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Certain Features of Lateral Pressures of Soils

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In my paper I intend to give a brief review of experimental investigations of two types of retaining structures carried out in recent years in the United States and in England.

In the first part of my report I shall describe special features of lateral pressures on inclined struts of wide open braced cuts. In the second part, I shall outline some aspects of the design of anchored sheet pile bulkheads.

In both cases, the interaction between the retaining structure and the soil is so complex that up to now it has not permitted rigorous mathematical analysis thereof. Observations and measurements on construction sites have invariably preceded even approximate theoretical explanations of corresponding modern methods of design proposed by various investigators. Therefore, both in the first and in the second parts of my paper I shall at first briefly touch upon preceding studies of this kind.

(1) Lateral Pressures of Sandy and of Silty Soils Against Inclined Struts of Sheet Pile Walls

The practical experiences during the construction of the first New York subways during the first decade of our century already showed that the lateral pressures of soils against the upper rows of struts exceeded appreciably the values obtained from design diagrams of the customary triangular shape.

Measurements performed during the following quarter of a century on subway construction sites in New York, Berlin and Munich confirmed that in such cases the actual lateral pressures of sandy soils have a parabolic form *(Fig. 1)*.

On the basis of these measurements, Terzaghi *(Ref. 1)* proposed in 1941 a design diagram which had the form of a trapezoid. This diagram was established in a purely empirical manner as an envelope of pressure diagrams obtained by measurements on various sections of the Berlin subway.

Other studies showed that the increase of the lateral pressures on the upper struts of braced cuts is related to the nature of the deformations of the sheet pile wall as shown diagrammatically on Figure 1.

A rigorous theoretical solution of the problem has not yet been obtained and is greatly hindered by the fact that at different depths in the soil two physically different phenomena, i.e., wedging and slipping, develop simultaneously.

In its lower part, the soil behind the sheeting expands; its density decreases and surfaces of sliding develop in it. In the upper part, however, under normal conditions of deformations of the braces of the cut, a densification of the soil takes place and a wedging of its hard particles, i.e., the so-called "arching action" *(Ref. 2)*.

In spite of the absence of a mathematically rigorous justification, Terzaghi's design trapezoid *(Fig. 1)* fully justified itself in practice when cuts were braced by horizontal struts. However, experience with inclined struts proved to be somewhat different in certain cases.

When the width of a cut is very large, the bracing of its walls by horizontal struts *(Fig. 1)* is not practical. For that reason, one frequently uses the method shown *(Fig.*

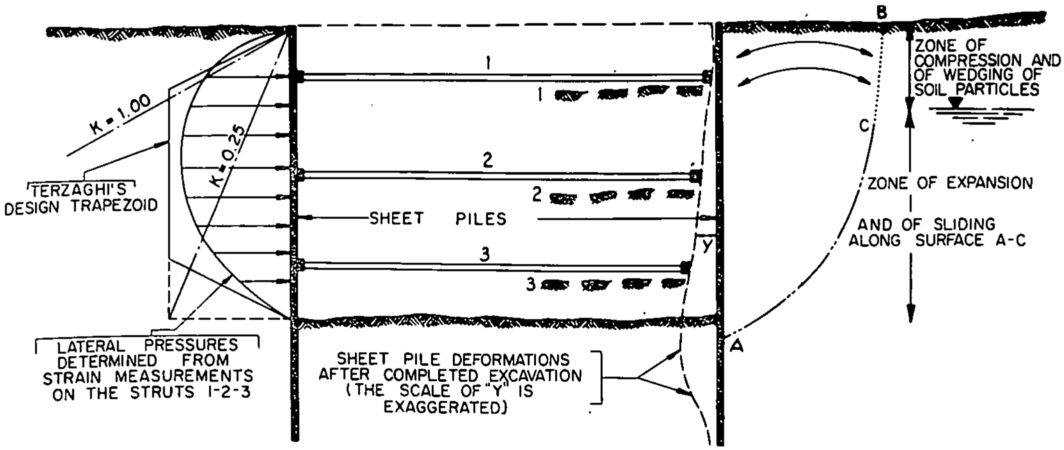

FIGURE 1

2-1), that is the excavation is performed within sheet pile walls approximately along the natural slope b-b, with a step-wise lowering of the ground water level by well points. After that, the central part of the foundation of the structure is concreted and the upper inclined strut N_1 is installed. The lower horizontal strut H_1 is installed after further lowering of the water level and excavation of the sloped soil.

The horizontal component L_1 of the pressure on the upper strut N_1 is determined in the usual way from the design diagram of lateral soil pressures accepted for the case handled. The vertical component V_1 is determined as a function of L, and of the inclination of N_1. Usually the value V_1 equals only part of all tangential active stresses T_a which are developed along the

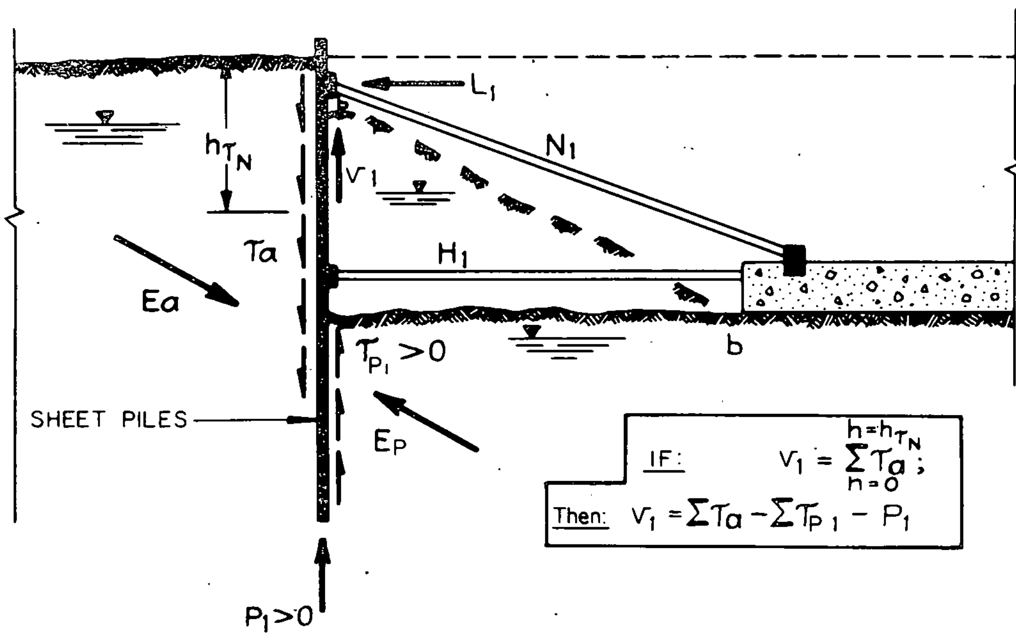

FIGURE 2-1

face of the sheet pile wall by the inclined active pressure of the soil E_a, i.e.:

$$V_1 = \sum_{h=0}^{h=hT_N} T_a \qquad (1)$$

The rest of the active tangential stresses T_a is balanced by passive tangential stresses T_{P1} *(Fig. 2-1)* and by the resistance P_1 of the tip of the sheet piling, i.e.:

$$V_1 = \Sigma T_a - \Sigma T_{P1} - P_1 \qquad (2)$$

If for any reason the tangential passive stresses T_{P2} and the resistance of the tip of the sheeting P_2 will be reduced to zero then, as shown *(Fig. 2-2)*, the value of the vertical component V_2 of the inclined strut N_2 will be greatly increased as compared to the usual value V_1, since:

$$B_2 = \Sigma T_a \qquad (3)$$

The horizontal component L_2 of the inclined strut will be increased correspondingly and equilibrium will become possible only as a result of the development in the upper part of the soil behind the sheeting of lateral pressures of a passive character which have to be greater than the values of the design trapezoid of Terzaghi shown in Figure 1. The strut N_2 itself will then be subjected to compression which will exceed considerably the design values, a circumstance which may have dangerous consequences.

With the help of the two following diagrams, let us now consider the question as to what kind of circumstances may produce in practice such an undesirable situation. If soft deposits extend below the level of the bottom of the excavation, then the foundation has to be supported by piles which, as well as the sheet pile wall, are then driven into a more compact underlying layer as this is diagrammatically shown *(Fig. 2-3)*.

The driving of piles for the outer parts of the foundation is carried out after completion of the excavation. If the saturated soil below the level of the bottom of the excavation is primarily of a non-cohesive type, e.g., it consists of silty fine-grained sands or of rock flour, then the vibrations which accompany the driving of piles in the immediate vicinity of the sheet pile wall produce a temporary but considerable decrease of the forces of inner friction in the soil.

As a consequence, this produces also a decrease of its tangential passive stresses T_{P3} along the surface of the sheeting and of the resistance P_3 of the tip of the sheeting. This decrease of T_{P3} and of P_3 has to be balanced by an appreciable increase of

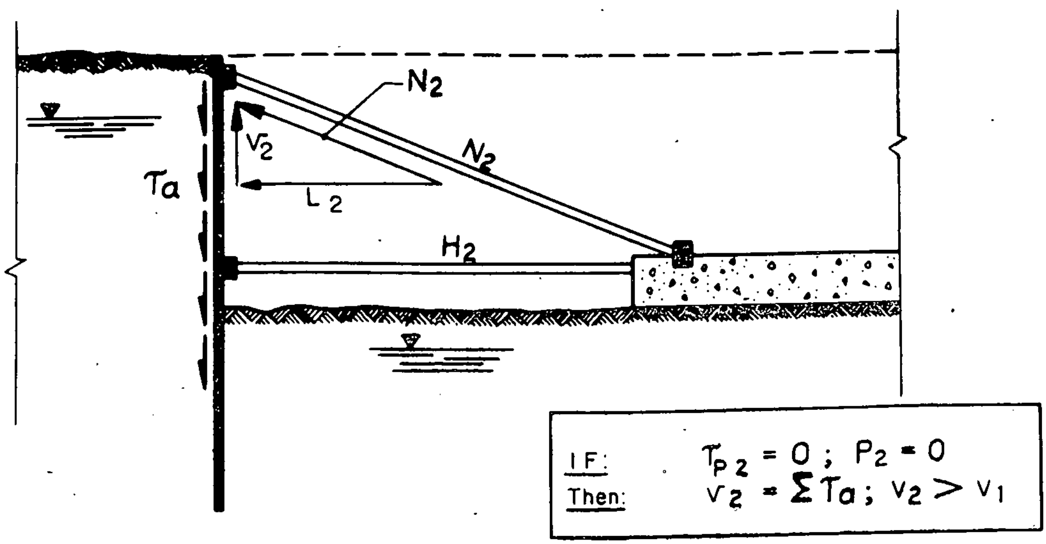

FIGURE 2-2

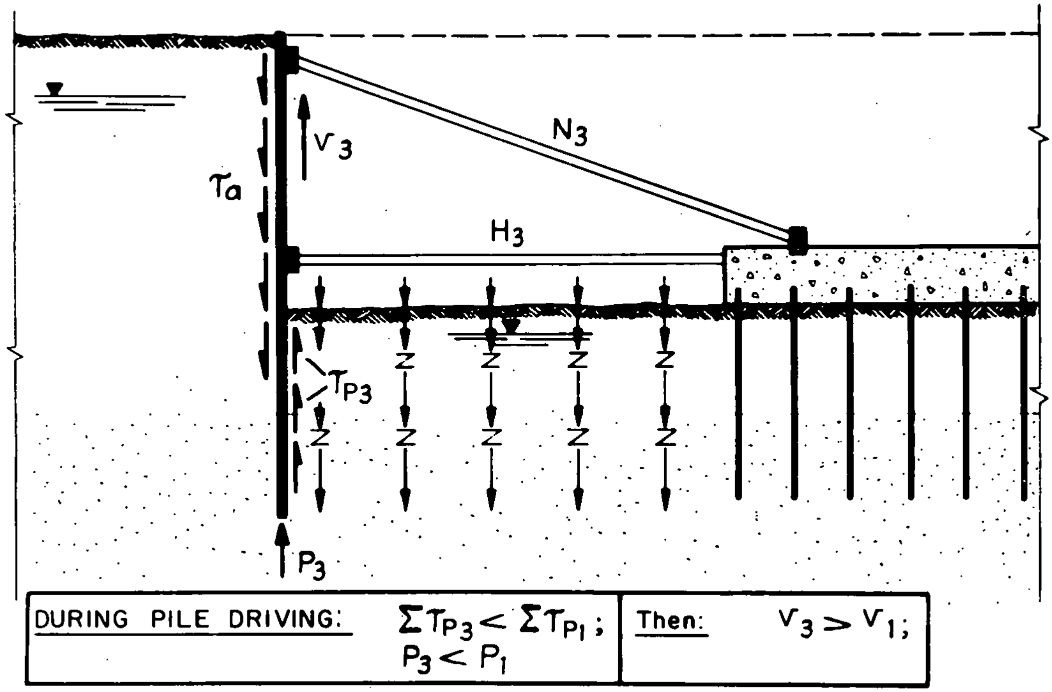

FIGURE 2-3

the vertical component V_3 and hence also of the axial pressure N_3 on the inclined strut.

Another circumstance with similar undesirable consequences is diagrammatically indicated *(Fig. 2-4)*. The difference between the active and passive lateral pressures (p_a—p_b) along the span "L" of the sheet pile wall will develop in it bending moments M. These moments may be increased by an eccentricity of the tangential active stresses T_a along the sheeting caused by the lateral displacement Δy of the upper part of the sheet pile wall towards the interior of the cut during its excavation and prior to the installation of the upper inclined strut N_1, see Figure 1.

Given sufficiently large values of the span "L," of the eccentricity "y" and of the pressures (p_a—p_b), the development of a moment M_o in the sheeting is possible, whereby this moment M_o may exceed the moment M_{PL}, after which a flow and plastic deformations of the material of the sheeting may take place. As a result the sheet piling is no longer in a position to transmit axial loads to the underlying compact soil. Therefore the inclined strut N_4 has to resist the greater part of the active tangential stresses T_a.

In my capacity of associate of the firm of King & Gavaris, consulting engineers, I participated in the study of a case which occurred in a deep cut (h = 17 m) in waterlogged glacial deposits of varved clays, silty sands and of rock flour where apparently took place simultaneously the conditions illustrated both by Figure 2-3 and by Figure 2-4, i.e: the driving of piles, the decrease of the passive resistance p_p and the corresponding increase of the bending moment M_o accompanied by a decrease of the tangential passive stresses T_p. The situation was aggravated by the fact that only the individual master piles and not the entire sheet pile row reached the underlying layer of compact sand and gravel.

What should be the preventive measures in such cases? First of all, the ground water lowering should be performed not only by stepped rows of well points inside of the cut, but also with the help of individual deep wells with pressure pumps at their bottoms placed outside of the sheet pile wall. Second, additional rows of struts should be

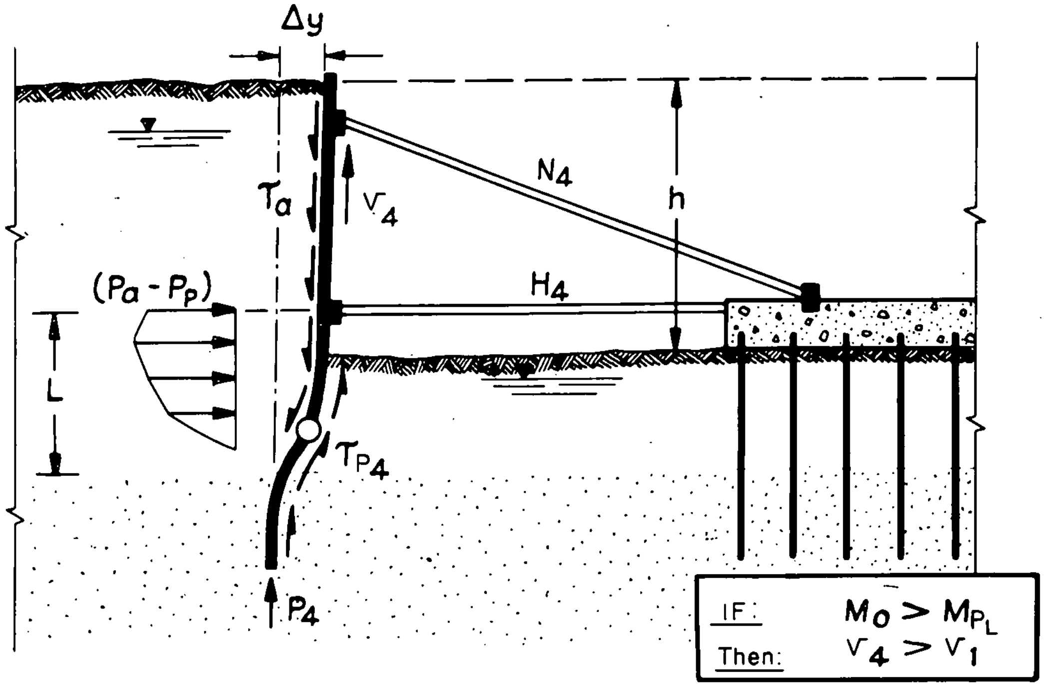

FIGURE 2-4

installed having the purpose to decrease the span "L" and to relieve the upper row of inclined struts. Third, consideration should be given in design even if only approximately to all these special factors when computing and detailing the structure.

A rigorous mathematical analysis presents here even greater difficulties than in the design of horizontal struts. For that reason additional measurements on construction sites of this type are greatly to be desired.

(2) Design of Anchored Sheet Pile Bulkheads

A number of experiments performed during the past 15 years have resolved certain contradictions between various design methods of anchored sheet pile bulkheads.

The oldest and best known of these methods did not consider any fixation of the lower part of the sheeting in the soil, see diagram of lateral pressures *(Fig. 3-B)*, and the corresponding curve (1) of the diagram of bending moments *(Fig. 3-A)*. In reality such diagrams are possible only in two cases: first in the case of absolute rigidity, i.e., non-flexibility of the sheeting, see the straight line (1) *(Fig. 3-C)*, and second, in the case of limit equilibrium of the entire soil into which the sheeting is driven, i.e., when the safety factor of the passive resistance of the soil has a value equal to unity along the entire depth of embedment of the sheeting. This latter case obviously is not permissible for the design of any kind of structure.

At the beginning of the 1930's, the German engineer Blum *(Ref. 3)* developed a method for the evaluation of the fixation of the lower part of the sheeting and proposed a simplified method of design with the help of the so-called "equivalent beam." The method consisted in the determination of the point at which the bending moments are equal to zero.

An imaginary "hinge" was assumed at that point and the part of the sheeting above it was designed as a statically determinate beam on two supports. The point of zero moment, i.e., the hinge, was again determined on the basis of a state of limit equilibrium of the soil along the entire depth of embedment of the sheeting. As a result,

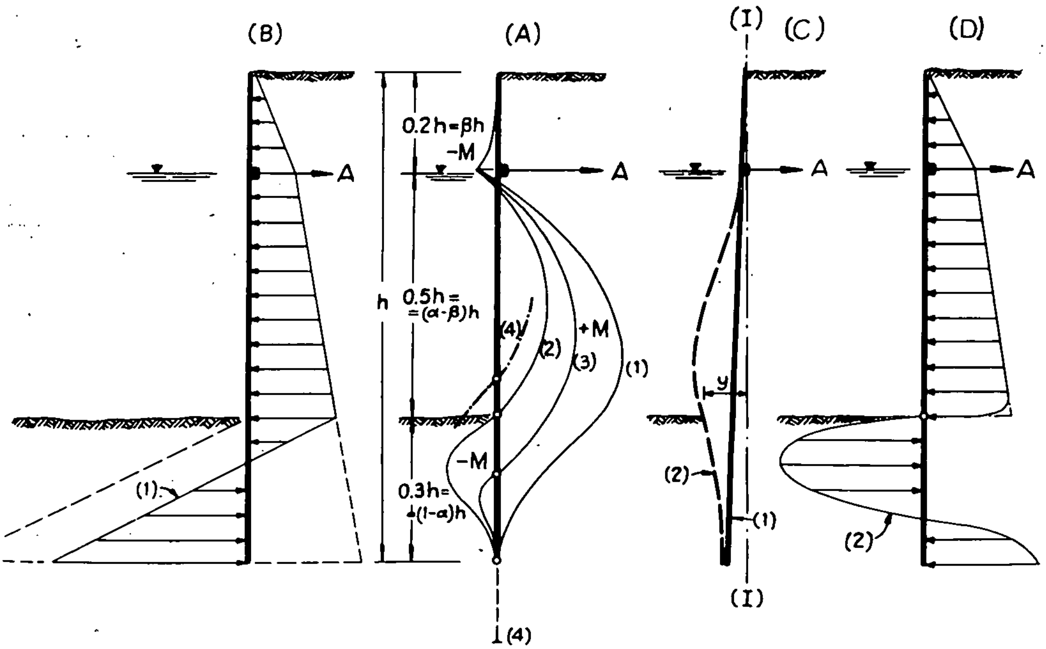

FIGURE 3

only the value of the angle of internal friction of the soil influenced the location of the hinge and the required depth of embedment reached appreciable values. The degree of flexibility of the sheeting was *not* taken into consideration.

In 1943-49, I had the opportunity to direct model tests at Princeton University for the Bureau of Yards and Docks of the U. S. Navy. These tests showed *(Ref. 4)* that:

1. The redistribution of active pressures as suggested by the Danish rules does not actually occur.
2. Consideration of passive pressures above the anchor level according to the 1938 suggestion of the German scientist Ohde corresponds to reality only in the case of absolute rigidity of the anchor and of its supports and only in the case of dredging of the soil in front of the sheet pile bulkhead and not of backfilling behind the bulkhead.
3. The fixation of the lower part of the sheeting embedded in sand takes place under a much smaller depth of penetration than followed according to Blum, see curve (2) *(Fig. 3-A)*.
4. At the same time the "hinge" is located in the immediate vicinity of the dredge level of a sandy harbor bottom in front of the sheeting if the latter has a flexibility which corresponds to steel sheet piles having a cross-section in the form of the letter "Z" as used in the USA.

The latter two circumstances are explained by much higher passive resistance of the sandy soil *(Fig. 3-D)* than was considered possible previously. This is brought about by the deflection "y" of the sheeting *(Fig. 3-C)*. Further tests of my associates at Princeton in 1951-53 *(Ref. 6)* have confirmed this explanation.

Measurements in the field also showed *(Ref. 8)* that an increase of the depth of embedment raises the level of the "hinge," see the curve (4) *(Fig. 3-A)*.

In 1952-57 the talented young English scientist, Peter Rowe, published *(Refs. 5, 7)* data concerning his tests which confirmed the results obtained by us at Princeton and which developed them appreciably further.

Having decreased the scale of his model, approximately to 1:25, whereas the model at Princeton had the scale of 1:10, and using dry sand instead of the submerged sand with which we worked, Peter Rowe was able to increase more than tenfold the number of tests he carried out as compared to our own. Further, he established definite relationships between the flexibility of the sheeting, the density of the sand into which the sheeting is embedded and the maximal bending moment (+ M) of the sheeting *(Fig. 4)*. To that end Rowe introduced the coefficient of flexibility ρ:

$$\rho = \frac{H^4}{EI} \tag{4}$$

where:

- H = length of the sheeting in feet *(see Fig. 3)*.
- E = modulus of elasticity of the material of the sheeting in psi.
- I = moment of inertia of the sheeting in inches^4 per foot width of the sheeting.

The computation of this coefficient in the metric system should not present difficulties.* The moments measured were expressed by Rowe in per cent of the value of the moment M_1 without any kind of fixation of the lower end of the sheet pile, see curve (1) *(Fig. 3-A)*. Typical results are shown on Figure 4 for coefficients $\alpha = 0.7$ and $\beta = 0.2$, i.e., for the same depth of embedment and location of the anchor as shown in Figure 3. At the same time the surface surcharge $p_s = 0$.

* 1 foot = 0.305 m.; 1 inch = 2.54 cm..; 1 lb. = 0.454 kg.

Peter Rowe suggested plotting on the same diagram the curves of resistance moments of various types of sheet pile sections for a given permissible value of their bending stress. The intersection of such a curve of resistance with a curve of experimental moments for the given type of soil indicates the most rational use of permissible stresses for a given sheet pile section. A design is carried out in practice by trial selections of sections and their verification on a diagram of the type shown on Figure 4.

The curve (A) on Figure 4 shows the curves of resistance moments to bending of steel sheet piles of the USA sections MZ-27, MZ-32 and MZ-38 for a permissible stress of 22,000 psi = 1560 kg/cm^2.

On the same Figure 4 is shown the value of the moment M_2 (= 45% of M_1) obtained in accordance with the simplified method of the "equivalent beam" with a hinge at the level of the dredged bottom of the harbor, see curve (2) *(Fig. 3)*. This line intersects almost at the same point the resistance curves (A) and the average experimental curve of Rowe. Thus the results of all these tests fully agree with each other.

The method of Rowe greatly facilitates understanding of the fundamentals of the problem. Thus if we plot on Figure 4 the resistance curve (B), for the same sheet pile sections for which we have plotted the curve (A), but for the additional deflection which would correspond to the first limit of elasticity of the metal of 33,000 psi = 2360

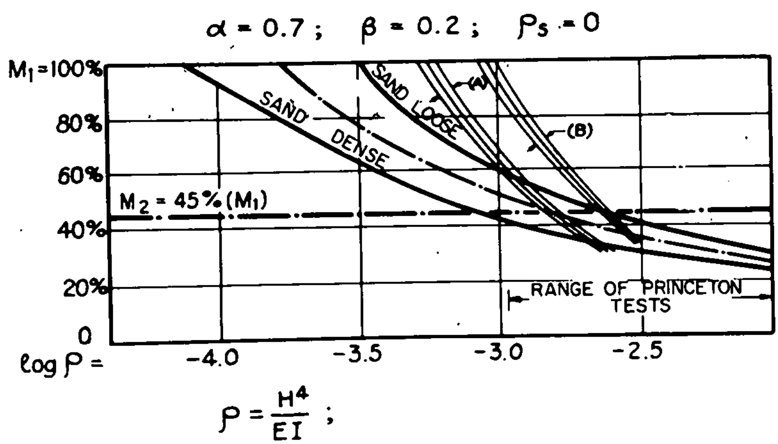

FIGURE 4

kg/cm², then the corresponding experimental moment will decrease from 45 per cent of M_1 *(Fig. 4-A)* to 36 per cent of M_1 *(Fig. 4-B)*. In other words, this indicates the presence of an additional factor of safety equal to $45/36 = 1.25$ if one designs a steel sheet pile wall for the stresses of 1560 kg/cm² usually considered permissible in the United States.

Rowe worked out experimentally a system of curves similar to those of Figure 4 for clays as well *(Ref. 7)*. However, more complicated soil conditions have not yet been taken care of by this semi-empirical method.

Measurements which I carried out in a number of harbors have shown that soil conditions corresponding to simplified laboratory conditions of model tests should be considered an exception rather than the rule. There are reasons to assume that the assumption for the design of average values of coefficients obtained from tests with two different soils will not always be a correct solution. Thus, for instance, as shown *(Fig. 5)*, a sheet pile wall driven through a layer of plastic clay into compact sand will be subjected to an appreciably larger curvature and therefore also to an appreciably larger bending moment so that its "hinge" will be located lower than in the case of a wall which does not reach sand. At the same time, however, the lateral displacement of the latter at the dredge level will be larger than the displacement of the first, i.e., $y_c > y_s$.

Attempts of mathematical analysis of the problem combined with the use of coefficients of lateral subgrade reactions of different soils, determined on the basis of laboratory tests, are being made but have not yet led to practicably usable solutions.

In any case, any theoretical solution will need verification by full-scale field measurements. The location of the point of zero moment, i.e., of the imaginary "hinge," is easily determined *(Refs. 8, 9)* and therefore represents a reliable check criterion of the

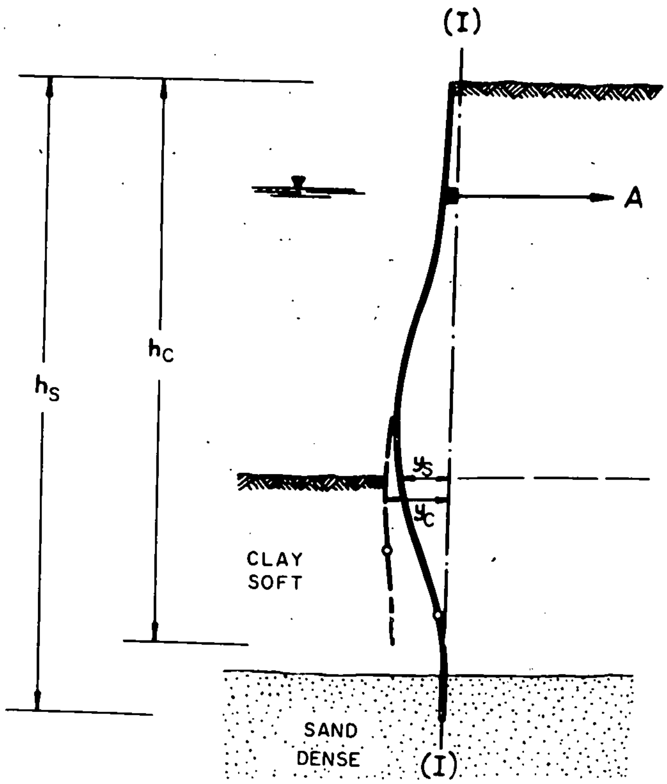

FIGURE 5

degree to which any theory corresponds to real conditions. Therefore, I consider that the simplified method of the "equivalent beam" with modifications reported by me at last year's conference in Brussels *(Ref. 9)* represents a practical, feasible approach to a final solution of the design problems for this type of structures.

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Critical Elements of Design and Construction of Heavy-Duty Flexible Pavements

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The increase in weight of aircraft has made it necessary to devote special attention to the design and construction of flexible pavements that will perform satisfactorily under these unprecedented aircraft loads and operational characteristics.

The load on a single wheel of an aircraft can now be as high as about 60,000 to 70,000 lbs. The number and alignment of the wheels vary with the gross weight and type of aircraft. In order for the wheels to carry such loads without using excessively large tires, the inflation pressures of the tires must be much higher than those required in the past. Tire pressures are now approaching 300 psi and this intensity of contact loading makes it necessary to give special attention to the upper portion of the pavement.

In addition to the great increase in load weights and tire inflation pressures, the operational characteristics of the aircraft while on the ground must be considered. Almost all recently developed aircraft can be steered by mechanical means, which results in operation of the newer aircraft in a path much narrower than that of the older aircraft which are steered by rudder alone. Such concentrated traffic, referred to as channelized traffic, has the effect of deteriorating the pavement in a shorter period of time than traffic distributed over a wider