the well foundation. In this discussion,the criteria adopted for settlement analysis is presented with a view to share the writer's experience with others.

Gomti aqueduct consisting of 12 bays each of 32.5 m span is founded on 37 m deep 12 m x 27 m D-shaped wells. The strata along the well comprises interbeddings of clay, sand and sand mixed with silt along with traces of Kankar. The strata below the well foundation is normally loaded clay of low plasticity.

Total settlement in the clayey foundation is the sum of immediate settlement and the consolidation settlement. The immediate settlement was worked out from load settlement curve after making allowance to local leave which occurred due to the removal of the vertical loading during well sinking operation. Thin settlement for prototype D-shaped well was calculated to be 5.7 cm for design loads.

The consolidation settlement was computed from e-p curves obtained on undisturbed samples of foundation clay for load in excess of the existing surcharge at foundation level. The cross-section of the river indicated that at the location of various piers, the river bed varied between El.98 and 106.0 m. The settlement calculated by Terzaghi's one dimensional consolidation theory at El98,100,102,103.5 and 106.0 m, was found to 11.5,9.4,8.2, 7.4 & 5.3 cm respectively.

Since this is a case of hydraulic structure, excessive settlements can not be tolerated. If the immediate settlement calculated for the entire design load is added to consolidation settlement, the total settlement would have been in excess of the permissible settlement. However, 80% of the total design load comprised dead load of foundation and superstructure causing 3.8 cm settlement at the end of construction itself and as such, immediate settlement due to the water load would be only 1.9 cm. Further, the calculations for consolidation settlement indicated that because of change in river bed elevation,the settlement would vary from 5.3 to 11.5 cm, nevertheless, the differential settlement in the adjacent piers would be smaller. Assuming that the differential settlement to be of the order of 50% of the total sttlement as per conventional assumption, the maximum differential settlement would be 6-7 cm which would cause maximum angular distortion (δ/L) of 1/500 and the same was considered as acceptable. The possibility of differential settlement due variation of strata below the pier, was considered remote as any such tendency was likely to be resisted by lateral resistance arising from such deep embedment of wells. Further, in order to minimize deviation of aqueduct floor from design level as a result of settlement, the piers were raised by about 6 cm above the required level before placing aqueduct section.

A. Van Wambeke (Oral discussion)

We have been discussing the problem of the effect of differential settlements on a structure and underlining the various difficulties that arise during their estimation.

I would like to recall the heritage of the late Mr Louis Menard, which is a method for calculating these settlements using three essential parameters.

- the heterogeneity of the soil, taken from the heterogeneity index of the specific settlements (settlement of a strip footing 1 m wide at 1 m depth derived from the pressuremeter data for each borehole)
- the structural rigidity given as a coefficient Kn
- the sensitivity of the structure to the effects of differential settlement: tolerances are adapted as a function of structural rigidity and the materials used (say, concrete, steel, etc ....)

This method is the only evaluation of the phenomenon in existance, even though it is a pragmatic quantification of the ground conditions.

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M. Wallays (Written discussion)

STRUCTURE-FOUNDATION: SOIL INTERACTION AND SETTLEMENT CRITERIA

Kerisel's oral intervention and the afterwards oral discussion

Kérisel a judicieusement proposé de définir les critères de la limite de la déformation acceptable des constructions en fonction de la courbure plutôt qu'en fonction du tassement différentiel.

Ce point de vue est inattaquable. En effet, la résistance des matériaux montre que, notamment pour les poutres et les dalles, les contraintes, et donc aussi celles provoquant des désordres, sont liées à la courbure. Il en est de même lorsqu'il s'agit d'une structure fondée sur semelles isolées. En effet, chacun sent, et le calcul demontrerait,que la situation de la fig.1a, qui correspond à une plus grande courbure, est plus dangereuse que celle de la fig. 1b, qui correspond à une plus faible courbure, alors que le plus grand tassement différentiel relatif est le même.

Comme cela a été souligné, il est exact que,si généralement la prévision du tassement différentiel est déjà difficile, voire très difficile dans certains cas, la prévision de la courbure l'est encore bien davantage, puisqu'elle correspond à un degré supplémentaire de différenciation.

On ne progressera cependant dans l'établissement de critères précis concernant la limite de la déformation acceptable des constructions,que si lors de travaux importants ou délicats, on prévoit de mettre en place les points de mesure,de manière à obtenir des données valables sur les courbures prises dans différents plans.

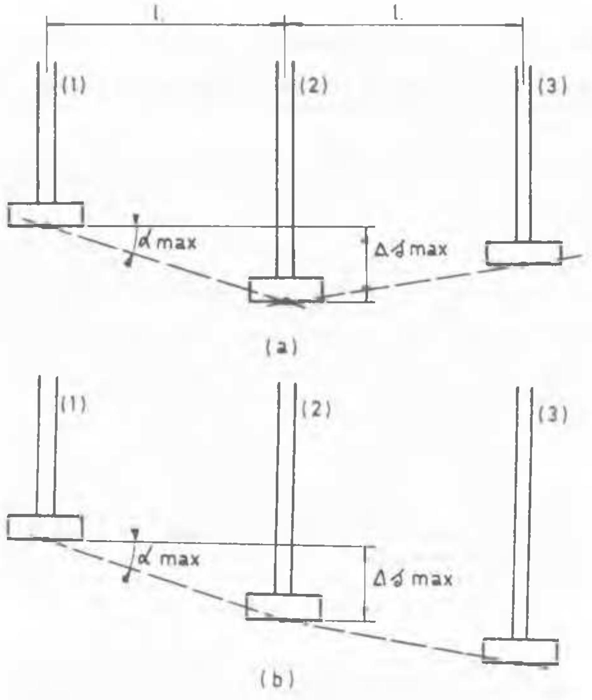

Fig. 1

M. Bozozuk, Panelist

SIMPLE METHODS OF ESTIMATING ENGINEERING BEHAVIOUR
Méthodes Simples pour Estimer le Comportement Géotechnique

For a given soil-structure interaction problem there may be simple methods of estimating engineering behaviour that do not require the use of a computer. Some methods do produce good results, but accuracy depends upon the problem, the complexity of the soil, and personal experience with the method. Terzaghi and others gave many simple models, some of which are illustrated by the following examples.

The contact pressure under a rigid raft foundation on clay can be estimated by the following rule of thumb: Edge pressure is about three times that at the centre, provided applied stresses are within the allowable working stress of the soil, i.e., there is no yielding by the soil.

A second problem is one of estimating excess pore water pressures beneath an embankment on sensitive marine clay (Law and Bozozuk, 1979). An elastic analysis is applied using a hand calculator and the influence charts published by Poulos (1967) to determine the increase in vertical stress, taking into account the hard stratum at depth. The appropriate factors are depth to hard stratum (Z), thickness of the compressible layer (h), width of the loaded area (B), and Poisson's ratio. For the typical embankment shown in Figure 1, part of the foundation soil is stressed beyond its undrained

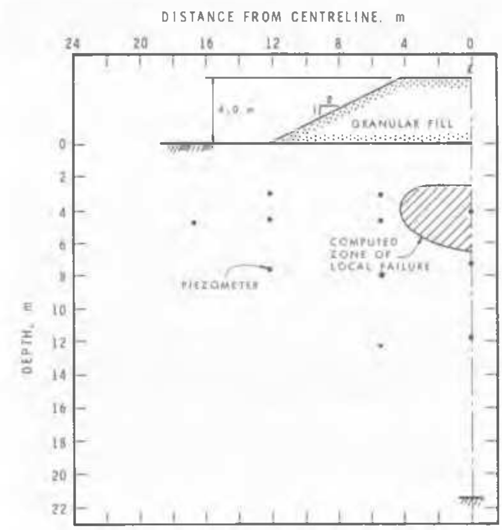

Figure 1. Zone of Local Failure Under the 4.0 m High Boundary Road Embankment

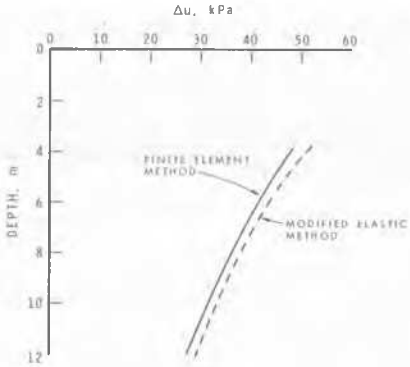

Figure 2. Estimated Excess Pore Water Pressures Below Centre of Embankment

strength. The elastic analysis is therefore not completely applicable and the method was modified, leading to the following equation for excess pore water pressures below the centreline of the embankment:

$$\Delta U = \Delta\sigma_v + \left\{\frac{1 + K_o}{2}\right\}\sigma'_{vo} - S_u$$

where $\Delta\sigma_v$ = change in vertical stress determined from the influence charts

σ'_{vo} = initial in situ vertical effective stress

K_o = coefficient of earth pressure at rest

S_u = undrained shear strength.

The estimated pore water pressures were compared with those calculated by means of a finite element analysis, taking into account embankment rigidity. The modified elastic method compared very well below the centre (Figure 2) and, in fact, was quite good below the shoulder (Figure 3).

A third example deals with estimating the bearing capacity of friction piles in silt or clay soils (Bozozuk, Keenan and Pheeney, 1979).

The ultimate bearing capacity is given by

$$Q_a = Q_s + Q_p - W_p$$

where Q_s = bearing capacity due to shaft friction

Q_p = bearing capacity at tip of pile

W_p = weight of pile.

The bearing capacity of the pile tip can be estimated from

$$Q_p = N_q \sigma'_v A_p$$

The unit shaft friction is given by

$$q_s = k_s \gamma' z \tan \delta'$$

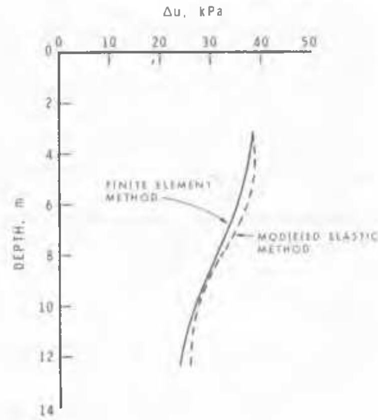

Figure 3. Estimated Excess Pore Water Pressures Below Shoulder of Embankment

where k_s = relation between horizontal effective stress acting on the pile and the vertical effective stress in the soil at that point

z = depth below ground surface

γ' = effective unit weight of soil

$\tan \delta'$ = coefficient of friction between the pile and the soil

Intergrating this equation over the full depth of the pile gives

$$Q_s = \frac{1}{2} C \beta (L^2 - D^2)$$

where C = circumference or perimeter of pile shaft in contact with the soil

L = buried length of pile

D = depth below which positive skin friction is mobilized. (If there are no voids around the pile shaft, or no negative skin friction, $D = 0$.)

β is a constant that can be evaluated from the following equation if the parameters are known

$$\beta = k_s \gamma' \tan \delta'$$

Since β is constant for a pile subject to compression or pull-out loads, the following parametric equation relates the mobilized shaft friction for the two types of loading

$$P_P = \left\{\frac{1 + \nu}{1 - \nu}\right\} P_T$$

where P_P = total shaft friction load due to positive skin friction (compression pile)

P_T = ditto (tension pile)

ν = Poisson's ratio of pile material.

To illustrate, a Hercules H800, 30 cm diameter precast concrete pile was driven 18 m into marine clay. The pull-out load was 51.4 t, the estimated P_p = 70 t and the measured P_p = 80 t.

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C. Viggiani, Co-Reporter

SIMPLE METHODS FOR SOIL-FOUNDATION-STRUCTURE INTERACTION ANALYSIS

SYNOPSIS The role of simple and sophisticated soil-foundation-structure interaction analyses is discussed on the basis of simple examples. It is argued that sophisticated analyses should not be used as a substitute of engineering judgement, and that a proper use of simple methods of analysis may still be very valuable.

INTRODUCTION

In carrying out any analysis of soil-structure interaction, one should always keep in mind that the system to be analysed (fig. 1) is com= posed by three mutually interacting components:

- the subsoil
- the foundation
- the superstructure

The unknown quantities, hence, are not only the contact stresses between soil and foundation, but also the forces acting on the foundation.

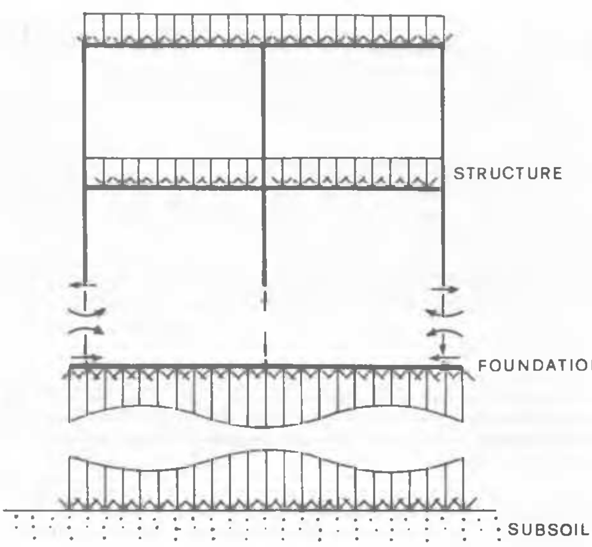

Fig. 1. Schematic representation of soil-foundation-stru= cture interaction.

There is a widespread belief that the uncerta= inties in modelling and analysing are mainly concentrated in the subsoil. As in the ancient geographical maps of black Africa, in the sub= soil region one could write: "hic sunt leones" (here are dragoons).
However, it has been recently realized (Burland et al., 1977) that dragoons are by no means li= mited to the subsoil; the behaviour of a buil= ding structure is also very complex and diffi= cult to model, due to the combined influence of factors as the construction sequence, the presence of non-bearing components (infilling panels, claddings), the creep of materials.

The analysis can be carried out with different objectives: first, the evaluation of the form and magnitude of the relative deflection, used to assess the likelihood of damage and to inve= stigate the merits of alternative foundation and structural solution; second, the prediction of forces and stresses within the structure.
It is important to distinguish between the two objectives, since a simplified analysis may reveal quite suitable for one of them and com= pletely misleading for the other one.

SOIL-FOUNDATION INTERACTION

Concerning the second, more specialised objec= tive, the present days routine procedure is that of neglecting the influence of the super= structure. It is assumed that the loads acting on the foundation are not influenced by the re= lative settlement; accordingly, they are calcu= lated by a structural analysis with fixed sup= ports.
The soil-foundation interaction is then analy= sed under a known system of loads.

Considerable efforts have been spent in refining this step of the analysis, and available methods range from the simple Winkler model (linearly elastic unconnected springs) to sophisticated non-linear elasto-visco-plastic finite elements analyses.

A relatively refined linearly elastic subsoil model is represented in fig. 2; an elastic layer of finite thickness H resting on a rigid base and whose modulus increases linearly with depth (Esposito et al., 1978). Such a model is charac= terized by four parameters, namely the relative stiffness RS = $E_f h^3/E_o L^3$, the relative thickness

H/B, the degree of heterogeneity E_o/mL and the Poisson ratio of the soil ν.

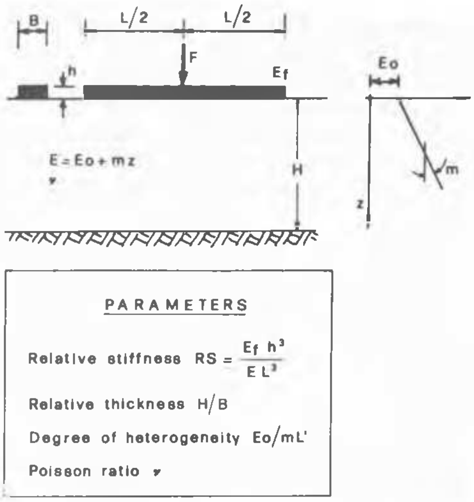

Fig. 2. Subsoil modelled as an elastic layer of finite thickness, resting on a rigid base and whose mo= dulus increases linearly with depth.

It may be shown, however, that the results are not very sensitive to variations of parameters values, except the relative stiffness RS. In fig. 3, for instance, the values of the ben= ding moment at mid span of a foundation beam loaded by a concentrated force at the center are reported. Similar results are obtained with different load distibutions, though the sensiti=

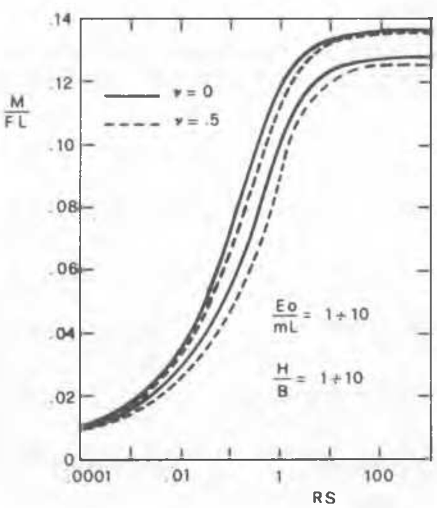

Fig. 3. Maximum bending moment in a foundation beam loaded by a concentrated force at mid span and resting on a soil modelled as in fig. 2.

vity of results to parameters values is somewhat higher in the case of uniformly distributed load

Let us consider now (fig. 4) a relatively rigid beam loaded by eleven equal and equally spaced concentrated forces. Rather surprisingly, the beam undergoes a hogging deflection, with the concavity downward.
If the two forces at the beam ends are removed, the deflection pattern changes completely. The bending moments change in sign and take values

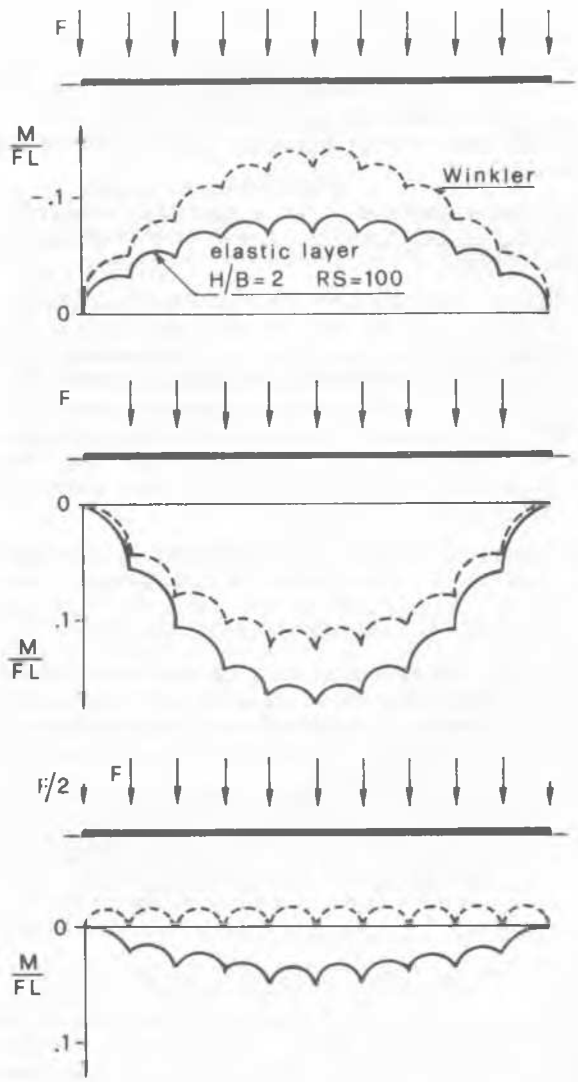

Fig. 4. Bending moments in a relatively stiff foundation beam subjected to slightly different load distri= butions and resting on different soil models. The equivalence between the two soil models is obta= ined by equating the average settlement.

larger than before, though the total acting load is smaller.
If the two forces at the beam ends are given an intensity equal to half that of the other for= ces, the bending moments tend to disappear.
It is to be underlined that two different soil models, as the Winkler springs bed and the ela= stic layer, predict the same behaviour even if the values of the bending moments obtained by the two models are rather different.

As it is well known, the Winkler model is not suited for the analysis of uniformly distribu= ted loads; this is the reason of the above dif= ferences, since an increasing number of concen= trated loads tend to coincide with a uniformly distributed load.
As a matter of fact, for the usual loading con= dition with a few widely spaced forces, the two models give practically coincident results, provided the parameters values are properly se= lected.

The results reported, and a thorough analysis of many similar results, bring to the following conclusions:

- within the framework of linearly elastic models, the results of soil-foundation in= teraction analysis are not very sensitive to parameters values, so that a proper se= lection of these values appears possible. The only notable exception is the relative stiffness RS; in a certain range, its in= fluence is very significant, thus requiring a very precise assessment;
- apparently minor modifications of the app= lied loads, under some circumstances, can affect significantly the results, irrespec= tive of the adopted subsoil model.

The above comments are very significant, if one bears in mind that the influence of the super= structure actually affects the relative stiff= ness RS and the distribution of the forces ac= ting on the foundation.
As a consequence, it appears that there is no need of further improvements of the soil-foun= dation interaction model, if the influence of the superstructure is not accounted for.

INFLUENCE OF THE SUPERSTRUCTURE

Method of the "equivalent stiffness"

Some Authors (Meyerhof, 1953; Sommer, 1965; De Simone, 1966; Mazzolani, 1967; Koenig, Sherif, 1975) suggest to make an allowance of the in= fluence of the superstructure by simply increa= sing the stiffness of the foundation to an "equivalent stiffness" accounting, in some way, for the existence of the superstructure.
The method is appealing for its simplicity and relatively widespread. Unfortunately, a more careful scrutiny reveals some serious drawbacks.

Firstly, the suggestions for the evaluation of the equivalent stiffness of the foundation-su= perstructure system are rather indefinite.
According to Koenig and Sherif (1975) the equi= valent stiffness is to be evaluated by simply summing up the moments of inertia of the foun= dation and of the floors if no doweling (shear resisting connections) exists; if the construc= tion is strengthened by walls running parallel to the bending axis, doweling can be assumed and the Steiner theorem (transfer formula) is to be applied.
El Kadi (1968), on the basis of some case histo= ries, claims that only the basement and two to three floors are involved.

Bearing in mind the sensitivity of results to the values of the relative stiffness, these un= certainties seriously affect the reliability of the method.
Furthermore, as it will be shown later, in some instances the method of equivalent stiffness leads to completely wrong results, if used for the prediction of forces and stresses in the structure.

It seems, therefore, that the use of an equiva= lent stiffness to this aim should be abandoned.

Method of the "limit situations"

It may be shown (Pozzati, 1953; Viggiani, 1978; Caputo, 1980) that the actual loads transmitted by the superstructure to the foundation are bounded by the values pertaining to two limit situations (fig. 5):

- the stiffness of the superstructure is very small in comparison to the stiffness of the foundation;
- the stiffness of the superstructure is much larger than the stiffness of the foundation.

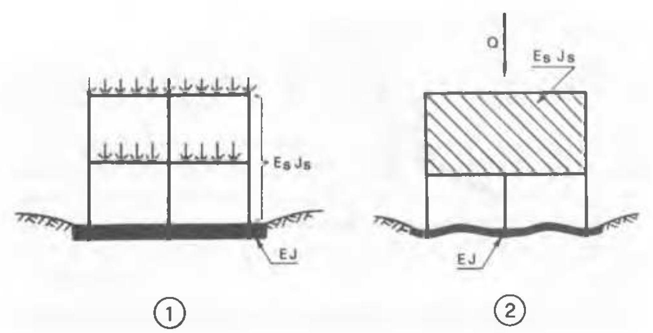

Fig. 5. Limit situations of foundation-structure relative stiffness. 1: EJ $\gg E_sJ_s$. 2. $E_sJ_s \gg$ EJ.

In these situations, the loads are easily obtained by straightforward calculations.

In the situation 1, being the structure very flexible, section forces within it are independent of relative settlements. The "routine procedure" previously outlined applies.
The loads on the foundation are obtained by a conventional structural analysis with fixed supports; they depend on the characteristics of the structure and on the external load distribution, but are independent of soil-foundation interaction.

In the situation 2, the loads transmitted by the superstructure to the foundation are obtained by imposing that the vertical displacements at the base of the columns are either equal (centered load resultant) or linearly variable (eccentric load resultant), and by satisfying global equilibrium. The loads depend on soil-foundation interaction, on the load resultant and its position, on the columns position; they are independent of the characteristics of the structure and of the external load distribution.

If the subsoil is modelled as a homogeneous elastic layer of finite thickness (model of fig. 2 with m = 0) the extensive tabulation published by Koenig and Sherif (1975) can be employed. In these tables, among other results, the values w_{ij} of the displacement induced in a point i of the foundation by a unit force acting in the point j of the same foundation beam can be found, as a function of H/B, L/B and RS.
If the load resultant Q is centered, one can write, for the n points at the base of the columns:

$$w = \sum_{j=1}^{n} Q_j \, w_{ij} \qquad (1)$$

where w is the settlement; and the equilibrium condition:

$$\sum_{j=1}^{n} Q_j = Q \qquad (2)$$

thus obtaining (n+1) equations in the (n+1) unknowns Q_j, w.

If the load resultant Q is eccentric, one can write:

$$w_i = \sum_{j=1}^{n} Q_j \, w_{ij} = w_o + \alpha_o \, x_i \qquad (3)$$

where w_o and α_o are respectively the displacement of the left extremity of the foundation and the rigid rotation of the system, and x_i is the distance of the point i from the left extremity. The equilibrium requires that:

$$\sum_{j=1}^{n} Q_j = Q \qquad (4)$$

$$\sum_{j=1}^{n} Q_j \, x_j = Q \, x \qquad (5)$$

Eqs. (3) to (5) are (n+2) equations in the (n+2)unknowns w_o, α_o and Q_j.

Some very simple examples are reported to illustrate the potential of the method.

Let us consider first a two bays portal frame with pin jointed columns, and four different load distributions (a to d in fig. 6). All the load distributions have a centered resultant; hence, the solution for the limit situation 2 (rigid superstructure) is the same, and is obtained by eqs. (1) and (2).

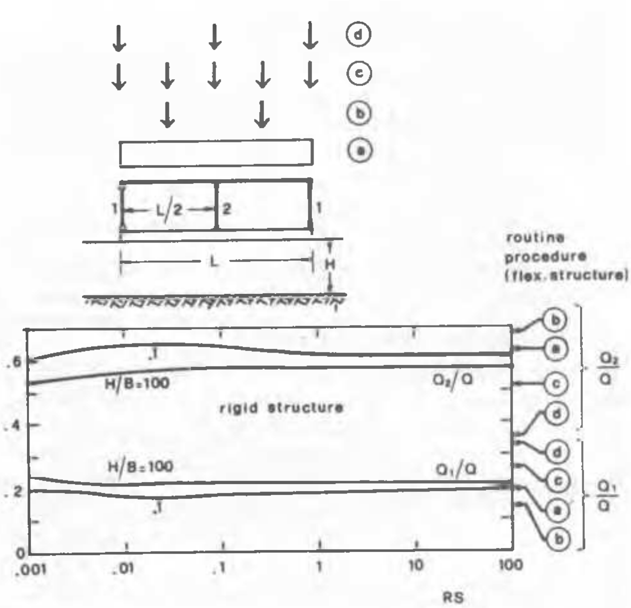

Fig. 6. Limit situations solution for a simple structural scheme.

The values of the columns loads Q_1 and Q_2 have been calculated for different values of the relative stiffness foundation-soil, and are plotted in fig. 6 against RS. It may be observed that they are practically unaffected by the subsoil (values of RS and of H/B).

The values of the columns loads in the other limit situation 1 (flexible superstructure) are independent of the subsoil, but are different for the different load distributions; they are reported by arrows on the right side of the diagram in fig. 6.

It may be seen that, for the load distribution a (uniformly distributed load), the two limit situations are practically coincident, irrespective of the subsoil characteristics; this means that the influence of the superstructure is not relevant in this case.
Completely different conclusions may be drawn for other load distribution; for instance, in the load distribution d (three concentrated forces on the columns), the range bounded by the two limits is very broad, and hence the influence of the superstructure is likely to be significant.

Furthermore, if the structural scheme is but slightly modified by adding two small cantile= vers at the ends of the foundation beam (fig. 7), the results obtained are once more signi= ficantly altered. The figure shows that there is, in this new scheme, a marked influence of RS and that the relations between the two li= mit situations are different from those of fig. 6. The influence of the superstructure, as depicted by the wide range between the two limits, appears significant.

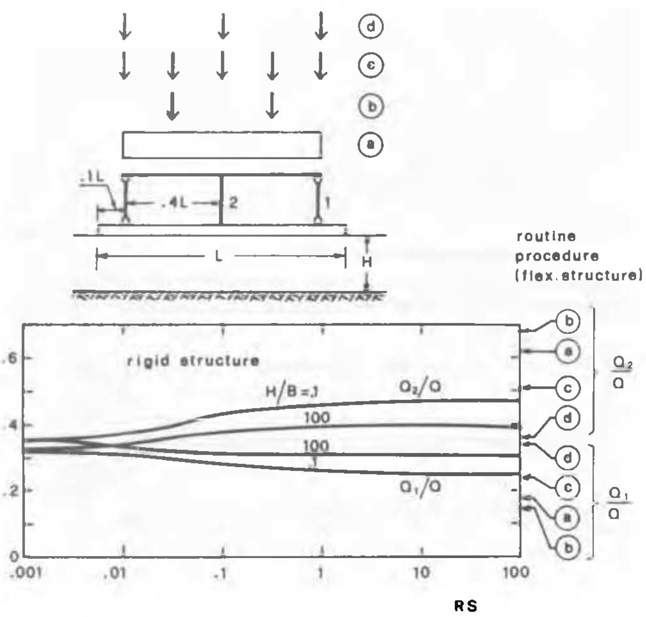

Fig. 7. Limit situations solution for a simple structu= ral scheme.

The simple examples shown, and many other sche= mes that have been analysed by the method of limit situations, allow some comments.

The influence of the superstructure on the soil-foundation interaction depends on many factors. Among them, the geometry of the structure and its characteristics; the characteristics of the subsoil; but also the distribution of external loads. At the writer's knowledge, this last factor had not yet been signaled.

The significance of each single factor may be large; as a matter of fact, apparently minor modifications of loads and/or geometry may si= gnificantly affect the results. The variety of possible combinations is such that no generali= zation is possible.

The method of limit situations has the advanta= ge of giving an insight of the relevance of the problem.
Actually, if the range of results bounded by the two limits is narrow, the influence of the superstructure is likely to be negligible. In these cases the routine procedure, neglecting it, may be adequate.

If, on the contrary, the range is broad, the influence of the superstructure is likely to be significant.
In some cases the actual situation may be reco= gnized to be close to the upper or lower limit. For instance, continuous foundations of framed structures for apartment buildings, as a rule, are close to limit 1 (flexible superstructure) both because of statics requirements and wide= spread preactice of providing relatively rigid foundations. In this sense, some "a posteriori" arguments in favour of the routine design pro= cedure may be found.

Foundations of reinforced concrete sylos cells groups, on the contrary, may be close to limit situation 2 (rigid superstructure).

In intermediate situations, one can't help but performing an analysis explicitly accounting for the superstructure.
In principle, such an analysis is both possible and relatively straightforward by a suitable computer program coupling a model of the subsoil (e.g., the elastic layer) to a suitable ideali= zation of the superstructure. Studies of incre= asing sophistication have been reported, inclu= ding time effects, non linearity and change of the stiffness during the construction; very ge= neral computer programs have been written.

However, the limitations of knowledge about ground and structure behaviour should always be considered, bearing in mind that a more so= phisticated analysis does not means necessarily a more reliable prediction.
With a statemente that is only apparently para= doxical, Burland (1981) expressed this concept by saying that "a design that relies on a very precise calculation is a bad design".

MISUSES OF SIMPLE METHODS

It has been stated that the equivalent stiffness approach, if used for the prediction of forces and stresses within the structure, may in some instances lead to completely wrong results. This claim may be substantiated with some sim= ple examples.

Let us consider the reinforced concrete sylo schematically represented in fig. 8, and let the stiffness of the superstructure be such that the equivalent stiffness of the overall structure, evaluated by one of the criteria suggested by various Authors, is rather high (say RS = 100).

The case is very close to the limit situation 2; furthermore, for this geometry and load di= stribution, the range bounded by the two limits is very narrow (see fig. 6). As a consequence, the loads and stresses obtained considering limit situation 2 (rigid superstructure) are very nearly exact.

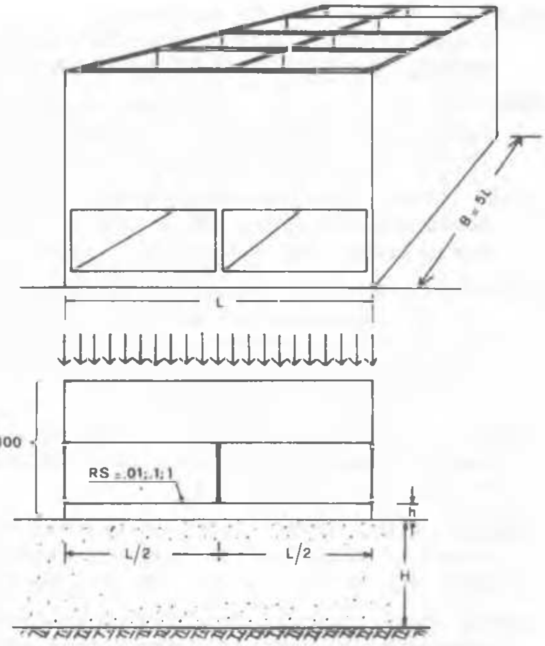

Fig. 8. Scheme of a reinforced concrete sylo

	RS	Q_1/Q	Q_2/Q
RIGID STRUCTURE	.01	.251	.498
	.1	.245	.510
	1	.244	.512
EQUIVALENT STIFFNESS	100	.188	.624

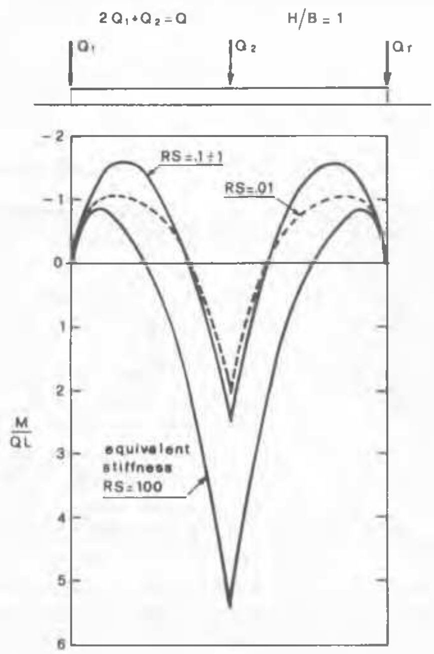

Fig. 9. Bending moments in the foundation raft of the sylo represented in fig. 8.

In fig. 9 the bending moments in the foundation plate, for three different values of the relative stiffness foundation-soil, are reported. They are completely different from the values obtained by the equivalent stiffness criterion, also reported in fig. 9. The latter, hence, are to be considered completely wrong.

The differences are still larger for the slightly different geometrical scheme of fig. 10.

These results constitute a warning against an improper use of simple methods of analysis. In this connection it is interesting to note that the same method of the equivalent stiffness, if used in the evaluation of the form and magnitude of relative deflections, seems to offer considerable merits and potential (Burland et al., 1977).

CONCLUDING REMARKS

Engineering analyses of a complex phenomenon, as the soil-foundation-structure interaction, are always more or less simplified, since they account for a few only of the many factors that affect the system behaviour (geometry, constitutive relations of the materials, construction sequence, applied loads, interface characteristics etc.). Furthermore, the difficulty of measurement and selection of the relevant parameters and the essentially random character of some factors (construction sequence; influence

	RS	Q_1/Q	Q_2/Q
RIGID STRUCTURE	.01	.348	.304
	.1	.345	.310
	1	.344	.312
EQUIVALENT STIFFNESS	100	.188	.624

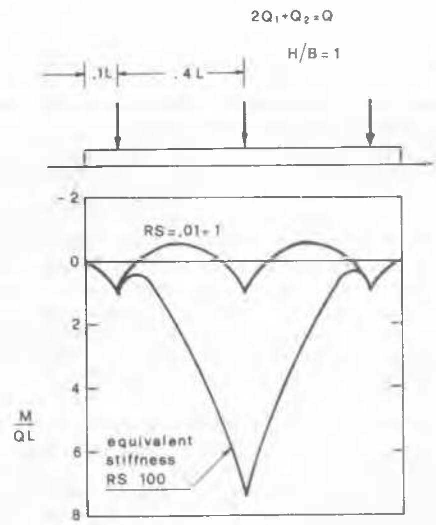

Fig. 10. Bending moments in the foundation raft of the sylo represented in fig. 8, with two cantilever extensions added at the sides of the raft.

of non bearing components; subsoil geometry and heterogeneity) hinder a completely reliable de= terministic prediction of the system behaviour.

In the writer's opinion, the present state of the Art in soil-structure interaction studies is such that the role of engineering judgement in the selection of the most suited model and of the relevant parameters values is still very significant. In this situation, a proper use of simple methods of analysis is believed to be valuable.

Sophisticated analyses should not be seen as a specific design tool or a substitute for engi= neering judgement. Their role in foundation engineering is rather that of producing parame= tric and sensitivity studies, in order to deve= lop a deeper understanding of the influence of the various factors.

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Kien Du Thinh (Oral discussion)

THE BUILT-IN STIFFNESS OF SPREAD FOOTINGS
La Fixité des Semelles Isolées

The problem of built-in stiffness of spread footings is very often met in frames.

In engineering practice the idealized situation of hinged (e.g. zero stiffness) or fixed end (e.g. infinite stiffness) supports is usually assumed without any quantitative consideration of the embedment depth of the footings. The results of the structural analysis depend very much on a correct estimation of the support conditions. The great variation of the bending moments $M_A \div M_D$ of a frame ABCD with the built-in stiffness S, especially in the range $0.1 \div 10$ [kNm/1], is shown in figure 1. Note that M_B changes the sign, e.g. a risk of putting re-inforcements on the wrong side exists.

The built-in stiffness determines also the dynamic behaviour of a frame and the results of stability analyses. In the case of prefabricated frame constructions, which used to have hinged corners, a stability analysis is meaningless (figure 2) unless the built-in stiffness is known.

In order to solve the problem, we have carried out tests with model footings in fine, silty

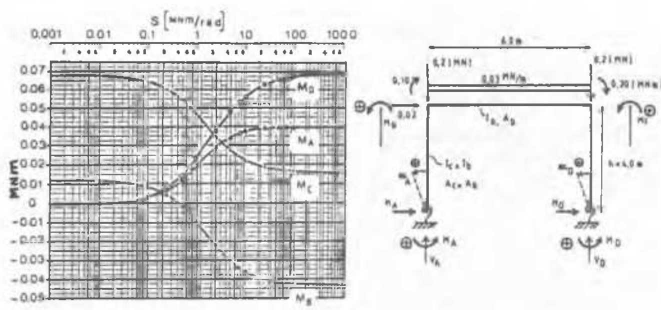
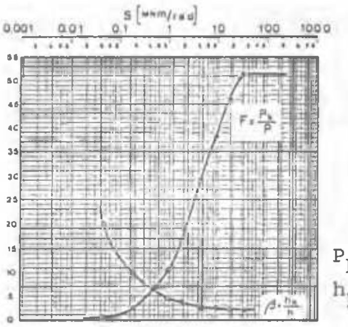


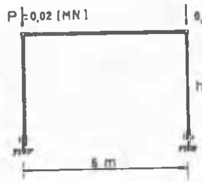
Fig 1 Bending moments vs. built-in stiffness

sand. The model footings are square or rectangular in shape and have an area of 900 cm^2.

The non-linear decrease of the built-in stiffness with the loading moment and its increase with the embedment depths D = 0(+), 1B(o), 2B(*), (B = width of footing) is illustrated in figure 3.

Further analyses show that the moment-rotation





P_k ... buckling load

h_k ... buckling height

Fig 2 Safety against buckling vs. built-in stiffness

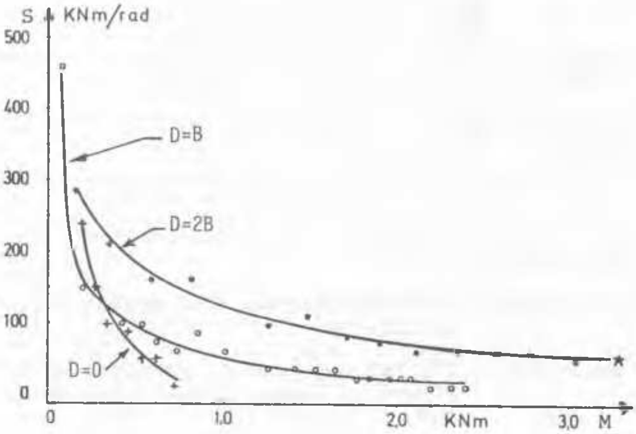


Fig 3 Variation of built-in stiffness S with loading moment M and embedment depth D. (Test results with square footing)

Z.J. Lu (Oral discussion)

THE INFLUENCE OF DEPTH ON THE BEARING CAPACITY OF ANCHOR SLABS
(Additional Remarks to Paper 5/34, Vol. 2)

1. Theory in Literatures and Controversy

For over a hundred years, anchor slabs have been used in the construction of wharfs. According to Terzaghi (1942), the bearing capacity of anchor slabs whose depth of embeddment is shallow ($H < 4.5b$) may be determined by the theory of Coulomb's passive earth pressure, while the bearing capacity of deep anchor slab is determined in the same way as a footing embedded at the same depth. It was thus believed that, theoretically, the bearing capacity of anchor slabs should be proportional to the square of their depth of embeddment.

During the seventies, Neely (1973), Ranjan(1974) and Das(1975, 1977) had proved ,respectively, in their model tests that the bearing capacity of anchor slabs increases with the 1.7^{th} -1.9^{th} power of depth factor (H/b). However, some model tests had revealed that there is a critical depth beyond which the bearing capacity will cease to increase. Neely(1975) disagreed to the critical depth, and he believed that the bearing capacity will continue to increase with depth. Therefore, the question was brought up whether there is a critical depth and what the critical depth will be. This remains to be a matter of controversy.

2. In-situ test results

In the period of 1977-1980, several groups of in-situ test were carried out under guidance of the author at six different engineering sites. A part of the results can be seen in the attached figure, in which the bearing capacities of 1m×1m anchor slabs embedded at different depths are given. In view of the facts that no excessive deformation should be allowed in an anchor slab structure, the author has adopted a displacement of 100mm to be the criterion of ultimate bearing capacity.

relationship may be expressed with accuracy as a hyperbolic function.

The secant built-in stiffness modulus can thus be written

$$S = \frac{M}{\alpha} = \frac{1}{a\alpha + b} \qquad \text{(figure 4)}$$

where M = moment
α = rotation of the footing
a,b = constants

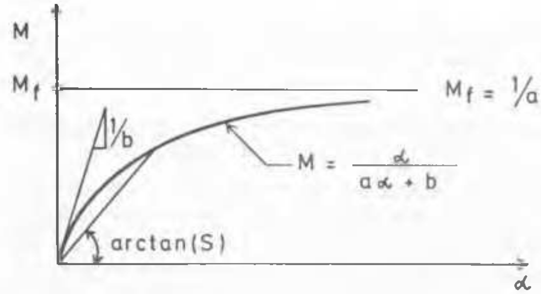
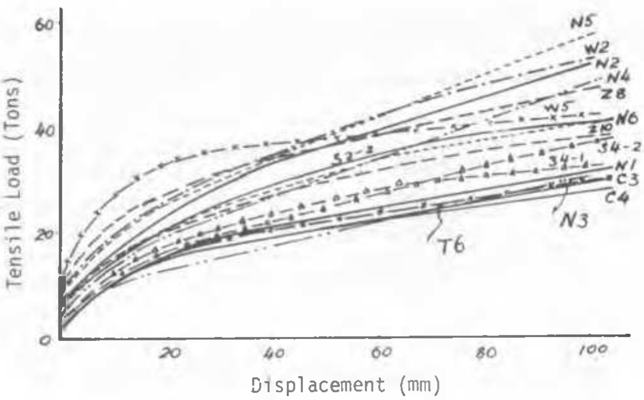


Fig 4 Moment-rotation relationship

This formulation is advantageous because of (I) its simplicity and (II) the clear physical meaning of the coefficients a and b. The failure moment is related to 1/a, and the initial built-in stiffness is equal to 1/b. Both coefficients can be determined by an in-situ test with the Ménard Pressuremeter.



Load-Displacement Curves of 1m×1m Anchor Slabs (In-situ Test Results)

Despite the scattering of results, it still can be seen from the figure that, the influence of depth is by far smaller than expected. The ultimate bearing capacity of anchor slabs is by no means proportional to the square of their depth, nor does it increase linearly with depth. Table I, Fig.I

3. The author's opinion

Therefore, the in-situ tests arrived at a conclusion that, both the classical theory of bearing capacity and the empirical formulae from model tests are invalid for deep seated anchor slabs.

The classical theories of bearing capacity were derived with the traditional assumption that the vertical compressive stress in soils is equal to it's overburden pressure. But if arch action happens in the deep layer of soil, the assumption becomes invalid. In the author's opinion some sort of arch action must have happened during the process of deformation, and the stress distribution become very complicate arround a deep anchor slab. Research works on this topic are continuing.

Table: Results of in-situ test on 1m×1m Anchor Slabs

Site of Test	Test No..	Depth of Embeddment H(m)	Ultimate Bearing Capacity(t)	Remarks
Chang-chow	C3	3.0	28.0	
	C4	3.0	30.8	
Taiyuan	T6	6.0	29.0	
Nanping	N1	3.0	33.0	
	N2	3.0	52.0	
	N3	5.48	31.2	
	N4	5.48	48.2	
	N5	9.05	57.6	
	N6	9.05	40.8	
Siping	S4-1	2.3	34.0	
	S4-2	2.3	37.9	
	S2-2	3.3	41.6	
Zhang-chung	Z10	4.0	39.2	
	Z8	4.0	47.3	
Wu-chang	W2	4.0	52.7	
	W5	6.7	42.0	

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Ch. Veder (Oral discussion)

USE OF DIAPHRAGM WALL PANELS TO TRANSMIT THE THRUST OF THE ARCH OF A BRIDGE CONSTRUCTED OF REINFORCED CONCRETE (AUGARTEN BRIDGE; GRAZ; AUSTRIA)

GENERAL In situ measurements of the stresses transmitted to the soil by the laterally loaded diaphragm-wall panels made it possible to determine the distribution of skin friction and bottom stresses.

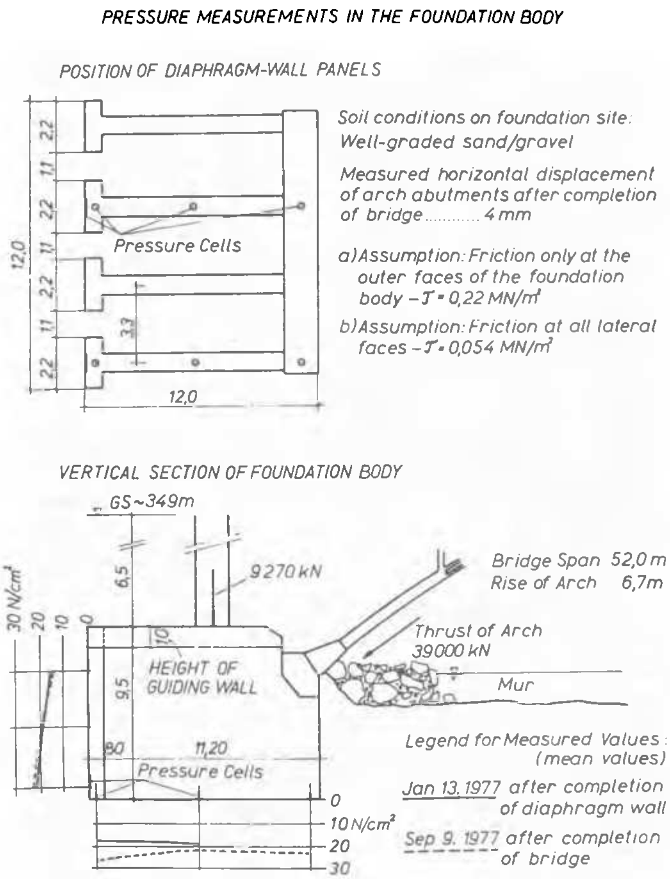
The Mur River passes through the town of Graz with a length of 14 km. To accommodate increasing traffic, an additional bridge was built, the Augarten Bridge. While the existing 8 bridges are girder bridges, spanning the river either self-supporting or with a central pier, a steel-reinforced concrete arch construction was chosen for the new bridge.

SUBSOIL CONDITIONS From the street surface down, the subsoil consisted to 4.70 m of man-made fill, from 4.70 to 24.00 m of densely deposited, well-graded sand and gravel (max. grain ⌀ approx. 20 cm; $\varphi = 33°$; $\gamma = 1.8\ t/m^3$; $\gamma' = 1.0\ t/m3$), and from 24.0 m down of densely deposited, overconsolidated, silty sand.

The ground-water table fluctuates approx. with the water level of the Mur River, at a mean depth of about 8 m below street surface.

DECRIPTION OF THE BRIDGE

Total length 65.2 m Arch thrust 39 000 kN



Width	18.5 m	Vertical load	
Span of arch	52.0 m	on abutment	9 270 kN
Rise of arch	6.7 m	Resultant	44 000 kN

The arch was centered after the System Cruciani.

CONSTRUCTION The foundations consist of 4 diaphragm-wall panels for each abutment; see Fig. On the landward side each panel was provided with a 2.2 m long transverse panel; on the side facing the river a transverse tie beam transmits the arch thrust of the bridge to the 4 panels.

To determine the magnitude and distribution of stresses transmitted from the construction to the soil by the diaphragm-wall panels, pressure cells of the system Glötzl, which function after the principle of movable membranes, were placed at the bottom of the diaphragm walls and the landward, vertical, transverse panels.

The first stress-strain measurements were carried out in January 1977, immediately after the concrete was poured into the bentonite-supported trenches, the second measurements in September 1977, that is after the thrust of the bridge arch had taken full effect.

As illustrated, after the concrete was poured the stresses at the bottom amounted to 0.18 to 0.20 MN/m^{2}2, at the transverse panels to 0.17 MN/m^2 (at the top) and 0.21 MN/m^2 (at the bottom), thus about corresponding to the weight of the concrete, part of which lies below the ground-water table.

The second measurements, taken after the thrust of the arch had taken full effect, showed a stress increase by an average of 0.03 MN/m^2, but only at the bottoms; practically no difference could be registered at the transverse panels

These observations correspond to those made at vertical load-bearing elements, namely that a relatively small load applied at the head of the elements is mainly taken up by skin friction and only to a very minor degree by bottom stress.

First assumption: The resultant of 44000 kN = 44 MN ist taken up by the outer faces of the foundation body alone, that is surface F_1 = 2 x (8.50 x 12) = 205 m^2 ; resultant friction

$$\tau_1 = \frac{44}{205} = 0.22 \text{ MN/m}^2.$$

Second assumption: The resultant is taken up by all side faces, that is surface F_2 = surface F_1 multiplied by 4 = 820 m^2; resultant friction τ_2 = 0.054 MN/m^2.

Since there is an adequate distance between the panels, i.e. 3.3 m mean distance, that is four times the 0.8-m-thickness of the panels, the determination of skin friction stress may be based on the second assumption.

The tables of ÖNORM B4440 list for "Large Bore Piles", reaching in densely deposited, cohesionless soil with a STP-value N_{30} = 30 to 50 to depths of 2 to 8 m below foundation surface, a skin-friction stress of 0.07 MN/m^2. Thus the values registered at the Augarten Bridge were far below the standard permissible values, as proved by the perfect behavior of the construction; after trial loading, an opposite displacement of the arch abutments by 8 mm was measured.

CONCLUSION For all projects with similar conditions, the diaphragm wall offers the technically as well as economically satisfactory foundation method.

B.R. Thamm (Oral discussion)

EARTH PRESSURE ON BRIDGE ABUTMENTS FOR HINGED BRIDGE SUPERSTRUCTURES

Abutments of hinged bridge superstructures have two main advantages when comparing with conventional bridge abutments:

- The abutment can be more slender since the horizontal forces from the superstructure are balanced directly by the backfilling. Thus the design of such abutments could be more economical.

- There is no need for a construction allowing for movements of the superstructure, since these movements are compensated by the backfill.

These advantages of bridge abutments with hinged bridge superstructures cannot fully be achieved because of great uncertainties with respect to magnitude and distribution of base and earth pressures and their interaction according to the movements of the abutment. Furthermore a hinged bridge superstructure undergoes daily and seasonal changes in temperature which results in movements of the abutment against or from the backfill.

To investigate the problem field measurements were undertaken for a 20 m single span bridge with a concrete abutment.

Fig. 1. View of mobil inclinometer

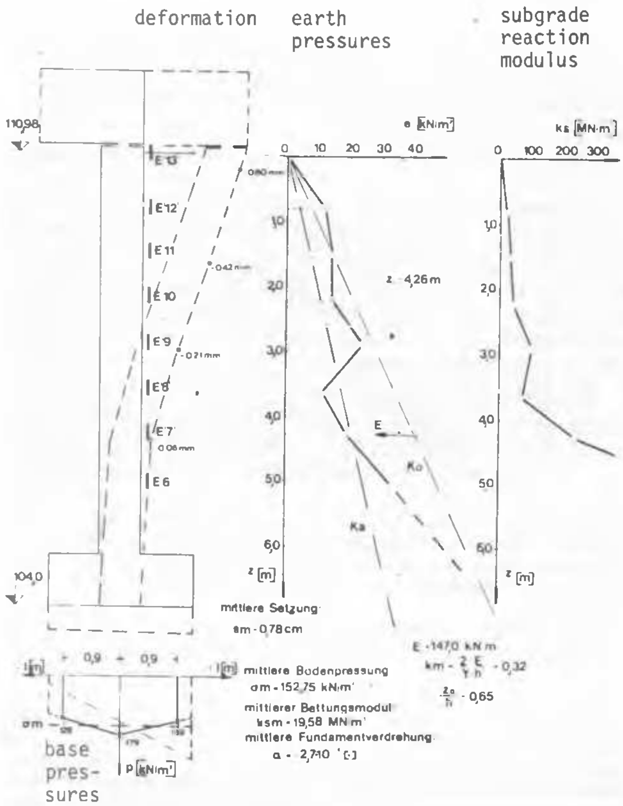


Fig. 2. Results when backfill is reaching hinge level

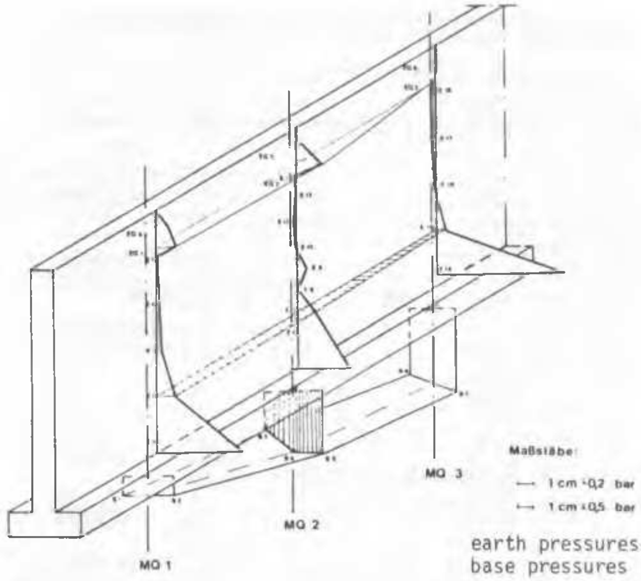


Fig. 3. Isoparametric view of base and earth pressures for end of backfill

Due to small changes in movements of the abutments a special measuring device (Fig. 1) had to be developed to guarantee an accuracy of about ± 0.02 mm (per 1 m).

The device consists of a mobil inclinometer which fits onto steel balls arranged on a plate suited at selected points of the abutment wall. During backfilling readings of base and earth pressures as well as deformations were taken.

Results are shown in Fig. 2 for the moment when the backfill is just reaching hinge level.

At this stage the abutment had moved against the backfill showing earth pressures greater than K_0-condition in the upper part. These pressures, however, will almost immediately be reduced and base pressures will increase at the front and decrease at the backfill side of the foundation.

After placement of the fill above hinge level base and earth pressures will rearrange again. In an isoparametric view (Fig. 3) the earth and base pressures are shown for the three lines of pressure cells placed behind the abutment. The wing is omitted at the left side for better understanding.

Earth pressures are high at the base and at hinge level, whereas in the middle of the abutment height less pressures than in the active case were measured. K_m is close above K_a. The base pressures show an almost flexible response of the whole abutment.

The backfill of the abutment was just finished before the conference. Longterm investigations of earth pressure changes due to temperature changes are planned in the future.

T. Matsui (Oral discussion)

EARTH PRESSURE ON PASSIVE PILES IN A ROW DUE TO LATERAL SOIL MOVEMENTS

I wish to discuss about the interaction between soil and piles in a row. It is well known that some of piles are subjected to lateral earth pressures due to soil movements. One of the most important points on such passive piles may be to accurately estimate the lateral earth pressure acting on piles due to soil movements.

We have already presented a theoretical equation to estimate the lateral earth pressure on the piles, considering pile interval as accurately as possible, and its validity was discussed for some field data (Ito and Matsui, 1975). In these a few years, we carried out a series of model tests on piles in a row for both clay and sand samples, to increase its reliability for various conditions of soils and piles. Fig.1 shows a schematic view of the test equipment used. This apparatus consists of a soil container box with model piles in a row and a laterally loading system of a pair of bellofram sylinders, which provides different pressures on both sides of soil mass, followed by lateral soil movements. Measurements of earth pressure on piles and soil displacement were carried out by load cells and displacement guages.

Fig.2 illustrates the plan views of the deformation behavior of soils around piles in two test cases, which were observed through a glass plate in the upper plate of the container box. Initial marks were located almost straight on mesh lines parallel to the direction of soil deformation. In this figure, the irregularity of the flow lines occurs mainly within a zone surrounded by brocken lines, which represent the plastic zone of the soil, assumed in our theoretical analysis.

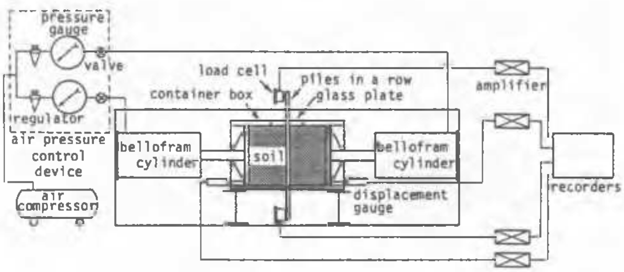


Fig.1 Schematic View of Test Equipment

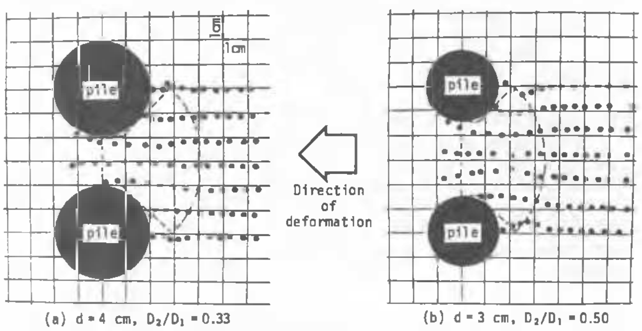


Fig.2 Plan View of Deformation Behavior of Soil around Piles

Therefore, the assumption on the plastic condition set up in our theory may be acceptable.

Fig.3 shows the relations between the interval ratio and the lateral force acting on a pile at a critical state, which is divided by the interval between piles, for both clay and sand. These points in this figure are the measured values, and solid curves represent the theoretical ones due to our theory. It is confirmed that all measured points, which include the results for various kinds of soil strengths, pile diameters and intervals between piles, give very good agreements with the theoretical curves. Thus, it can be concluded that the lateral earth pressure on passive piles when the soil just around piles becomes a plastic state can be estimated by our theoretical equation, with a sufficient reliability over a wide range of the interval between piles.

Finally, we have developed some design methods for such passive piles as stabilizing piles against landslide (Ito, Matsui and Hong, 1981a, b), foundation piles of landing pier in harbor Ito, Matsui and Hong, 1979) and so on, using our theoretical equation of lateral earth pressure. Consequently, the reliability of those design methods may be increased, I believe.

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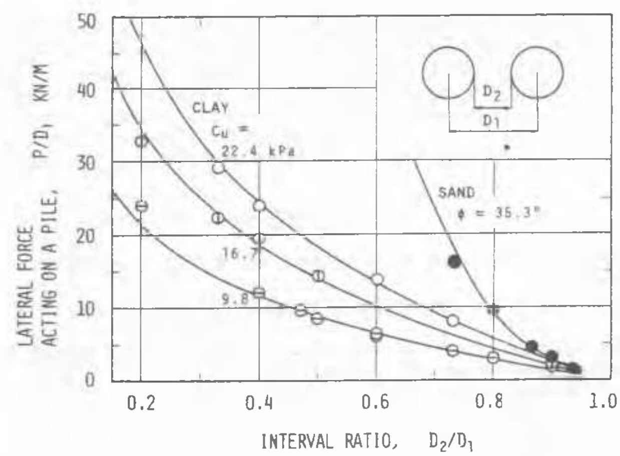


Fig.3 Relation between Lateral Force Acting on a Pile and Interval Ratio for Clay and Sand Specimens

T.K. Chaplin (Oral discussion)

Paper 5/10 by Broms, Ivmark & Mattson, on tubular elements as foundations (proposed by Mattson) is novel; it raises modelling and practical points.

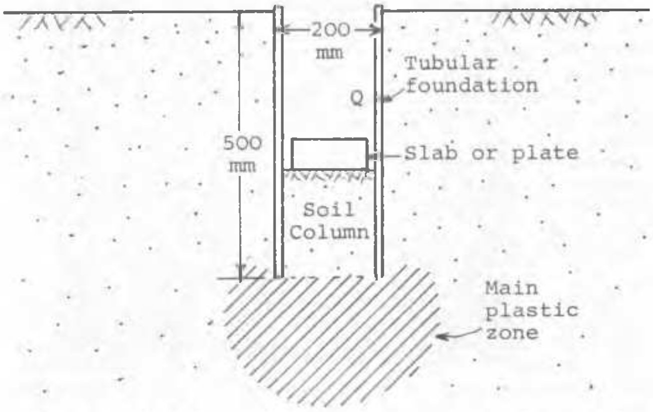


Fig.1 Model of Mattson's Proposed Foundation

As in Fig.8 of the paper, Fig.1 shows a model foundation 200 mm diam. by 500 mm deep. At the base, plastic yielding would occur; a model test should be designed to get detailed strains here, for comparison with finite element analysis as the General Report (Section 3.2) suggests.

Away from the cylinder, it would be helpful to know where deformation gives *localised shearing.* It seems to need, say, 200+ grains along a main dimension in sand (one diameter in triaxial test samples). Dilatancy could well be strong near a model cylinder. Farther away, except at small strains, localised shearing might arise in dense sand at any scale. Localised shearing might also occur in over-consolidated and quick clays. At full scale, strains might be restricted to soil near the foundation. To detect localised shearing, curvature-change tubes might be placed under and near a smaller foundation, Fig.2, or inclined in two sets for a building, Fig.3.

The nature of the soil filling inside a cylinder would presumably affect the column height needed to transfer full load from a slab or plate. The consolidation settlements of a soft clay column might cause trouble if the slab or plate rested directly on it without a granular layer below. The method seems attractive for soft clays, as tubular elements could be forced to large depths, excavated until slight heave begins, and finally completed with granular layer and bearing plate.

If the plug starts moving down before the final level is reached, or driving resistance is too high, the plug would be bored out as needed. In quick clay, special care would be essential.

With larger foundations, it is hard to imagine dense sand giving a peak resistance like a 200 mm diam. model, and in any case design would be based on a settlement criterion in practice. In tests by the writer, model piles of 12.7 mm diam. (0.5 in) in dense sand had no peak at less than 300 mm embedment. Scale effects may give large differences between model tests and full-scale foundations. Reports on this promising new system will be awaited with much interest.

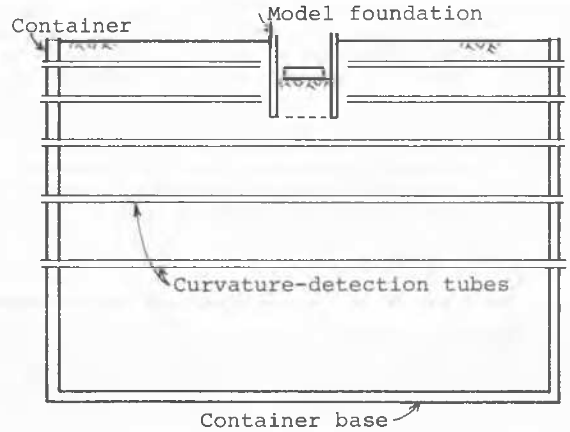
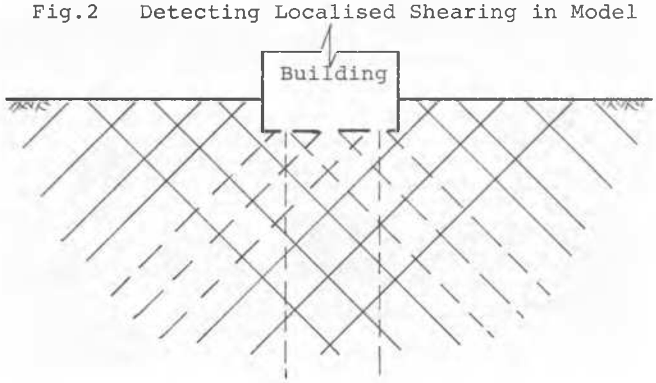


Fig.2 Detecting Localised Shearing in Model



Sets of inclined curvature-detection tubes

Fig.3 Localised Shearing Check under Building

F. Baguelin, F. Bourges, R. Frank and C. Mieussens (Oral discussion)

DIMENSIONNEMENT DES PIEUX SOUMIS AUX POUSSEES LATERALES DE SOL
Design of Piles Submitted to Lateral Soil Pressures

Une méthode prévisionnelle de calcul des pieux soumis aux poussées latérales de sol a récemment été recommandée dans l'ensemble des Laboratoires des Ponts et Chaussées (Bourges, Frank et Mieussens, 1980).

Cette méthode repose sur deux principes :
1°) la pression de réaction p(z) sur le pieu est une fonction de $\Delta y = y(z) g(z)$ ou y(z) est le déplacement horizontal d'équilibre sol-pieu cherché et q(z) est le déplacement horizontal "libre" du sol (Fig.1), principe proposé en 1973 tant par Marche, que par Poulos.

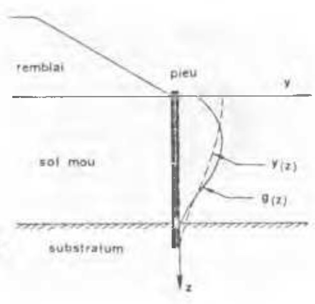


Fig.1 - Déplacement libre g(z) du sol et déplacement y(z) avec un pieu.

2°) la prise en compte de lois de réaction $p = f(\Delta y)$ non linéaires dans chaque couche de sol (nécessitant la résolution mathématique du problème par un processus itératif de convergence programmé sur ordinateur).

Trois problèmes se posent pour l'àpplication de cette méthode de prévision :
- la prévision des déplacements horizontaux libres du sol $g(z)$;
- la prévision des lois de réaction $p = f(\Delta y)$;
- les conditions aux limites en tête et surtout en pointe.

L'objet des récentes recommandations des LPC a justement été de proposer des règles précises de détermination de ces paramètres, indispensables à une méthode de prévision.

En ce qui concerne $g(z)$, la méthode retenue est issue de l'analyse des mesures de déplacements en pied d'une dizaine de remblais en vraie grandeur. Cette méthode permet de prévoir le déplacement horizontal "immédiat" correspondant à la mise en place du remblai, ainsi que l'évolution dans le temps. Il s'agit d'une méthode empirique et relativement grossière. Elle est néanmoins évolutive et peut être calée avantageusement par des mesures en place correspondant au projet en question.

Les lois de réaction non linéaires sont construites à partir d'essais pressiométriques :

- essais au pressiomètre standard Ménard (fig.2) qui fournissent des profils de E_M (module pressiométrique) et de p_l (pression limite). Le module de réaction $(E_s)_M = k_s.B$ (B : largeur du pieu) est calculé à partir de E_M et la pression ultime sur le pieu est prise égale à p_l.
- essai au pressiomètre autoforeur (PAF). On utilise directement la courbe d'expansion (p : pression dans la sonde ; $\Delta V/V_o$ volume injecté relatif) obtenue à l'essai de référence, pour construire la courbe de réaction, en doublant les déplacements par rapport à l'assimilation directe des deux courbes effort-déformation ($p-p_o$, $\Delta V/2V_o$) et (p, $2\Delta y/B$) (fig. 3) pour tenir compte de la lenteur relative du chargement du pieu par rapport à la vitesse de l'essai de référence.

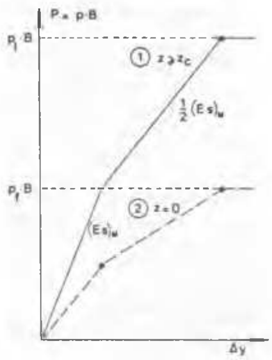


Fig.2 - Courbe de réaction: (1) au-dessous de la profondeur critique - (2) en surface.

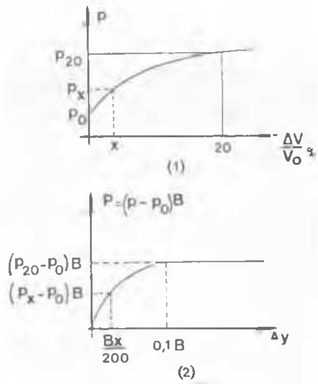


Fig.3 - Correspondance entre la courbe d'expansion du PAF (1) et la courbe de réaction du pieu (2).

Enfin, pour ce qui est des conditions en pointe, qui jouent un rôle important dans ce problème, quelle que soit la rigidité relative sol-pieu, on recommande d'introduire des lois de mobilisation du moment fléchissant et de l'effort tranchant en pointe, en fonction respectivement de $y'_p - g'_p$ et $y_p - g_p$. En attendant plus de données expérimentales sur ce type de lois, des schématisations simples sont proposées.

Cette méthode a été intégralement appliquée pour un certain nombre d'études courantes, ainsi que pour un pieu expérimental largement instrumenté (Bigot, Bourges et Frank, 1981). Elle a donné dans l'ensemble satisfaction. Cependant, un certain nombre de problèmes méritent encore largement d'être approfondis et devront, à notre sens, faire l'objet de recherches futures :
- influence de la vitesse de chargement sur les lois de réaction : $p = f(\Delta y)$;
- effets de groupe sur ces lois de réaction et sur le déplacement $g(z)$ du sol à prendre en compte (effet d'écran) ;
- lois de mobilisation des efforts en pointe $T_p = f_t(y_p - g_p)$ et $M_p = f_m(y'_p - g'_p)$;

  et enfin,
- constatations en pied d'ouvrages et élaboration de modèles théoriques pour évaluer les déplacements $g(z)$ à partir des caractéristiques élémentaires du sol mou.

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A.S. Nene (Written discussion)

Discussion on "PERFORMANCE OF PRE-STRESSED ANCHORS UNDER SLOW REPEATED LOADING" by T.H. Hanna and M.J. Al-Mosawe

The authors are to be congratulated for presenting a new aspect of anchor foundation. Andreadis et al (1) have reported the results of repeated load tests on non-prestressed anchors. In their paper they have reported about a high factor of safety (greater than 10) recommended by some design organisations, for long-term behaviour of anchor foundations subjected to repeated loads. In this context, it is felt that more weight should have been given to results of anchors at load levels between 10% to 50% of ultimate static load capacity of dead anchors.

Fig. 2 indicates that curves for load levels (80% Pv-30% Pv) and (60% Pu - 0% Pu) intersect at load cycles 100. Can it be concluded that for a low number of load cycles (say <100), the higher the upper limit of load level, the higher is the displacement and for high load cycles, the higher the difference in the load level, the higher is the displacement.

More information on this aspect will help in deciding the factor of safety taking also into account the number of load cycles.

In Fig. 3 of the Authors' paper the loading level 60% Pu-0% Pu appears to be misprinted as 40% Pu - 0% Pu.

Fig. 8 shows load-displacement curves for prestressed anchors, subjected to repeated and alternating loads. In this figure results of dead anchors are presented. For better comparison, the results of prestressed anchors should have been given.

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Andreadis, A.et al (1978. Embedment Anchors Subjected to Repeated Loading. Techn. Note, Geotechnical Division. ASCE, Vol. 104, GT7, pp 1033-1036.

M.P. Luong (Oral discussion)

DEFORMABILITE RELATIVE ET CONDITIONS D'INTERFACE SOL-STRUCTURE DES BUSES METALIQUES ENTERREES

Le principal intérêt des buses métalliques enterrées dans un remblai réside dans leur flexibilité relative qui amène à les considérer plutôt comme un coffrage destiné à canaliser les contraintes dans le massif de sol environnant que comme un ouvrage possédant une capacité portante intrinsèque.

La charge due au poids des terres réellement transmise à une structure placée dans un remblai dépend en partie de sa déformabilité relative vis-à-vis de celle des remblais environnants.

L'interaction sol-structure traduit ici l'aspect hyperstatique des efforts sur la structure et le mode de redistribution des sollicitations résultant du couplage des déformabilités de la structure et du sol avoisinant. Une illustration simple de cette interaction est proposée par les Fig. 1a et 1b qui suggèrent avec évidence les conditions de charge active et de charge passive des terres correspondant respectivement à un effet de voûte positif et négatif qui soulage ou surcharge la structure enfouie.

L'observation du comportement des ouvrages flexibles placés dans les remblais bien compactés à forte densité suggère le mécanisme de ruine de la structure par écrasement de la paroi soumise à un effort normal N excessif.

Une approche numérique par la méthode des éléments finis se révèle ici un outil puissant et souple pour analyser correctement l'interaction entre la structure flexible placée dans un remblai déformable. Elle permet en effet de tenir compte des zones de caractéristiques mécaniques différentes, de simuler les conditions d'interface sol-structure et d'obtenir une image plus complète des performances de la structure en cours de sa construction, aussi bien qu'après lorsqu'elle est en service.

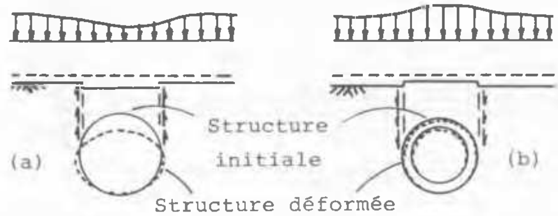

Fig.1 - Effet de voûte positif (a) et négatif (b).

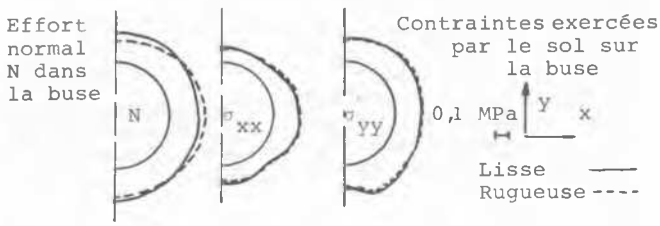

Fig.2 - Influence de la nature de l'interface sol-structure enfouie.

L'influence de la nature de l'interface lisse ou rugueuse est montrée sur la Fig. 2 où

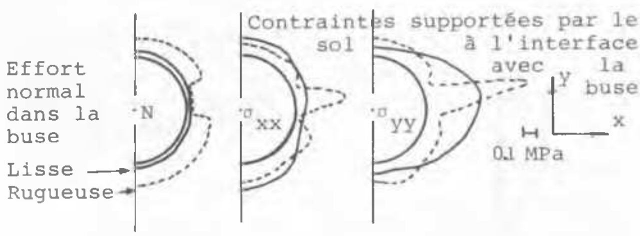

Fig.3 - Report des contraintes dans le sol de remblai grâce à un dispositif contractable.

montre la Fig. 3 obtenue à partir d'un calcul en éléments finis.

Les résultats numériques montrent l'importance de l'interface lisse ou rugueuse sur le comportement sol-structure enterrée et aussi de sa déformabilité relative. Ils permettent, en outre, une évaluation quantitative correcte des efforts dans la structure et dans le sol ainsi que des déplacements en tous points.

Une vérification expérimentale avec le L.C.P.C., le SETRA et le L.M.S., est en cours sur le site de Dourdan avec les buses métalliques Arval de 2,50 m de diamètre sous un recouvrement de plus de 15 m de remblai (fig. 4).

L'intérêt technique de ces aspects complexes, mais relativement bien élucidés d'interaction sol-structure enterrée s'est ainsi révélé stimulé par une augmentation du coût, un nombre croissant d'ouvrages et une importance accrue des dimensions de certaines structures construites ou en projet.

Fig.4 - Site expérimental de Dourdan.

Th. Dietrich (Written discussion)

ON THE KINEMATICAL PREREQUISITES OF THE APPLICATION OF WINKLER'S PRINCIPLE TO EMBEDDED ROD-LIKE STRUCTURES

Professor Poulos in his General Report on Session 5 (Poulos, 1981) considered my arguments (Dietrich, 1981) to be valid in case of an embedded rod-like structure loaded by a concentrated load rather, than with such a structure loaded by a distributed load. But since the application of Winkler's principle to embedded rod-like structures is justified by the occurence of (in the limit) plane motion, and since this is more likely to occur under distributed load than under concentrated load, the opposite is true. However, plane motion - or generally - pseudo plane motion of the second kind (Truesdell, Toupin (1960)) will prevail, and Winkler's principle will apply eventually, with any distribution of the transversal load of a flexible rod-like structure embedded in psammic halfspace (or an other material halfspace of similar properties), if only the intensity of the load is kept small enough.

ERRATA

In my paper (Dietrich, 1981), replace the last sentence of the first column on page 104 and the consecutive first sentence of the second column by "Let $w(P_1)$ denote the magnitude of the displacement of P_1 (where $P_1 = A_1 \cap L$). $w(P_1)$ is obtained by integrating the strains ϵ_1 of the elements of L along L from P_1 to the boundary infinitely below."

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Poulos, H.G. (1981) Soil-structure Interaction -General Report (Preliminary). Proc. 10th ICSMFE, 69 - 96

Dietrich, T. (1981) on the validity of Winklers principle. Proc. 10th ICSMFE, vol 2, 103 - 108

Truesdell, C./Toupin, R.A. (1960) The Classical Field Theories. In Encyclopedia of Physics. Ed. F. Flügge, Springer, Berlin, p. 329.

J. Feda (Written discussion)

ON THE MODELLING OF SOILS-STRUCTURE INTERACTION

If the soil-structure interaction is to be taken into account, a suitable geomechanical model of the foundation soil should be created. Since a geomechanical model is an abstract /incomplete/ image of the real foundation soil, some features of the latter one are omitted. There are, therefore, usually several alternatives how to model one and the same foundation soil, when solving the same boundary value problem.

The plurality of geomechanical models reaises two questions:

- Which models in the given set are equivalent?
- Which of them is the optimum one?

The optimum geomechanical model should enable both the mathematically accessible solution of the given boundary value problem and, in the same time, adequately reliable and realizable choice of its mechanical parameters. This second criterion used to be decisive one.
The specific feature of the soil-structure interaction problems lies in the fact, that the choice of the type of the model is more important that the magnitude of its parameters, on

contrary to the common deformation and stability geomechanical problems, where the opposite is true.

The analysis of the equivalence of geomechanical models can be carried out more exactly than the optimation procedure. The ambiguousness presents the criterion of equivalence. One may postulate that equivalent models should yield the same prognosis of the stress and/or strain fields, and namely in some preselected points, curves or regions.

Fig. 1 presents the classification of geomechanical models. The most common models are the time dependent deformation models which will be commented further on. Such models are incomplete in two respects: they do not enable the solution of either rheological or strength problems. This is a serious handicap, as proved by boussinesqian models where the omission of the limit stress condition raises the contact pressure to infinite values at the foundation edges.

Winklerian models /besides the one-parametric common one there is a series of two-parametric, like Pasternak´s, Vlasov´s etc. or multi-parametric, like Kolář-Němec´s/ are preferred by the designers, since they do not increase the width of the /diagonal/ strip of the global stiffness matrix when the soil-structure interaction is taken into account. Their parameters can be found from the settlement of the soil surface /they are, therefore, termed two-dimensional models/ but the magnitude of these parameters depends on the extent of the loaded area and on the magnitude of load. They are, therefore, unsuitable from the geomechanical standpoint unless some equivalent relation between winklerian and boussinesquian models could be found /as in case of Gibson´s soil but more realistic/.

Boussinesqian models are represented by half-spaces of different nature - both homogeneous and nonhomogeneous, isotropic and anisotropic, linear and nonlinear etc.

Hybrid models consists of two models, one for the calculation of the stress field, another for the calculation of the deformation fields.

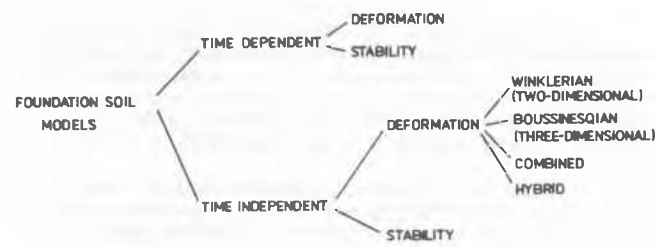

Fig. 1 Classification of models of the foundation soil.

Combined models are models combining winklerian and boussinesqian models. Such a combination can be realized either in a series /each of two half-spaces take over the whole load, e.g. Shtaerman´s model/ or in parallel /Repnikov´s or Klein´s models - each component takes over one part of the load/. The parallel combination of the Winkler and Boussinesq half-spaces seems to be the most perfect geomechanical model which may be well physically interpreted. It corresponds to the linear isotropic nonhomogeneous soil with linearly increasing deformation modulus with depth and with its non-zero surface value. It can be found that such a model is approaching to the Winkler model with increasing the extent of the loaded area and vice-versa, to the Boussinesq model if the extent of the loaded area is decreased.

Sometimes, an oversimplified model yields a realistic prediction of the actual behaviour owing to the mutual compensation of errors. In harmony with the arguments of Leroueil and Tavenas in their paper to the first session of this conference, such models are inappropriate and should be avoided.

P. Gussman and W. Lutz (Written discussion)

STABILITY OF SLURRY TRENCHES UNDER GROUND WATER CONDITIONS

The authors would like to report briefly on the stability of slurry trenches, where ground water levels are to be considered.

This, together with considerations of surface loads adjacent to trenches, has been the subject of research activities in the Institute of Soil Mechanics and Foundation Engineering, Stuttgart.

With the 3-dimensional failure-model of fig. 1 we can find the factor of safety according of Fellenius' rule for a suspension propped slurry trench. Regarding only the equilibrium of forces together with suitably reduced shear forces in the lateral surfaces of the wedge, the problem can be solved in a simple way.

But we should remember the fact that, for kinematical models like this, the answer is only correct if the chosen geometry is fully varied. This means a variation of the slope angle, the depth and the length of the failure wedge. (The variation of the length is trivial because it is identical to the length of the trench.)

In fig. 2 the factor of safety is plotted against the depth of the wedge for four different heights of ground water surfaces. (The slope angle is already varied.)

The general character of all curves is as follows: The factor of safety increases with depth in the first part, goes down to a minimum

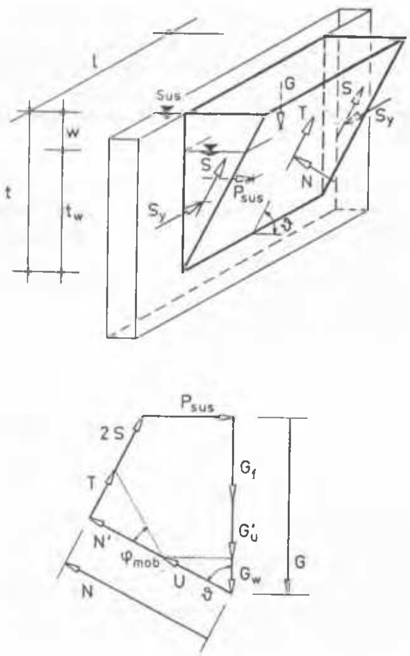

Fig. 1

in the second part and rises again in the last part. The location of the minimum, which is decisive for the design of the trench, is however not coincident with the maximum depth of the trench.

The theoretical computations were able to be verified by the experimental results of a small-scale slurry trench model, but this research work is yet to be finished.

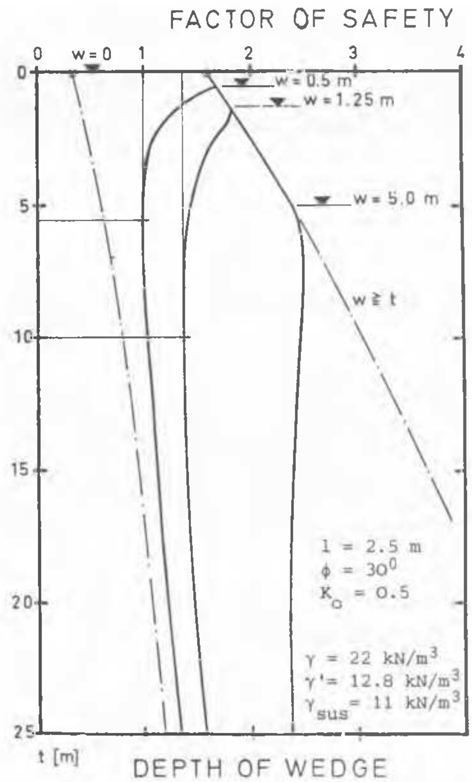

Fig. 2: Example

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Gussmann,P./Lutz,W. (1981). Schlitzstabilität bei anstehendem Grundwasser. Geotechnik 2,1981.

G.X. Zhang, N.R. Zhang and F.L. Zhang (Written discussion)

SETTLEMENT PREDICTION BY INSITU SHEAR WAVE VELOCITY FOR SANDY COHESIVE SOILS

Discussion and addendum to Paper 5/58

The authors agree with the General Reporter of Session V that no account is taken of the rigidity of the foundation or the structure in calculating the stresses in the soil beneath the foundation in the Paper and that the results could be improved if the contact pressures instead of the structural loads were used to calculate the stresses.

The authors would like, however, to point out that although a linear settlement analysis may do equally well as a nonlinear one in predicting average settlements, very different moments in the structure could possibly be induced by minute changes in the trends of differential settlements predicted by the different models as shown in Fig. 14(b) of the Paper.

Fig. 15 is presented to further illustrate this point. In this figure, two opposite trends of differential settlements, one concave when the load was smaller and the other convex when the load was greater than the present overburden pressure are shown. Although there is still

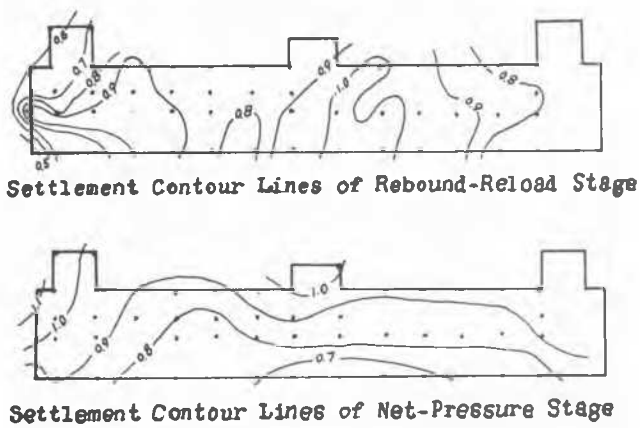

Fig. 15 Observed Settlement Contour Lines (cm) of Different Loading Stages

much dispute on how to explain this phenomena correctly and fully, there is no doubt that it has something to do with the nonlinear property of soils.

To improve the accuracy of settlement predictions, the authors have revised their original proposal of a single valued empirical coefficient α in eq.(1) to two separate values of α to be used for pressures smaller and greater than the present overburden or preconsolidation pressure. The ratio between these two separate values of α has been taken to be equal to the ratio between the moduli of rebound and virgin compression obtained from conventional consolidation tests.