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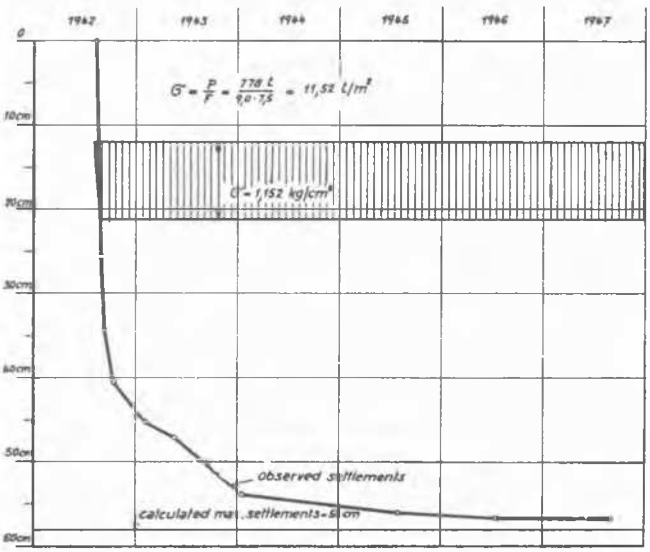

Shelter B.

FIG.5

four samples have been examined, the settlements are calculated approximatively according to the theory of Steinbrenner. 1)

For the singular layers the modulus of plasticity V is determined by the aid of the Oedometer tests. This modulus must be found out for the difference between the real soil charge and the charge resulting from the shelter. The maximal settlements can be determined in calculating separately the pressions and the settlements of the singular layers, adding them up afterwards for the total settlement.

Fig. 1 shows the soil profil for refuge A, fig. 2 represents an Oedometer-test for the depth 10,20 - 10,40 m (V = 15 kg/cm2) and fig.3 illustrates the calculated maximal settlements and the observed settlements.

Fig. 4 shows the soil profil for refuge B and fig. 5 the calculated maximal settlements and the observed settlements.

From the fig. 3 and 5 we learn that in all these cases, basing on the approximative theory of Steinbrenner, good results can be obtained.

SUMMARY.

For simple cases (provided that undisturbed samples have been taken and examined in the laboratory) the settlements which may happen can be calculated according to the approximative theory of Steinbrenner in a sufficiently exact way.

REFERENCES.

1) W. Steinbrenner: Tafeln zur Setzungsberechnung. Bodenmechanik und neuzeitlicher Strassenbau.
Schriftenreihe der Strasse 3. Volk und Reich Verlag, Berlin, 1936.

2) C.F. Kollbrunner: Fundation und Konsolidation, Bd. I. S. 374 ff. Schweizer Druck- u. Verlagshaus, Zürich, 1946.

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SUB-SECTION VI b

MEASUREMENTS OF STRESS DISTRIBUTION IN THE CONTACT FACE

VI b 1 TESTS FOR THE DETERMINATION OF THE DISTRIBUTION OF SOIL REACTIONS UNDERNEATH BEAMS RESTING ON SOIL

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INTRODUCTION.

To obtain a more exact idea about the law of distribution of soil reactions underneath beams resting on soil, a certain number of tests have been performed in the laboratory on steel beams resting on Rhine sand.

DESCRIPTION OF THE TESTING EQUIPMENT.

As container was used a very rigid box from 2,05 m. x 1,05 m. A view of the box is given on fig. 1, a scheme on fig. 2. The box was placed under a 10 tons press. The soil used is Rhine sand, with the granulometric distribution given on fig. 3.

The sand is put in the box in successive layers of 0,15 m thickness after compaction; the necessary precautions were taken to prevent the disturbance of the structure of a previous layer by introducing new layers. By means of the press and the device indicated on fig. 4, each layer was subjected in each point to a pressure of 2 kg/cm^2. Thus there was a certitude that the soil previous to the tests has been subjected to a pressure of 2 kg/cm^2; for stresses less than this value the settlements don't depend on the constant of compressibility C, but are determined by the constant of expansion A.

The total thickness of the sand in the

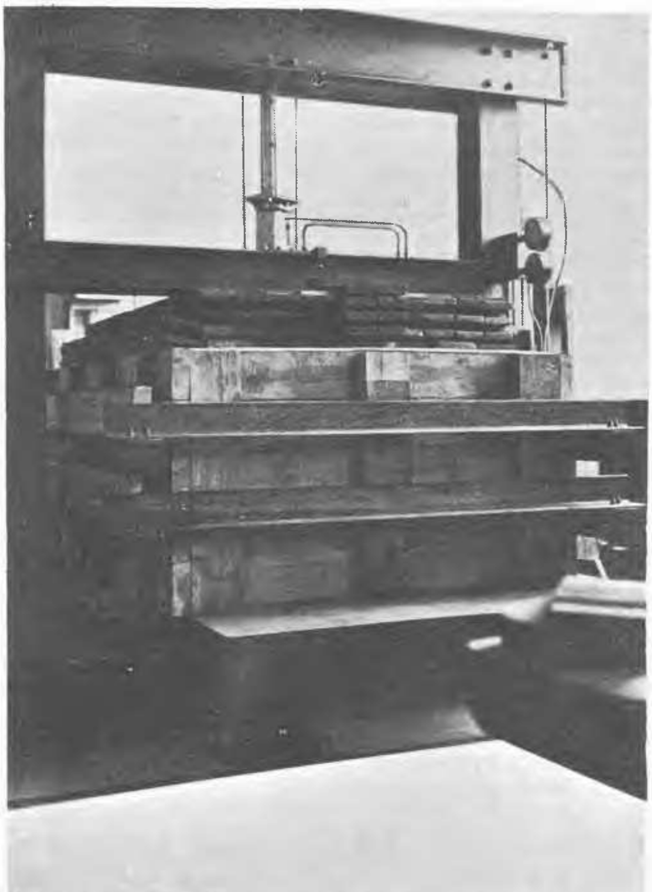


FIG.1

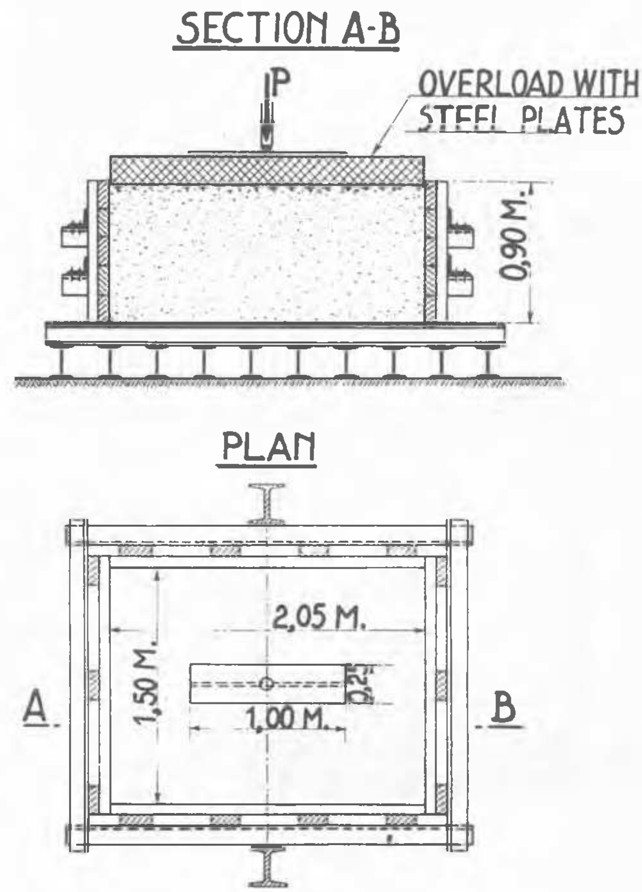


FIG.2

box was 0,90 m.

The successively used steelbeams had a length of 1 m, and the lower girder had a width of 0,25 m; they were put on the surface of the sand, their longitudinal axis being parallel to that of the box, as indicated on fig. 2.

In the real problems, it is rare that a beam is founded on the surface of the soil; usually the base of the beam is at least 1 m underneath the surface. To approach as much as possible the real conditions, the surface of the sand, not occupied by the beam, was loaded by steel weights, to obtain around the beam a uniform overload of 1,6 t/m^2, what nearly corresponds to the weigth of 1 m of soil.

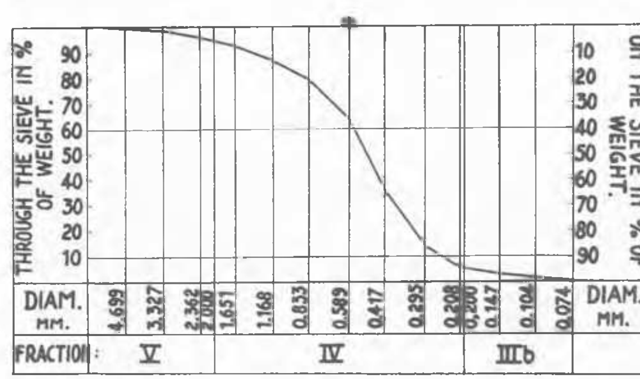
Some physical and mechanical characteristics of the sand, as placed in the box, are the following:

watercontent w = 2 à 5 %
mean percentage of voids n = 42.6 %
dry weight γ_s = 1520 kg/m^3
angle of internal friction by n = 39.7 % $\Phi = 32°$.

Several consolidation tests were run on samples taken in the box, the values obtained for the constant of compressibility C vary between 117 and 165, and those of the constant of expansion A between 1230 and 2660, around a mean value of 1640.

VALUE OF THE MODULUS OF ELASTICITY E_s.

Knowing the values of the constants of compressibility C and expansion A, it is necessary to deduct from these values the modulus of elasticity E_s.



Granulometric Size. Rhine sand

FIG.3

The modulus of elasticity E_s of the sand is given by the formula $E_s = A\,(p + p_0)$; p being the effective stress existing at the given point and on the given plane and p_0 a constant. As the stress p is variable from one point to another, the modulus of elasticity is also variable. It will be necessary to try to compute a mean value of the modulus, valid for the zone of the box influenced by the beam.

Let p_s be the stress at the surface of the layer and p_f the stress at the base. The index 0 indicates the values corresponding to the not loaded state of the beam. The volumetric weight is taken as 1.67 t/m^3, and for A

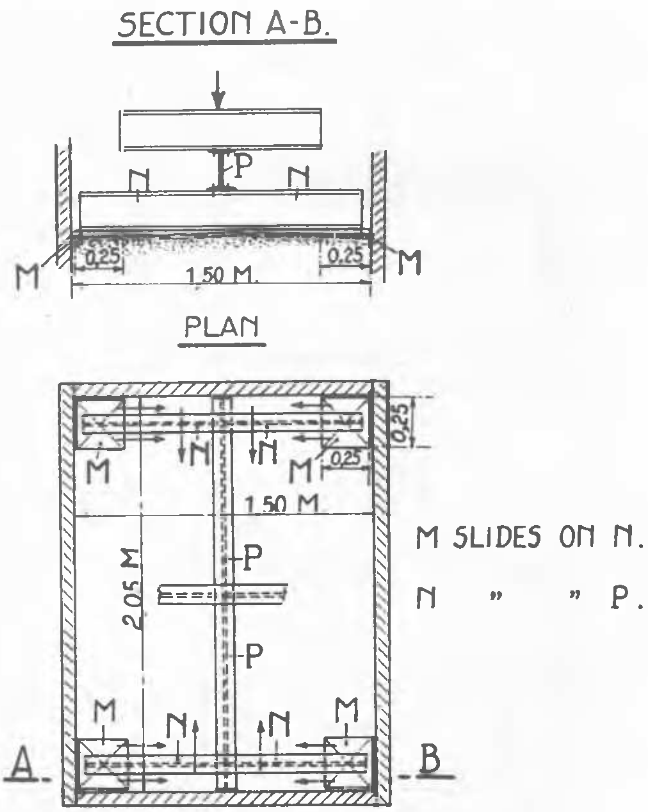


FIG.4

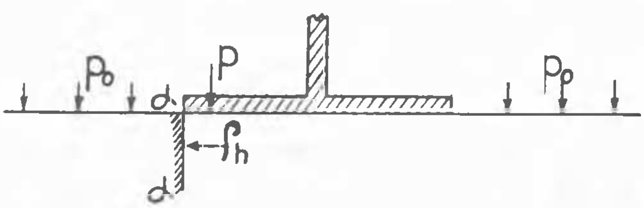


FIG.5

is introduced the mean value A = 1640. The constant p_o is neglected against p. The sand-layer having a thickness of 90 cm and being covered by steel weights producing a stress of 1,6 t/m^2, one finds for the not loaded state:

p^o_s =1,6 t/m^2

$E^o_{s,s}$ = 1640 x 1,6= 2620 t/m^2=262 kg/cm^2

p^o_f= 1,6 +0,9 x 1,67= 3,1 t/m^2

$E^o_{s,f}$=1640 x 3,1= 508 kg/cm^2.

The mean value $E^o_{s,m}$ is

$$E^o_{s,m} = \frac{262 + 508}{2} = 385 \text{ kg/cm}^2.$$

If Δp figures the increase of stress produced at the given point by the load of the beam, the modulus of elasticity E_s of the soil at the given point, after application of the load, is given by the formula E_s= A (p_o +Δp), with the condition that every lateral displacement at the given point is prevented. This should be the case, if, the box being indeformable, the increase p were uniformly applied over the whole surface of the box. In reality the load is only applied on a restricted surface; thus when the value of the load is increasing necessarely values will be obtained at which in certain regions considerable horizontal displacements will be produced. To compute from which values of the load horizontal displacements can start, consider the border of the beam surrounded by an overload p_o = 1,6 t/m^2 (fig. 5). The vertical plane will not undergo an horizontal displacement as long as the ratio $\frac{\rho h}{p_o}$ doesn't transgress a certain value K_o, which is called the coefficient at rest. The sand having been previously compacted under a pressure of 2 kg/cm^2, it seems that one can adopt K_o = 1. This means that, as long as the stress ρ_h on the plane $\alpha\alpha$ remains less than p_o, there will be no horizontal displacement of the plane.

The maximum value of the reaction p soil-beam corresponding to a given value ρ_h, is given by the formula

$$p=\rho_h \, tg^2 (45^o + \frac{\varphi}{2}) + 2 c \, tg (45^o + \frac{\varphi}{2})$$

Thus, as long as $p \leq p_o \, tg^2 (45^o + \frac{\varphi}{2}) + 2c \, tg(45^o + \frac{\varphi}{2})$,

there will be no lateral displacement of the plane $\alpha\alpha$: on the contrary, when

$$p \geqq p_o \, tg^2(45^o + \frac{\varphi}{2}) + 2 c \, tg (45^o + \frac{\varphi}{2})$$

a lateral displacement on the border of the beam becomes unavoidable.

With φ= 30° c = 0,02 kg/cm^2 p_o= 1,6 t/m^2,

one obtains p= 1,6 x 3 x $\frac{2 \times 0,2}{0,576}$ = 5,495 t/m^2.

Admitting that the reactions soil - beam are uniformly distributed, one obtains a load P'

P' = pΩ= 5,495 x 0,25 x 1,00 = 1,37 t.

For loads larger than P" = 1,37 t., it will be necessary to take into account the production of horizontal displacements. It can be noted that the value of P' is many times smaller than the ultimate bearing capacity of the soil, and even much smaller than the load P" producting plastic zones at the border of the beam.

As a conclusion, the values given by the formula E_s = A (p_o +Δp) are only valid for sollicitations smaller than 1,37 t. For this last value the modulus of elasticity has approximately its highest value; for values P>P' the modulus of elasticity doesn't certainly more increase with the applied loads. Nevertheless for values of P, a little higher than P' and much smaller than P", the possible decrease of $E_{s,m}$ will be scarcely noticeable.

Finally the most probable values of $E_{s,m}$ are the following:

P (t)	0	0,5	1,0	1,37	1,5	2,0	3,0	5,0
$E_{s,m}$ (kg/cm^2)	385	401,5	483,5	544	544	544	544	544

In the range of the considered sollicitations, the mean modulus of elasticity is situated with great probability between 385 and 544 kg/cm^2.

TESTS WITH A BEAM DIL 25.

A first serie of tests was performed on a beam DIL 25 with the following characteristics:

length ℓ = 1,00 m b = 0,25 m $\frac{\ell}{b}$ = 4
h = 0,25 m

I_{xx}=12.714 cm^4 $\frac{I_{xx}}{v}$= 1,017 cm^3 P= 82,87 kg/m

S = 105,57 cm^2 E_a(adopted) = 2.000.000 kg/cm^2

This beam was loaded by means of the piston of the press by a central load, the value of which was varied between 0 and 5 tons. The measured quantity is the stress of the extreme inferior fibre of the central section. The measures were taken woth a strain indicator (fig.6.).

The stresses obtained by multiplying the measured dilatation with the modulus of elasticity E_a of the steel, are given in the tabel I in function of the corresponding sollicitations. For the same sollicitation, tabel I gives two values A and B, which are the extremes of a serie of measures taken by successive loadings and unloadings.

On the other hand the stress at the given point was calculated by the different method at disposition, to compare the computed values with the measured ones.

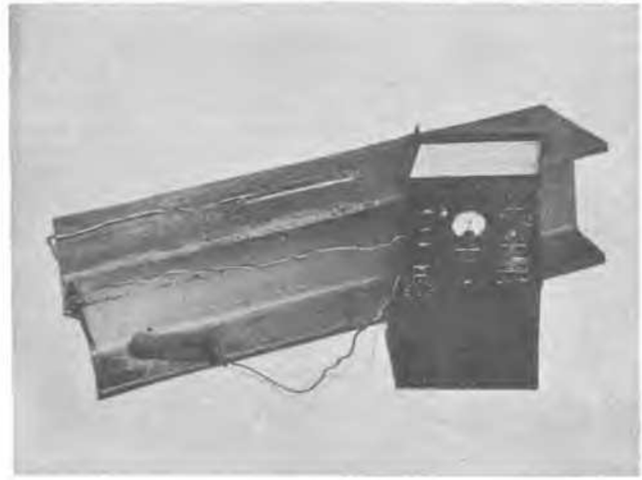


FIG.6

TABLE I

P(t)	computed σ kg/cm^2		measured σ kg/cm^2		
	method K	parabola of 2nd degree	method of Schiel	A	B
0,5	6,15	7,36	8,76	7,5	8
1,0	12,3	14,72	17,52	15,-	16
1,5	18,45	22,08	26,28	22,5	24
2,0	24,6	29,44	35,04	30	32
2,5	30,75	36,8	43,8	38	39,5
3,-	36,90	44,16	52,56	45,5	47,5
4,-	49,2	58,88	70,08	60	63
5,-	61,5	73,6	87,6	75	78,5

Using the method of the distribution of a parabola of the 2nd degree, it is easily found that the law of distribution of the soil reactions can be assimilated to a parabola of the 2nd degree, determined by the ratio $\frac{p_C}{p_m} = 0,6$,

where

p_C = the reaction at the center.

p_m = the mean value of the reactions.

The moment in the central section M_C is given by:

$$M_c = -\frac{b\ell^2}{16} \cdot p_m \left(3 - \frac{p_c}{p_m}\right) \qquad (2)$$

$$M_c = -2,4 \frac{b\ell^2}{16} \cdot p_m$$

The stress at the point considered is

$\sigma = M_C : I/v$ (3)

The values of σ so obtained are given in the 3rd column of the tabel I.

When the method of the modulus of soil reaction K is applied, the value of K is given by the formula

$$k_m = 1,33 \frac{E_s}{\sqrt[3]{b^2 \cdot \ell}}$$

$$k_m = \frac{E_s}{29,8} \qquad (4)$$

The moment in the central section is given by

$$M'_C = \frac{P\ell}{2\,m'_C} \qquad (5)$$

The method of the modulus of soil reaction gives easily m'_C = 4,00

thus

$M'_C = -\frac{P \times 1,00}{2 \times 4,00} = -\frac{P}{8}$ t.m. and $\sigma = M'_C : I/v$

The so calculated values of σ are given in the 2 nd column of the table I.

The preceding calculations showing that one is in the case of a beam with practically an infinite stiffness, the formulas established by Schiel for this case are appicable.

One finds thus

M_C = - 0,1786 P (P in t, M_C in t.m.) (6)

The corresponding values of σ are given in the 4 th column of table I.

TESTS WITH A T BEAM OF GREAT STIFFNESS.

A second serie of tests was run on a T beam, with the section shown on fig. 7, and with the following characteristics:

ℓ = 1,00 m b = 0,25 m h = 0,1755 m

I_{xx} = 949 cm^4 $\frac{I_{xx}}{v}$ = 353 cm^3.

The T beam was subjected to the same tests as the DIL beam. The measured values for the stress in the extreme inferior fibre in the central section are indicated in table II.

TABLE II

P(t)	computed σ (kg/cm^2)		measured σ (kg/cm^2)	
	method K	parabola of the 2nd degree	A	B
0,5	17,05	20,42	22,4	23,8
1,0	33,7	40,5	44,9	47
1,5	50,7	60,4	67,3	70
2,0	67,7	80,5	89,8	92,5
2,5	84,5	101	112,2	115
3,0	101,5	121	134,7	138
3,5	118,9	141	157,1	160
4,0	135,2	161	179,5	184
4,5	152,1	181	202,-	207
5,0	169,1	201	224,4	229,5

Distribution as a Parabola of the second degree.

By adopting this method one gets easily the figures of table III.

TABLE III

P (t)	P_m (t/m2)	probable mean value of $E_{s,m}$ kg/cm2 (page 144)	$\frac{P_C}{P_m}$ for I =949 cm^4	$M_{C,2}$ in t/m. form (2)	σ kg/cm^2
0	0	385	0,69	0	0
0,5	2	401,5	0,6925	0,0721	20,42
1,0	4	483,5	0,715	0,143	40,5
1,5	6	544	0,735	0,213	60,4
2	8	544	0,735	0,284	80,5
2,5	10	544	0,735	0,355	101,-
3	12	544	0,735	0,426	121,-
4	16	544	0,735	0,567	161,-
5	20	544(?)	0,735	0,709	201,-

Method of the modulus of soil reaction.

One gets the figures of table IV.

TABLE IV

P (t)	P_m (t/m2)	probable mean value of E_s kg/cm^2 (page 144)	probable mean value of K_m kg/cm^3 (form 4)	$\frac{al}{2}$	m'_C	M'_C t/m form.(5)	σ kg/cm^2
0	0	385	12,9	0,711	4,15	0	0
0,5	2	401,5	13,45	0,72	4,155	0,0601	17,05
1,0	4	483,5	16,2	0,756	4,18	0,119	33,7
1,5	6	544	18,3	0,775	4,193	0,179	50,7
2,-	8	544	18,3	0,775	4,193	0,2385	67,7
2,5	10	544	18,3	0,775	4,193	0,298	84,5
3,-	12	544	18,3	0,775	4,193	0,358	101,5
4,-	16	544	18,3	0,775	4,193	0,477	135,2
5,-	20	544(?)	18,3	0,775	4,193	0,597	169,1

For comparison the computed values of σ are also indicated on table II.

TESTS WITH A T BEAM OF LOW STIFFNESS.

A third serie of tests was performed on a T beam, with the section indicated on fig. 8, and with the following characteristics:
ℓ = 1,00 m b = 0,25 m h = 0,0665 m
I_{xx} = 54,7 cm^4 $\frac{I_{xx}}{v}$ = 48,7 cm^3.

The values of the stress in the inferior fibre of the central section, measured with the strain-indicator, when the load was several times increased and decreased between 0 and 5 tons, are given in the table V. For the dame value of the load, the readings on the strain-indicator were rather variable from one cycle to another; thus only the central values are given.

Distribution as a Parabola of the second degree.

If $\frac{P_C}{P_m} > 1{,}5$, a reduced lenght ℓ'' is considered, defined by

$$\ell'' = \ell \sqrt{\frac{\frac{P_c}{P_m}}{3\left(\frac{P_c}{P_m} - 1\right)}} \qquad (7)$$

TABLE V

P (t)	σ computed method K	(kg/cm²) parabola of the 2nd degree	σ measured (kg/cm²)
0,5	96,4	90,6	80
1	183,5	172	160
1,5	267	249	230
2	357	332	305
2,5	444	414	381
3	534	497	460
3,5	622	569	539
4	710	642	619
4,5	801	733	690
5	892	824	770

on the other hand $p_C = \frac{3}{2} p_m \frac{\ell}{\ell''}$ (8)

and $M_C = - \frac{b\,\ell''^2}{16} p_C$ (9)

The values of the table VI are thus easily obtained.

TABLE VI

P (t)	p_m (t/m²)	probable mean value of E_s (kg/cm²) (page144)	$\frac{p_C}{p_m}$ for I = 54,7 cm⁴ (tractions existing)	ℓ'' (m) form (7)	p_C (t/m²) form(8) (tractions zero)	$M_{C,2}$ (t.m.) formula (9)	σ (kg/cm²)
0	0	385	1,561	0,961	0	0	0
0,5	2	401,5	1,598	0,944	3,18	0,0442	90,6
1,0	4	483,5	1,72	0,891	6,74	0,0839	172,-
1,5	6	544	1,81	0,859	10,50	0,1212	249,-
2	8	544	1,81	0,859	14,-	0,1619	332,-
2,5	10	544	1,81	0,859	17,50	0,2020	414,-
3	12	544	1,81	0,859	21,-	0,2421	497,-
4	16	544	1,81	0,859	28,-	0,3135	642,-
5	20	544	1,81	0,859	34,9	0,403	824,-

Method of the modulus of soil reaction K.

If $\frac{a\ell}{2} > \frac{\pi}{2}$, the fictive length $\lambda = \frac{\pi}{a}$ is introduced.

For the rest the figures of the table VII are easily obtained.

TABLE VII

P (t)	p_m (t/m²)	probable mean value of E_s (kg/cm²) (page144)	probable mean value of K_m (kg/cm²) (form.4)	$\frac{al}{2}$	m'_C	M'_C (t.m.) (form(5))	σ (kg/cm²)
0	0	385	12,9	1,455	5,305	0	0
0,5	2	401,5	13,45	1,471	5,338	0,0469	96,4
1,0	4	483,5	16,2	1,54	5,60	0,0893	183,5
1,5	6	544	18,3	1,585	5,75	0,1301	267
2,0	8	544	18,3	1,585	5,75	0,1738	357
2,5	10	544	18,3	1,585	5,75	0,216	444
3,0	12	544	18,3	1,585	5,75	0,26	534
4,0	16	544	18,3	1,585	5,75	0,346	710
5,0	20	544	18,3	1,585	5,75	0,434	892

The computed values of σ are also indicated for comparison on table V.

CONCLUSIONS OF THE TESTS.

An attentive inspection of the tables I, II and V gives the following conclusions:

1) From all the used methods of computation, this of the parabolic distribution of the second degree, gives results which approach best the results of the tests, and this is true whatever is the stiffness of the used beams.

2) For beams with a high relative stiffness, the method of the modulus of soil reaction furnishes values which are clearly too small; this corresponds to the theoretical deductions.

3) The results of the tests are situated between the values obtained with the method of the modulus of soil reaction and those given by the method of Schiel-Wieghardt; this corresponds to the theoretical deductions.

4) For beams with a very low relative stiffness, the method of the modulus of soil reaction K and also this of the parabolic distribution of the second degree give values

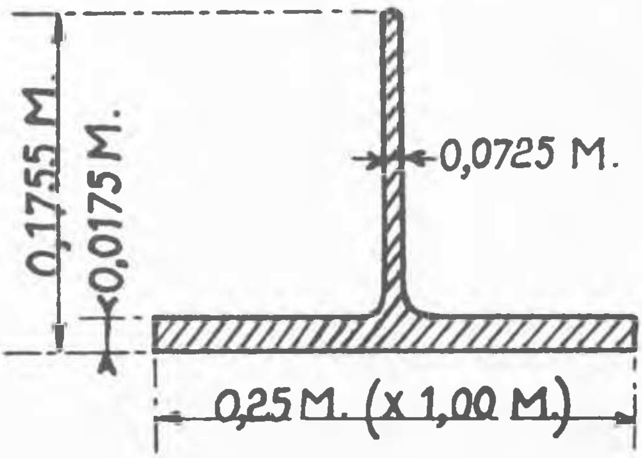


FIG.7

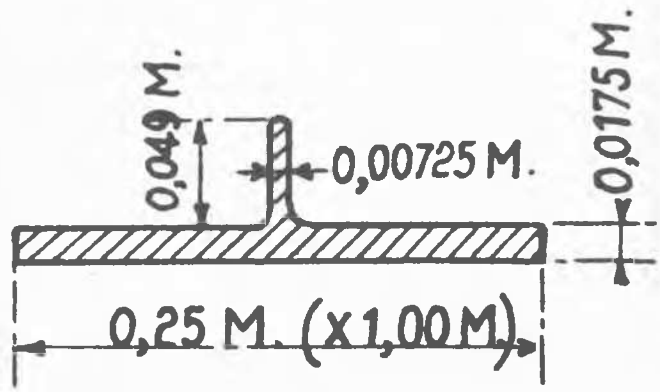


FIG.8

higher than those of the tests; the values obtained by the method K are higher than those of the parabolic distribution. These results concord again with the theoretical deductions.

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SUB-SECTION VI c

INFLUENCE OF GROUNDWATER

VI c 1 INFLUENCE OF GROUND WATER LEVEL OSCILLATION ON SUBSIDENCE OF STRUCTURES

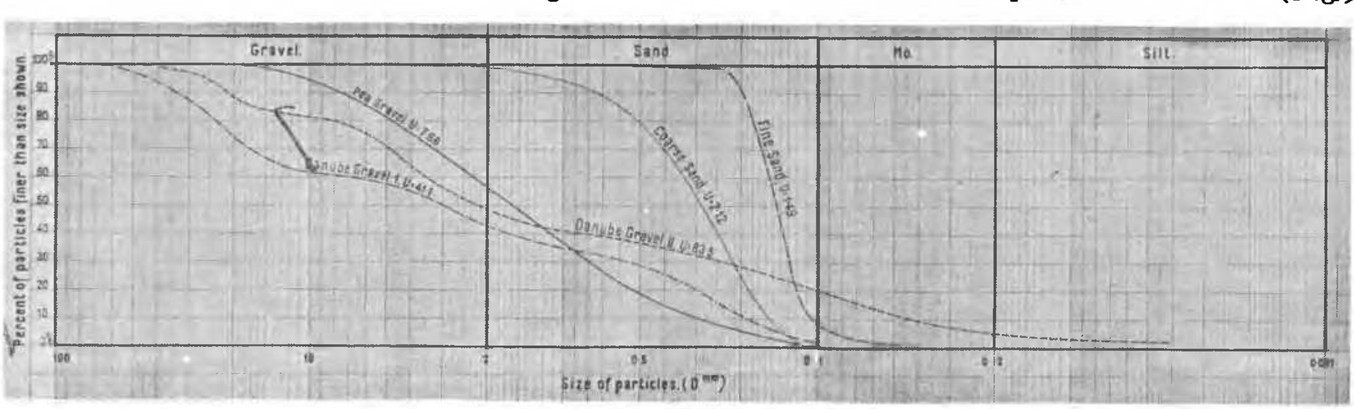
Prof. Dr. JÓZSEF JÁKY

Structures built on loose soil near to rivers suffer suddenly great and unequal subsidence when flooded. In Budapest, in 1940 and 1941, the Electric Works, apartment houses and other structures along the banks of the Danube suffered heavy damages and cracks when the level of the Danube rose over two metres above the flood mark. The author, using an apparatus designed for this purpose (Fig.1.) determined by laboratory tests the percentage of maximum settling (ε %) of different kinds of granular soils, in relation to load and also the ways of protection. Grain size distribution curves of the tested materials are shown on Fig. 2.

The empirical relations gained by experiments are as follows.

Compressing granular material in the cylinder shown on Fig. 1. with pressure p kg/cm^2 then flooding it with rising water by opening the valve in the lower part of the apparatus, the soil sample ceases to be consolidated and settles under constant load.

1) The specific rate of settling (ε %) varies with the weight of the structure in accordance to Gauss' probability curve. Increasing the load, settling becomes greater, at a critical p_0 kg/cm^2 it reaches the maximum and then diminishes so that if $p \to \infty$ it becomes $\varepsilon \to 0$. (Fig.3)



Grain Size Distribution Curves.

FIG.2